resisted by concrete. The area of hoop steel may be obtained by equating the strain energy stored in hoop steel and in the concrete in principal compression to the work done by the torsion in rotating the beam.[3] Bach and Graf at Stuttgart have experimentally shown that hoops without longitudinal reinforcement are ineffective against torsion and that an equal volume of longitudinal steel should be provided.

For the same bending stress, the principal tensile stress is a function of shear stress. Thus torsion is a primary design factor if torsional shear stress exceeds transverse shear stress. However, if the latter exceeds the former, then transverse shear is a primary design factor.

It has been suggested[2] that if torsion be the primary design factor, reinforcement be designed for it after taking due account of the torsion-taking-capacity of unreinforced section, while entire Transverse shear be taken on additional reinforcement. But if torsion is only a secondary design factor, reinforcement be designed for transverse shear after taking due account of the transverse-shear-taking-capacity of the unreinforced section, while entire torsion be taken on additional reinforcement.

Suggested 'Step by Step' Procedure

Step 1 Estimate V_u (load-factored dead and live load shears less any relief in shear due to prestress and sloping soffit, assuming unit load factor on prestress) and T_u (load factored torsional moment, due to dead and live load and any other applied load) at the section.

Step 2 Evaluate equivalent ultimate transverse shear stress (v_u) and the ultimate torsional shear stress (σ_u)

v_u may be estimated from $v_u = \dfrac{V_u}{bd}$

where b = breadth of rib

d = '0.8 of effective depth' in case of reinforced concrete, and

= '0.8 of overall depth or the effective depth, whichever greater' in case of prestressed concrete

and σ_u may be found using the appropriate formula given in colume 3 in Table 24.2 of formulae for shear stress due to torsion (elastic and ultimate) for different shapes of sections.

It may be noted that σ_u is constant throughout the section, but v_u (the greatest ultimate transverse shear stress) may occur at about the mid-height fibre or the mid-width fibre depending on whether it is vertical transverse shear or horizontal transverse shear. In case both these shears (V_{uv} and V_{uh}) coexist, then $v_{uv} = \dfrac{V_{uv}}{bd}$ and $v_{uh} = \dfrac{V_{uh}}{DB}$ (where D = overall depth, $B = 0.8$ of breadth of rib) and use bigger of v_{uv} and v_{uh}

Step 3 Check that $(v_u + \sigma_u) \not> 0.15$ of standard 28-day concrete cylinder crushing strength

If this is not so, then increase section, recalculate v_u and σ_u, and ensure the above stipulation before going to Step 4 ahead.

Step 4 Now it is either $v_u > \sigma_u$ or $\sigma_u > v_u$; proceed correspondingly,

... if $v_u > \sigma_u$ (i.e., transverse shear governs, then design for full T_u and only for part of V_u)

then $A_t = \dfrac{T_u p_t}{f_{sy} 0.8 x_1 y_1}$

and $A_v = \dfrac{(V_u - V_c)}{f_{sy} d} p_v$

where V_c = (10 bd) in kg, b and d in cm.

(V_c = shear capacity of concrete alone)

... if $\sigma_u > v_u$ (i.e., torsion governs, then design for full V_u and only for part of T_u)

then $A_t = \dfrac{(T_u - T_c)}{f_{sy} 0.8 x_1 y_1} p_t$

where T_c (the torsion capacity of concrete alone) may be found using the appropriate formula given in column 3 in Table 24.2 of formulae for shear stress due to Torsion on different shapes of sections, assuming the limiting torsional shear stress $\sigma = 0.1$ of standard cylinder strength,

and $A_v = \dfrac{V_u p_v}{f_{sy} d}$

where p_t = pitch of torsion hoops (effectively closed stirrups)

p_v = pitch of vertical stirrups (for vertical transverse shear)

A_t = twice the section area of torsion hoop leg, hoop placed at pitch p_t ($1/2 A_t$ is section area of one horizontal or vertical leg of hoop),

A_v = section area of all vertical shear stirrup legs at the section, placed at pitch p_v,

f_{sy} = yield stress in hoop and stirrup reinforcements,

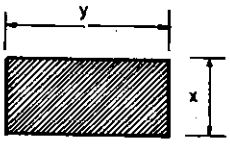
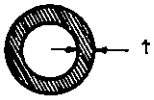
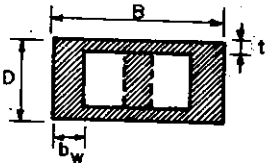
x_1, y_1 = the lengths of the short and long sides of the torsion hoop,

d = as defined above in Step 2.

Step 5 Provide longitudinal reinforcement perpendicular to the hoops, inside them, and proportionately distributed along the hoop perimeter); section area of this longitudinal reinforcement is,

$$A_l = A_t \frac{x_1 + y_1}{p_t}$$

Table 24.2 *Relation between torsional shear stress σ and torsional moment T*

Column 1	Column 2	Column 3
Section	in *ELASTIC phase* T and σ are elastic (working-load) values	*At ULTIMATE* T and σ are the ultimate values
	$\sigma = \dfrac{16T}{\pi D^3}$ at surface at any point	$\sigma = \dfrac{12T}{\pi D^3}$ at every point in section
 x = small side y = long side	at middle of long side, on surface, $\sigma = \dfrac{T}{\alpha_1 x^2 y}$ at middle of short side, on surface, $\sigma = \dfrac{T}{\alpha_2 x y^2}$ α_1 and α_2 from Table 2.1 in Reference 2 $\sigma = 0$ at corners and cg (ALSO SEE NOTE* below)	$\sigma = \dfrac{2T}{x^2(y - x/3)}$ at every point in section
	$\sigma = T/2At$ at any point in section, A = mean of the areas enclosed by inner and outer boundaries of the tube of thickness t	Same formula as for Elastic phase (col. 2)
 Neglect Any Intermediate Webs	at middle of long side: $\sigma = \dfrac{T}{2t(D-t)(B-b_w)}$ at middle of short side: $\sigma = \dfrac{T}{2b_w(D-t)(B-b_w)}$	$\sigma = \dfrac{T}{(b_w+t)(D-t)(B-b_w)}$ at every point in the section
T or L section	See Reference 2 (ALSO SEE NOTE* below)	See Reference 2

**NOTES*

(i) *Rectangular section (Fig. 24.8)*

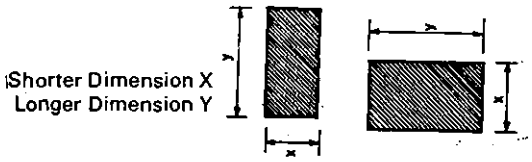


Fig. 24.8

Torsional constant is,

$J = \gamma x^3 y$, where γ depends on the ratio of $\dfrac{y}{x}$

This was dicovered by St. Venant and the γ values are tabulated below

y/x	1	1.5	2	3	5	>5
γ	0.14	0.20	0.23	0.26	0.29	0.33

A formula which will give values within 4% of these is

$$\gamma = 0.33 - 0.21\frac{x}{y}\left(1 - \frac{x^4}{12y^4}\right)$$

(ii) *T, I or L sections (Fig. 24.9)*

Torsional constant is

$J = \gamma_1 b_w^3 h + \gamma_2 h_t^3(b_t - b_w) + \gamma_3 h_b^3(b_t - b_w)$

The γ's depend on the individual y/x ratio (i.e., ratio of longer side to smaller side) for each rectangle where it should be remembered that y and x are the longer and shorter dimensions respectively, e.g., for the top flange y is $(b_t - b_w)$ and x is h_t.

A different value will be found if the section is divided up as follows (Fig. 24.10):

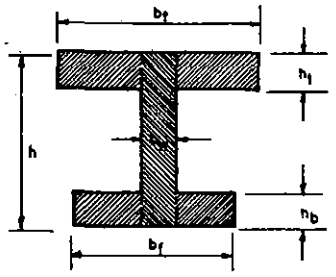


Fig. 24.9

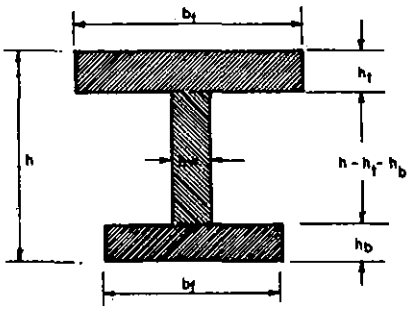


Fig. 24.10

$$J = \gamma_1 h_t^3 b_t + \gamma_2 h_b^3 b_t + \gamma_2 b_w^3 (h - h_t - h_b)$$

Where torsion clauses are based on the work of Prof. Cowan, it is the latter method of dividing up the section which is suggested. Where proportioning the torsional moment between individual rectangles, the widest rectangle should be taken as long as possible. To be consistent therefore, if b_w is greater than h_t and h_b use the first method of dividing up the section. If not, then the use the second method.

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1. Swami, N, Behaviour and Ultimate Strength of Prestressed Concrete Hollow Beams under Combined Bending and Torsion—(Magazine of *Concrete Research*, vol. 14, no. 40, C and CA., London), March 1962.
2. Cowan and Lyalin, *Reinforced and Prestressed Concrete in Torsion*, Edward Arnold, London.
3. Cowan, "Torsion of Rectangular Elastic Isotropic Beam Reinforced with Helics of another Material", *Appl. Scientific Research*, (Delft) section A, 3, 1952.
4. Raina, VK, "Elastic and Ultimate Design Against Diagonal Cracking in Prestressed Concrete Members Under Combined Torsion, Flexure and Shear", Seminar on Problems in Prestressing, ING of IABSE Madras, February 1970, IRC, New Delhi.
5. ACI and AASHTO Design Specifications.

CHAPTER 25

Design of a Reinforced Concrete Section Subject to Combined Axial Thrust and Any-Axis Bending—Simplified Practical Method

Synopsis

Most of the methods available for designing reinforced concrete sections subjected to axial thrust and bending about any axis—orthogonal or oblique, are at best terribly theoretical and at worst inaccurate. This leaves a practising designer to the mercy of 'established' graphs and charts whose scope is very limited, in that, they are not available for all shapes of sections. This chapter very simply derives the general expression for stresses for the general cases of combined axial thrust and oblique bending in a section of *any* shape. The design procedure is then illustrated, step by step, through five typical examples.

NOTES 1. The method outlined in this presentation is applicable even if the section is prestressed. In such a case the prestress force and moment may be added respectively to the axial thrust and bending moment (and the concept becomes akin to that in partial —prestressing where reinforcement is provided to carry certain amount of tension).

2. The method outlined in this presentation is 'exact' and yet very 'simplified'.

3. The author acknowledges with grateful thanks Engr. S.G. Athavale who initially evolved the basic content of the theoretical derivation presented here.

25.1 INTRODUCTION

Under a purely axial load (zero eccentricity), an RC section suffers only direct compression. As the eccentricity is introduced and increased, additional bending stresses (tensile and compressive) are created and so long as the resultant stresses (calculated using full section properties) are within permissible limits, cracking of concrete is insignificant. However, if the resultant maximum tensile stress in concrete exceeds its permissible value, the section is then assumed to have cracked and full section properties cannot be assumed effective any more. The effective section then is less than the full physical section as the concrete below the neutral axis (i.e., in tension) is considered ineffective. The thrust P, applied at an eccentricity e from the physical centroid of the overall section (e = resultant applied bending moment/P) is withstood only by the effective section whose centroid is no more the physical centroid of the overall section. If e' is the shift between these two centroids, then the thrust P in fact acts at an eccentricity of only $(e - e')$ from the centroid of the effective section.

In a section subjected to a thrust combined with two orthogonal moments (causing bending about an oblique axis), in working-load design one of the approaches (though erroneous) is to first assume a monoaxial bending with thrust, and on the section properties of this monoaxially cracked section then superimpose the effect of the other orthogonal moment and later finally apply some stress correction—if at all. Obviously, such a method is less powerful because it is an indirect procedure in attempting to arrive at the final bending behaviour of the section—not to speak of ignoring the e' effect in an oblique direction as well as the effect of rotation of neutral axis.

It is better to have approximate solution to an exact problem instead of an exact solution to an approximate problem.

25.2 THEORY

Consider the section as shown in Fig. 25.1, subjected to a thrust P and a resultant bending moment M_R (vectorial resultant), such that this combination is equivalent to P applied at a distance $e = \frac{M_R}{P}$ from the physical centroid of the overall section, as shown. Let f be the extreme compressive stress, n the distance of the neutral axis below the extreme compressed fibre, p the stress at the level of the centroid of the effective section, x, y, and $\bar{y}$ the various distances as shown, and dA the transformed area of the element shown shaded (Fig. 25.1).

Force in the element = stress × area

$$= \frac{f}{n}(x + \bar{y})dA \qquad (25.1)$$

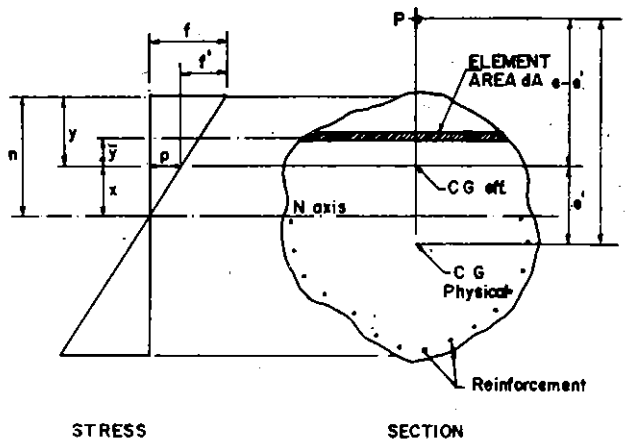


Fig. 25.1

and also $$= \left(p + \frac{f'}{y}\bar{y}\right) dA \qquad (25.2)$$

Its moment about centroid of effective section

$$= \text{force} \times \text{arm}$$
$$= \left(p + \frac{f'}{y}\bar{y}\right) dA \cdot \bar{y} \qquad (25.3)$$

Equating external and internal forces,

$$P = \Sigma \frac{f}{n}(x + \bar{y})dA$$
$$= \frac{f}{n}(\Sigma x dA + \Sigma dA \cdot \bar{y}) \qquad (25.4)$$

But $\Sigma dA \cdot \bar{y}$, the moment of area about its own centroid, is equal to zero, so that,

$$P = \frac{f}{n}(\Sigma x dA + 0)$$
$$= \frac{f}{n} x \Sigma dA$$

$= p.A_{\text{eff}}$ ($\because \frac{f}{n} x = p$ and ΣdA = total effective area $A_{\text{eff.}}$)

Thus $$p = \frac{P}{A_{\text{eff}}} \qquad (25.5)$$

Equating moments due to external and internal forces bout the centroid of effective section,

$$P(e - e') = \Sigma \left(p + \frac{f'}{y}\bar{y}\right) dA \cdot \bar{y}$$

$$= p\Sigma dA \cdot \bar{y} + \frac{f'}{y}\Sigma dA \cdot (\bar{y})^2 \qquad (25.6)$$

but $\Sigma dA \cdot \bar{y} = 0$, and $\Sigma dA \cdot (\bar{y})^2$ = second moment of effective area about its own centroid i.e., I_{eff}, thus,

$$P(e - e') = 0 + \frac{f'}{y} I_{\text{eff}}$$

so that $$f' = \frac{P(e - e')}{I_{\text{eff}}} y \qquad (25.7)$$

Since total stress f at any fibre distant $\pm y$ from centroid of effective section comprises of direct compression p and bending stress $\pm f'$, thus,

$$f = p \pm f'$$

substituting from Eqs. (25.5) and (25.7) we get;

$$f = \frac{P}{A_{\text{eff}}} \pm \frac{P(e - e')}{I_{\text{eff}}} y \qquad (25.8)$$

However, it should be noted that this formula is applicable when the cracked section is symmetrical about the load line. (Load line is the prependicular from load point P to the axis of resultant moment.) In other words, this formula is applicable when the neutral axis is parallel to the axis of the resultant moment so that load line would be perpendicular to neutral axis itself. This is so in the case of circular sections, solid or annular, under monoaxial, biaxial or multiaxial bending. This is so also in the case of rectangular, square and triangular (equilateral and isosceles) sections, solid or box, so long as the resultant moment acts about an axis which falls perpendicular to the axis of symmetry of the effective section.

When the cracked section is not symmetrical about the load line, i.e., the load line is not a principal axis, then the neutral axis is not parallel to the axis of the resultant moment. The simple superimposition of bending stresses calculated individually with respect to the two orthogonal axes is obviously invalid if these axes are not principal axes, because then it is the case of the effective section being asymmetrical about these axes. In such a case, the effective section is subjected to bending about two orthogonal axes — neither of which is a principal axis — and the superimposition of stresses has then to be on the following lines:

If $T - T$ and $L - L$ are the two orthogonal axes through the physical centroid of the overall section $b \times d$, M_T and M_L are moments about these axes and P the axial thrust, then the extreme fibre stresses in the cracked section are

given by,

$$\frac{P}{A_{\text{eff}}} \pm \left[\frac{P(e_L - e'_L) - P(e_T - e'_T)\left(\frac{I_{LT_{\text{eff}}}}{I_{\text{eff}_T}}\right)}{I_{\text{eff}_L} - \left(\frac{I^2_{LT_{\text{eff}}}}{I_{\text{eff}_T}}\right)}\right] \times$$

$$\left\{\left(\frac{b}{2} - e'_L\right) \text{ or } \left(\frac{b}{2} + e'_L - \text{ cover to reinforcement}\right), \text{ depending on which fibre}\right\}$$

$$\pm \left[\frac{P(e_T - e'_T) - P(e_L - e'_L)\left(\frac{I_{LT_{\text{eff}}}}{I_{\text{eff}_T}}\right)}{I_{\text{eff}_T} - \left(\frac{I^2_{LT_{\text{eff}}}}{I_{\text{eff}_L}}\right)}\right] \times$$

$$\left\{\left(\frac{d}{2} - e'_T\right) \text{ or } \left(\frac{b}{2} + e'_T - \text{ cover to reinforcement}\right), \text{ depending on which fibre}\right\}$$

where e_T and e_L are the load eccentricities from $T-T$ and $L-L$ axes, respectively $\left(e_T = \frac{M_T}{P}\right.$ and is measured along $L-L$ axis, $e_L = \frac{M_L}{P}$ and is measured along $T-T$ axis$\left.\right)$, e'_T and e'_L are the distances from the T and L axes to the cg of the effective section.

I_{eff_T} and I_{eff_L} are the second moments of area (moments of inertia) of the effective section about axes parallel to T and L axes passing through its centroid.

$I_{LT_{\text{eff}}}$ is the product of inertia of the effective section about axes parallel to L and T axes passing through its centroid. The inclination ϕ of the axis of the resultant moment to $L-L$ axis is given by,

$$\tan\phi = \frac{M_T}{M_L}$$

The inclination ϕ' of the N-axis is given by,

$$\tan\phi' = \frac{M_T \cdot I_L - M_L \cdot I_{LT}}{M_L \cdot I_T - M_T \cdot I_{LT}}$$

(I_{LT} represents product of inertia)

$\phi = \phi'$ only when $I_L = I_T$ and $I_{LT} = 0$

To locate the inclination ϕ in a cracked section it is sufficient to initially adopt the properties of uncracked section in the above formula, taking $I_{LT} = 0$. However, as has been shown in Examples 4 and 5 ahead, the value of ϕ can be finally verified by the following formula,

$$\tan\phi = \frac{P(e_T - e'_T)I_{\text{eff}_L} - P(e_L - e'_L)I_{LT_{\text{eff}}}}{P(e_L - e'_L)I_{\text{eff}_T} - P(e_T - e'_T)I_{LT_{\text{eff}}}}$$

or $$\tan\phi = \frac{(e_T - e'_T)I_{\text{eff}_L} - (e_L - e'_L)I_{LT_{\text{eff}}}}{(e_L - e'_L)I_{\text{eff}_T} - (e_T - e'_T)I_{LT_{\text{eff}}}}$$

25.3 EXAMPLES

Example 1

Case of a solid rectangular RC section subjected to axial thrust and monoaxial bending about a side. Consider a solid rectangular RC section (Fig. 25.2) of size 120 cm × 60 cm. Modular ratio $m = 14$. Axial thrust $P = 98$ tonnes and bending moment $M = 83.5$ tm about axial parallel to its breadth.

Step 1 Assume 40 cm^2 of mild steel in compression as well as in tension flanges, as shown.

Step 2 Assume neutral axis at 55 cm from extreme compression fibre.

Step 3 $A_{\text{eff}} = (60 \times 55) + (14 - 1)40 + (14 \times 40)$

$= 3300 + 520 + 560 = 4380$ cm^2

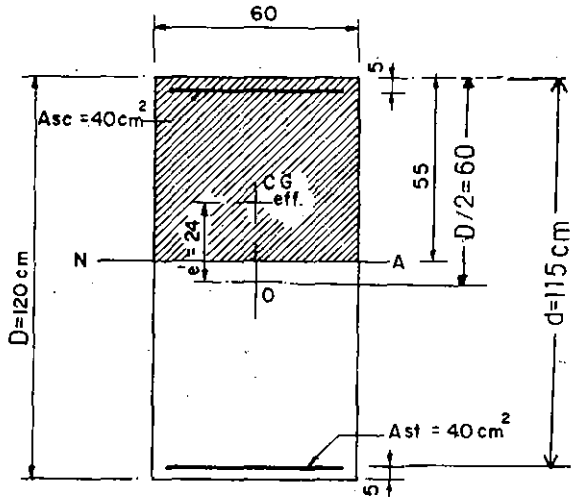


Fig. 25.2

Step 4 Distance of CG_{eff} from physical centroid (take moments of effective areas about physical centroid)

$$e' = \frac{(3300 \times 32.5) + (520 \times 55) - (560 \times 55)}{4380} = 24 \text{ cm.}$$

$$e - e' = \frac{83.5 \times 100}{98} - 24.0 = 85.2 - 24.0 = 61.2 \text{ cm}$$

Step 5 $I'_{\text{eff}} = 60 \times \frac{55^3}{12} + 60 \times 55 \times 8.5^2 + 520 \times 31^2 + 560 \times 79^2$

$= 50{,}70{,}000$ cm^4

Step 6 Distance of neutral axis below CG_{eff}

$$\text{At } y \text{ below } CG_{eff}, \text{ stress} = \frac{P}{A_{eff}} - \frac{P(e-e')y}{I_{eff}}$$

This stress is zero at neutral axis, so that diatance of neutral axis below CG_{eff} will thus be given by:

$$y = \frac{I_{eff}}{A_{eff}(e-e')}$$

$$\text{i.e., } \frac{50,70,000}{4380 \times 61.2} = 18.0 \text{ cm compared to } (24-5)$$

$$= 19 \text{ cm, assumed, hence OK}$$

Step 7 (i) Max. compressive stress in concrete,

$$= \frac{P}{A_{eff}} + \frac{P(e-e')\left(\frac{D}{2}-e'\right)}{I_{eff}}$$

$$= \frac{98,000}{4380} + \frac{98,000 \times 61.2}{50,70,000}\left(\frac{120}{2} - 24\right)$$

$$= 22.4 + 42.5 = 64.9 \text{ kg/cm}^2$$

OK for the concrete chosen.

(ii) Max. tensile stress in steel,

$$= m\left\{\frac{P}{A_{eff}} - \frac{P(e-e')\left(d - \frac{D}{2} + e'\right)}{I_{eff}}\right\}$$

$$= 14\left\{22.4 - \frac{98,000 \times 61.2(115-60+24)}{50,70,000}\right\}$$

$$= 14(22.4 - 93.4) = -994 \text{ kg/cm}^2$$

OK for steel chosen.

Example 2

Case of a solid circular RC section subjected to axial thrust and bending.

Consider a solid circular RC section (Fig. 25.3) of 120 cm dia. Modular ratio $m = 18$. Axial thrust $P = 32$ tonnes and bending moment $M = 30.5$ tm.

Step 1 Assume 35 cm^2 of mild steel uniformly distributed along the periphery.

Step 2 Assume neutral axis at 36 cm from extreme compression fibre.

Step 3 Reading the values of constants from Fig. 25.4 for section properties of circular segment above neutral axis,

$$\text{for } \frac{x}{d} = \frac{36}{120} = 0.3,\ A = 0.65,\ B = 0.20$$

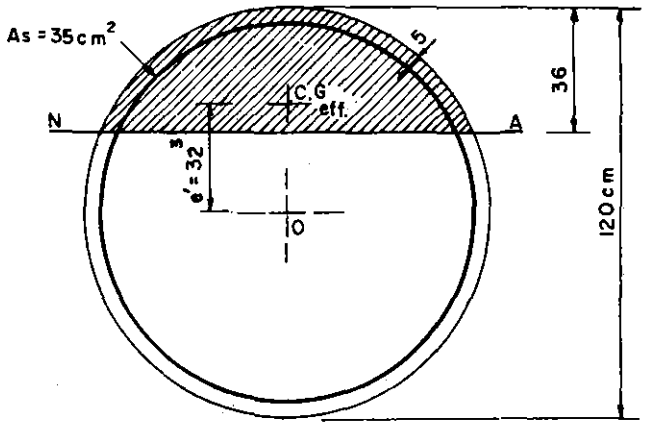


Fig. 25.3

and $C = 0.02$

$\therefore\ \bar{x} = 0.65 \times 60 = 39$ cm (distance of cg of segment from centre of circle)

$A = 0.20 \times 120^2 = 2880$ cm^2 (section area of segment)

and $I = 0.02 \times 60^4 = 2,59,200$ cm^4 (second moment of area of segment about its own centroid).

Step 4 For convenience taking transformed area of steel as $(m-1)A_s$, both in tensile as well as in compressive zones, the effective area is,

$$A_{eff} = 2880 + (18-1)35$$

$$= 2880 + 595 = 3475 \text{ cm}^2$$

Step 5 Distance of CG_{eff} from physical centroid of whole section is

$$e' = \frac{2880 \times 39}{3475} = 32.3 \text{ cm (by taking moments of effective areas about physical centroid)}$$

$$\therefore\ e - e' = \frac{30.5 \times 100}{32} - 32.3 = 95.3 - 32.3$$

$$= 63 \text{ cm}$$

Step 6 $I_{eff} = 2,59,200 + 2880 \times 6.7^2 + \left(595 \times \frac{55^2}{2} + 595 \times 32.3^2\right)$

$$= 19,08,000 \text{ cm}^4$$

Step 7 Distance of neutral axis below CG_{eff} (as in Example 1) is,

$$\frac{I_{\text{eff}}}{A_{\text{eff}}(e-e')} = \frac{19,08,000}{3475 \times 63} = 8.7 \text{ cm}$$

compared to $(32.3 - 24.00) = 8.3$ cm assumed, OK

Step 8 (i) Max. compressive stress in concrete,

$$= \frac{P}{A_{\text{eff}}} + \frac{P(e-e')(D/2-e')}{I_{\text{eff}}}$$

$$= \frac{32000}{3475} + \frac{32000}{19,08,000} \times 63\left(\frac{120}{2} - 32.3\right)$$

$$= 9.2 + 29.2 = 38.4 \text{ kg/cm}^2, \text{ OK}$$

(ii) Max. tensile stress in steel,

$$= m\left(\frac{P}{A_{\text{eff}}} - \frac{P(e-e')(d-D/2+e')}{I_{\text{eff}}}\right)$$

$$= 18\left(9.2 - \frac{32000 \times 63(115-60+32.3)}{19,08,000}\right)$$

$$= 18(9.2 - 92.0) = -1490 \text{ kg/cm}^2, \text{ OK}$$

Example 3

Case of a hollow circular RC section subjected to axial thrust and bending.

Consider a hollow circular RC section (Fig. 25.5) of outer dia. 600 cm and inner dia. 360 cm. Modular ratio $m = 18$. Axial thrust $P = 630$ tonnes and bending moment $M = 1850$ tm.

Step 1 Assume 100 cm^2 of mild steel at outer as well as inner faces of hollow section, uniformly distributed along the peripheries.

Step 2 Assume neutral axis at 150 cm from extreme compression fibre.

Step 3 Taking constants from Fig. 25.4 for the two circular segments (bounded by outer and inner circles above neutral axis,

Outer circle

$$\frac{x}{d} = \frac{150}{600} = 0.25$$

$$\therefore A = 0.70; \quad B = 0.15; \quad C = 0.01$$

Inner circle

$$\frac{x}{d} = \frac{30}{360} = 0.083$$

$$\therefore A = 0.905; \quad B = 0.03; \quad C = 0 \text{ (approx.)}$$

so that $\bar{x} = 0.70 \times 300 = 210$ cm (for bigger segment)

$\bar{x} = 0.905 \times 180 = 163$ cm (for smaller segment)

Area $= 0.15 \times 600^2 = 54,000$ cm^2 (for bigger segment)

Area $= 0.03 \times 360^2 = 3890$ cm^2 (for smaller segment)

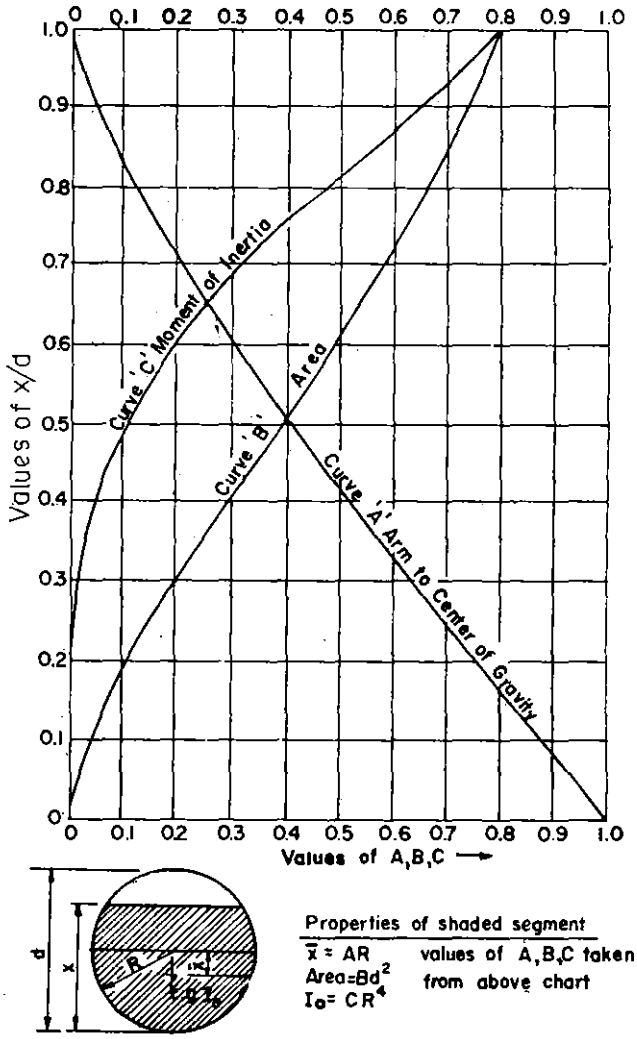


Fig. 25.4 *Constants for Properties of Circular Segments (Ref. page 703 of RC Fundamentals by Phil M Ferguson, 2nd Edition, Wiley Eastern.)*

$I = 0.01 \times 300^4 = 810,00,000$ cm^4 (for bigger segment)

$I = 0$ (approx.) (for smaller segment)

Step 4 For convenience, taking transformed area of steel as $(m-1)A_s$, both in tensile and compressive zones,

$$A_{\text{eff}} = 54,000 - 3890 + (18-1)(100+100)$$
$$= 53510 \text{ cm}^2$$

Step 5 Distance of CG_{eff} from physical centroid:

$$e' = \frac{(54,000 \times 210) - (3890 \times 163)}{53510} = 200 \text{ cm}$$

$$\therefore (e - e') = \frac{1850 \times 100}{630} - 200 = 294 - 200$$
$$= 94 \text{ cm}$$

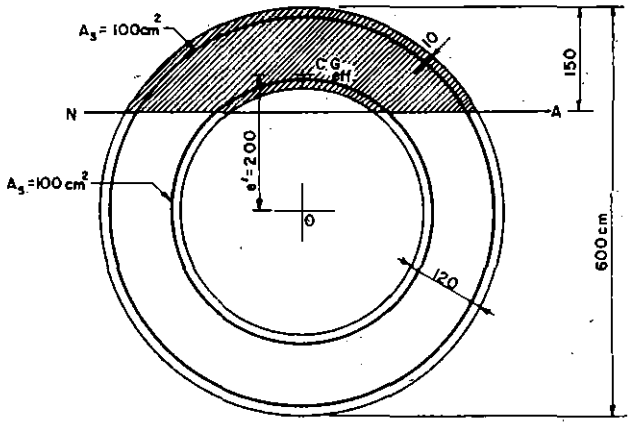


Fig. 25.5

Step 6 $I_{\text{eff}} = 810,00,000 + 54,000 \times 10^2 - 3890 \times 37^2$

$$+(18-1)100\left(\frac{290^2+190^2}{2}\right) + (18-1)(100+100)200^2$$

$= 25,79,00,000 \text{ cm}^4$

Step 7 Distance of neutral axis below CG_{eff} is given by,

$$\frac{I_{\text{eff}}}{A_{\text{eff}}(e-e')} = \frac{25,79,00,000}{53510 \times 94}$$

= 51.2 cm compared to 200 − 150 = 50 cm assumed, OK

Step 8 (i) Max. compressive stress in concrete

$$= \frac{P}{A_{\text{eff}}} + \frac{P(e-e')(D/2-e')}{I_{\text{eff}}}$$

$$= \frac{630,000}{53,510} + \frac{630,000 \times 94(600/2-200)}{25,79,00,000}$$

$= 11.8 + 23.0 = 34.8 \text{ kg/cm}^2$, OK

(ii) Max. tensile stress in steel

$$= m\left(\frac{P}{A_{\text{eff}}} - \frac{P(e-e')(d-D/2+e')}{I_{\text{eff}}}\right)$$

$$= 18\left(11.8 - \frac{6,30,000 \times 94(590-300+200)}{25,79,00,000}\right)$$

$= 18(11.8 - 112.4) = -1810 \text{ kg/cm}^2$, OK

Example 4

Case of a solid rectangular section subjected to axial thrust and biaxial bending.

Consider a solid rectangular RC section (Fig. 25.6) of size 75 cm× 200 cm Modular ratio m = 14. Axial thrust P = 160 tonnes. Bending moments M_T = 180 mt. and M_L = 50 mt. TT and LL axes are orthogonal axes parallel to breadth and length of section, respectively.

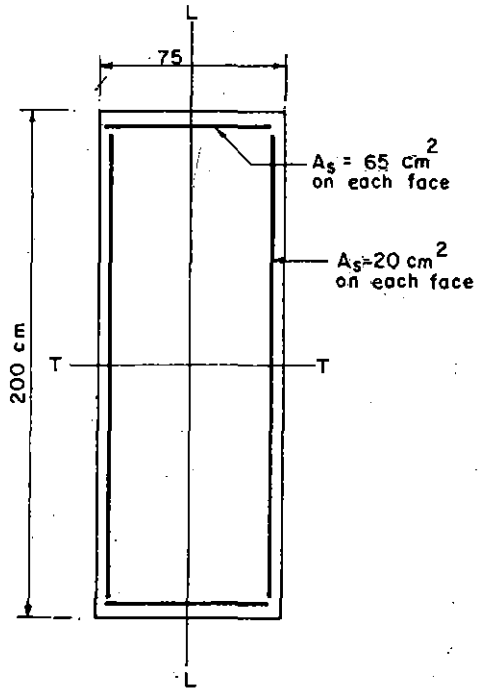


Fig. 25.6

Step 1 Assume 65 cm² of mild steel along each breadth face and 20 cm² of mild steel along each length face as shown.

Step 2 Properties of uncracked section

$$I_{TT} = \frac{75 \times 200^3}{12} + (14-1)\left\{65 \times 95^2 + \frac{20 \times 190^2}{12}\right\} \times 2$$

$= 500,00,000 + 168,20,000$

$= 668,20,000 \text{ cm}^4$

$$I_{LL} = 200 \times \frac{75^3}{12} + (14-1)\left\{20 \times 32.5^2 + 65 \times \frac{65^2}{12}\right\} \times 2$$

$= 70,40,000 + 11,44,000$

$= 81,84,000 \text{ cm}^4$

$I_{TL} = 0$

Step 3 *Location of neutral axis:* For the initial trial, using the properties of uncracked section,

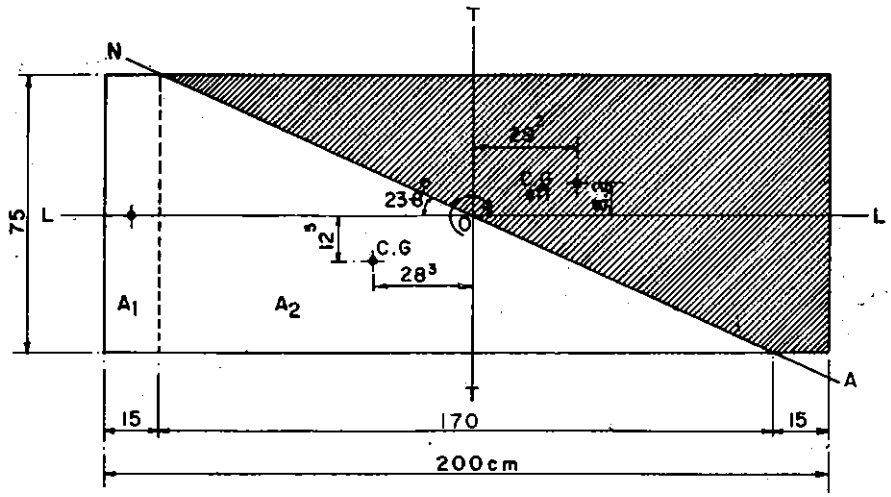


Fig. 25.7

$$\tan\phi = \frac{M_T I_{LL}}{M_L I_{TT}} = \frac{180 \times 81,84,000}{50 \times 668,20,000} = 0.44$$

$$\therefore \phi = 23.8°$$

Assume neutral axis to pass through physical centroid (point O) of uncracked section at an inclination of 23.8° to LL axis (Fig 25.7).

Step 4 Taking transformed area of steel as $(m-1)A_s$, both in tensile and compressive zones,

$$A_{\text{eff}} = (15 \times 75) + \left(75 \times \frac{170}{2}\right) + (14-1)(65+20)2$$

$$= 1125 + 6375 + 2210 = 9710 \text{ cm}^2$$

Step 5 Ordinates of CG_{eff} *wrt* the physical centroid: (take moments of effective area about TT and LL axes respectively).

$$e'_T = \frac{(1125 \times 92.5) + (6375 \times 28.3)}{9710} = 29.2 \text{ cm}$$

$$e'_L = \frac{6375 \times 12.5}{9710} = 8.2 \text{ cm}$$

Step 6 $\therefore e_T - e'_T = \dfrac{180 \times 100}{160} - 29.2 = 83.3$ cm

$$e_L - e'_L = \frac{50 \times 100}{160} - 8.2 = 23.05 \text{ cm}$$

Step 7 See Fig. 25.7.

$$I_{\text{eff}_T} = \frac{500,00,000}{2} + 168,20,000 - 9710 \times 29.2^2$$

$$= 335.4 \times 10^5 \text{ cm}^4$$

$$I_{\text{eff}_L} = \frac{70,40,000}{2} + 11,44,000 - 9710 \times 8.2^2$$

$$= 40.12 \times 10^5 \text{ cm}^4$$

$$I_{LT_{\text{eff}}} = 1125(63.3)(-8.2) + 6375(4.3)(-0.9) + 2210(-29.2)(-8.2)$$

$$= -0.80 \times 10^5 \text{ cm}^4$$

Step 8 Check for $\tan\phi'$

$$\tan\phi' = \frac{(83.3 \times 40.12) - 23.05(-0.8)}{(23.05 \times 335.4) - 83.3(-0.8)}$$

$= 0.43$ compared to 0.44 assumed, OK

Step 9 Check for position of neutral axis:
Stress anywhere along the neutral axis must be zero.

As a check, stress at point O (Fig. 25.7) must be zero as it lies on the assumed neutral axis in this trial. Let us find it out:

$$\text{Stress at point O} = \left(\frac{160,000}{9710}\right) - \left(\frac{160,000\left\{23.05 - 83.3 \times \dfrac{(-0.80)}{335.4}\right\}}{\left(40.12 - \dfrac{0.8^2}{335.4}\right) \times 10^5}\right) 8.2$$

$$- \frac{160,000\left\{83.3 - 23.05 \times \dfrac{(-0.8)}{40.12}\right\}}{\left(335.4 - \dfrac{0.8^2}{40.12}\right) \times 10^5} \times 29.2$$

$= -2.0$ kg/cm^2 compared to zero if neutral axis really passed through point O. This means a fresh trial is necessary. The new neutral axis will be parallel to the assumed one but bodily slightly displaced.

For the purposes of this illustration if the assumed neutral axis is taken as final, then the extreme stresses are as follows:

Step 10 (i) Max. compressive stress in concrete,

$$= \frac{160,000}{9710} + \frac{160,000\left\{23.05 - 83.3 \times \frac{(-0.8)}{335.4}\right\}}{\left(40.12 - \frac{0.8^2}{335.4}\right) \times 10^5}(37.5 - 8.2) + \frac{160,000\left\{83.3 - \frac{23.05(-0.8)}{40.12}\right\}}{\left(335.4 - \frac{0.8^2}{40.12}\right) \times 10^5}(100 - 29.2)$$

$$= 16.5 + 27.2 + 28.3 = 72 \text{ kg/cm}^2$$

(ii) Max. tensile stress in steel,

$$= 14\left\{16.5 - \frac{27.2(37.5 + 8.2 - 5)}{29.3} - \frac{28.3(100 + 29.2 - 5)}{70.8}\right\}$$

$$= 14\{16.5 - 39.6 - 49.8\}$$

$$= -1020 \text{ kg/cm}^2$$

Example 5

Case of a hollow rectangular section subjected to axial thrust and biaxial bending. Consider a hollow rectangular RC section (Fig. 25.8) of outer dimensions 140 cm × 295 cm with 42 cm thick walls. Modular ratio $m = 11$. Axial thrust $P = 345$ tonnes. Bending moments $M_L = 280$ mt and $M_T = 225$ mt. TT and LL axes are orthogonal axes parallel to breadth and length of section, respectively.

Step 1 Assume reinforcement as shown in Fig. 25.8.

Step 2 Properties of uncracked section,

$$I_{LL} = \left(295 \times \frac{140^3}{12}\right) - \left(211 \times \frac{56^3}{12}\right) + (11 - 1)\left\{55 \times 65^2 + 20 \times 33^2 + 45 \times \frac{130^2}{12} + \frac{15 \times 66^2}{12}\right\} \times 2$$

$$= 676,00,000 - 31,00,000 + 64,60,000$$

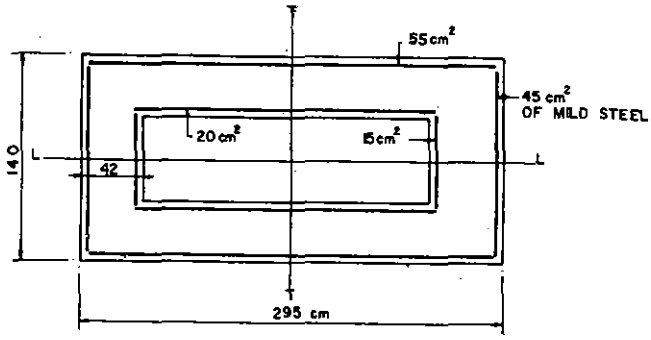


Fig. 25.8

$$= 709.6 \times 10^5 \text{cm}^4$$

$$I_{TT} = \left(140 \times \frac{295^3}{12}\right) - \left(56 \times \frac{211^3}{12}\right) + (11 - 1)\left\{45 \times 142.5^2 + 15 \times 110.5^2 + 55 \times \frac{285^2}{12} + 20 \times \frac{221^2}{12}\right\} \times 2$$

$$= 29,84,00,000 - 437,50,000 + 309,60,000$$

$$= 2856.1 \times 10^5 \text{ cm}^4$$

$$I_{TL} = 0$$

Step 3 Location of neutral axis.

For the initial trail, using the properties of uncracked section,

$$\tan\phi = \frac{M_T \cdot I_{LL}}{M_L \cdot I_{TT}} = \frac{225 \times 709.6}{280 \times 2856.1} = 0.20$$

$$\therefore \phi = 11°$$

Assume neutral axis inclined at 11° to LL axis and intersecting TT axis at O′ to 20 cm above physical centroid, O.

Step 4 Taking transformed area of steel as $(m - 1)A_s$, both in tensile and compressive zones,

$$A_{\text{eff}} = (20.5 \times 295) + \left(59 \times \frac{295}{5}\right) - \left(29.1 \times \frac{145.5}{2}\right) + (11 - 1)(55 + 20 + 45 + 15) \times 2$$

$$= 6050 + 8700 - 2110 + 2700$$

$$= 15340 \text{ cm}^2$$

Step 5 Ordinates of CG_{eff} *wrt* physical centroid:

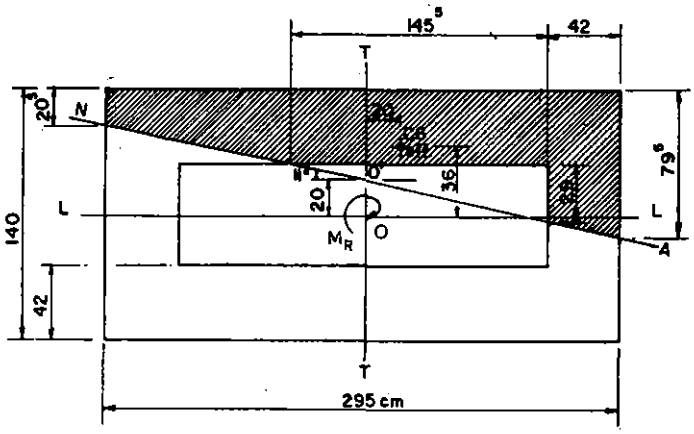


Fig. 25.9

Take moments of effective area about LL and TT axes, respectively

$$e'_L = \frac{(6050 \times 59.75) + (8700 \times 29.83) - (2110 \times 18.3)}{15340}$$

$$= 38.0 \text{ cm.}$$

$$e'_T = \frac{(8700 \times 49.17) - (2110 \times 57)}{15340}$$

$$= 20.0 \text{ cm.}$$

Step 6 $e_L - e'_L = \dfrac{280 \times 100}{345} - 38.0 = 43.2$ cm

$$e_T - e'_T = \frac{225 \times 100}{345} - 20.0 = 45.2 \text{ cm}$$

Step 7 See Fig. 25.9.

$$I_{\text{eff}_L} = \left(295 \times \frac{20.5^3}{12} + 6050 \times 21.75^2\right)$$

$$+ \left(295 \times \frac{59^3}{36} + 8700 \times 8.17^2\right)$$

$$- \left(145.5 \times \frac{29.1^3}{12} + 2110 \times 19.7^2\right)$$

$$+ (64,60,000 + 2700 \times 38^2)$$

$$= 148.22 \times 10^5 \text{ cm}^4$$

$$I_{\text{eff}_T} = \left(20.5 \times \frac{295^3}{12} + 6050 \times 20^2\right)$$

$$+ \left(59 \times \frac{295^3}{36} + 8700 \times 29.17^2\right)$$

$$- \left(29.1 \times \frac{145.5^3}{36} + 2110 \times 37^2\right)$$

$$+ (309,60,000 + 2700 \times 20^2)$$

$$= 1221.5 \times 10^5 \text{ cm}^4$$

$$I_{TL_{\text{eff}}} = 6050\,(21.75)(-20) + 8700(-8.17) \times (29.17) - 2110(-19.7)(37) + 2700(-38.0)(20.0)$$

$$= -9.80 \times 10^5 \text{ cm}^4$$

Step 8 Check for $\tan \phi'$:

$$\tan \phi' = \frac{45.2(148.22) - 43.2(-9.80)}{43.2(1221.5) - 45.2(-9.80)} = 0.135$$

$\therefore \phi' = 8°$ compared to $11°$ assumed, OK good enough.

Step 9 Check for position of neutral axis.
Stress anywhere along neutral axis should be zero.
Check at O′.

$$\text{Stress at O}' = \frac{345,000}{15340} - \left(\frac{345,000\left\{45.2 - 43.2\dfrac{(-9.80)}{148.22}\right\}}{\left(1221.5 - \dfrac{9.8^2}{148.22}\right) \times 10^5}\right) \times 20.0$$

$$- \left(\frac{345,000\left\{43.2 - \dfrac{45.2(-9.80)}{1221.5}\right\}}{\left(148.22 - \dfrac{9.8^2}{1221.5}\right) \times 10^5}\right)(-20.0 + 38.0)$$

$= 22.5 - 2.7 - 18.3 = 1.5$ kg/cm², compared to zero

if the neutral axis really passed through point O′. This means a fresh trial is necessary. The new neutral axis will be almost parallel to the assumed one but bodily slightly displaced.

Step 10 For the purposes of this illustration if the assumed neutral axis is taken as final then the extreme stresses are as follows:
(i) Max. compressive stress in concrete:

$$= \frac{345,000}{15340} + \frac{345,000\left\{145.2 - 43.2\dfrac{(-9.80)}{148.22}\right\}}{\left(1221.5 - \dfrac{9.8^2}{148.22}\right) \times 10^5}(147.5 - 20.0)$$

$$+ \frac{345,000\left\{143.2 - 45.2\dfrac{(-9.80)}{1221.5}\right\}}{\left(148.22 - \dfrac{9.8^2}{1221.5}\right) \times 10^5}(70.0 - 38.0)$$

$= 22.5 + 17.5 + 32.5 = 72.5 \text{ kg/cm}^2$

(ii) Max. tensile stress in steel:

$$= 11\left\{22.5 - 17.5\frac{(147.5 + 20.0 - 5)}{127.5} - 32.5\frac{(70 + 38 - 5)}{32.0}\right\} = 1150 \text{ kg/cm}^2$$

CONCLUSION

The work presented here is of immense practical application which only practising designers can best appreciate. It is, of course, based on elastic analysis.

CHAPTER 26

Post-tensioned Prestressing of Concrete

26.1 GENERAL PRINCIPLES OF PRESTRESSED CONCRETE

One of the best definitions of prestressed concrete is given by the ACI Committee on Prestressed Concrete:

> "Prestressed concrete: Concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are countered to a desired degree. In reinforced-concrete members the prestress is commonly introduced by tensioning the steel reinforcement."

The basic principle of prestressing is not limited to structures in concrete; it has been applied to steel construction as well. When two plates are joined together by hot-driven rivets or high-tensile bolts, the connectors are highly prestressed in tension and the plates in compression, thus enabling the plates to carry tensile loads between them.

Whether prestressing is applied to steel or concrete, its ultimate purpose is twofold—first, to induce desirable strains and stresses in the structure; second, to counterbalance undesirable strains and stresses. In prestressed concrete, the steel is pre-elongated so as to avoid excessive lengthening under service load, while the concrete is precompressesd so as to prevent cracks under tensile stress. Thus an ideal combination of the two materials is achieved.

26.2 STAGES OF LOADING

One of the considerations peculiar to prestressed concrete is the plurality of stages of loading to which a member or structure is often subjected. Some of these stages of loading occur also in non-prestressed structures, but others exist only because of prestressing. For a cast-in-place structure, prestressed concrete has to be designed for at least two stages: the initial stage during prestressing and the final stage under external loadings. For precast members, a third stage, that of handling and transportation, has to be investigated. During each of these three stages, there are again different periods when the member or structure may be under different loading conditions. These will be analyzed below.

Initial Stage

The member or structure is under prestress but not subjected to any superimposed external loads. This can be further subdivided into the following periods, some of which may not be important and hence may be neglected in certain designs.

Before Prestressing: Before the concrete is prestressed, it is quite weak in carrying load; hence the yielding of its supports must be prevented. Provision must be made for the shrinkage of concrete if it might occur. This is often significant because any shrinkage cracks will destroy the capacity of the concrete to carry tensile stresses.

During Prestressing: This is a critical test for the strength of the tendons. Oftentimes, the maximum stress to which the tendons will be subjected throughout their life occurs at that period. It occasionally happens that an individual wire may be broken during prestressing, owing to defects in its manufacture. But this is seldom significant, since there are often many wires in a member. If a bar is broken in a member with only a few bars, it should be properly replaced. For concrete, the prestressing operations impose a severe test on the bearing strength at the anchorages. Since the concrete is not aged at this period while the prestress is at its maximum, crushing of the concrete at the anchorages is possible if its quality is inferior. Again, unsymmetrical and concentrated prestress from the tendons may produce overstresses in the concrete. Hence the order of prestressing the various tendons must often be studied before hand.

At Transfer of Prestress: For pre-tensioned members, the transfer of prestress is accomplished in one operation and within a short period. For post-tensioned members, the transfer is often gradual, the prestress in the tendons being transferred to the concrete one by one. In both cases, there is no external load on the member except its own weight. Thus, the initial prestress, with little loss as yet taking place, imposes a serious condition on the concrete and often controls the design of the member. For economic reasons the design of a prestressed member often takes into account the weight of the member itself in holding down the cambering effect of prestressing. This is done on the assumption of a given condition of support for the member. If that condition

is not realized in practice, failure of the member might result. For example, the weight of a simply supported prestressed girder is expected to exert a maximum positive moment at midspan which counteracts the negative moment due to prestressing. If the girder is cast and prestressed on soft ground without suitable pedestals at the ends, the expected positive moment may be absent and the prestressing may produce excessive tensile stresses on top fibres of the girder, resulting in its failure. Likewise, if the supporting staging is very flexible (not massive and rigid), the cambering effect due to prestress may still not allow the self weight of the structure to act freely since the precompressed staging may, despite its decompression, still keep touching the soffit of structure, thereby allowing only a part of dead load to act. This may cause severe tension in top fibre.

Decentering and Retensioning: If a member is cast and prestressed in place, it generally becomes self-supporting during or after prestressing. Thus the falsework can be removed after prestressing, and no new condition of loading is imposed upon the structure. Some concrete structures are retensioned, i.e., prestressed in two or more stages. Then the stresses at various stages of tensioning must be studied.

Intermediate Stage

This is the stage during transportation and erection. It occurs only for precast members when they are transported to the site and erected in position. It is highly important to ensure that the members are properly supported and handled at all times. For example, a simple beam designed to be supported at the ends will easily break if lifted farther away from its ends.

Not only during the erection of the member itself, but also when adding the superimposed dead loads, attention must be paid to the conditions of support and loading. This is especially true for a cantilever layout, when partial loading may result in more serious bending than a full loading.

Final Stage

This is the stage when the actual working loads come on the structure. As for other types of construction, the designer must consider various combinations of live loads on different portions of the structure with lateral loads such as wind and earthquake forces, and with strain loads such as those produced by settlement of supports and temperature effects. For prestressed concrete structures, especially those of unconventional types, it is often necessary to investigate their cracking and ultimate loads, in addition to the working load.

These will be discussed as follows:

Working Load: To design for the working load is a check on excessive stresses and strains. It is not necessarily a guarantee of sufficient strength to carry overloads. However, an engineer familiar with the strength of prestressed-concrete structures may often design conventional types and proportions solely on the basis of working-load computations.

Cracking Load: Cracking in a prestressed-concrete member signifies a sudden change in the bond and shearing stresses. It is sometimes a measure of the fatigue strength. For certain structures, such as tanks and pipes, the commencement of cracks presents a critical situation. For structures subject to corrosive influences, for unbonded tendons where cracks are more objectionable, or for structures where cracking may result in excessive deflections, an investigation of the cracking load seems important.

Ultimate Load: Structures designed on the basis of working stresses may not always possess a sufficient margin for overloads. This is true, for example, of prestressed-concrete members under direct tensile loads. Since it is desirable that a structure does possess a certain minimum overload capacity, it is often necessary to determine its ultimate strength. In general, the ultimate strength of a structure is defined by the maximum load it can carry before collapsing. However, before this load is reached, permanent yielding of some parts of the structure may already have developed. Although any strength beyond the point of permanent yielding may serve as additional guarantee against total collapse, some engineers consider such strength as not usable and prefer to design on the basis of usable strength rather than the ultimate strength. However ultimate strength is more easily computed and is more commonly accepted as a criterion for design.

In addition to the above normal loading conditions, some structures may be subject to repeated loads of appreciable magnitude which might result in fatigue failures. Some structures may be under heavy loads of long duration, resulting in excessive deformations due to creep, while others may be under such light external loads that the camber produced by prestressing may become too pronounced as time goes on. Still others may be subject to undesirable vibrations under dynamic loads. These are special conditions which the engineer must consider for his individual case.

26.3 PRESTRESSED VESRSUS REINFORCED CONCRETE

As it is assumed that readers are already acquainted with reinforced concrete, it will be interesting to compare prestressed concrete with it. The most outstanding difference between the two is the employment of materials of higher strength for prestressed concrete. In order to utilize the full strength of the high-tensile steel, it is necessary to resort to prestressing to prestretch it. Pretressing the steel and anchoring it against the concrete produces desirable strains and stresses which serves to reduce or eliminate cracks in

concrete. Thus the entire section of the concrete becomes effective in prestressed concrete, whereas only the portion of section above the neutral axis is supposed to act in the case of reinforced concrete.

The use of curved tendons will help to carry some of the shear in a member. In addition, precompression in the concrete tends to reduce the diagonal tension. Thus it is possible to use a smaller section in prestressed concrete to carry the same amount of external shear in a beam.

High-strength concrete, which cannot be economically utilized in reinforced-concrete construction, is found to be desirable and even necessary with prestressed concrete. In reinforced concrete, using concrete of high strength will result in a smaller section calling for more reinforcement and will end with more costly design. In prestressed concrete, high-strength concrete is required to match with high-strength steel in order to yield economical proportions. Stronger concrete is also necessary to resist high stresses at the anchorages and to give strength to the thinner sections so frequently employed for prestressed concrete.

Each material or method of construction has its own field of application. When welding was first developed in the 1930s, some engineers were over enthusiastic and believed that it would replace riveting altogether, which it has not done even yet. Prestressed concrete is likely to have a similar course of development. Not for a long time will it be used in as great quantity as reinforced concrete. But a new type of construction, basically sound in its strength and economy, is likely to have a rapid rate of growth and to be adaptable to new and unprecedented situations and requirements.

The advantages and disadvantages of prestressed concrete as compared with reinforced concrete will now be discussed in respect to their serviceability, safety and economy.

Serviceability: Prestressed-concrete design is more suitable for structures of long spans and those carrying heavy loads, principally because of the higher strengths of materials employed. Prestressed structures are more slender and hence more adaptable to artistic treatment. They yield more clearance where it is needed. They do not crack under working loads, and whatever cracks may be developed under overloads will be closed up as soon as the load is removed, unless the load is excessive. Under dead load, the deflection is reduced, owing to the cambering effect of prestress. This becomes an important consideration for structures such as long cantilevers. Under live load, the deflection is also smaller because of the effectiveness of the entire uncracked concrete section, which has a moment of inertia 2 to 3 times that of the cracked section. Prestressed elements are more adaptable to precasting because of the lighter weight.

So far as serviceability is concerned, the only shortcoming of prestressed concrete is its lack of weight. Although seldom encountered in practice, there are situations where weight and mass are desired instead of strength. For these, plain or reinforced concrete could serve just as well and at lower cost.

Safety: It is difficult to say that one type of structure is safer than another. The safety of a structure depends more upon its design and construction than upon its type. However, certain inherent safety features in prestressed concrete may be mentioned. There is partial testing of both the steel and the concrete during prestressing operations. For many structures, during prestressing, both the steel and the concrete are subject to the highest stresses that will exist in them during their life of service. Hence, if the materials can stand prestressing, they are likely to possess sufficient strength for the design service loads.

When properly designed by the present conventional methods, prestressed-concrete structures have overload capacities similar to and perhaps slightly higher than those of reinforced concrete. For the usual designs, they deflect appreciably before ultimate failure, thus giving ample warning before impending collapse. The ability to resist shock and impact loads and repeated working loads has been shown to be as good in prestressed as in reinforced concrete. The resistance to corrosion is better than that of reinforced concrete for the same amount of cover, owing to denser concrete and the non-existence of cracks. If cracks should occur, corrosion can be more serious in prestressed concrete. Regarding fire resistance, high-tensile steel is more sensitive to high temperatures, but, for the same amount of minimum cover, prestressed tendons can have a greater average cover because of the spread and curvature of the individual tendons.

Prestressed-concrete members do require more care in design, construction, and erection than those of ordinary concrete, because of the higher strength, smaller section, and sometimes delicate design features involved. Although prestressed-concrete construction has been practised only since the late 1940s, it is possible to conclude from experience that the life of such structures can be as long if not longer than that of reinforced concrete.

Economics

From an economic point of view, it is at once evident that smaller quantities of materials, both steel and concrete, are required to carry the same loads, since the materials are of higher strength. There is also a definite saving in stirrups, since shear in prestressed concrete is reduced by the inclination of the tendons, and the diagonal tension is further minimized by the presence of prestress. The reduced weight of the member will help in economizing the sections; the smaller dead load and depth of members will result in saving materials from other portions of the structure. In precast members, a reduction of weight saves handling and

transportation costs.

In spite of the above economies possible with prestressed concrete, its use cannot be advocated for all conditions. First of all, the stronger materials will have a higher unit cost. More auxiliary materials are required for prestressing, such as end anchorages, and grouts. More formwork is also needed, since non-rectangular shapes are often necessary for prestressed concrete. More labour is required to place steel in prestressed concrete, especially when the amount of work involved is small. More attention to design is involved, and more supervision is necessary; the amount of additional work will depend upon the experience of the engineer and the construction crew, but it will not be serious if the same typical design is repeated many times.

From the above discussion, it can be concluded that prestressed concrete design is more likely to be economical when the same unit is repeated many times or when heavy loads and long spans are encountered. It should also find suitable application when combined with precasting or semi-precasting such as composite or lift-slab construction. Each structure must be considered individually. The availability of good designers, of experienced crews, of pre-tensioning factories and construction gear and of competitive bidding often helps to tip the balance in favour of prestressed concrete.

For more details on the subject of relative economics, see Chs. 18 and 42 of this book.

26.4 SYSTEMS

Linear or Circular Prestressing: Circular prestressing is a term applied to prestressed circular structures, such as round tanks, silos, and pipes, where the prestressing tendons are wound around in circles. As distinguished from circular prestressing, the term linear prestressing is often employed to include all other structures such as beams and slabs. The prestressing tendons in linearly prestressed structures are not necessarily straight; they can be either bent or curved, but they do not go round and round in circles as in circular prestressing.

Pre-Tensioning and Post-Tensioning: The term pre-tensioning is used to describe any method of prestressing in which the tendons are tensioned before the concrete is placed. It is evident that the tendons must be temporarily anchored against some abutments or stressing beds when tensioned and the prestress transferred to the concrete after it has set. This procedure is employed in precasting plants or laboratories when permanent beds are provided for such tensioning; it is also applied in the field where abutments can be economically constructed. In contrast to pre-tensioning, post-tensioning is a method of prestressing in which the tendon is tensioned after the concrete has hardened. Thus, the prestressing is almost always performed against the hardened concrete, and the tendons are anchored against it immediately after prestressing. This method can be applied to members either precast or cast in place.

End-Anchored or Non-End-Anchored Tendons: When post-tensioned, the tendons are anchored at their ends by means of mechanical devices to transmit the prestress to the concrete. Such a member is termed end-anchored. Occasionally, though rarely, a post-tensioned member may have its tendons held by grout with no mechanical end anchorage. In pre-tensioning, the tendons generally have their prestress transmitted to the concrete simply by their bond action near the ends. The effectiveness of such stress transmission is limited to wires and strands of small size. Anchorages have been developed for pre-tensioning so as to permit the use of tendons of larger diameter.

Bonded or Unbonded Tendons: Bonded tendons denote those that are bonded throughout their length to the surrounding concrete. Non-end-anchored tendons are necessarily bonded ones; end-anchored tendons may be either bonded or unbonded to the concrete. In general, the bonding of post-tensioned tendons is accomplished by subsequent grouting; if unbonded, protection of the tendons from corrosion must be provided by galvanizing, greasing, or some other means. Sometimes, bonded tendons may be purposely unbonded along certain portions of their length (generally in pre-tensioning).

Precast, Cast-in-Place, Composite Construction: Precasting involves the placing of concrete away from its final position, the members being cast either in a permanent plant or somewhere near the site of the structure, and eventually erected at the final location. Precasting permits better control in mass production and then is often economical. Cast-in-place concrete requires more form and falsework per unit of product but saves the cost of transportation and erection, and it is a necessity for large and heavy members. In between these two methods of construction, there are tilt-up wall panels and lift slabs which are constructed at places near or within the structure and then erected to their final position; no transportation is involved for these. Oftentimes, it is economical to precast part of a member, erect it, and then cast the remaining portion in place. This procedure is called composite construction. The precast elements in a structure of composite construction can be more easily joined together than those in a totally precast structure. By composite construction, it is possible to save much of the form and falsework required for total cast-in-place construction. However, the suitability of each type must be studied with respect to the particular conditions of a given structure.

Partial or Full Prestressing: A further distinction between the types of prestressing is sometimes made depending upon the degree of prestressing to which a concrete member is

subject. When a member is designed so that under the working load there are no tensile stresses in it, then the concrete is said to be fully prestressed. If some tensile stresses will be produced in the member under working load, then it is termed partially prestresed. For partial prestressing, additional rebars are frequently provided to reinforce the portion under tension. In practice, it is often difficult to classify a structure as being partially or fully prestressed since much will depend upon the magnitude of the working load used in design. For example, highway bridges may be designed for full prestressing, though actually they are subject to tensile stresses during the passage of heavier vehicles. On the other hand, roof beams designed for partial prestressing may never be subjected to tensile stresses since the assumed live loads may never act on them.

Protection of Tendons

A fundamental requirement for prestressing tendons is that they must be protected against corrosive attack in order to maintain the integrity of the structure. Where the cables are contained within ducts inside a concrete section this protection is provided by pumping cement grout into the duct after prestressing has taken place. Where the tendons are external to the section other options become available, including providing an added casing of concrete directly around the tendons, or using a polyvinylchloride coating to enclose the cable which has been prepacked with grease.

Tendon Profiling

Because of the changes in stress induced by the application of loading at different sections along a beam, the optimum use of prestressing requires that the position of the tendons should change within the section from one point to another along the length. In post-tensioning this is achieved without difficulty by fixing the cable ducts to the required profile before concreting. Where pre-tensioning is used this obviously cannot be done, since the tendon is stressed prior to concreting. The tendon can be pulled or deflected out of a straight line to approximate to the ideal profile but this harping adds complexity to the stressing procedure and is only reckoned as appropriate for large units. An alternative technique for simulating the effect of a profile is to break the bond between selected tendons and the surrounding concrete near the ends of the unit. This 'debonding' is achieved by placing sleeves over the selected tendons and thus rendering them ineffective in those zones. A plastic hosepipe is commonly used for this purpose.

Parasitic Forces

In statically-indeterminate structures, parasitic forces are set up by the prestressing force in addition to the moments induced by the eccentricity of the prestressing cable (measured from the centroid of the section) along the member. An exception to this occurs where the shape of the cable profile is such that it would not modify the support reactions — a so-called 'concordant' profile — which is rarely achieved in practice. This coincides with the resultant line of thrust.

Parasitic forces develop because the structure is not free to deform as the prestressing forces alone would dictate. The deformation of the structure is constrained by the supports, and constraining forces therefore develop at these supports, modifying the reactions to external loading and inducing a pattern of moments in the structure: i.e. parasitic moments, or secondary moments.

For design purposes it is necessary to assess the effective eccentricity of the cable at points along the structure, this eccentricity not necessarily being equal to the physical eccentricity of the cable measured from the centroid of the section. The effective eccentricity can be assessed by treating the forces exerted on the structure due to the prestressing force as a loading case, thus producing bending-moment and shearing-force diagrams. The effective bending moment at any section, divided by the prestressing force, then gives an effective eccentricity at that point. Details of analysis are given ahead.

Sections at which Stresses should be Checked

Although consideration of the stresses at known critical cross-sections (e.g., at midspan and near the supports) may show satisfactory stresses at all stages, where prestressing cables are curved it is necessary to check the stresses at various sections throughout the span, and this should be done even otherwise.

Shear-Lag Effect on Section Properties

Difficulties can arise in calculating the stresses produced by prestressing forces and applied loading on a voided slab of box deck near the supports. Section properties are well defined within the span. Near the supports, however, the situation is complicated by shear-lag and by the presence of a diaphragm which is solid.

The effect of shear lag is to remove the side cantilevers from the effective section resisting the bending moments. For prestressing calculations the gross area — i.e., including the cantilevering slabs — still nearly applies to the 'average' stress induced by prestressing forces.

At the face of the diaphragm member, the section is again well defined. Any tapering of the voids that is adopted to assist in providing shearing resistance or a more gradual distribution of stress between the fully voided section and the solid section at the centre-line must, of course, be reflected in the section properties assumed at this point.

At the centre-line of the pier the real section is solid, but

the uniform stress due to the prestressing force requires a certain distance to redistribute itself over the solid section in place of the voided section for the rest of deck. It is therefore questionable whether stress calculations that are based on the solid section give valid answers. As the size of void increases it becomes clearer that stresses evaluated at the face of the diaphragm are those which are most relevant for design purposes.

An additional effect of the diaphragm is to modify the longitudinal bending moments because the support reactions are distributed through the width of the diaphragm, rather than acting at a point, as is usually assumed when calculating moments and shearing forces on the deck. This means that any stress calculations that are based on peak values of the bending moments tend to be academic, and do not reflect the real stress situation in the bridge. This shortcoming is generally accepted as a contribution towards conservatism in design, but where calculations for severe temperature stresses must also be taken into account, demanding substantial residual stresses under applied loads it seems appropriate to consider this width of support.

Also, see Ch. 19 in this book for Shear-Lag.

Loss of Prestress

Although the force transmitted to a structure at the anchorage points may be well defined at the time stressing of the tendons takes place, these stresses will be modified by the subsequent behaviour of the structure. The prestressing tendons and the concrete are sustaining each other in an equilibrium state of stress, arising from the strains locked in by the stressing operations. Steel subjected to locked-in strain experiences some loss of stress with time, this phenomenon being known as relaxation. Concrete is subject to shrinkage and to minor changes in the stress/strain relationship with time, these effects being known as creep. In the process of stressing, the loads imposed by successive tendons as they are stressed cause the structure to deform elastically, thereby modifying the locked-in strain in those tendons that have previously been tensioned. All of these effects reduce the locked-in forces, thus resulting in a loss of prestress. An additional loss of prestress in post-tensioned tendons arises from the friction between the cable and the duct, which leads to a progressive reduction in the actual force in the cables as the distance from the jacking point increases.

The total loss of prestress is usually substantially greater with pre-tensioning than with post-tensioning because the whole prestressing force is applied to the member simultaneously in pre-tensioning, and the demand for rapid fabrication usually leads to the stress being transferred into the member while the concrete is still relatively 'green' and thus more susceptible to losses due to creep. Also, pre-tensioning is affected by the entire shrinkage of the concrete, whereas in the case of post-tensioning some of this shrinkage takes place prior to the commencement of prestressing, and this part does not, therefore, affect the prestressing force.

Fuller details are given ahead.

Anchor Blocks (End Blocks)

At the point where a prestressing cable is anchored the full prestressing force in that cable is transmitted to the structure. This represents a substantial force concentrated on a very small area, and early prestressed concrete design and construction produced examples of failure in the region of anchorages, due to the splitting of the concrete. Similar problems can still overtake the unwary designer. The effect of these large anchorage forces is not unlike that of driving a wedge into the structure, and the shape of many prestressing anchorages encourages the analogy.

Several theories were developed as a basis for the design of anchorage zones (Freysinnet, Guyon, Magnel, Borsch, Bleich, Siever, Morsch, etc.), producing significant differences in the results. The Cement and Concrete Associsation (London) undertook a substantial series of tests, presenting its findings in two research reports, the first dealing with single anchorages and the second with group effects.

The prime requirement resulting from the calculations derived from these research reports is the need for heavy reinforcement perpendicular to the axis of prestressing cable and immediately behind the anchorage, to withstand the splitting forces arising from the wedge-like action of the anchorage.

An additional requirement in the region of prestressing anchorages is the need to examine the way in which the forces disperse into the full section of the deck. For example, in a box-section deck, groups of anchorages are usually concentrated in concrete 'anchor blocks' formed by an enlargement of the web. The dispersal of these large concentrated forces into the full cross-section gives rise to shearing and tensile stresses around the blocks. Each case requires individual assessment, assumptions being made as to the flow of the forces, and reinforcement being provided to cater for the shearing and tensile stresses that result. Within the anchor blocks themselves the design approach can be based on deep-beam theory.

The substantial amounts of reinforcement often necessary in the region of anchorages call for particular care in detailing. It is all too easy to adopt an arrangement of reinforcement which fits well on a drawing but proves almost impossible to assemble on site. If there is a conflict between the need to provide the theoretical amount of reinforcement required and the resulting production of an arrangement so congested as to prevent good concreting in

that region, it is obviously even more important that well-compacted concrete should support an anchorage than that the calculated area of reinforcement should be present.

The fixing of closely-spaced reinforcement can usually be facilitated by detailing the steel in such a way that the reinforcement required in the bottom of the deck to deal with slab or diaphragm action can be fixed first. Anchorage-block reinforcement is then assembled on top of this mat, followed by the top surface reinforcement for the diphragm or slab, which is assembled on top of the bursting steel. The system of stirrups adopted must permit this sequence.

For bursting steel itself many designers favour the use of a spiral immediately around the anchorage. This follows from the development of an early form of anchorage for small prestressing systems, which was constructed with fine aggregate concrete inside a contiguously wound helix. The attraction of the spiral is its unquestionable effectiveness in all directions within the planes of the splitting force, and the fact that it unites a significant lump of concrete with the anchorage. Only in smaller stressingg systems is this spiral adequate on its own; however, a system of links, U-bars and straight bars will often be needed to supplement the spiral where conventional links are used, care should be taken to detail reinforcement to a shape which avoids a double thickness of steel at the overlap closing the link. A close spacing of reinforcement is needed in an anchorage zone.

Details of designing reinforcement in anchorage zones are given ahead.

Shear

The capacity of a prestressed concrete member to resist shear depends on the amount of bending moment acting in conjunction with shearing force at the section. This is best appreciated by considering the case of a simply-supported beam subject to concentrated loads at third points.

Adjacent to the supports, the limiting shear capacity of a beam depends on its ability to sustain the diagonal compressive force arising from the shearing force, without causing buckling of the section.

Moving away from the supports into the area where the pattern of shearing stresses is fully developed (clear of the influence of the support reaction), but where bending stresses are modest, any distress arising from excessive shear will be shown by the formation of diagonal cracks near the neutral axis of the member —where the shearing stresses are a maximum—and extending towards the boundaries of the section as the load increases; until failure occurs either by the shear stirrups yielding or the web buckling in compression.

In the vicinity of the two applied loads is a region of shearing stress associated with high bending stresses. In this area failure of the section will be heralded by the formation of cracks beginning at the extreme tensile fibre and extending upward into the member as the load increases. The propagation of these cracks impinges on the cross-sectional area available to provide effective shearing resistance, and thus reducing the shear capacity. Once a critical point is reached, the shearing and bending effects combine to extend the crack to the point where failure occurs. In the case of a continuous beam it is normal to find areas of high shearing stresses and high bending stresses together, so that this mode of failure becomes critical in such cases.

Between the two applied loads mentioned above, nominal shearing stress will be combined with high bending stresses. Failure in this zone would occur essentially as a result of pure bending.

The approach necessary to calculate the shear capacity differs according to the zone concerned. Inclined prestressing cables have a marked effect on the principal stresses in a web subjected to shear and are therefore significant in assessing the shear capacity where bending does not produce flexural cracks. The moment at which flexural cracks commence to form is unaffected by the inclination of the stressing cables so that, where the shear capacity is limited by flexural cracking, inclined tendons do not help.

For a detailed discussion on design against 'shear' (and 'Shear and Torsion') refer to Ch. 24 in this book.

Ultimate Moment of Resistance

The basic aim in prestressed concrete design is to maintain the section in compression under all conditions of loading. However, where overload does occur, tensile stresses can develop to such an extent that the section begins to crack, and the whole basis of evaluating the capacity of the section to resist load changes.

There are fundamental differences between the behaviour of bonded and unbonded tendons in ultimate load conditions. Since a bonded tendon is united with the concrete section at all points along its length, the concrete and the prestressing tendon will continue to act together, responding to the same strain-distribution curve at each section, until failure takes place. With unbonded tendons, whereas the concrete section will be responding to the varying systems of force and distributions of strain at individual sections, the tendon is stressed by a strain that is locked into it by being held at points far removed from each other along the length of the member. It is, therefore, insensitive to particular distributions of strain of individual sections so that, although the concrete at a particular section may be strained to a point, approaching its ultimate capacity, the prestressing tendon crossing the section may only be stressed to a level

approximating to its normal working load.

The general approach to assessing the ultimate resistance of a prestressed concrete member is to seek, by trial and adjustment, a balance between the internal and external forces based on an assumed distribution of strain.

For details about calculating the absolute ultimate moment of resistance of a section refer to Ch. 27 in this book, where details are also given about how to estimate the actually attainable moments of resistance in keeping with compatibility of deformations and consequent moment-redistribution.

Description of the Construction Phases

Where a construction is not entirely cast *in situ* on a single support system, the consulting engineer must specify to the contractor, in a detailed manner, the different construction phases in order that there is no ambiguity on the job site. This specification shall necessarily include those phases which may influence the stability of the structure, either during construction or in service. It is not always essential to define those which are part of the concreting sequences, form stripping and tensioning ... which, considered in relation to any one phase, have little or no effect on the calculations.

Particular attention must be paid to the clamping and unclamping of temporary bearings and blocking pieces.

Order of Tensioning

The order in which cables are tensioned forms an integral part of the design. It will be drawn up after verification of the provisional phases of construction. Particular attention shall be given to the necessary concrete strength with respect to the more highly stressed sections of the structure, and the anchorage areas.

When the structure is simple, the tensioning sequence will figure on the cable layout. In the case of large structures comprising several cable layouts, or for structures to be built in several phases, the tensioning sequence will be included in the specification for the construction stages.

NOTE 1. For some details about prestressing steel (wires, strands and bars) and prestressing anchorages, see Ch. 6 in this book.

2. While some design details are given in their relevant contexts in other chapters in this book, details as per ACI practice are available in APPENDIX 6 to this book.

Losses in Prestress

The prestress imparted to 'post-tensioned' prestressed concrete structure suffers a reduction owing to various causes. These causes are:

A. *Frictional resistance* Felt by the extending tendon at the time the tendon is tensioned, *and the 'slip'* of the tensioned tendon at the anchorages at the instant it is locked in the anchorages to release the stressi jacks.

B. *Shrinkage of concrete* over an extended peri of time.

C. *Creep of concrete* over an extended period time.

D. *Elastic shortening of concrete* each time tendon is stressed.

E. *Relaxation of the tendon steel* over a period c time.

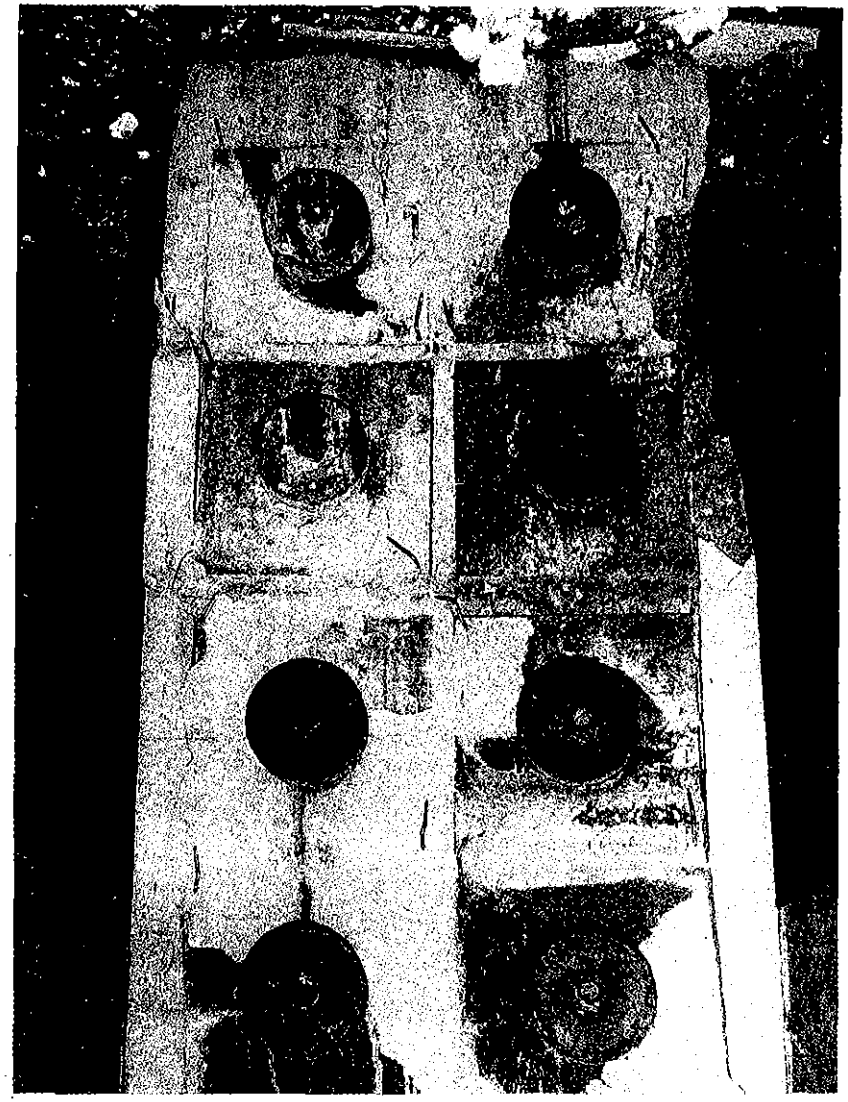
In pre-tensioned prestressed concrete: Loss 'A' i obviously absent, loss 'B' is relatively more (sinc even the shrinkage cracks that have occurred prio to transfer, absorb some of the already applie prestress unlike in post-tensioning where such cracks get closed during tensioning process and only then the pressure gauge registers the dormant prestress), loss 'D' is almost double of that in the case of post-tensioning (since shortening caused by the transfer of each tendon is felt by all tendons as all tendons are transferred simultaneously, unlike in post-tensioning where shortening caused by prestress in any particular tendon is felt only by those of the tendons already stressed, so that the last stressed tendon suffers nothing, the first stressed tendon suffers due to all the subsequent tendons and variation in-between is almost linear), and loss 'E' is almost identical to that in post-tensioning.

A. Loss due to Friction and Slip

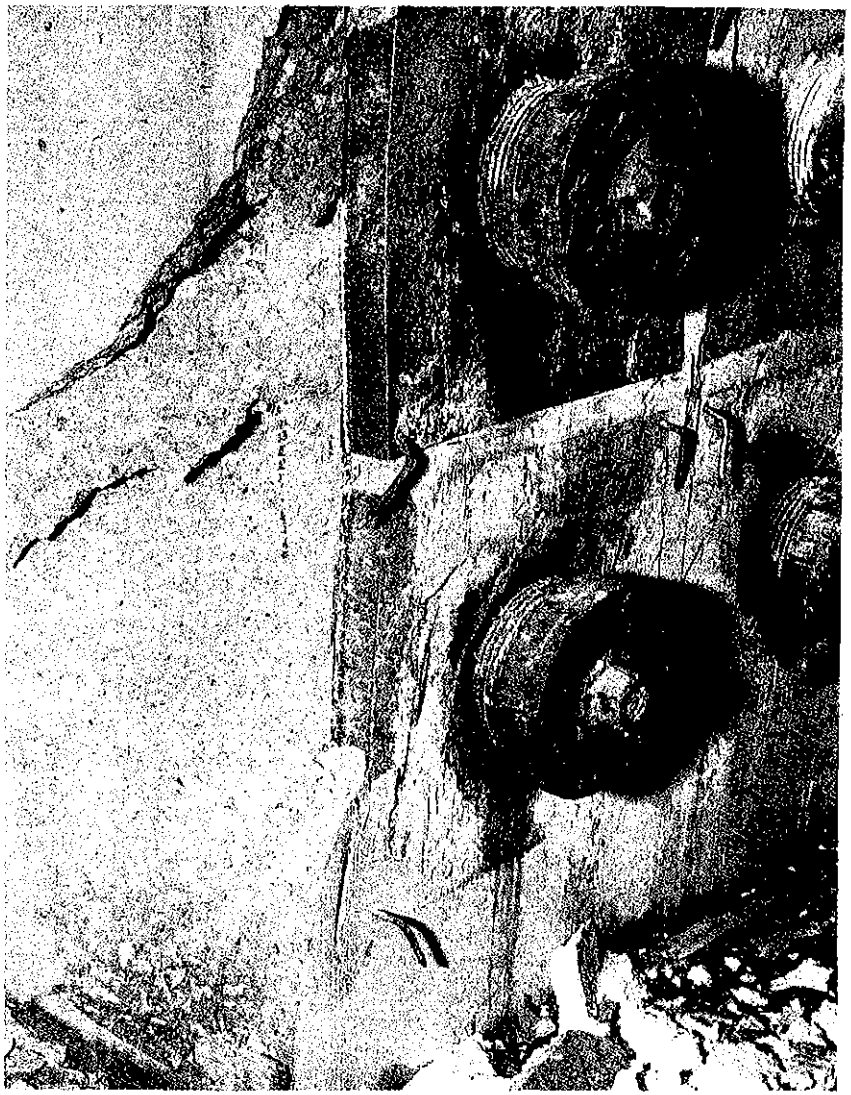
By doing the necessary friction calculation and the slip-effect calculation in advance of the design of the structure, the actual initial prestress forces in each tendon, at each section after friction and slip effects, are calculated and only these values used in design. This takes care of friction and slip loss.

Details of these calculations are given ahead. For the purpose of only a general 'feel', for instance in a symmetrically parabolically profiled tendon (with a central straight stretch) in a simple span of about 35 m, stressed from both ends equally and simultaneously, the friction will reduce the force in the tendon from $0.75F_u$ at jacking ends to about $0.64F_u$ at midspan; also that the slip effect could travel up to 10 m, or more from jacking-ends towards mid-span (depending on tendon profile), reducing the force at the jacking-end from $0.75F_u$ before slip to about $0.65F_u$ (or lower) after slip (F_u = ultimate tensile force possible in the tendon). Also, the extension in a tendon is usually of the order of about 4.5-5.0 mm per meter of extending length between null point to jacking point (assuming average initial stress in cable to be about 0.65 of ultimate and $E = 2 \times 10^6 \text{kg/cm}^2$). This again is only a rough figure for a

Prestressing operation



Distress in anchorage zone noticed while prestressing (caused by crushing of improperly vibrated concrete)



Another view showing distress in anchorage zone concrete

general idea, exact figure for each stressing-end depending on actual prestress force distribution along tendon length (taking due account of actual friction losses), actual location of the null point (the point in the cable where forces from the two jacking-ends are equal and hence does not 'move') and the correct E value of the tendon-steel.

It is preferable to stress each tendon from both its ends (simultaneously and equally) unless for instance the method of construction prevents it, e.g. lack of jacking space, or use of coupler, etc. For one thing, this permits fuller use of cable steel. One-end stressing in a structure symmetrical about its mid-span, will require checking stresses even at the mirror-image sections owing to unequal prestress forces at such sections, which only adds to the design labour. However, the biggest advantage of both-end stressing is the much better chance of salvaging the structure should an inadvertent mortar or grout leak block the tendon for some portion within its length. In such a case with live anchorages at both ends, most of the tendon length can still be tensed and, despite the rock-anchor type behaviour of cable within the blockage, changes remain for the anchor force distributions from the opposing sides within the blockage, to intersect. The overall stresses at this critical section within the blockage in such a case, can then be rechecked backwards using the actual (even though) reduced value of the cable force at the section. Again, two-end stressing is advantegeous in short cables where the magnitude of slip—which is independent of tendon length—can be a significant proportion of the total elongation. Here, anchoring at only one end is done first and then slip loss is taken up by compensatory (further) stressing at the other end and only then is that end anchored. (See ahead for discussion on 'cable blockage'.)

B. Loss due to Shrinkage of Concrete

This loss in a tendon, stressed at a particular age of concrete, is the product of the Residual Shrinkage Strain in concrete from that day onwards and E the modulus of elasticity of cable steel, and this product, a stress, can then be expressed as a percentage of average initial stress in the tendon. As an example, if the tendon is stressed when concrete is 7 days old, it is possible to estimate the loss in this tendon due to shrinkage of concrete during any required time interval therebeyond (e.g., between 7 to 14 days, 14 to 28 days, etc.) knowing residual shrinkage strains of concrete on those days.

Values for these are given for example in the IRC Specifications, but their exactitude is only academic. Total residual shrinkage strain value beyond, say 7 days (when usually most of the prestress is applied in many practical cases), is more important. Its product with E of cable steel, expressed as a percentage of the average initial stress in the tendon (usually 0.64 of ultimate) is of the order of 6 whether the construction is wholly cast *in situ* or partly precast. Half of this may be assumed to occur within the first month and balance thereafter (with a fifth in the second month, another like amount in the third month and the balance thereafter). For cables stressed at concrete age 28 days or more, the above figure of 6 may be taken as 3, remembering that hair-splitting exactitude in estimating these figures is immaterial so long as total of losses 'B', 'C', 'D', and 'E', referred to earlier, is taken as stated in the Table 26.1 given ahead. (Loss due to shrinkage of concrete reduces if humidity increases.)

C. Loss due to Creep of Concrete

This loss in a tendon, stressed at a particular age of concrete, is the product of the Residual Creep Strain in concrete from that day onwards and E of the cable steel. However, since creep is caused if either the loading is long term or the rate of loading is slow, its residual strain is always expressed per certain weighted average compressive stress (f_{av}) in concrete at the level of centroid of the appropriate cables under the appropriate loading condition, usually 100 kg/cm^2. Therefore the above product has to be multiplied accordingly by 100 and divided by the actual f_{av} (kg/cm^2) as prevalent. Noting that f_{av}, being an intermediate fibre level stress, can therefore generally not be more than about 80% of maximum permissible (extreme fibre) compressive stress, it may be taken as ranging between 95 to 115 kg/cm^2, cast in situ to precast construction. Consequently it will only be a slight over estimation of the loss if, on paper, f_{av} is directly taken as 100 kg/cm^2. This will only ease the computations without any unnecessary (and at any rate a pointless) hair-splitting. (Anyone who has time to waste and has lost track of the aim of this exercise within its practical meaning, may indulge in mensurational paper work here, but he will not be doing anything worthwhile by hair splitting.)

Here also, it is possible to estimate the loss in a tendon (due to creep of concrete) during any required time interval after stressing the tendon if only the residual creep strains of concrete on those days are known. Values for these are given for example in the IRC Specifications (in terms of concrete strengths, which at any rate can be approximated to the age of concrete and the strain values related to concrete age). Again, the total residual creep strain value beyond say 7 days (when usually most of the prestress is applied in many practical cases) is more important. Its product with E of cable steel (after adjustment of $100/f_{av}$ explained earlier) expressed as a percentage of the average initial stress in the tendon (usually 0.64 of ultimate) is of the order of 10, whether the construction is wholly cast in situ or partly precast.

A fifth of this may be assumed to occur within the first

month and the balance thereafter (with a fifth in the second month, another like amount in the third month and balance two-fifths thereafter). For cables stressed at concrete age 28 days or more, the above figure of 10 may be taken as 8. (Loss in prestress due to creep of concrete initially decreases if humidity increases and subsequently increases if the humidity continues to increase.)

D. Loss due to Elastic Shortening of Concrete

As explained in the beginning, in post-tensioned concrete this loss may be assumed to vary linearly from the maximum in the cable stressed first to zero in the cable stressed last. Hence, may be taken as half of the maximum, latter being the product of 'strain in concrete due to a weighted average compressive stress in concrete at the level of centroid of all tendons under all dead loads and all initial prestress' and E of the cable steel.

i.e., $$\frac{1}{2}\left(\frac{f'_{av}}{E_{concrete}}\right)E_{steel}$$

i.e. $$\frac{1}{2}mf'_{av}$$

Value of f'_{av}, as against that of f_{av} mentioned earlier in reference to loss due to creep of concrete, is obviously higher because of the stated loading condition. This value may safely be taken as 120 kg/cm^2 whether the construction is cast in situ or precast.

The above loss is assumed to occur at the very time an individual cable is stressed.

The above stress loss of $(1/2\ mf'_{av})$, when expressed as a percentage of the average initial stress in the tendon (usually 0.64 of ultimate), is of the order of 3.5, assumed to occur at time of stressing the tendons.

E. Loss due to Relaxation of Tendon Steel

Cable steel under constant tension tends to relax in as much as its stretched fibres creep away from each other. This reduces the prestressing force in the tendon and the magnitude of reduction depends on how much stress-relieving has gone into the steel in its manufacturing process. It ranges between 4 to 7 per cent of the (average) initial prestressing force (or stress) in the tendon and most of it is assumed to occur in the first 1000 hours after tensioning the tendon, with majority of it occurring in its earlier stages.

Suggested values of loss of prestressing force in stage-I tendons (PSI) stressed at concrete age 7 days and in stage-II tendons (PSII) stressed at concrete age 28 days, expressed as percentage of the (average) initial prestressing force in the cables: (Post-tensioned concrete) are given in Table 26.1.

Reference may also be made for another simple approach given for post-tensioned as well as pre-tensioned concrete in the AASHTO Design Specifications, which, in principle, are reproduced below with courtesy and acknowledgement: "Loss of prestress due to all causes, excluding friction, may be determined by the following method. The method is based on normal weight concrete and one of the following types of prestressing steel: 250- or 270-ksi, seven-wire, stress-relieved strand; 240-ksi stress-relieved wires; or 145- to 160-ksi smooth or deformed bars. For data regarding the properties and effects of lightweight aggregate concrete and low-relaxation tendons, refer to documented tests or see authorized suppliers.

TOTAL LOSS

$$\Delta f_s = SH + ES + CR_c + CR_s$$

where Δf_s = total loss excluding friction in pounds per square inch

SH = loss due to concrete shrinkage in pounds per square inch

ES = loss due to elastic shortening in pounds per square inch

CR_c = loss due to creep of concrete in pounds per square inch

Table 26.1

Prestress	Loss between (% of av. P_i)	Due to shrinkage of concrete (% of av. P_i)	Due to creep of concrete (% of av. P_i)	Due to elastic shortening of concrete (% of av. P_i)	Due to relaxation of steel (% of av. P_i)	Total loss (% of av. P_i) P_i = initial prestressing force after friction and slip
Stage I (applied at 7 days)	7—28 days	3	2	3.5	6	14.5
	after 28 days	3	8	—	—	11.0
	Total	6	10	3.5	6	25.5
Stage II (applied at 28 days)	after 28 days	3	8	3.5	6	20.5

NOTE: Loss of prestressing force on account of 'curvature-friction and wobble-friction' and 'slip at anchorages at the time of releasing the tensioning-jacks' is accounted for through the friction-effect calculations, and only the 'after friction and slip' prestressing force values used as P_i values.

CR_s = loss due to relaxation of prestressing steel in pounds per square inch

Shrinkage of Concrete: Pre-tensioned members

$$SH = 17{,}000 - 150\ RH$$

Post-tensioned members ~

$$SH = 0.80(17{,}000 - 150\ RH)$$

where RH = mean annual ambient relative humidity in per cent

Elastic shortening:

Pre-tensioned members

$$ES = \frac{E_s}{E_{ci}} f_{cir}$$

Post-tensioned members

$$ES = 0.5\frac{E_s}{E_{ci}} f_{cir}$$

where E_s = modulus of elasticity of prestressing steel strand, which can be assumed to be 28 × 10^6 psi.

E_{ci} = modulus of elasticity of concrete in psi at transfer of stress, which can be calculated from:

$$E_{ci} = 33w^{3/2}\sqrt{f'_{ci}}$$

in which w is the concrete unit weight in pounds per cubic foot and f'_{ci} is in pounds per square inch (cylinder crushing strength).

f_{cir} = concrete stress at the centre of gravity of the prestressing steel due to prestressing force and dead load of beam immediately after transfer; f_{cir} shall be computed at the section or sections of maximum moment. (At this stage, the initial stress in the tendon has been reduced by elastic shortening of the concrete and tendon relaxation during placing and curing the concrete for pre-tensioned members, or by elastic shortening of the concrete and tendon friction for post-tensioned members. The reductions to initial tendon stress due to these factors can be estimated, or the reduced tendon stress can be taken as 0.63 f'_s for typical pretensioned members.)

Creep of Concrete: Pre-tensioned and post-tensioned members.

$$CR_c = 12f_{cir} - 7f_{cds}$$

where f_{cds} = concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied.

Relaxation of Prestressing Steel:

Pretensioned members:

250- to 270-ksi Strand

$$CR_s = 20{,}000 - 0.4ES - 0.2(SH + CR_c)$$

Post-tensioned members:

250- to 270-ksi Strand

$CR_s = 20{,}000 - 0.3FR - 0.4ES - 0.2(SH + CR_c)$

240- ksi Wire

$CR_s = 18{,}000 - 0.3FR - 0.4ES - 0.2(SH + CR_c)$

145- to 160-ksi Bars

$CR_s = 3{,}000$

where FR = friction loss stress reduction in psi below the level of 0.70 f'_s at the point under consideration.

ES, SH and CR_c = appropriate values as determined for either per-tensioned or post-tensioned members.

In lieu of the preceding method, the following estimates of total losses may be used for prestressed members or structures of usual design. These loss values are based on use of normal weight concrete, normal prestress levels, and average exposure conditions. For exceptionally long spans, or for unusual designs, the method given above must be used.

Estimate of Prestress Losses

	Total loss	
	f'_c = 4,000 psi	f'_c = 5,000 psi
Pretensioning (Strand)	—	45,000 psi
Post-Tensioning*		
Wire or Strand	32,000 psi	33,000 psi
Bars	22,000 psi	23,000 psi

* *friction losses not included.*

f'_c = 28-day standard cylinder crushing strength of concrete

f'_s = ultimate tensile stress of prestressing steel.

26.5 CABLE FRICTION CALCULATIONS

1. If the force in the cable at its stressing-end (i.e., at its jacking-end) is P_0 (generally 0.75 of ultimate, but never more than 0.80 of ultimate), then allowing for loss in this force on account of frictions due to curvature of cable profile

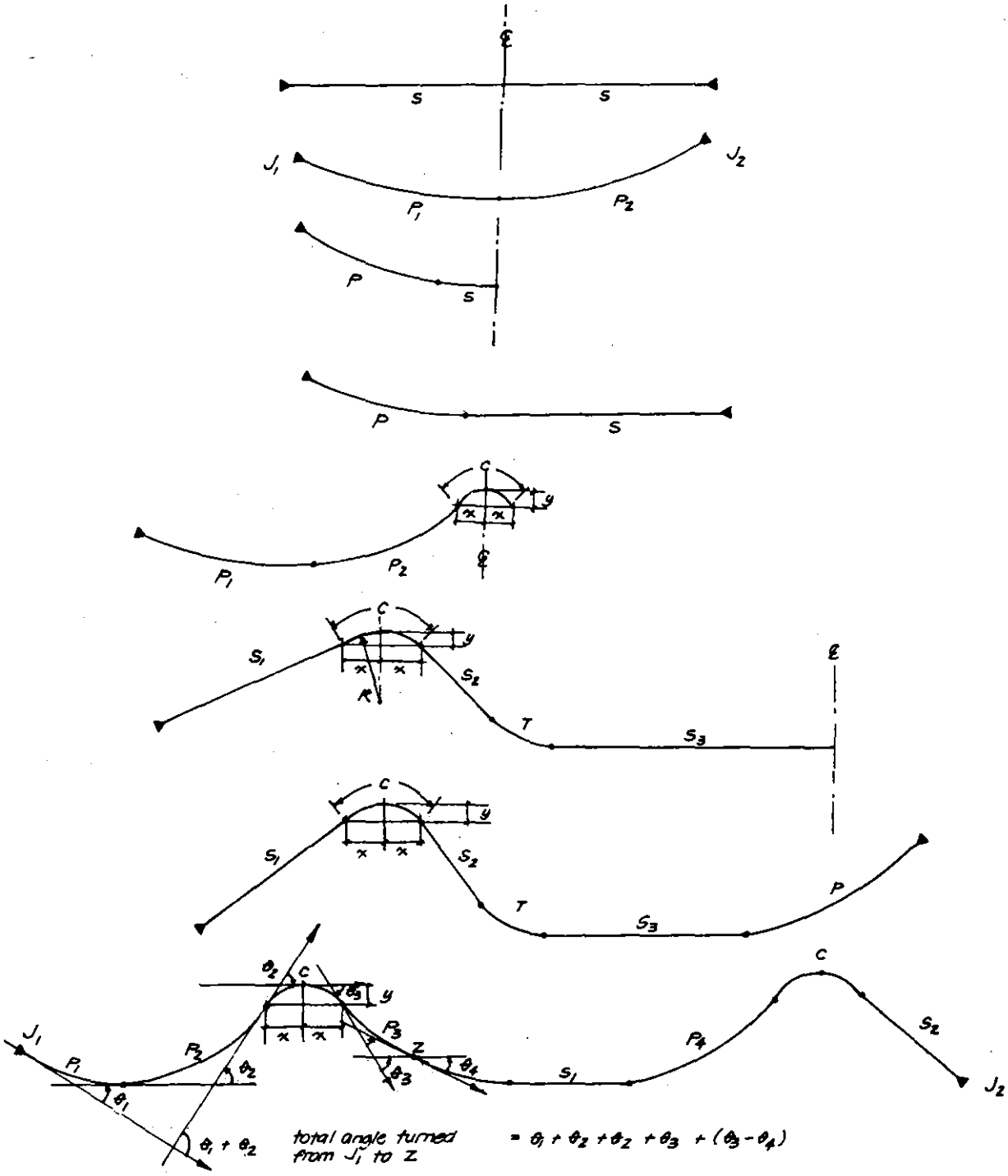


Fig. 26.1(a) ▶ = jacking point
S = straight
P = parabolic
C = circular
T = curved 'transition'

x and y to be so fixed as to give R the radius of the circular curve as not less than the permitted minimum bending radius for the cable and duct adopted.

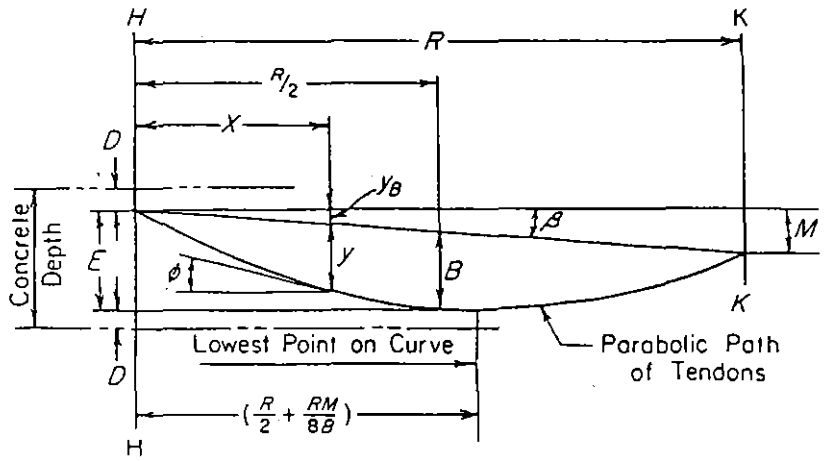


Fig. 26.1(b) *Geometry of parabolic path of tendons where ends of parabola are at different elevations*

$$Y = 4B\frac{X}{R}\left(1 - \frac{X}{R}\right)$$

$$Y_B = M\frac{X}{R}$$

$$\tan\phi = \frac{4B}{R}\left(1 - \frac{2\lambda}{R}\right)$$

L = Length of tendon along parabolic curve

$$= R\left(\sec.\ \beta + \frac{2.67n^2}{\sec.^3\phi}\right)$$

$$n = \frac{B}{R}$$

w_T = uniform upward load applied to concrete by tendons (Force F)

$$= \frac{8BF}{R^2}$$

V_H and V_K = concentrated downward loads applied at supports by tendons = $F \tan\phi$

At $H, X = O$ and $\tan\phi = \dfrac{4B + M}{R}$

At $K, \bar{X} = R$ and $\tan\phi = \dfrac{M - 4B}{R}$

and wobble effect up to a point distant x from the jacking point, **the force in the cable at that point** will be:

$$P_x = P_0 \cdot e^{-(\mu\theta + kx)}$$

where θ = total angle turned, in elevation and plan, between the two points

x = distance between the two points, measured along the cable profile (but since rise of cable is small compared to its projected length in plan, x may easily be taken as distance in plan between the two points)

μ = curvature friction coefficient

k = wobble friction coefficient

Values of μ and k depend on type of duct housing the cable (e.g. plane, corrugated, galvanized, flexible, rigid, etc.) and, to some extent, even on the amount of care exercised in handling the duct and how symmetrically and untustled the tendon elements stay inside the duct. As an example, for a rigid, corrugated and unrusted duct, housing high tensile steel wires assumed placed along a circle in cross-section, with the duct remaining unsquashed all along its length, μ may be taken as 0.25 per radian and k as 0.003 per meter length. Duct manufacturers give μ and k values for different types of ducts while the codes usually specify the upper bound values.

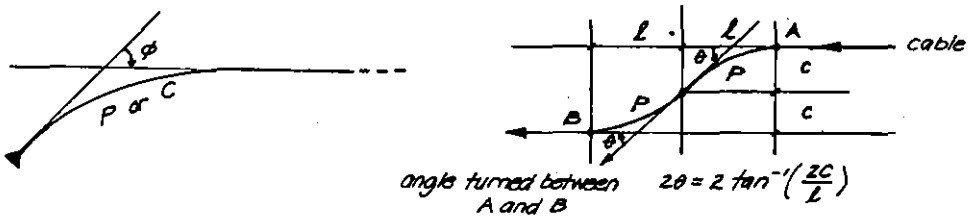


Fig. 26.2

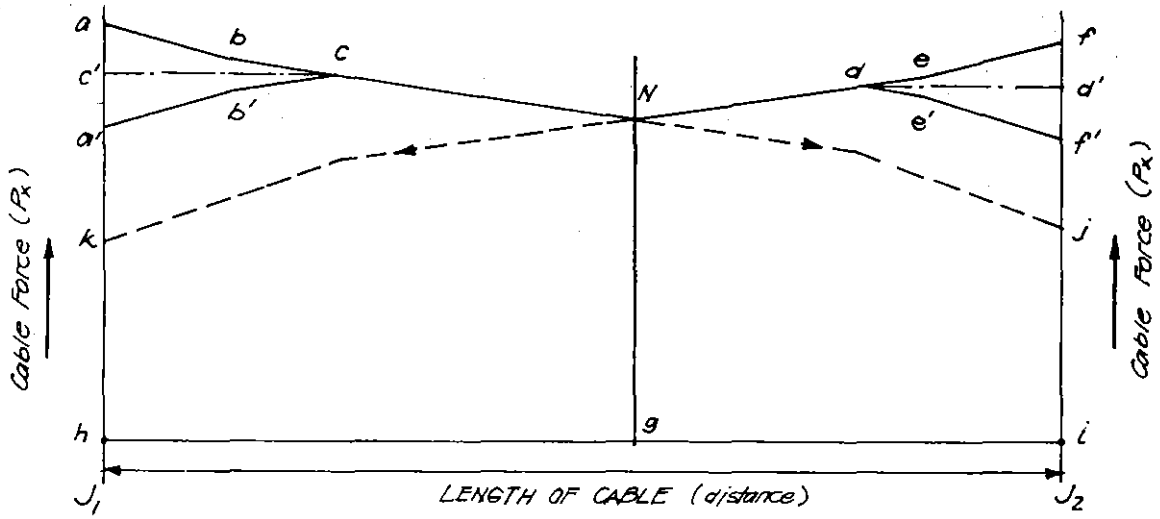


Fig. 26.3

2. **Angles turned through in elevation and plan** between the jacking point and any point in the cable length can be found out from geometry of the cable profile in elevation and plan. Some of the usual cable profiles in elevation, adopted for various cables in various situations are indicated ahead~ (not drawn to same horizontal and vertical scales). Fig. 26.1.

In plan the cable may turn in profile, depending on the situation, and some of these profiles are indicated below (drawn to different scales in length and breadth directions) ~ Fig. 26.2.

3. **Evaluating 'change in cable length'** [either the 'cable elongation' owing to stressing or the 'cable slip' owing to small reverse movement at the instant the stressing jack is released and the cable is locked in its anchorage]:

If δ_x be the movement (i.e., the change of length) in a small cable length dx, then

$$\frac{\delta x}{dx} = \frac{\text{change in length}}{\text{original length}} = \text{strain} = \frac{F_x}{AE}$$

where F_x is the cable force at that section, and A and E the section area and modulus of elasticity of cable. Then,

$$\delta_x = \frac{F_x dx}{AE}$$

Summing up on both sides,

$$\Sigma\delta_x = \Sigma\frac{F_x dx}{AE} = \frac{1}{AE}\Sigma(F_x dx)$$

LHS represents the total movement, and $\Sigma(F_x dx)$ represents the area of (Force $\times$ length) diagram, causing the movement.

Hence, in Fig. 26.3 (for explanation see ahead):

(i) Cable elongation at jacking end J_1

$$= \frac{1}{AE} \text{ (area } abc\ Ngha)$$

(ii) Cable elongation at jacking end J_2

$$= \frac{1}{AE} \text{ (area } fedNgif)$$

(iii) Cable slip at end $J_1 = \frac{1}{AE}$ (area $abcb'a'a$)

(iv) Cable slip at end $J_2 = \frac{1}{AE}$ (area $fede'f'f$)

4. **Estimation of**

- Initial force P_i in the cable at any section (after friction and slip)
- Null point N (-location of)
- Cable elongation at each stressing-end
- Jack pressure at each stressing-end
- Slip at each stressing-end

For each chosen trial cable profile, assumed stressed from both ends J_1 and J_2 simultaneously and equally, proceed step-wise as follows:

Step 1 Work out inclination of tangent to cable profile with horizontal at various points in the cable profile from its geometry. Also work out the angle turned through in plan by the cable between various points.

Step 2 Assume J_1 as the stressing-end and J_2 as the dummy-end, work out total angle turned through as well as distance from J_1 up to *each* point in the cable (moving towards J_2), thus work out value of $(\mu\theta + kx)$ up to each point with respect to J_1 and hence the P_x value $\left(= P_0 e^{-(\mu\theta + kx)}\right)$ at each of these points. This can best be done in a tabular format. Plot these P_x values at the respective

sections along the cable length, *line abc Nj* in Fig. 26.3.

Step 3 Assume J_2 as the stressing-end and J_1 as the dummy-end, repeat Step 2, and we plot the line *fedNk* in Fig. 26.3.

Step 4 Point of intersection of the above two lines, N, is then the *NULL POINT*.

Step 5 Cable extensions (as explained earlier):

- Cable elongation at $J_1 = \frac{1}{AE}$ (area *abcNgha*)
- Cable elongation at $J_2 = \frac{1}{AE}$ (area *fedNgif*)

Step 6 Jack-pressures (assuming 96% jack efficiency, usual):

at J_1 : $\frac{\text{force } ha}{\text{ram area of Jack} \times 0.96}$

at J_2 : $\frac{\text{force } if}{\text{ram area of Jack} \times 0.96}$

Step 7 (i) Unless the prestressing system adopted is free of slip (e.g. in a 'nut' or 'button head' system), a certain amount of slip is bound to occur when locking the cable into its anchorage (generally 6 to 10 mm). Normally, initial prestressing force in a cable should not exceed 0.7 of its ultimate at any section, but a Jacking Force P_0 of up to 0.8* of ultimate is allowed so long as after slip-effect the prestressing force along the cable at no section exceeds 0.7 of ultimate, and this slip-effect is brought on (by releasing the jack and locking the cable) within about 2 minutes after reaching 0.8 of ultimate force in the cable at the jacking point.

(ii) Assuming that the slip effects from the two jacking ends die out at sections c and d, then, as explained earlier:

$$\text{slip at } J_1 = \frac{1}{AE} \text{ (area } abcb'a'a)$$

$$\text{slip at } J_2 = \frac{1}{AE} \text{ (area } fede'f'f)$$

It has to be understood that the effect of slip is to lower the lines abc and fed to their mirror-images $a'b'c$ and $f'e'd$, shown in Fig. 26.3.

Reason: On account of slip up to c (consider stressing-end J_1 for example) all points in the cable from a up to c move towards c and, therefore, relative to c carry no force i.e. have the same force as at c. This lowers line abc to horizontal $c'c$. In addition, the physical slip movement from a to c attracts opposing friction, which, in magnitude is same as between c and a, only reversed this time. This lowers $c'c$ to $a'b'c$ (mirror image of abc wrt $c'c$).

(iii) So, for the first trial, locate points c and d where the vertical ordinates equal 0.7 of ultimate force in cable and work out the magnitudes of the slips at the two ends. If these are equal to or more than the normal slips associated with the prestressing system, then OK, and specify the same as required slips in the drawing. Otherwise, re-select locations of points c and d (at ordinates less than 0.7 of ultimate force value) till the estimated slip values equal the above referred normal slip values. This line $a'b'cNde'f'$ is established.

Step 8 Vertical ordinates below line $a'b'cNde'f'$ at various sections may be read off and these then represent the P_i (initial prestress force) in the cable at these sections, and their products with $\cos\theta$ and $\sin\theta$ at the respective sections represent the horizontal and the vertical components of the initial prestress force in the cable at those sections (θ being inclination of tangent to cable profile in elevation with the horizontal at the section). "$P_i \cos\theta e$" represents the primary initial prestress moment due to the cable at any section where e is eccentricity of the cable wrt centroid of the section.

NOTE: Repeat above steps for each cable and then sum up "$P_i \cos\theta e$" values and "$P_i \sin\theta$" values of all cables in each stage of prestress at each section for (first calculating secondary prestress values in indeterminate structures and then for) stress calculations.

5. If the **cable is stressed only from one end**, say J_1, then in Fig. 26.3 the prestressing force distribution before slip will be the ordinates below $abcj$ and the Null point will shift to j itself. Slip at the jacking end will be $\frac{1}{AE}$ (area $abcb'a'a$), and ordinates below line $a'b'cj$ will represent the P_i values after friction and slip. Extension at stressing end will be $\frac{1}{AE}$ (area $abcjiha$).

6. If the **cable is short or friction is too large,** the slip effects from the two stressing ends may even overshoot N (Fig. 26.3). In such a case, the mirror images of lines abN and feN (drawn wrt horizontal through N) will then bodily lower down further by constant ordinates X and Y, respectively, such that

$\frac{1}{AE}(X \cdot gh) + \frac{1}{AE}$ (area between abN and its mirror image line) = magnitude of normal slip (from where X can be evaluated and the ordinates P_i read off)

and $\frac{1}{AE}(Y \cdot gi) + \frac{1}{AE}$ (area between feN and its mirror image line) = magnitude of normal slip (from where Y can be evaluated and the ordinates P_i and off).

In such a case if the cable was stressed from only one

* Preferably not more than 0.75 of ultimate force in the cable.

end, say from end J_1, and $\frac{1}{AE}$ (area between abj and its mirror image line) was less than the magnitude of the normal slip, then mirror image line of abj (wrt horizontal through j) would bodily lower further by a constant ordinate Z, such that:

$\frac{1}{AE}(Z \cdot hi) + \frac{1}{AE}$ (area between abj and its mirror image line) = magnitude of normal slip (from where Z can be evaluated and then the ordinates P_i read off).

26.6 EFFECT OF CABLE-BLOCKAGE

Sometimes a cable in a short zone of its length may get blocked due to accidental leakage of slurry from concrete at the time of casting or even from leakage of grout from a neighbouring cable.

This blockage might become a solid lock which might be realised only at the time of attempting to stress this particular cable. (Refer to explanation given in Sec. 26.4 under 'Losses in Prestress' of this chapter as to how 'both-end stressing' can be particularly advantageous in such an emergency.)

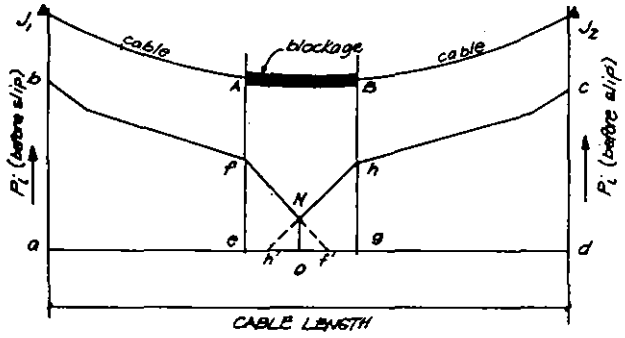


Fig. 26.4

In such a case proceed as follows:

Step 1 Try 'see-saw' stressing, i.e., carefully 'pull' the cable from one end at a time (keeping the other end 'locked' in the jack at that time) and release, taking care not to increase the pulling force beyond 0.75 to 0.77 of ultimate force possible in the cable. By this alternate see-saw stressing and releasing process try to break the grout-block. The process can take considerable time and, unless the blockage is not too hard and or is reasonably small in length, it may not be possible to break it. At any rate, in the process, the approximate locations of points A and B (limits of the blockage) can be reasonably predicted since the extending cable length up to its jacking point can be roughly estimated from the available extension, knowing that the rate of extension is usually about 4.5–5 mm per meter of the extending length for an average cable force of 0.6 of ultimate. If the blockage is released, carry out the scheduled stressing, but if blockage remains intact, then proceed to Step 2 ahead.

Step 2 Stress the cable from each end, gradually and simultaneously (taking precaution the blockage does not suddenly break with a jerk), raising the pulling force to 0.8 of the ultimate force. Note down the extension at each end, lock the cable at each end and note down the slips, as usual. Thus ordinates ab and dc (before slips), each equal to 0.8 of ultimate, are plotted (see Fig. 26.4).

- Knowing the approximate locations of blockage points A and B (which, where micro-drilling tools and expertise are available, can be located reasonably accurately) and hence the values of $(\mu\theta + Kx)$ from J_1 to A and from J_2 to B, the ordinates ef and gh Fig. 26.4 (being the cable forces at A and B can be quickly evaluated and plotted.
- The blockage acts like a rock anchor for the embedded cable from A towards B and from B towards A. If the force ef dies down to zero at f' and gh dies down to zero at h', then force $ef = f_b \times p_m \times$ (length ef') and force $gh = f_b \times p_m \times$ (length gh')
  where: f_b = average bond stress afforded by the grout in the blockage and p_m = effective perimeter of cable section gripped.

  Thus the lengths ef' and gh' can be estimated and luckily if lines hh' and ff' intersect, the minimum prestressing force ON (at the null point N) can be worked out and this value of force in this particular cable used in checking the overall stresses at this section. Chances are the stresses will turn out acceptable. Even if the extreme fibre stress turns out to be slightly tensile, the non-prestressed reinforcement within the tensile stress block can be checked to carry this tension force (at a reasonable over-stress as acceptable). Thus, the structure may be salvaged.

26.7 DESIGN OF CABLE-ANCHORAGE ZONES (END BLOCKS) IN POST-TENSIONED PRESTRESSED CONCRETE

Owing to extremely high bursting and spalling stresses set up in concrete immediately in front of a prestressing anchorage at the (very) time of tensioning the prestressing cable, design of reinforcement in the anchorage zone is of paramount importance. Actually, the stresses far exceed any elastically permissible values and this complicates the

analysis. The stress situation is somewhat akin to that in concrete above (or below) a bearing plate or those in the immediate neighbourhood of a concrete hinge bearing (on either side of its throat). The prestressing force emanating from the anchorage travels into the concrete member initially in the shape of trajectories that set up strees-eddies in three dimensions within a certain distance — generally taken equal to three-quarters to full depth of beam at anchoring face. The zone within this distance is sometimes referred to as the 'lead-in' zone. The prestress force is assumed to come on the full 'effective section' only from the end of the lead-in zone.

If the section of the beam at the end of the lead-in zone is assumed rectangular (Case I), then the centroid of prestress force diagram coincides with that of the prestress stress diagram at that section. However, if the section is not rectangular (Case II), then the prestress stress diagram and the section area diagram may be multiplied together to draw the prestress force diagram at that section. Generally it is enough to assume the situation as Case I without any significant effect on the final detailing. Then the following step by step procedure may be followed for designing the anchorage zone reinforcement:

Step 1 Draw the prestress 'stress' diagram at the end of the lead-in zone.

Step 2 Divide the area fo this diagram into n equal area compartments, n being the number of cables at the end of the lead-in zone, stressed equally. Then if the cables at the end of the lead-in zone fall in line with the centroids of their respective strees-compartments, it is a case of Linear Force Distribution, otherwise it is a case of Non-linear Force Distribution. If it is the former, then go directly to each individual anchorage and draw for each its surrounding prism whose side-dimension $2a$ may be taken as the leasser of (a) twice the distance of the anchor centreline from the nearest 'free-edge' and (b) twice its distance from the nearest boundary of its 'neighbouring prism'. (Thus the prism side-dimension $2a$ for each individual anchor in this case is established.) However, if the case is one of 'Non-linear Force Distribution', then proceed by the method of 'successive resultants' { i.e., locate the resultants R_1, R_2, etc., of every two successive anchors down the elevation, treating the odd left out anchor (if any) as its own resultant (Fig. 26.5); then locate the resultants R_1', R_2',... of every two successive resultants already established (R_1' being the resultant of R_1 and R_2, R_2 being the resultant of R_3 and R_4 and so on) down the elevation; repeat the same process to locate the still bigger resultants R_1'', R_2'',... of every 2 successive earlier resultants R_1' and R_2', R_3' and R_4' and so on. Continue this all the way till one final total resultant R is located. Draw its surrounding prism whose side dimension is twice its distance from the nearest free-edge; then, within this prism, draw the prisms for each of its two immediate sub-resultants, noting that the prism side-dimension of a sub-resultant is the lesser of twice its distance from the nearest free-edge or twice its distance from the nearest boundary of its parent-prism'; proceed in this way backward to the sub-resultants of each of the just dealt sub-resultants, until, in the end, the prism side-dimension $2a$ for each individual anchor is established} until the prism side-dimension $2a$ is established for each anchor.

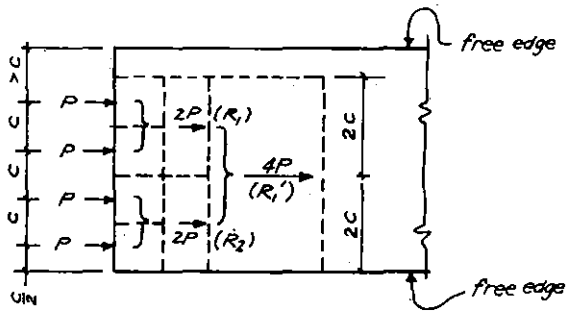


Fig. 26.5

Step 3 Establish the value of the concentration ratio $\frac{2a'}{2a}$ for each 'anchor'. ($2a'$ is the width of the loaded area of an anchorage, which for circular anchors may be taken as the 'side-dimension' of 'equivalent-area square'.)

Step 4 Estimate the bursting force T behind each anchor from the following formula:

$$T = 0.30P\left(1 - \frac{2a'}{2a}\right)$$

where P is the jacking force in the anchorage.

Step 5 Area of reinforcement steel required against bursting behind an individual anchor, is:

$$A_s = T/f_s$$

where f_s = working (elastic) tensile stress permitted in the reinforcement.

This A_s is to be provided at the centroid of the bursting force distribution behind the anchor, which may be assumed at $\frac{2a}{4}$ from the face of anchor, along its axis. This A_s is generally provided partly in the form of (a) a helix (of length $2 \times \frac{2a}{4}$ i.e.

of length a) placed touching the anchor face, and (b) 2, 3, or 4 grill meshes as close to the anchor as detailing would permit duly spaced between each other. If diameter of helix bar is d, its pitch is p, then area of steel provided by it against bursting is,

$$A_h = \left(\frac{\pi}{4}d^2\right) 2\frac{a}{p}$$

If the overall detailing of each grill mesh is such that behind each anchor we get 2 horizontal bars above and 2 below it and 2 vertical bars on either side of it (i.e., 4 bars in each direction, per mesh), and if say 4 grill meshes are provided one after another as described above, then section area of each bar in each mesh is,

$$= \frac{(A_s - A_h)}{4} \times \frac{1}{4}$$

Small bar diameters must be adopted, preferably 6 mm for helix and 10 mm for mesh. Also low tensile strength reinforcement is preferable. All this helps limit crack width.

Step 6 In addition to the above detailed reinforcement against bursting, anti-spalling reinforcement needs to be provided within the distance between the concrete face and the far end of the anchor. Normally 2 successively placed grill meshes, made of 10 mm diameter reinforcement bar, so as to give at least 4 horizontal and 4 vertical bars about each anchor, per grill, will suffice. This may be suitably increased proportionately if jacking force per anchor exceeds about 100 T.

The C and CA method of design of reinforcement in the anchorage Zone (after establishing the correct $2a$ value by Guyon's method explained in Step 2 above) is summarised below (for each anchor):

- See Fig. 26.6
- Prestressing force at anchoring $= P$
- Crushing stress $f_c = \frac{P}{2a^2}$ (kg/cm^2)
- Tensile stress in concrete

$$fy = f_c\left[0.4625\left(\frac{a'}{a}\right)^2 - 1.3\frac{a'}{a} + 1.1\right] \text{ kg/cm}^2$$

- Bursting force $T = P\left(0.5 - 0.4\frac{a'}{a}\right)$, shown shaded.
- Permitted tensile stress in concrete $f_t = 0.39\sqrt{u_t}$ kg/cm^2
- u_t = standard cube crushing strength of concrete at transfer in kg/cm^2
- Bursting reinforcement $A_{st} = \frac{T}{f_{st}}\left[1 - \left(\frac{Kf_t}{f_y}\right)^2\right]$

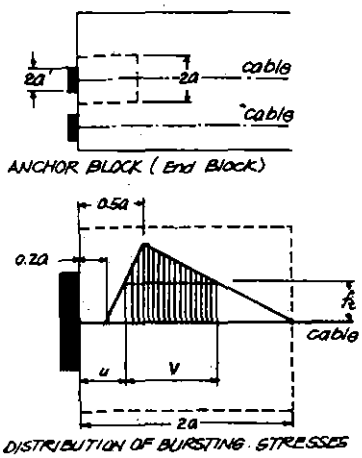


Fig. 26.6

where $K = 0.5\frac{a'}{a} + 1.12$

- Location of bursting steel ~ within dimension v:

$$u = \left(0.2 + \frac{0.3Kf_t}{f_y}\right)a \text{ and } v = 1.8a\left(1 - \frac{Kf_t}{f_y}\right)$$

However, additional reinforcement, the anti-spalling reinforcement, should be provided as explained in Step 6 above.

Typical Detailing at Certain Anchorages ~

(a) Fully embedded (dead) anchorages (Fig. 26.7): When a passive (dead) anchorage is embedded in the concrete it develops deformations beneath anchorage due to compression at tensioning and tensile stresses behind the anchorage.

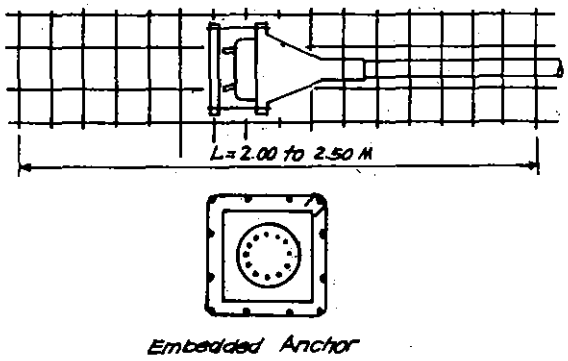


Fig. 26.7

To avoid cracking, it is necessary to include bars parallel to the cables to hold the concrete.

To avoid problems it appears that the steel should take up 20 to 30% of the anchorage force.

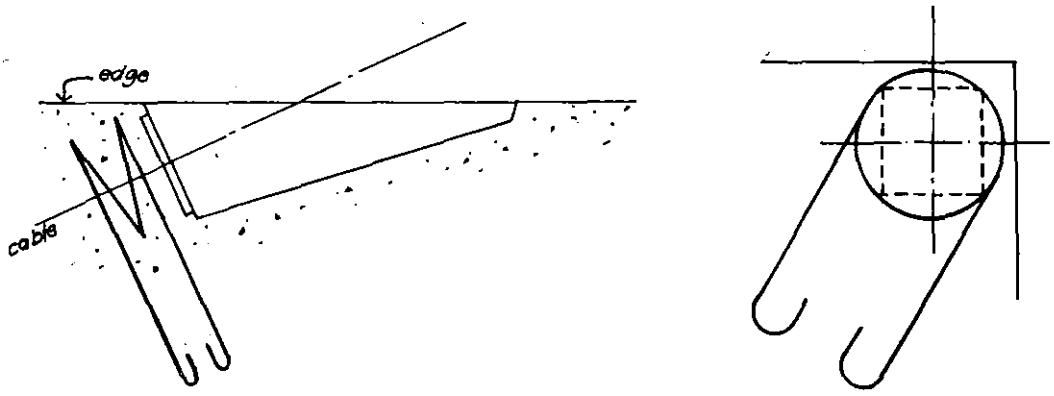


Fig. 26.8

(b) Anchorages at the 'edges' of structures (Fig. 26.8): It is necessary to provide special reinforcement for anchorages which are close to the surface of the concrete, by stirrups, tying the anchorage into the body of the structure.

(c) Anchorages in the thickening of webs (Fig. 26.9): This type of anchorage involves the following stress:
- a tensile stress in that part of the web situated beyond the anchorage. This is the same tensile stress as that of the embedded anchorage.
- moments in the web due to the eccentricity of the tendon.
- a shear force between the web and the thickening.

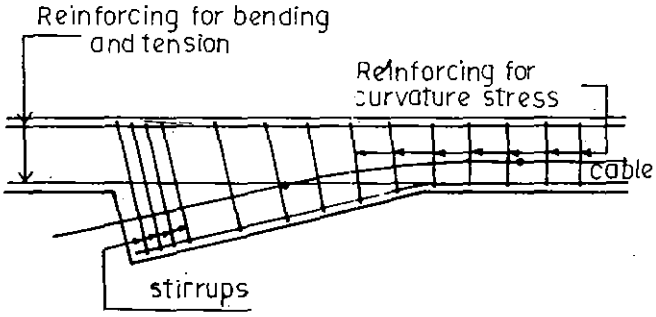


Fig. 26.9

To counter these stresses it is necessary to provide longitudinal reinforcement in the web together with stirrups in front of the anchorage to resist splitting on the web at the thickening.

Finally, in the cable's curved portion it is necessary to resist the side thrust of the cable. These forces are taken up by stirrups which should continue for some distance beyond the zone of curvature (shown in Fig. 26.9). This is because of inaccuracies at the site which may considerably alter the position of the zone of deviation.

26.8 CONCORDANT CABLE PROFILES IN CONTINUOUS PRESTRESSED CONCRETE BEAMS

A concordant cable is one which coincides with the resultant line of thrust at every section. (Resultant line of thrust is the locus of the centroids of the prestress force diagrams, section to section, and its position at any section is given by the 'total' prestressing moment divided by the prestressing force at that section, measured w.r.t.c.g. of the section.) Position of resultant line of thrust is given by $\frac{m_t^p}{P}$ with respect to the centroid of section. Position of cable is given by $\frac{m_o^p}{P}$ with respect to the centroid of section, m_o^p being the primary moment, $P \cdot e^*$, at the section. If the two have to be the same, then:

$$\frac{m_o^p}{P} = \frac{m_t^p}{P}$$

But m_t^p (total prestress moment) = $m_o^p + \sum_{s=1 \text{ to } n} m_s p_s$

where : $m_o^p =$ primary prestress moment and

$\sum_{s=1 \text{ to } n} m_s p_s =$ secondary prestress moment

Hence $\frac{\Sigma m_s p_s}{P} = 0$

Since $P \neq 0$, and m_s is a function of p_s (moment diagram due to unit p value), thus p_s must be zero. Then, from the general equation of compatibility ($V_{ro} + V_{rs} p_s = 0$) it follows that: $V_{ro} = 0$.

* "$P \cos\theta \cdot e$" in fact, as explained earlier.

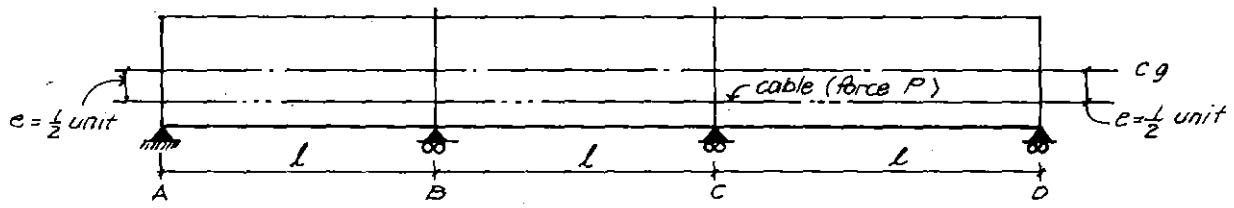


Fig. 26.10

This means:

$$\int \frac{m_r m_o^p}{EI} ds + \int \frac{s_r s_o^p}{GA} ds + \int \frac{n_r n_o^p}{EA} ds = 0,$$

$r = 1$ to n

$n =$ no. of statical indeterminacies.

This then is the general condition of concordance. In beams, where shear and thrust energies are particularly insignificant, the second and third terms may be ignored, so that in continuous prestressed concrete beams the general condition of cable-concordance reduces to:

$$\int \frac{m_r m_o^p}{EI} ds = 0$$

i.e., 'that cable profile is concordant whose primary prestress moment m_o^p diagram when mated with each of the $m_1, m_2, \ldots, m_n$ diagrams of the continuous beam, with $1/EI$ diagram, adds up to zero'.

Example on Plotting Resultant Line of Thrust

The above 3-span continuous beam (Fig. 26.10), of constant cross-section, is prestressed by means of a straight cable as shown. Locate the resultant line of thrust (a purely academic case, only to explain the method):

Procedure:

(i) Calculate n (statical indeterminacies),

(ii) Introduce biactions at the 'n' releases,

(iii) Draw m_o^p diagram (the BMD on S_o due to applied load of prestress),

(iv) Draw $m_1, m_2, m_3 \ldots$ diagrams (due to unit biactions, on S_o)

(v) Find $V_{10} \left(= \int \frac{m_1 m_0}{EI} ds\right)$,

$V_{20} \left(= \int \frac{m_2 m_0}{EI} ds\right)$,

$V_{11} \left(= \int \frac{m_1 m_1}{EI} ds\right)$, and

$V_{12} \left(= \int \frac{m_1 m_2}{EI} ds\right)$

(vi) Find p_s from $V_{ro} + V_{rs} p_s = 0$

(vii) Find $m_t^p (= m_0^p + m_1 p_1 + m_2 p_2)$ at each section.

(viii) Find $\frac{m_t^p}{P}$ at these sections, which gives ordinates of the Resultant Line of Thrust at those sections, wrt centroid of sections.

Solution: $n = 3(M - N + 1) - R = 3(6 - 4 + 1) - 7 = 2$, introduce 2 releases and, therefore, 2 biactions p_1 and p_2. Let the releases be moment releases by the sides of the intermediate supports, as shown in Fig. 26.11. EI is constant.

$$EI\, V_{10} = \Sigma m_1 m_o^p = 2\left\{\frac{l/2}{3}\left(0 + \frac{P}{2}1 + 4\frac{P}{2}\frac{1}{2}\right)\right\} = \frac{Pl}{2}$$

$$EI\, V_{20} = \Sigma m_2 m_o^p = \frac{Pl}{2}$$

$$EI\, V_{11} = \Sigma m_1 m_1 = 2\left\{\frac{l/2}{3}\left(0 + 1 + 4\frac{1}{2}\frac{1}{2}\right)\right\} = \frac{2l}{3}$$

$$EI\, V_{12} = \Sigma m_1 m_2 = \frac{l/2}{3}\left(0 + 0 + 4\frac{1}{2}\frac{1}{2}\right) = \frac{l}{6}$$

$$EI\, V_{22} \equiv EI\, V_{11} = \frac{2l}{3}$$

$$EI\, V_{21} \equiv EI\, V_{12} = \frac{l}{6}.$$

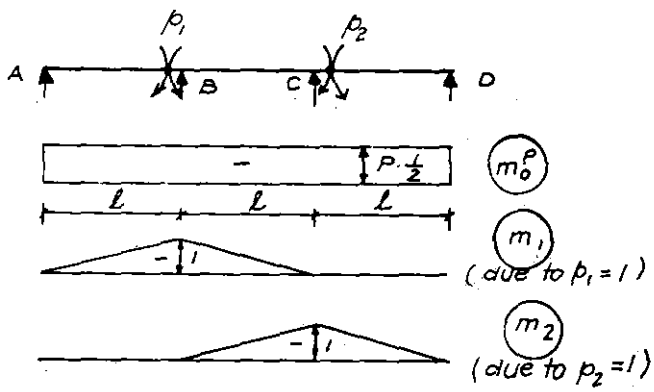


Fig. 26.11

- Substituting in the compatibility equations

$$V_{10} + V_{11} p_1 + V_{12} p_2 = 0$$

and $\quad V_{20} + V_{21}p_1 + V_{22}p_2 = 0$

We have: $\dfrac{Pl}{2} + \dfrac{2l}{3}p_1 + \dfrac{l}{6}p_2 = 0$

and $\quad \dfrac{Pl}{2} + \dfrac{l}{6}p_1 + \dfrac{2l}{3}p_2 = 0$

giving: $\quad p_1 = p_2 = -\dfrac{3}{5}P$

- Thus total prestressing moments m_t^p are:

$$m_{t_A}^p = (m_0^p + m_1 p_1 + m_2 p_2)_A$$
$$= -P/2 + 0 + 0 = -P/2$$
$$m_{t_B}^p = (m_0^p + m_1 p_1 + m_2 p_2)_B$$
$$= -P/2 + (-1)\left(-\frac{3}{5}P\right) = P/10$$
$$m_{t_C}^p = P/10 \text{ and } m_{t_D}^p = -P/2$$

- This 'total' prestressing moment diagram is shown in Fig. 26.12, and dividing its ordinates by the prestressing force at each section (P, in this case) we obtain the profile and location of the resultant line of thrust as shown in the second part of Fig. 26.12.

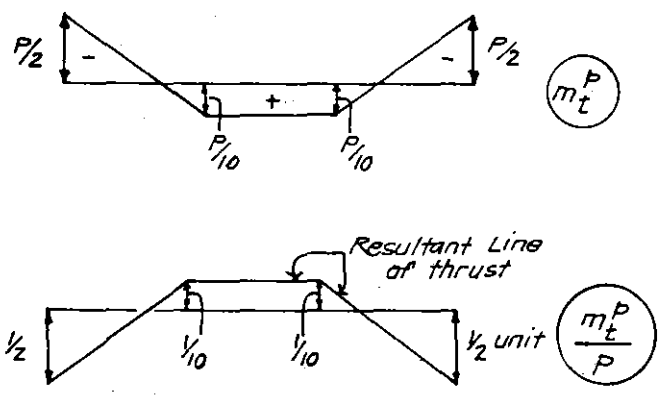


Fig. 26.12

26.9 NON-CONCORDANCE AND ESTIMATION OF SECONDARY PRESTRESS MOMENTS, SHEARS AND REACTIONS IN CONTINUOUS PRESTRESSED CONCRETE BEAMS

In practice when carrying out a workman-like practical design of a continuous prestressed concrete beam-type bridge-deck, the continuous cable profile is fixed from practical considerations of detailing and ease rather than purely from theoretical considerations (of plotting cable zones with their theoretical upper and lower limits, then 'fishing' for a concordant profile and then linearly transforming the same, should the former lie outside the physical depth of the beam!). Consequently, in most practical designs, the continuous cable profile is not concordant, i.e., does not coincide with the resultant line of thrust (see description given earlier in 26.8 in which the concordant cable has been explained in some detail). The eccentricity between the two at any section sets up another moment (and its associated shears and consequent support reactions). This is generally referred to as 'parasitic' (parasite-like) or 'secondary' prestress moment, causing parasitic shears and reactions. These would be zero if the tendon profile only coincided with the resultant line of thrust. ***It must be remembered that while the simple prestress moment resulting from the mere product of 'horizontal prestressing force and its eccentricity with respect to the centroid of the concrete section' (called 'primary' prestress moment, m_o^p) is a purely internal entity—and, if inclined, has its primary prestress shear, but never any support reactions, the secondary prestress moment and shear are associated with their balancing support reactions (i.e., external reactions) and are thus an external entity like any external load.*** Since these secondary moments cause vertical reactions at supports, the total overall beam length may be imagined as a 'simple' long beam between its two ends, subjected to upward or downward vertical loads at the intermediate support locations (equal to the support reactions due to secondary prestress moments). ***Hence, the secondary prestress moment diagram between two successive supports is always linear.*** At those supports where secondary prestress reaction opposes the self dead load reaction, it should be ensured that the latter is more so that there is no 'lifting up' of the beam at such supports (as otherwise this would change the support conditions for various loadings). Generally, the self dead load reactions are higher anyway. ***It must be understood that since these secondary prestress moments, shears and reactions, are directly proportional to EI (like the fixity moments due to any applied loading in an indeterminate structure), their magnitude reduces when cracking sets-in and EI reduces. Beyond elastic limit, with each additional hinge forming, as the structure cracks and tends to come closer and closer to a statically determinate form, the magnitude of these parasitic values keeps reducing till they disappear when the structure becomes determinate.*** In a simply supported prestressed concrete beam (or in any statically determinate prestressed concrete structure) there is only primary prestress moment (and shear and thrust), causing no support reactions (primary prestress being an internal entity) and no secondary prestress effect. Having understood that the secondary prestress effect is like any other externally applied loading on the indeterminate structure, it can easily be evaluated, say by the Flexibility Method of influence coefficients (explained earlier in Ch. 20) taking the primary prestress moment

• EXAMPLE

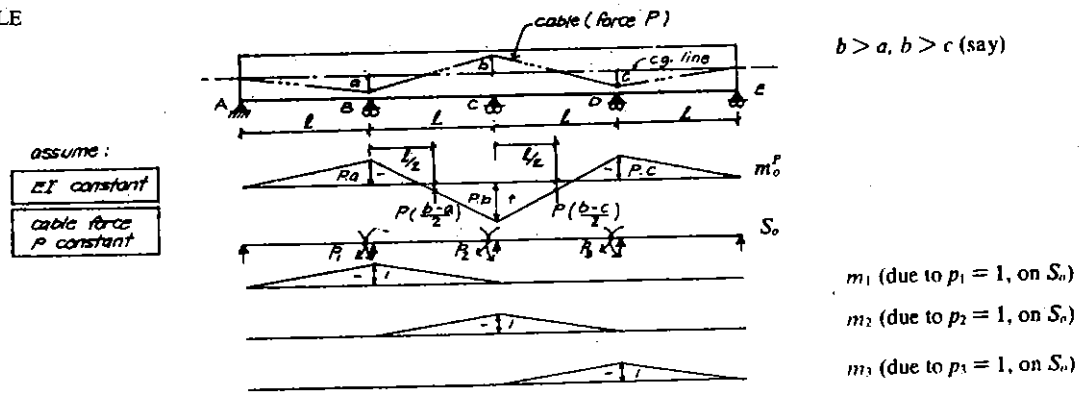


Fig. 26.13

diagram, m_o^p, as the m_o (statical) moment diagram due to the 'applied loading'.

Hence, in a practical case of a continuous prestressed concrete beam, in order to work out the total prestress moments: $m_t^p (= m_o^p + \sum_{r=1}^{n} m_r p_r^p)$, total prestress shears: $s_t^p (= s_o^p + \sum_{r=1}^{n} s_r p_r^p)$ and the support reactions due to secondary prestress, exactly the same procedure as explained in the earlier referred Flexibility Method may be followed, taking the primary prestress moment diagram m_o^p (the diagram showing the products of horizontal prestress force and its eccentricity wrt to centroid of the concrete section, at various sections) directly as being the statical moment diagram m_o due to applied loading (prestress in this case) on the statically made determinate structure. Secondary prestress moment and shear 'at any section' would then be simply: $\sum_{r=1}^{n} m_r p_r^p$(i.e., $m_1 p_1 + m_2 p_2 + \ldots + m_n p_n$) and $\sum_{r=1}^{n} s_r p_r^p$(i.e., $s_1 p_1 + s_2 p_2 + \ldots + s_n p_n$), respectively, with $m_1, m_2, \ldots$ and $s_1, s_2, \ldots$ being the ordinates of these diagrams 'at the section' and p_r^p(i.e., $p_1, p_2, \ldots, p_n$) being the biaction p-values (under prestress case) solved out of the n compatibility equations.

The corresponding support reactions can then readily be worked out knowing that the secondary prestress moment diagram is linear in between successive supports. The same procedure holds good for prestressed concrete frames.

It is of interest to remember that in a 2-span practical prestressed concrete continuous beam, the secondary prestress moment at the central support is of the same sign as the primary prestress moment there (i.e., sagging type, causing compression at top fibre and tension at bottom fibre) and can be almost equal in magnitude. Hence its usefulness should not be underestimated. In terms of effect, it is not just secondary. At span points, it can of course be opposite in effect to the primary prestress moment. Again, in a 3-span case too it assists the primary moment at the intermediate supports but opposes it in the span regions. Parasitic prestress moments, shears, and reactions (and thrusts in frames) must be thoroughly accounted for.

Interesting Special (Academic) Case (Fig. 26.13)

In a continuous psc beam if the cable profile is 'linear between successive supports' and of 'zero eccentricity at ends', then the total-moment due to prestress at any section is zero ($m_t^p = 0$), or, in other words, the resulting line of thrust coincides with the centroidal axis of beam, or that it is axial. Thus, in such a case the prestress causes pure compression P/A and no bending and not even shear.

It should also be noted that one of these two cable-profiles is a linear transformation of the other and the two have same resultant line of thrust. (See ahead for the 'principle of Linear Transformation' and its effect of not altering the total prestress moment.)

Refer to Fig. 26.13:

• $n = 3(8 - 5 + 1) - 9 = 3$, so the beam of 4 spans needs 3 releases to make it statically determinate (S_o). Let us introduce 3 moment releases as shown in Fig. 26.13: p_1, p_2 and p_3, m_o^p, m_1, m_2 and m_3 are shown in Fig. 26.13.

• $$EI\,V_{10} = \Sigma m_1 m_0 = \frac{l/2}{3}\left\{0 + Pa \times 1 + 4 \times \frac{1}{2} \times \frac{Pa}{2}\right\} + \frac{l/2}{3}\left\{Pa \times 1 + 0 + 4 \times (-\frac{1}{2}) \times P(\frac{b-a}{2})\right\} = \frac{Pl}{6}(4a - b)$$

$$EI\,V_{20} = \Sigma m_2 m_0 = \frac{l/2}{3}\{0 + Pb \times (-1) + 4 \times \left(-\frac{1}{2}\right) \times P\left(\frac{b-a}{2}\right)\} + \frac{l/2}{3}\{-Pb + 0 + 4 \times \left(-\frac{1}{2}\right) \times P\left(\frac{b-c}{2}\right)\} = \frac{Pl}{6}(a - 4b + c)$$

$$EI\,V_{30} = \Sigma m_3 m_0 = \frac{l/2}{3}\{0 + Pc + 4 \times \left(-\frac{1}{2}\right)$$

$$\times P\left(\frac{b-c}{2}\right)\} + \frac{l/2}{3}\{Pc + 0 + 4 \times \frac{1}{2} \times \frac{Pc}{2}\}$$

$$= \frac{Pl}{6}(4c - b)$$

$$EI\ V_{11} = \Sigma m_1 m_1 = 2 \times \frac{l/2}{3}\left\{0 + 1 + 4 \times \frac{1}{2} \times \frac{1}{2}\right\}$$

$$= \frac{2l}{3}, \equiv EI\ V_{22} \equiv EI\ V_{33}$$

$$EI\ V_{12} = \Sigma m_1 m_2 = \frac{l/2}{3}\{0 + 0 + 4 \times \frac{1}{2} \times \frac{1}{2}\}$$

$$= \frac{l}{6}, \equiv EI\ V_{21} \equiv EI\ V_{23} \equiv EI\ V_{32}$$

$$EI\ V_{13} = \Sigma m_1 m_3 = 0, \equiv EI\ V_{31}$$

- Substituting in the 3 compatibility equations:

$$V_{10} + V_{11}p_1 + V_{12}p_2 + V_{13}p_3 = 0$$
$$V_{20} + V_{21}p_1 + V_{22}p_2 + V_{23}p_3 = 0$$
$$V_{30} + V_{31}p_1 + V_{32}p_2 + V_{33}p_3 = 0$$

We get: $p_1 = -Pa$, $p_2 = Pb$, $p_3 = -Pc$

- Hence : , $m_{t_A}^{p} = (m_0 + m_1p_1 + m_2p_2 + m_3p_3)_A$

$$= 0 + 0 + 0 + 0 = 0$$

Similarly $m_{t_B}^{p} = -Pa + (-1)(-Pa) + 0 + 0 = 0,$

$$m_{t_C}^{p} = 0,\ m_{t_D}^{p} = 0,\ m_{t_E}^{p} = 0,$$

and infact m_t^P at any section is found to be equal to zero.

- Hence the position of resultant line of thrust ($= m_t^p/P$ at any section wrt cg of the section) is also equal to zero wrt cg of each section, i.e. coincides with cg line.
- ***This means that the resultant line of thrust is coinciding with the section cg line all along, i.e., is axial, which is the same as if the cable itself was placed at zero eccentricity wrt cg of section at each section.***
- ***It should also be noted that even the total shear due to prestress ($s_t^p = s_o + \sum s_r p_r$) in the present type of cable is zero at each section, eventhough the cable looks inclined in each span.***

26.10 LINEAR TRANSFORMATION OF CONTINUOUS PRESTRESSING TENDON PROFILES IN CONTINUOUS BEAMS

In continuous prestressed concrete beams designed theoretically, if the 'theoretically established' tendon profile lies outside the beam in certain zones, it can be brought back into the beam by the principle of linear transformation without changing the resultant line of thrust (i.e., without changing the total prestressing effect).

The principle of linear transformation of tendon profile in continuous beams states that:

"The resultant line of thrust remains unaltered, i.e., the total prestress moment, etc., remain unchanged even if the position of tendon is shifted up or down, vertically, at any or all the intermediate supports, by any amount, so long as the position of the tendon at the two ends is kept unchanged and the new profile in each span is obtained by pure linear transformation of the previous profile" (Fig. 26.14).

Proof

Consider, for example, a 3-span continuous prestressed concrete beam with a given cable profile as shown in Fig. 26.15(a). If this cable profile is linearly transformed as shown in Fig. 26.15(b) (downward by $2a$ at one intermediate support and by $2b$ upward at the other) let us calculate and see if the total prestressing moments, etc., remain unchanged, the transformation having been done according to the above-stated principle (Fig. 26.14).

NOTE For ease, assume EI is constant, horizontal prestressing force P constant throughout, and spans equal.

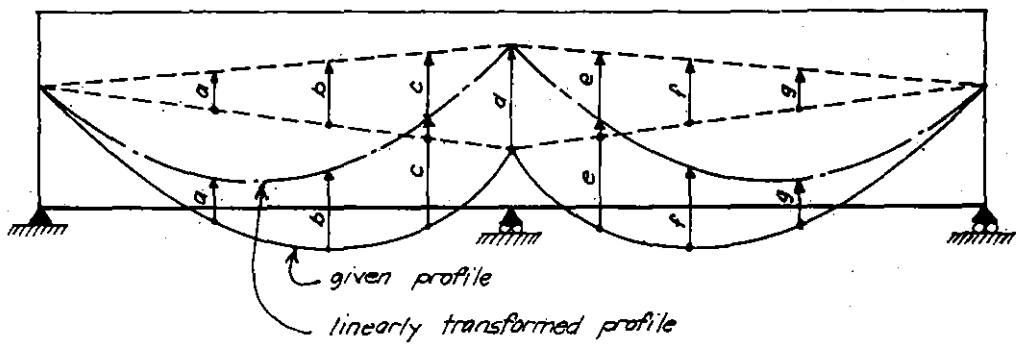


Fig. 26.14

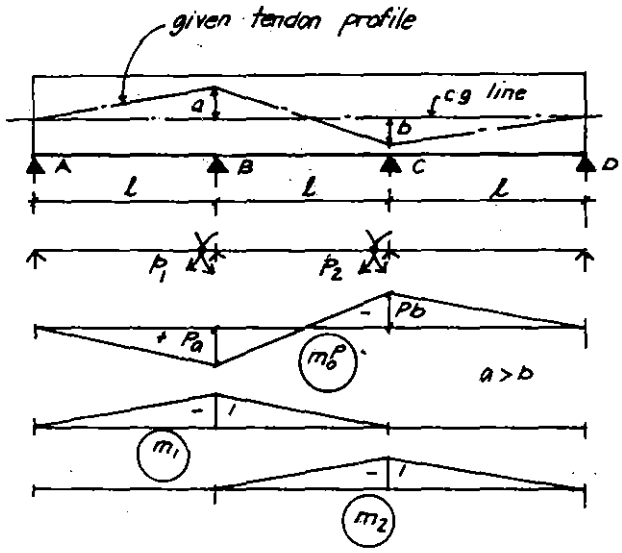


Fig. 26.15(a)

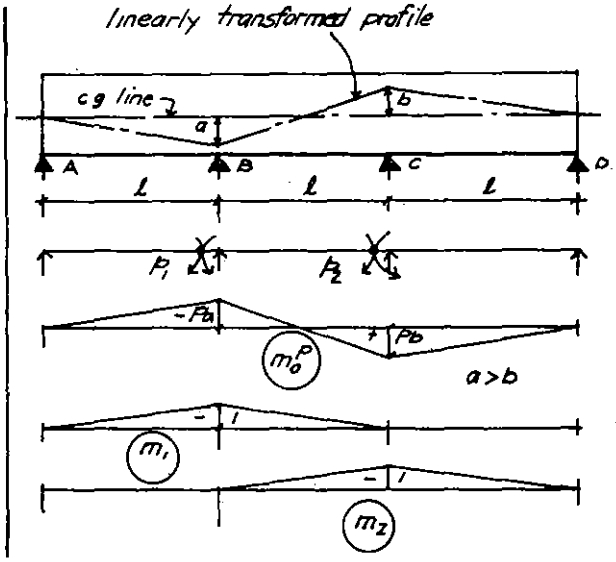


Fig. 26.15(b)

$$V_{10} + V_{11}p_1 + V_{12}p_2 = 0$$
$$V_{20} + V_{21}p_1 + V_{22}p_2 = 0$$
$$V_{10} = \int \frac{m_1 m_o^p}{EI} = \frac{1}{6EI}\left(0 - Pa - 4\frac{Pa}{2}\frac{l}{2}\right) + \frac{l}{6EI}\left(-Pa - 4\frac{1}{2}P\frac{(a-b)}{2}\right)$$
$$= \frac{Pl}{6EI}(b - 4a)$$
$$V_{20} = \int \frac{m_2 m_o^p}{EI} = \frac{Pl}{6EI}(4b - a)$$
$$V_{12} = V_{21} = \int \frac{m_1 m_2}{EI} = \frac{l}{6EI}$$
$$V_{11} = V_{22} = \int \frac{m_1 m_1}{EI} = \frac{2l}{3EI}$$

Substituting in the above two equations, and solving these, we get:

$$p_1 = Pa, \quad p_2 = -Pb$$

so that total p.s. moment at, say B, is
$m_{t_B}^p = m_0^p + m_1 p_1 + m_2 p_2 = Pa - 1Pa + 0 = 0$
likewise $m_{t_{B-C}}^p = m_0^p + m_1 p_1 + m_2 p_2$

$$= P\frac{(a-b)}{2} + \left(-\frac{1}{2}\right)Pa + \left(-\frac{1}{2}\right)(-Pb) = 0$$

and $m_{t_c}^p = -Pb + 0 + (-1)(-Pb) = 0$
also, p.s. shear, say at just to left of B, is $s_{t_{B_L}}^p = s_0^p + s_1 p_1 + s_2 p_2$

$$= \frac{Pa}{l} + \left(-\frac{1}{l}\right)Pa + 0 = 0$$

likewise, say at middle of span $A - B$
is $s_{t_{A-B}}^p = \frac{Pa}{l} + \left(-\frac{1}{l}\right)Pa = 0$

and at A, reaction = shear = $Pa/l + \left(-\frac{1}{l}\right)Pa = 0$

$$V_{10} + V_{11}p_1 + V_{12}p_2 = 0$$
$$V_{20} + V_{21}p_1 + V_{22}p_2 = 0$$
$$V_{10} = \int \frac{m_1 m_o^p}{EI} = \frac{Pl}{6EI}(4a - b)$$
$$V_{20} = \int \frac{m_2 m_o^p}{EI} = \frac{Pl}{6EI}(a - 4b)$$
$$V_{12} = V_{21} = l/6EI$$
$$V_{11} = V_{22} = 2l/3EI$$

Substituting in the above two equations, and solving these, we get:

$$p_1 = -Pa, \quad p_2 = Pb$$

so that $m_{t_B}^p = -Pa - 1(-Pa) + 0 = 0$

$$m_{t_{B-C}}^p = -P\frac{(a-b)}{2} + (-1/2)(-Pa) + (-1/2)Pb = 0$$

and $m_{t_c}^p = Pb + 0 + (-1)Pb = 0$
also p.s. shear, say at just to left to B,

is $$s_{t_{B_L}}^p = s_o^p + s_1 p_1 + s_2 p_2 = -\frac{Pa}{l} + (-1/l)(-Pa) + 0 = 0$$

at A, shear = reaction = $-\frac{Pa}{l} + (-1/l)(-Pa) = 0$
likewise, say at middle of span $A - B$
Shear: $s_{t_{A-B}}^p = -\frac{Pa}{l} + (-1/l)(-Pa) + 0 = 0$
Hence, it can be seen that total moments, shears, and reactions etc., due to prestress remain same despite the linear transformation.

NOTE Regarding linear transformation of cable profiles in Prestressed Concrete Frames: There being no intermediate support in the transom of a single span frame, the entire tendon profile in the transom is bodily lifted or lowered in its linear transformation by the same amount e. Consequently this P_e moment diagram for the transom (i.e., the effect of linear transformation alone) is rectangular. As for the leg cable profile, it may be only 'bodily rotated' for its linear transformation, thereby giving a triangular P_e moment diagram as a result of this transformation. The effect of the above moment diagrams (rectangular in transom and triangular in the legs) can be readily worked by the flexibility method and it is seen that:

(i) ***In case of vertical legs, only additional thrust is caused in transom alone, and no other change***

(ii) ***In case of inclined legs, additional thrust is caused in transom as well as in the legs, no other change.***

However, all this is only of academic interest. ***In any practical design the cable profiles are selected from convenience of detailing and simplicity and then for such prefixed tendon profiles first the friction calculations are done so that exact initial prestress force normal to section is known at each section. Its product with cable eccentricity at each section gives the m_o^p (primary prestress moment) at various sections. This diagram is used as the 'applied load' statical moment m_o^p diagram and the frame analysed for this applied loading by the Flexibility Method, and the secondary prestress: moments $(m_1p_1 + m_2p_2 + \ldots)$, shears $(s_1p_1 + s_2p_2 + \ldots)$ and thrusts $(n_1p_1 + n_2p_2 + \ldots)$ calculated after working out the influence coefficients and $p_1, p_2, \ldots$ values from the compatibility equations.***

26.11 LIMITS OF LINEAR TRANSFORMATION OF CABLE PROFILES IN CONTINUOUS BEAMS

Effect of tendon transformation on ultimate strength. Contrary to general belief, it has been shown[1] that linear transformation of cable profiles in continuous prestressed concrete beams can, in some cases, lead to reduction in the ultimate load carrying capacity of the beam. This is possible when the transformation results in bringing the tendon close to the compression edge (at the first forming hinge sections), thereby reducing the effective depth. As the steel area remains the same, the proportion of steel-area to effective concrete area increases, making the section 'over reinforced', which in turn reduces the rotation capacity of the hinge. This can lead to a premature collapse of the hinge, resulting in an early failure of the beam (see also Ch. 28).

Limits of linear transformation

However, it has been shown[1] that the ultimate strength may not be seriously affected so long as it is ensured that linear transformation does not reduce the effective depth at any critical section to less than a third of the overall depth of section (see also Chs. 27 and 28).

REFERENCE

1. Raina, V.K., "Ultimate Moment Distribution in Continuous Prestressed Concrete Beams", *Ph.D. Thesis*, University of London, 1966.

CHAPTER 27

Behaviour of a Structure during the Loading History all the Way up to Collapse, and Estimation of Moments Attained at 'Ultimate' taking 'Redistribution' into Account

Synopsis

After elastic design, simply ensuring the ultimate moment of resistance of a critical section equals a certain load factor times its working load moment, does not necessarily guarantee that the structure will sustain the desired ultimate load, unless the structure accepts 100% redistribution of moments at 'ultimate'. Therefore, it is important to investigate the actually attainable ultimate moments at critical sections taking into account the compatibility of deformations and check that the (thus calculated) actually mobilisable ultimate strength of every critical section is not less than the required (codified) applied ultimate moment in a given statically indeterminate structure.

Notation

$M_2(M_u)$ moment of resistance at limit L_2
M_1 moment of resistance at limit L_1
M_0 applied bending moment at which critical section cracks (limit L_0)
M_{ps} prestressing moment
P prestressing force
e_s eccentricity of P, wrt centroid of section
w dead load per unit run
M_{DL} dead load bending moment
EI_{un} uncracked flexural rigidity
θ rotation (θ_p, α: plastic rotation)
A cross-section area
Z section modulus
z distance between hinge section and the adjacent point of contraflexure
d effective depth
D overall depth
l span
l_p equivalent plastified length
W_u applied ultimate load
f_{cu} cube strength of concrete at 28 days (crushing)
f'_c cylinder strength of concrete at 28 days (crushing)
f''_c maximum flexural compressive stress in concrete
e_p effective prestrain in tendon
e_{cp} strain in concrete at level of tendon due to stress under prestress and dead load
e_{csu} strain in concrete at level of tendon due to applied load at limit L_2
e_{cse} strain in concrete at level of tendon due to applied load at limit L_1
e_c strain in concrete at any fibre in flexural compression
n neutral axis factor
p'' $\dfrac{\text{volume of binders per unit legth of beam}}{\text{volume of concrete in unit length of beam}} \times 100$
ϕ curvature (ϕ_2, ϕ_1, ϕ_0, ϕ_{ps}: due to M_2, M_1, M_0, M_{ps}, respectively)

Suffixes 2 (or u) and 1 (or e) refer to stages at limits L_2 and L_1 respectively.

Suffixes l and r refer to 'left' and 'right' of hinge. Suffixes c, ps, and ms refer to concrete, prestress and mild steel respectively.

27.1 DEFORMATION CHARACTERISTICS

The early attempts to formulate a design procedure for reinforced concrete recognised the inelastic non-homogeneous characteristics of the composite material. This early emphasis on the inelastic properties was discontinued when the elastic straight line theory was introduced in the early part of this century. There are only a few papers in the period 1910 to 1930 which discuss the inelastic properties of the material, but since then there has been an ever increasing interest in this subject. Evans and Hognestad have published excellent reviews of this work. There have been many alternative stress blocks suggested for the flexural compressive stress distribution in concrete, such as, a cubic parabola by Mensch, a quarter ellipse by Kempton-Dyson, a fifth degree parabola by Lyse, a combination of parabola and rectangle by Rüsch, and a combination of parabola and trapezium by Hognestad.

Structural analysis is the evaluation of the distribution and magnitude of the load-effects (viz., bending moments, shearing forces and twisting moments) due to the applied loads, while structural 'design' embodies the calculation of section sizes and properties of the members for resisting the load-effects acting on them. M. Guyon has phrased it as follows: "The first object is to 'know', within the limits of our ability to do so; and the second is to 'apply', without going beyond what we know."

The flexural compressive stress distribution in concrete alters greatly between the working load and ultimate load owing to its inelastic behaviour beyond the elastic limit. When a structure is designed on the basis of elastic theory such that at working load the maximum concrete stress is a third of the cube crushing stress, it does not infer that the structure can carry thrice the working load before it fails even if it is statically determinate, nor is it possible to predict the collapse load by elastic analysis because the elastic theory embraces only the initial portion of the total load-deformation curve. In a statically indeterminate structure loaded to collapse, the bending moment at deformed sections will not necessarily be proportional to the applied load. When a member is designed according to the elastic theory, the code of practice requires that its ultimate moment of resistance is not less than a certain load factor times the applied working load moment. After cracking sets in and some sections start yielding, redistribution of moments commences. At the collapse of a structure it is possible that some critical sections may be unable to contribute their full (i.e., absolute) ultimate moment of resistance owing to the lack of redistribution and the obligation to compatibility. In such a case, the structure will have collapsed when the applied moment at these sections was still less than that required by the code. Therefore, it is preferable in a structure, designed on an elastic basis, to estimate the actually 'feasible' ultimate moment distribution, and check that the (thus calculated) actually mobilisable ultimate moment at every critical section (after redistribution) is not less than the anticipated applied ultimate moment required by the code. (In determinate structures, however, 100% redistribution can be assumed directly, but what has been stated above still holds.)

27.2 COLLAPSE BEHAVIOUR OF CONTINUOUS BEAMS

Consider a continuous beam being loaded proportionally from zero load to collapse load. During the loading history, a stage is reached when some critical sections being to yield (limit L_1). With total load on the 'beam as a whole' increasing, the burden of resisting this additional load falls more and more upon other sections that have not yet yielded. The yielding sections act like hinges, in that their stiffness is much reduced and they visibly rotate. The hinging regions are commonly idealised as being effective at the maximum-moment section and are called *plastic hinges*.

The bending moments at the sections of reduced stiffness do not increase proportionally with load and hence an increase in load must be taken by a rapid increase in moment at other less highly stressed sections. This adjustment in the distribution of bending moment is commonly called the 'redistribution' of moments.

Each time a plastic hinge is formed, the number of statical indeterminacies reduces by one, until eventually the beam becomes statically determinate in respect of such loading as remains to be applied to it. Then the development of yet another plastic hinge will make the beam unstable — a 'mechanism' causing its collapse! However, a premature collapse (i.e., when the number of hinges formed is less than $n+1$, n being the number of statical indeterminacies) is possible when any hinge reaches its defomational limit earlier and/or any individual span becomes unstable. A complete or full redistribution (adaptation) refers to the formation of $n+1$ hinges at failure with each hinged section attaining its full (absolute) ultimate moment of resistance through the phenomenon of redistribution.

The essential difference between steel and concrete structures is that in the former, the stress increases at high strains (i.e., strain hardening), while in the latter, after a distinct maximum stress, the stress decreases with increasing strain (i.e., strain softening). As a result of this, steel beams generally do not have a distinct collapse but exhibit large changes in geometry. On the other hand, owing to strain softening in concrete, the moment at a section does not remain constant once the ultimate value is reached, but diminishes with increasing deformation. This causes a distinct collapse in case of concrete. When the moment at a particular section is decreasing while the applied load on the whole is increasing, the yoke of resisting this load falls more and more on less deformed sections. The sharper the falling of moment at one section, the quicker the increase at others, and hence greater the redistribution.

27.3 PROPOSED METHOD FOR ESTIMATING TRUE ULTIMATE BENDING MOMENT DISTRIBUTION

In reinforced and prestressed concrete members, a stress-strain curve cannot be generalised into a basic relationship, simply because concrete is a heterogeneous composite material, in which, both the quality as well as the quantity of the constituent materials can vary. Instead a 'moment-deformation' curve is more fundamental[1] in such a material, the deformation being either curvature or rotation. Again, a moment-curvature relationship is less generalisable as compared to a moment-rotation relationship because, as tests

have shown,[2] the former seems to vary from section to section in a member even when the sections are exactly identical in composition, orientation and loading. The same tests have confirmed that the latter appears to be much less variable, probably because it refers to an entire hinging-region rather than to a section in it.

The necessity to idealise a *moment-deformation* characteristic is obvious because the actual experimental curve cannot be available every time for every member. The simplest idealisation is a bilinear idealisation, which is assumed to consist of one straight line from the origin to the point representing moment and deformation at yield (Limit L_1), and a second straight line joining the points representing moment and deformation at yield and at ultimate (Limit L_2). However, in prestressed concrete cases, since cracking commences quite late in comparison with that in reinforced concrete, a closer approximation to the actual curve is a trilinear idealisation which takes account of the uncracked phase (Limit L_0) prior to yeild.

The proposed[1,2] method assumes a moment-curvature idealisation in the elastic phase and a moment-rotation idealisation in the plastic phase and takes account of both elastic as well as plastic deformations. The compatibility requirement at ultimate is considered under the cumulative effects of elastic and plastic conditions. The attainable plastic rotation at a hinge is split up into parts occurring to its left and right.

For the elastic deformations, two idealisations (a bilinear and a trilinear) are suggested, any one of which may be adopted. The bilinear moment curvature curve assumes a constant cracked EI (intermediate between the uncracked EI and the constant cracked EI of Baker's) equal to $\left[\dfrac{M_1 + M_0}{\phi_1 + \phi_0}\right]$ (slope OL_1 in Fig. 27.1), and neglects the initial rotation due to prestress. The trilinear idealisation, a more sophisticated approach, assumes uncracked EI for moments up to M_0 throughout and then a constant cracked EI (much smaller than that in the bilinear case) given by $\left[\dfrac{M_1 - M_0}{\phi_1 - \phi_0 + \phi_{ps}}\right]$ (slope L_0L_1 in Fig. 27.3) for moments between M_0 and M_1. The effect of initial rotation due to prestress is taken into account as well.

Based on these assumptions, the curvature distribution at the limit L_1 is represented by the area aa_1dbgc_1ca in Fig. 27.2. The moment-cuvature relationship is then represented by $aL_0L_1L_2$ in Fig. 27.3.

The plastic rotation in each of the proposed idealisations is estimated from the empirical formula recommended by Baker and Amarkone[3] viz.,

$$\theta_p = 0.4(e_{cu} - e_{ce})z/d$$

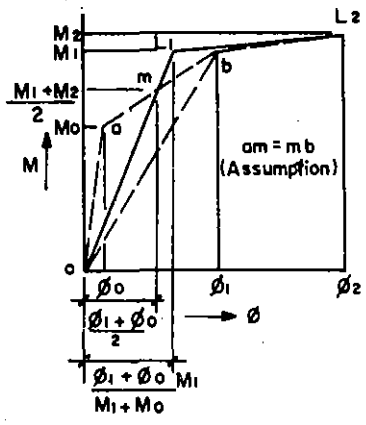


Fig. 27.1

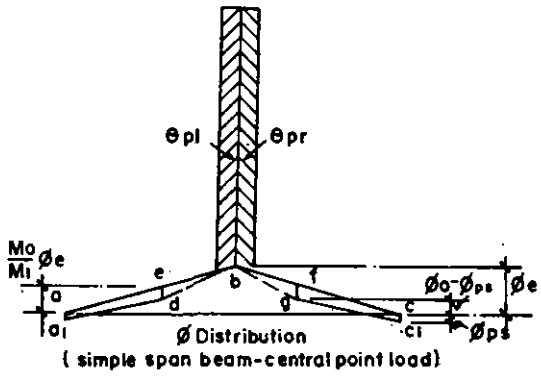


Fig. 27.2

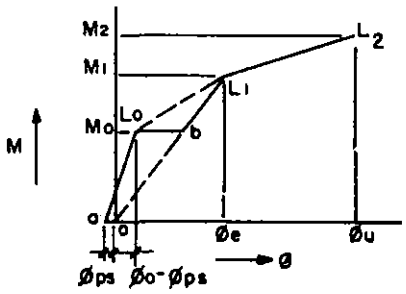


Fig. 27.3

The validity of this assumption was investigated[2] and found to involve a theoretical error of between 2–7%. The above formula is derived from its more general form

$$\theta_p = 0.8(e_{cu} - e_{ce})K_1K_3z/d$$

where K_1 is assumed to vary from 0.7 to 0.9 depending on type of steel and K_3 from 0.6 to 0.9 depending on grade of concrete. If the product of K_1 and K_3 is taken as 0.5

on an average, which is no less arbitrary than the choice from the above-mentioned ranges of K_1 and K_3, there is no justification for making a modification of 2–7% in the plastic rotation formula to suit the proposed idealisations.

Summarising, to estimate the ultimate moment distribution (and hence the ultimate load) in a given continuous beam, first the necessary calculations at the limits L_0, L_1 and L_2 may be done for each critical section (see sec. 27.6) and the appropriate EI values for each hinge region calculated. A suitable first-trial ultimate moment distribution may then be assumed, the cracked and uncracked parts marked off and the m diagrams drawn. The assumed ultimate moment diagram is correct provided the n compatibility equations are satisfied, otherwise a new trial is necessary. (The prestress moment diagram is considered in the case of a trilinear idealisation, as explained.)

In order to explain the compatibility calcutions based on this method, Examples 1 and 2 are given below. The former is based on the proposed bilinear idealisation and latter on the proposed trilinear idealisation. The same procedure applies to reinforced concrete beams also.

EXAMPLE 1 The 2-span beam is shown in Fig. 27.4.

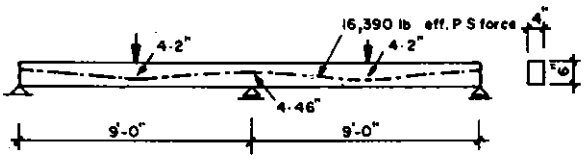


Fig. 27.4

From calculations at limits L_0, L_1 and L_2 for the 'mid-span' and 'support' sections the results are shown in Table 27.1.

At the collapse of the beam, assume that at the central support section 100% of its M_2 (i.e., 1.00 × 143,700 = 143,700 lb in.) is attained, while at the mid-span sections only 93% of their M_2 (i.e., 0.93 × 132,000 = 122,800 lb in.) is attained. (This in fact is the final of three trials which almost completely satisfies the involved compatibility equation, as will be seen ahead.)

Because of symmetry, only one symmetrical half need be considered Fig. 27.5(a) shows the assumed ultimate moment distribution, in which by marking off the ordinates pq and $p'q'$ = (M_1 values of the respective hinges) the M elastic diagram is curved out. Figure 27.5(b) represents the m_1 diagram in the span.

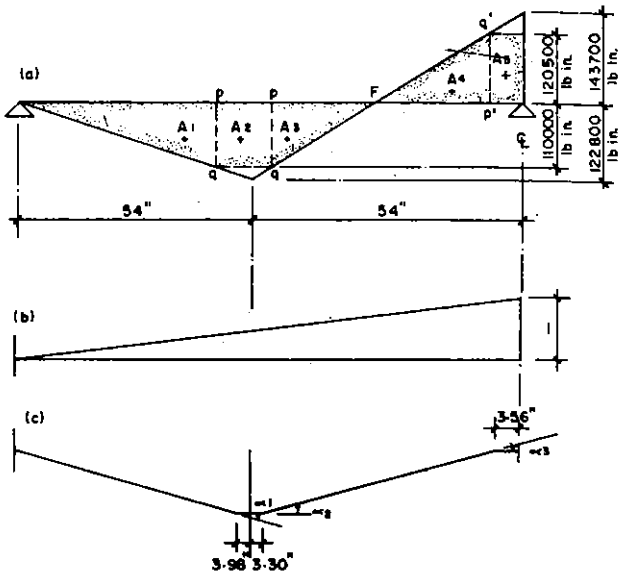


Fig. 27.5

The position of the point of contraflexure F is worked out from geometry. Similarly the areas A_1 to A_5, positions of their centroids, and the ordinates of the m_i diagram are worked out from geometry.

Attainable θ_{pl} and θ_{pr} at the hinges and their positions are worked out in Table 27.2.

Knowing the appropriate EI value in the span and support hinge regions (126 × 10^6 and 137 × 10^6 lb in.^{2}, respectively), values are substituted algebraically in the compatibility condition:

$$\int \left(\frac{M}{EI}\right)_{\text{elastic}} \cdot m_i \, dx + \Sigma \alpha m_i = 0$$

or

$$\Sigma \frac{A m_i}{EI} + \Sigma \alpha m_i = 0$$

The left-hand side equals 0.0014 or 1.4/1000, which is negligible, so that lhs. equals rhs. Hence compatibility may be assumed as achieved under the assumed ultimate

Table 27.1 *(lb and in. units)*

Section	M_0	M_1	M_2	e_{ce}	e_{cu}	ϕ_0 ($\times 10^{-3}$)	ϕ_1 ($\times 10^{-3}$)	$\frac{M_1 + M_0}{\phi_1 + \phi_0}$ ($\times 10^6$)	Max. θ_{pl} ($\times 10^{-3}$) radian)	Max. θ_{pr} ($\times 10^{-3}$) radian)
Mid-span	50,600	110,000	132,000	.002	.0038	0.120	1.160	126.0	0.1695 z_l	0.1697 z_r
Support	54,200	120,500	143,700	.002	.0039	0.130	1.150	137.0	0.1705 z_l	0.1705 z_r

Table 27.2 *Calculation of l_{pl}, l_{pr}, θ_{pl} and θ_{pr}*

	Midspan hinge	Central support hinge
z_l	54″	28.7″
z_r	25.3″	—
$l_{pl} = 0.5d\left(\frac{z_l}{d}\right)^{1/4}$	3.98″	3.56″
$l_{pr} = 0.5d\left(\frac{z_r}{d}\right)^{1/4}$	3.30″	—
Attainable θ_{pl}	$0.1695 \times 54 \times 10^{-3} \times \frac{122{,}800 - 110{,}000}{132{,}000 - 110{,}000}$ = 0.0053 rad. = α_1 in Fig. 27.5(c)	$0.1705 \times 28.7 \times 10^{-3} \times \frac{143{,}700 - 120{,}500}{143{,}700 - 120{,}500}$ = 0.0049 rad. = α_3 in Fig. 27.5(c)
Attainable θ_{pr}	$0.1695 \times 25.3 \times 10^{-3} \times \frac{122{,}800 - 110{,}000}{132{,}000 - 110{,}000}$ = 0.0024 rad. = α_2 in Fig. 27.5(c)	— —

moment distribution diagram, latter representing the actually mobilisable, i.e., 'attainable' moments at 'ultimate' (100% at the central support and only 93% of the absolute full value at the midspan sections).

Ultimate load W_u, applied at each load point, may now be calculated from the statical condition:

$$\left[\frac{W_u l}{4} + \frac{wl^2}{8}\right] - \left(\begin{array}{c}\text{fixed-end moment at}\\ \text{mid-span from ultimate BMD}\end{array}\right) = \left(\begin{array}{c}\text{actual attained ultimate}\\ \text{moment at mid-span}\end{array}\right)$$

so that, $W_u = 7080$ lb

EXAMPLE 2 The 3-span beam shown in Fig. 27.6.

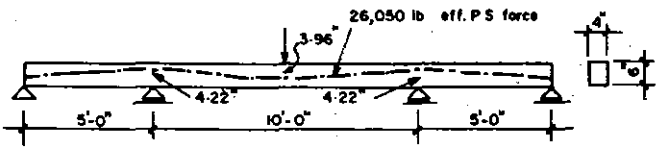


Fig. 27.6

From calculations at limits L_2, L_1 and L_0 for the loaded section (called *midspan* section hereafter) and the (intermediate) support sections, values are as in Table 27.3.

At the collapse of the beam, assume that at the midspan section 100% of its M_2 (i.e., 1.00 × 129,000 = 129,000 lb in.) is attained, while at each of the support sections only 92.4% of their M_2 (i.e., 0.924 × 161,840 = 149240 lb in.) is attained. (This again is the final of the various trials.)

Because of symmetry, only one m_i diagram need be worked with. Figure 27.7(b) shows the assumed ultimate moment distribution, in which, by marking off the respective M_1 ordinates in various hinge regions, the M elastic diagram is carved out. By further marking off the respective M_0 ordinates, the uncracked and cracked parts are marked. Figure 27.7(d) represents one m_i diagram. Figure 27.7(a) represents the prestress moment diagram.

The position of the points of contraflexure are worked out by geometry. Similarly, the areas A_1 to A_{10}, positions of their centroids, and the ordinates of the m_i diagram under the latter, are found by geometry.

Attainable θ_{pl} and θ_{pr} at the hinges are worked out in Table 27.4.

Knowing the appropriate EI values in each hinge region, values are substituted in the compatibility condition:

$$\Sigma\frac{Am_i}{EI} + \Sigma\alpha m_i = 0$$

Table 27.3 *(lb and in. units)*

Section	M_0	M_1	M_2	e_{ce}	e_{cu}	ϕ_0 ($\times 10^{-3}$)	ϕ_{ps} ($\times 10^{-3}$)	ϕ_1 ($\times 10^{-3}$)	EI_{un} ($\times 10^6$)	$\frac{M_1 - M_0}{\phi_1 - \phi_0 + \phi_{ps}}$ ($\times 10^6$)	Max. θ_{pl} ($\times 10^{-3}$ radian)	Max. θ_{pr} ($\times 10^{-3}$ radian)
Mid-span	64,500	117,900	129,000	.002	.0058	.138	.020	1.290	467	45.6	.452 z_l	.452 z_r
Supports	80,920	149,240	161,840	.0019	.0064	.173	.068	1.120	467	67.7	.432 z_e	.432 z_r

Table 27.4 *Calculation of θ_{pl} and θ_{pr}*

	Midspan hinge	Support hinge
z_l	27.8″	60″
z_r	27.8″	32.2″
Attainable θ_{pl}	$0.452 \times 27.8 \times 10^{-3} \times \dfrac{129{,}000 - 117{,}900}{129{,}000 - 117{,}900}$	$0.432 \times 60 \times 10^{-3} \times \dfrac{149{,}240 - 149{,}240}{161{,}840 - 149{,}240}$
	= .0126 rad	= 0
	= α_3 in Fig. 27.7(c)	= α_1 in Fig. 27.7(c)
Attainable θ_{pr}	$0.452 \times 27.8 \times 10^{-3} \times \dfrac{129{,}000 - 117{,}900}{129{,}000 - 117{,}900}$	$0.432 \times 32.2 \times 10^{-3} \times \dfrac{149{,}240 - 149{,}240}{161{,}840 - 149{,}240}$
	= .0126 rad.	= 0
	= α_4 in Fig. 27.7(c)	= α_2 in Fig. 27.7(c)
	($\alpha_3 = \alpha_4$ by symmetry)	($\alpha_2 = \alpha_5$ by symmetry)

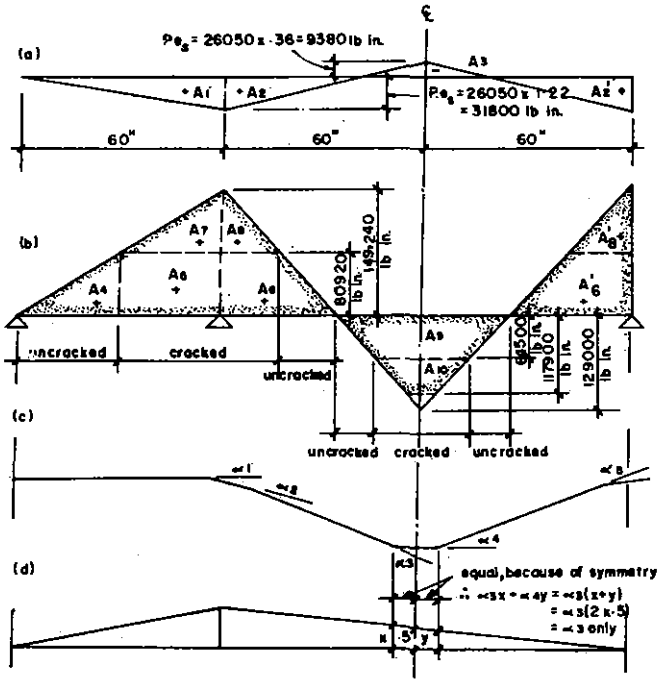


Fig. 27.7

and its lhs, algebraically, works out to :

$$\left\{\frac{A_1 m_i}{EI} + \left(\frac{A_2 m_i}{EI} + \frac{A_2' m_i}{EI}\right) + \frac{A_3 m_i}{EI} + \frac{A_4 m_i}{EI} + \frac{A_5 m_i}{EI} + \left(\frac{A_6 m_i}{EI} + \frac{A_6' m_i}{EI}\right) + \frac{A_7 m_i}{EI} + \left(\frac{A_8 m_i}{EI} + \frac{A_8' m_i}{EI}\right) + \frac{A_9 m_i}{EI} + \frac{A_{10} m_i}{EI} + (\alpha_3 x + \alpha_4 y)\right\} = 0.0003$$

which is negligible, so that lhs = rhs.

Hence compatibility may be assumed as achieved under the assumed ultimate moment distribution, latter indicating that only 92.4% of the absolute ultimate moment of resistance of the support sections is actually mobilisable at collapse of the structure. Ultimate load W_u, applied at the loaded section, may now be calculated, = 9150 lb.

Degree of accuracy A large number of 2 and 3 span prestressed concrete beams have been tested and also analysed.[2]

The following summary shows the degree of accuracy with which the proposed method predicted the ultimate load carrying capacity of these beams, the degree of accuracy being expressed as,

$$\frac{\text{Theoretically calculated ultimate load}}{\text{Experimentally measured ultimate load}}\% \pm \text{Standard deviation } \%$$

Idealisation	2-span beams	3-span beams
Bilinear	89.6% ± 7.9%	97.4% ± 5.7%
Trilinear	92.6% ± 5.9%	99.4% ± 7.6%

These results indicate that the degree of error involved, in general, is smaller in 3-span beams than in 2-span beams. This implies that more redistribution occurs in 2-span beams than can be predicted.

27.4 EFFECT OF TENDON TRANSFORMATION ON ULTIMATE STRENGTH

Contrary to the general belief, held by some, it has been shown[2] that linear transformation of cable profiles in continuous prestressed concrete beams can, in some cases, lead to reduction in the ultimate load carrying capacity of the beam. This is possible when the transformation results in bringing the tendon close to the compression edge at the first forming hinge section, thereby reducing the effective depth. As the steel area remains the same, the proportion of 'steel-area' to 'effective concrete area' increases, making the section over reinforced, which in turn reduces the rotation capacity of the hinge. This can lead to a premature collapse of the hinge, resulting in an early failure of the beam. See Ch. 28 for details.

However, it has been shown[2] that the ultimate strength may not be seriously affected so long as it is ensured that linear transformation does not reduce the effective depth at any critical section to less than a third of the overall depth of section.

Concluding Remarks

It is essential to investigate the truly attainable (mobilisable) ultimate bending moment distribution in an elastically designed structure in order to know its true ultimate load carrying capacity and the truly mobilisable moments of resistance (as against their 'absolute' values) at various critical sections.

The suggested method, based on compatibility of deformations, shows significant accuracy with which the suggested idealisations predict the actual measured ultimate load of 2- and 3-span continuous prestressed beams.

Though it is preferable to adopt a trilinear idealisation in all prestressed concrete cases, the adoption of a bilinear one does not lead to any significant degree of error.

NOTE A similar procedure can be applied to reinforced concrete.

27.5 IMPORTANCE OF WORKMAN-LIKE 'DETAILING'

As explained, at collapse-load condition, it is quite possible due to inability of certain hinges to continue to rotate and due to obligation to compatibility of deformations, that full redistribution of moments may not take place, i.e., all necessary hinges may not get the chance to fully mature. Hence, all the critical sections may not attain moment values equal to their full ultimate moment capacities. Either this should be investigated* and it should be ensured that these actually attainable moments at collapse stage (not the full ultimate moment capacities) are not less than the factored applied moments at each critical section, or at least very thorough attention should be given to the detailing of section dimensions and amount and flow of reinforcement and prestressing force, guarding against any abrupt changes, cut-offs, cut-outs, etc., that can severely impair the ultimate rotational capacity of a hinging region which can lead to incomplete redistribution and early ultimate failure.

* This can be done by various methods given for instance by Guyon, Macchi, Cooke, Raina (see method outlined in section 27.4), etc., but many designers may not be up-to-date on this. Hence, the need for a resolute and intensely workman-like detailing cannot be over emphasised.

27.6 ESTIMATION OF VARIOUS LIMITING MOMENTS AND STRESS-STRAIN RELATIONSHIP

1. The absolute value of the ultimate moment of resistance (Limit L_2) of a prestressed concrete section.
2. The moment of resistance of a prestressed concrete section at elastic limit (Limit L_1)
3. The moment of resistance of a prestressed concrete section at cracking limit (Limit L_0)
4. The stress-strain blocks for concrete in flexural compression.

1. Calculation at limit L_2 for finding M_2, e_{cu} and $n_u d$ at the 'collapse limit state' (see also Ch. 23 for load-factored type limit state allowed in the regular codes of practice).

(i) Assume a suitable first-trial $n_u d$.

(ii) Estimate e_{cu} and f_c'' from the empirical formulae (a) or (b), and (c) or (d), respectively

(a) $$e_{cu} = .0015\left(1.45 + 1.5\rho'' + \frac{0.7 - .1\rho''}{n_u} - f_e' \times 10^{-4}\right) \text{ but } \not> .01$$

(b) $$e_{cu} = .0042\left(\frac{d}{D} + 0.4p''\right)\frac{A_c \times 10^{-2}}{A_{ps} + 0.05A_{ms}}$$ but $\not> 0.01$ and $\not< 0.0035$

(c) $f_c'' = f_c'(.8 + .1/n_u)$, but $\not> f_c'$ where f_c' may be taken as .85 f_{cu}

(d) $f_c'' = 0.85 f_{cu}$

Formulae (a) and (c) are after Baker and Amarkone[3] while formulae (b) and (d) are after author[2] of this book. It may be seen that formulae (a) and (c) involve n_u and are, therefore, dependent on the trial and error calculations, while the proposed formulae have the advantage of involving only known parameters. It has been shown[2] that the proposed formulae are just as accurate,

$$\text{average } \frac{(b)}{(a)} = 1.04 \pm \text{ standard deviation of } 0.103$$

$$\text{average } \frac{(d)}{(c)} = 1.002 \pm \text{ standard deviation } 0.012.$$

(iii) Draw the strain-block in relation to the effective section, work out the location of compressive strain of 0.002, and draw the flexural compressive stress-block [see item 4 ahead].

(iv) Find out total tension T and total compression C in the effective section: if $T = C$, trial alright, otherwise, repeat assuming a fresh suitable value for $n_u d$ until $T = C$ finally.

(v) Calculate M_2, the (absolute) ultimate moment of resistance of the section, by taking moments of T and C forces about any fibre-level in the section.

2. *Calculation at limit* L_1 (for finding $M_1 \cdot e_{ce}$ and $n_e \cdot d$)

(a) (i) Assume a suitable first trail $n_e \cdot d$.

(ii) Assume total strain in the extreme tendon (or at c.g. of tendons, if bunched together) equal to its 0.1% proof strain value, so that $e_{cse} = 0.1\%$ proof strain value $-e_p - e_{cp}$), then if e_{ce} works out to 0.002 or less, then draw the corresponding stress-block and find out T and C. If $T = C$, then calculate M_1 by taking their moments about any fibre level in the section. If $T \neq C$, then may be by merely trying a fresh value of $n_e d$ (with e_{ce} still working out to less than or equal to 0.002), T may equal C, in which case the section is 'under reinforced' unless $e_{ce} = 0.002$ in which case reinforced' unless $e_{ce} = 0.002$ in which case the section is 'balanced' and M_1 can be found.

If instead it is impossible to get $T = C$ in the above trial without e_{ce} exceeding 0.002, it means that the section is 'over reinforced', in which case proceed as follows:

(b) (i) Assume a revised suitable first trial $n_e \cdot d$

(ii) Assume $e_{ce} = 0.002$, work out e_{cse} and check that the total strain $(e_p + e_{cp} + e_{cse})$ in the extreme tendon (or at cg of tendons, if bunched together) is less than 0.1% proof strain value. Draw the corresponding stress-block and work out T and C. If $T \neq C$, repeat with a fresh value for $n_e d$ until $T = C$ with $(e_p + e_{cp} + e_{cse})$ still < 0.1% proof strain value. Then calculate M_1 as explained above.

3. *Calculation at limit* L_0 *(for finding* M_0*)*

Where parasitic moments are absent, M_0 may be calculated from the limiting condition that cracking will commence when the resultant tensile stress equals the modulus of rupture (f_{m0d}):

$$\frac{M_0 + M_{DI}}{Z} - \left(\frac{P}{A} + \frac{P \cdot e_s}{Z}\right) = f_{m0d}$$

Where parasitic moments are present, by virtue of the fact that they start diminishing right from the instant of microcracking so that their amount actually present at the time of visible cracking is uncertain, the term Pe_s in the above formula (which now represents primary as well as secondary moments) becomes indeterminate. Hence, in the author's opinion, in such cases, M_0 may not be evaluated from the above formula. On the basis of test results, it is suggested in such cases to assume $M_0 = 0.55\ M_2$ if untensioned steel is not considered in calculating M_2 and $M_0 = 0.50 M_2$ if untensioned steel is considered.

4. *Stress strain blocks for concrete in flexural compression*

These are assumed as shown in Fig. 27.8. It is convenient to divide the effective concrete section into rectangles, so that when taking moments of their compressive forces, the lever arms are the same as of their stress-blocks.

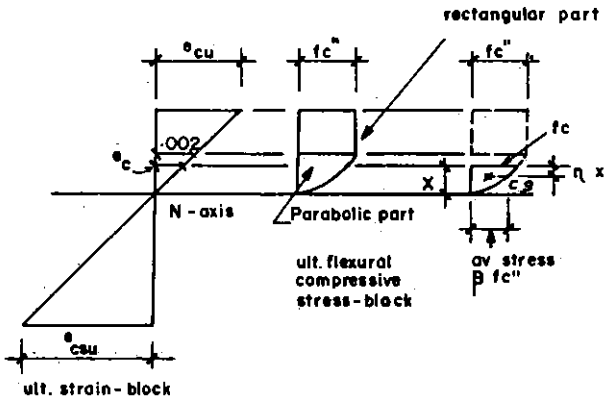


Fig. 27.8

For the rectangular part of the stress-block, or part thereof, 'average stress' equals $f''c$ and 'location of its centroid' lies at the middle of its depth.

For the parabolic part of the stress-block, or part thereof (i.e., $e_c \leq 0.002$), the 'average stress equals $\beta f_c''$ and 'location of its centroid below its top equals $\eta \cdot x$. The values of the coefficients β and η depend on the value of concrete strain e_c at the level of the top of the considered part of the parabolic stress-block. It may be shown[2] that,

$$\left.\begin{aligned} \beta &= \frac{e_c}{0.002}\left(1 - \frac{e_c}{0.006}\right) \\ \text{and } \eta &= \frac{4 - (e_c/.002)}{12[1 - (e_c/0.006)]} \end{aligned}\right\} \text{ for } 0 < e_c \leq 0.002$$

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1. Raina, V.K., "Estimation of Ultimate Load Carrying Capacity of Continuous Prestressed Concrete Beams in Flexure" *Consulting Engineer*, London, July, 1967.
2. Raina, V.K., "Ultimate Moment Distribution in Continuous Prestressed Concrete Beams", PhD Thesis, University of London, 1966.
3. Baker, A.L.L., and A.M.N. Amarkone, "Inelastic Hyperstatical Frames—Analysis and Application of the International Correlated Tests". CEB 9th Full Meeting, Ankara, September, 1964.

CHAPTER 28

Effect of Tendon Transformation on Ultimate Strength of Continuous Prestressed Concrete Beams

It is common knowledge in prestressed concrete that even if the position of the tendon in a continuous beam is shifted at any or all the intermediate supports (by any amount, vertically, up or down), he resultant line of thrust remains unchanged (i.e., the total prestress moment, etc. remains unaltered) so long as the position of the cable at the end support sections is unchanged and the new profile of the tendon in each span is obtained by pure 'linear transformation' of its parent profile. This is established even mathematically (see Linear Transformation in Ch. 26 of this book), but the fact remains that this principle holds so long as the beam is behaving purely elastically. After cracking sets in, and if the load is still increased, plastic rotations are created in the beam in its hinging regions, moments get redistributed, flexural rigidity (EI) of cracked sections reduces, and as a consequence, the prestress 'parasitic' moments begin to reduce until they disappear completely when all the n hinges are formed and fully developed (n being the number of statical indeterminacies of the structure). Should this full redistribution occur, then obviously the load taking capacity of the structure would be different in the two profile cases since the prestress primary moments (which are then the prestress total moments, parasitic ones having disappeared) are different for the two profiles. At least theoretically this could be the case, provided full redistribution takes place. (The latter is generally not true and hence some parasitic moments still exist at the instant of collapse, the hence the above stated principle of linear transformation of tendon profiles may continue to hold, at least nearly.)

An attempt has been made here to show theoretically that the ultimate carrying capacity of a continuous prestressed concrete beam does not remain unaffected with tendon transformation. Basing the analysis on the assumptions that, at ultimate of the beam, each critical section attains its full ultimate moment of resistance and that the sections are not over reinforced, a formula has been derived for the percentage change in the ultimate load carrying capacity caused by linear transformation of the parent cable. The derivation is based purely on an equilibrium concept. However, the above-mentioned assumptions may not hold good in some cases, which limits the applicability of this equilibrium method formula.

28.1 ANALYTICAL DERIVATION OF THE PROPOSED FORMULA

First, an expression is derived for the ultimate moment of resistance of a prestressed concrete section. The term representing force in the tendon is replaced by a mathematical equation expressing *force* in terms of *strain* in the tendon. Later, the variable part e_{csu} (in the strain $e_p + e_{cp} + e_{csu}$), called e in short, is expressed by a transcendental equation linking it with the location of the tendon in the section in order to make the cable force sensitive to linear transformation. Assuming the each critical section attains its full ultimate moment of resistance, an expression is derived for the maximum free BM that a span of a continuous beam can carry with parent cable as well as with transformed cable in terms of ultimate moments of resistance of the sections concerned. Formulae derived earlier for the latter are then substituted and the percentage change in the 'maximum free BMs' in the two cases worked out. Hence a percentage change in the ultimate loads that can be carried in the two cases, is obtained.

Though the analysis is carried out assuming a rectangular section of the beam, it is still applicable in the case of other sections so long as the effective section at ultimate is rectangular.

Derivation

Consider an intermediate span of any prestressed concrete continuous beam, as shown in Fig. 28.1. The cable profile in the chainline represents the parent cable, while that in the full-line represents a linearly transformed cable—transformations being $+\delta_1$ at (left hand support) section 1 and δ_3 at (right hand support) section 3. Consequently, the effective depths at sections 1 and 3 increase, while that at an intermediate section 2, distant βl from 1, decreases by δ_2 such that,

$$\delta_2 = (1 - \beta)\delta_1 + \beta\delta_3.$$

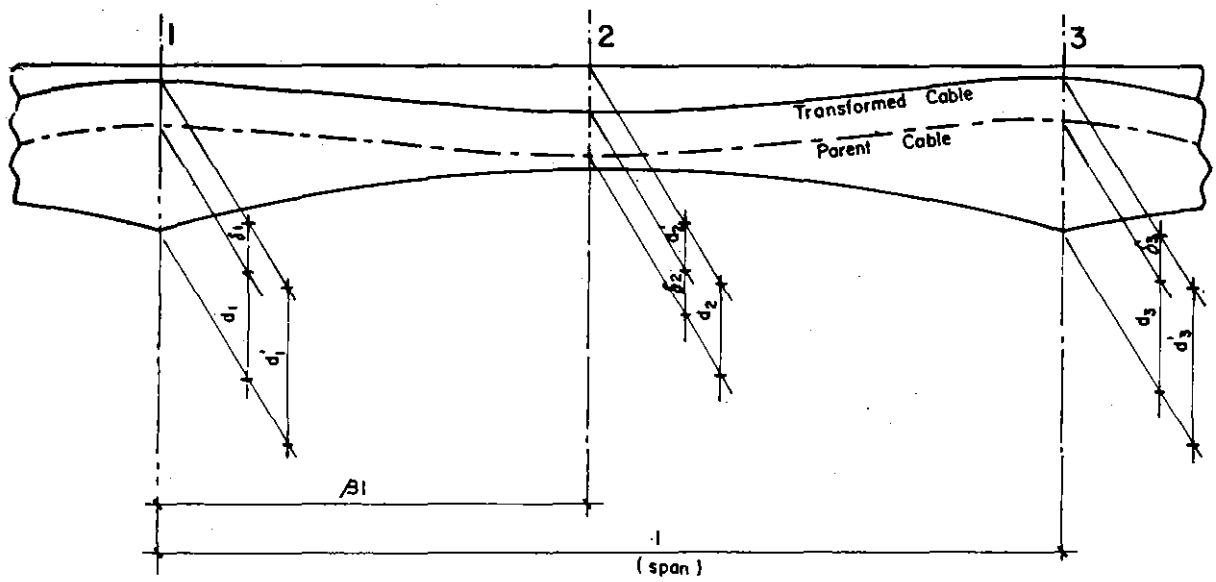


Fig. 28.1

Let

B = width of concrete section

A_s = area of cable steel in the section

A = area of untensioned steel in the section

D = eff. depth from compression edge to cg of untensioned steel

α = ratio of average flexural compressive stress in concrete at ultimate of f_c'', where $f_c'' = 0.85 f_{cu}$ (f_{cu} = 28 days cube strength of concrete)

γ = ratio of depth up to centre of compression at ultimate of depth up to neutral axis at ultimate. (NB : α and γ refer to 'total' flexural compression stress block at ultimate)

t_r = stress in cable steel in the section at ultimate of the section

t = stress in untensioned steel in the section at ultimate of the section

βl = distance of 'maximum span-moment section' from (left hand support) section 1

l = span

d_1, d_2, d_3 = eff. depths to tendon under parent profile case, at sections 1, 2 and 3

d_1', d_2', d_3' = eff. depths to tendon under transformed profile case, at sections 1, 2 and 3

M_1, M_2, M_3 = ultimate moments of resistance under parent profile case of sections 1, 2 and 3

M_1', M_2', M_3' = ultimate moments of resistance under transformed profile case of sections 1, 2 and 3

$e_{cu1}, e_{cu2}, e_{cu3}$ = e_{cu} values under parent profile case at sections 1, 2 and 3

$e_{cu1}', e_{cu2}', e_{cu3}'$ = e_{cu} values under transformed profile case at sections 1, 2 and 3

e_1, e_2, e_3, = e_{csu} (or e) values under parent profile case at sections 1, 2 and 3

e_1', e_2', e_3', = e_{csu} (or e) values under transformed profile case at sections 1, 2 and 3

d_0 = overall depth (general symbol)

NOTE See Ch. 27 for explanation of notation e_{cu}, e_p, e_{cp}, e_{csu}, f_c'', d, p'', A_c, A_{ps}, A_{ms}, $n_u d$, C and T.

$a = d_2/d_1$, $c = d_3/d_1$, $r = \delta_1/d_1$, $q = \delta_3/\delta_1$, $p = \delta_2/\delta_1$

Evidently, $\delta_1 = d_1' - d_1$, $\delta_2 = d_2 - d_2'$, $\delta_3 = d_3' - d_3$,

$$\left.\begin{aligned} &\text{and} \quad d_1' = (1+r)d_1 \\ &\qquad\quad d_2' = (a - pr)d_1 \\ &\text{and} \quad d_3' = (c + qr)d_1 \end{aligned}\right\} \qquad (A)$$

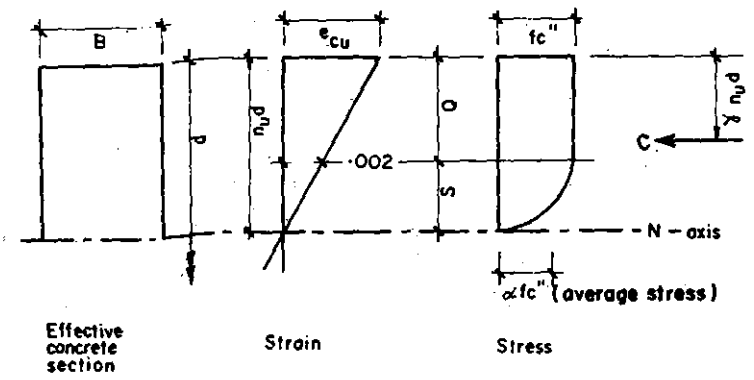
Values of e_{cu} and f_c'' may be found from the following proposed empirical formulae,

$$e_{cu} = 0.0042(d/d_0 + 0.4p'')\frac{A_c \times 10^{-2}}{A_{ps} + 0.05A_{m.s.}}$$

but $\not< 0.0035$ and $\not> 0.001$

$$f_c'' = 0.85 f_{cu}$$

These formulae have an advantage over those recommended by Baker and Amarkone, in that they are independent of trial and error calculations. Validity of the former in comparison with the latter has already been established by the author and some details and given in Ch. 27.



$$S = \frac{\cdot 002}{e_{cu}} \cdot n_u d$$

$$Q = n_u d - S = n_u d \left(\frac{e_{cu} - \cdot 002}{e_{cu}}\right)$$

$$fc'' = \cdot 85\ fcu$$

$$e_{cu} = \cdot 0042\left(\frac{d}{d_o} + \cdot 4p''\right)\frac{Ac \times 10^{-2}}{Aps + \cdot 05\,Ams}$$

but $\neq 0 \cdot 0035, \neq 0 \cdot 01$

$$\alpha = \left(1 - \frac{\cdot 0007}{e_{cu}}\right)$$

$$\gamma = \frac{\frac{(e_{cu} - 0 \cdot 002)^2}{2} + (e_{cu} - 0 \cdot 0013) \times 0 \cdot 0013}{e_{cu}(e_{cu} - 0 \cdot 0007)}$$

Fig. 28.2

The stress-strain block for concrete under ultimate flexural compression is assumed to be as shown in Fig. 28.2. Value of α may be obtained by equating the areas of the 'average' (equivalent) stress block and the assumed stress block:

$$n_u d \cdot \alpha f_c'' = (Qf_c'' + 2/3Sf_c'')$$

substituting for Q and S (Fig. 28.2) and simplifying, we get:

$$\alpha = \left(1 - \frac{0.0007}{e_{cu}}\right)$$

Value of γ may be obtained by equating the moments of areas of the 'average' (equivalent) stress block and the assumed stress block, say, about the compression edge:

$$(Qf_c'' + 2/3Sf_c'')\gamma n_u d = (Qf_c'')Q/2 + (2/3Sf_c'')(Q + 3/8S)$$

substituting for Q and S and simplifying, we get:

$$\gamma = \frac{\frac{(e_{cu} - 0.002)^2}{2} + (e_{cu} - 0.0013)0.0013}{e_{cu}(e_{cu} - 0.007)}$$

From the internal equilibrium condition $C = T$, at ultimate of a section,

$$(Bn_u d)\alpha f_c'' = A_s t_r + \Sigma At$$

$$\therefore \quad n_u = \frac{A_s t_r + \Sigma At}{B\alpha f_c'' d} \qquad (28.1)$$

Taking moment of internal forces, say about centre of compression, we get an expression for ultimate moment of resistance of the section, viz.,

$$M_u = A_s t_r(d - \gamma n_u d) + \Sigma At(D - \gamma n_u d) \qquad (28.2$$

At the ultimate of the section, the untensioned stee invariably yields and therefore carries its yield force. Th shift in the position of ultimate centre of compression fron that in the parent cable case to that in the transformed cabl case may be assumed to be small in comparison to the lever arm of the untensioned steel whose contribution to M_u and T is small enough. Therefore, the contribution of the untensioned steel to the 'change' in M_u under the two cases may be neglected. Accordingly, for the present purpose, the Eqs. (28.2) and (28.1) may be simply written as:

$$M_u = A_s t_r \cdot (d - \gamma n_u d) \qquad (28.3)$$

and

$$n_u = \frac{A_s t_r}{B\alpha f_c'' d} \qquad (28.4)$$

substituting Eq. (28.4) in (28.3) we get:

$$M_u = (A_s t_r)d - (A_s t_r)^2 \frac{\gamma}{B\alpha f_c''} \qquad (28.5)$$

$(A_s t_r)$, the tensile force in the tendons, depends on total strain $(e_p + e_{cp} + e_{csu})$ in them, of which e_{csu} depends on the location of the tendons in the section.

The load-strain curve of a high tensile steel tendon resembles a half range sine wave, and may be expressed in the form:

$$y = K\psi(x) \qquad (28.6)$$

making use of Fourier series. Since usually the normal prestressed concrete continuous beams are not over-reinforced, the total strain in cable steel at the ultimate of a hinge section may be assumed to be about 0.0095.

For one 0.276″ dia. HTS wire (that was used in the continuous beams tested by the author), a reasonably accurate relationship between the load y and the strain x, for strain values of about 0.0095, may be expressed as follows:

$$y = 14400 \sin \frac{\pi x}{0.03} \text{ (lb units)} \tag{28.7}$$

where 0.03 represents its ultimate strain (the half-range). Hence in general the load-strain relation for tendons may be written as:

$$y = b \sin(K_3 x) \tag{28.8}$$

where y = load of force $(A_s f_r)$
x = total strain in the tendon
b and K_3 = coefficients, depending on type of HTS, former also depending on the number of tendons.
y and b having same units.
For the particular 0.276″ dia. HTS wires, therefore:
b = $(14,400N)$ lbs, N being number of wires
K_3 = $\pi/0.03$

Substituting into Eq. (28.5), we get:

$$M_u = yd - y^2 \frac{\gamma}{B\alpha f_c''}$$
$$= bd \sin(K_3 x) - \frac{b^2\gamma}{B\alpha f_c''} \sin^2(K_3 x) \tag{28.9}$$

But $x = e_p + e_{cp} + e_{csu}$ (total strain in tendon), calling $(e_p + e_{cp})$ as K_1, and e_{csu}, as e, we have:

$$x = K_1 + e$$

$\therefore M_u = bd \sin(K_3K_1 + K_3 e) - \frac{b^2\gamma}{B\alpha f_c''} \sin^2(K_3K_1 + K_3 e)$ calling K_3K_1 as K_2 and $\frac{b^2\gamma}{B\alpha f_c''}$ as K_4, we have:

$$M_u = bd \sin(K_2 + K_3 e) - K_4 \sin^2(K_2 + K_3 e) \tag{28.10}$$

Thus:

$$\left.\begin{aligned}
M_1 &= bd_1 \sin(K_2 + K_3 e_1) - K_{4_1} \sin^2(K_2 + K_3 e_1)\\
M_2 &= bd_2 \sin(K_2 + K_3 e_2) - K_{4_2} \sin^2(K_2 + K_3 e_2)\\
M_3 &= bd_3 \sin(K_2 + K_3 e_3) - K_{4_3} \sin^2(K_2 + K_3 e_3)\\
M_1' &= bd_1' \sin(K_2 + K_3 e_1') - K_{4_1}' \sin^2(K_2 + K_3 e_1')\\
M_2' &= bd_2' \sin(K_2 + K_3 e_2') - K_{4_2}' \sin^2(K_2 + K_3 e_2')\\
M_3' &= bd_3' \sin(K_2 + K_3 e_3') - K_{4_3}' \sin^2(K_2 + K_3 e_3')
\end{aligned}\right\} \tag{28.10a}$$

From strain-compatibility across a section, based on linear strain distribution from compression-edge to tendon, we have,

$$e = e_{cu} \frac{d - n_u d}{n_u d}$$
$$= e_{cu} \left(\frac{1}{n_u} - 1\right)$$

Substituting Eq. (28.8) into (28.4) and then this into the above, we get:

$$e = e_{cu} \left(\frac{B\alpha f_c'' d}{b \sin(K_3 x)} - 1\right)$$

Calling $\frac{B\alpha f_c''}{b}$ as H and substituting for x as before, we get:

$$e = e_{cu} \left(\frac{Hd}{\sin(K_3K_1 + K_3 e)} - 1\right)$$

and $\because K_3K_1 = K_2$

$$\therefore e = e_{cu} \left(\frac{Hd}{\sin(K_2 + K_3 e)} - 1\right)$$

so that $\sin(K_2 + K_3 e) = \frac{Hde_{cu}}{(e_{cu} + e)}$ (28.11)

which is a *transcendental equation.*
Thus

$$\left.\begin{aligned}
\sin(K_2 + K_3 e_1) &= \frac{H_1 d_1 e_{cu1}}{(e_{cu1} + e_1)}\\
\sin(K_2 + K_3 e_2) &= \frac{H_2 d_2 e_{cu2}}{(e_{cu2} + e_2)}\\
\sin(K_2 + K_3 e_3) &= \frac{H_3 d_3 e_{cu3}}{(e_{cu3} + e_3)}\\
\sin(K_2 + K_3 e_1') &= \frac{H_1' d_1' e_{cu1}'}{(e_{cu1}' + e_1')}\\
\sin(K_2 + K_3 e_2') &= \frac{H_2' d_2' e_{cu2}'}{(e_{cu2}' + e_2')}\\
\sin(K_2 + K_3 e_3') &= \frac{H_3' d_3' e_{cu3}'}{(e_{cu3}' + e_3')}
\end{aligned}\right\} \tag{28.11a}$$

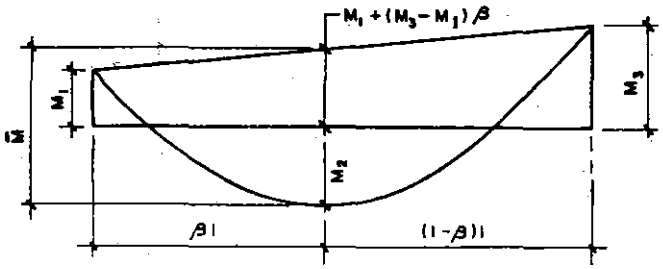
Assuming full redistribution of bending-moments at ultimate of the structure so that each critical section attains a BM equal to its M_u, and referring to Fig. 28.3, we have:
In parent cable case

$$\bar{M} = (1 - \beta)M_1 + M_2 + \beta M_3 \tag{28.12}$$

In transformed cable case

$$\bar{M}' = (1 - \beta)M_1' + M_2' + \beta M_3' \tag{28.13}$$

where $\bar{M}$ = maximum free BM due to total loading on span in parent cable case.



Moment diagram at failure
(parent cable case)

Fig. 28.3

$\bar{M}'$ = maximum free BM due to total loading on span in transformed cable case.

The 'change' in the maximum free BMs that the span is, therefore, able to carry, relative to parent cable case, is:

$$\frac{\bar{M}-\bar{M}'}{\bar{M}} = 1 - \frac{(1-\beta)M_1' + M_2' + \beta M_3'}{(1-\beta)M_1 + M_2 + \beta M_3}$$

$$= 1 - X \qquad (28.14)$$

where $$X = \frac{(1-\beta)M_1' + M_2' + \beta M_3'}{(1-\beta)M_1 + M_2 + \beta M_3} \qquad (28.15)$$

Substituting into Eq. (28.15) the Eq. (28.10a) and then into this new equation substituting Eq. (28.11a) and then into this still new equation substituting equation (A), and noting in general that

$K_4 H \left(= \dfrac{b^2\gamma}{B\alpha f_c''} \times \dfrac{B\alpha f_c''}{b}\right)$ is simply equal to $b\gamma$, we get after a very laborious simplification:

then $$\bar{M} = \bar{W}(1-\beta)\beta l + \frac{wl^2}{2}\beta(1-\beta) \qquad (28.17)$$

and $$\bar{M}' = \bar{W}'(1-\beta)\beta l + \frac{wl^2}{2}\beta(1-\beta) \qquad (28.18)$$

so that $$\frac{\bar{M}-\bar{M}'}{\bar{M}} = \frac{(1-\beta)\beta l(\bar{W}-\bar{W}')}{\bar{W}(1-\beta)\beta l + \frac{wl^2}{2}\beta(1-\beta)} \qquad (28.19)$$

Special case: Where the term $\dfrac{wl^2}{2}\beta(1-\beta)$ (own weight BM) is relatively negligible:

$$\frac{\bar{M}-\bar{M}'}{\bar{M}'} = \frac{\bar{W}-\bar{W}'}{\bar{W}} \qquad (28.20)$$

Connecting Eqs. (28.20) and (28.14), we get the percentage change in ultimate load carrying capacity of the span, below that of the parent-profile case viz. $\left(\dfrac{\bar{W}-\bar{W}'}{\bar{W}}\right)\cdot 100$, equal to $(1-X)\cdot 100$ where X is given by Eq. (28.16).

Procedure Thus the procedure for evaluating the percentage change in ultimate load carrying capacity, from this equilibrium method formula, may be summarised as follows:

Step 1 Find the load-strain $(y : x)$ relationship for a tendon in the from: $y = b\sin\dfrac{\pi x}{U}$, where b is as explained earlier, and U = ultimate tensile strain.

$$X = \frac{H_1'(1-\beta)\left[\dfrac{(1+r)^2(e_{cu1}')}{(e_{cu1}'+e_1')} - \dfrac{\gamma_1' e_{cu1}'^2(1+r)^2}{(e_{cu1}'+e_1')^2}\right] + H_2'\left[\dfrac{(a-pr)^2(e_{cu2}')}{(e_{cu2}'+e_2')} - \dfrac{\gamma_2' e_{cu2}'^2(a-pr)^2}{(e_{cu2}'+e_2')^2}\right] + H_3'\beta\left[\dfrac{(c+qr)^2 e_{cu3}'}{(e_{cu3}'+e_3')} - \dfrac{\gamma_3' e_{cu3}'^2(c+qr)^2}{(e_{cu3}'+e_3')^2}\right]}{H_1(1-\beta)\left[\dfrac{(e_{cu1})}{(e_{cu1}+e_1)} - \dfrac{\gamma_1 e_{cu1}^2}{(e_{cu1}+e_1)^2}\right] + H_2\left[\dfrac{a^2 e_{cu2}}{(e_{cu2}+e_2} - \dfrac{\gamma_2 e_{cu2}^2 a^2}{(e_{cu2}+e_2)^2}\right] + H_3\beta\left[\dfrac{c^2 e_{cu3}}{e_{cu3}+e_3} - \dfrac{\gamma_3 e_{cu3}^2 c^2}{(e_{cu3}+e_3)^2}\right]} \qquad (28.16)$$

NOTE In the above analysis b has been assumed constant at all sections.

However, should it vary for different sections, then its account can be taken in Eq. (28.10a) and a modified formula for X worked out.

Now if

w = equivalent uniformly distributed own weight of the beam per unit length

$\bar{W}$ = applied (equivalent point) load at section 2, corresponding to $\bar{M}$.

and $\bar{W}'$ = applied (equivalent point) load at section 2, corresponding to $\bar{M}'$

Step 2 Estimate e_{cu} and f_c'' at each critical section, in the parent cable case as well as in the transformed cable case, using the proposed empirical formulae:

$$e_{cu} = 0.0042(d/d_0 + 0.4p'')\frac{A_c \times 10^{-2}}{A_{ps} + 0.05A_{ms}},$$

but $\not< .0035$ and $\not> .01$

and $f_c'' = 0.85 f_{cu}$

Step 3 Estimate α, γ and H at each critical section, in the parent cable case as well as in the transformed cable case, using the following formulae:

$\alpha = (1 - 0.0007/e_{cu})$

$$\gamma = \frac{(e_{cu} - 0.002)^2/2 + (e_{cu} - 0.0013)0.0013}{e_{cu}(e_{cu} - 0.0007)}$$

and $H = B\alpha f_c''/b$($b = N$ times 'b for one tendon' found in Step 1 above, N being the number of tendons at the section).

Step 4 Plot 'Hd versus e' relationship from

$$Hd = (1 + e/e_{cu})\sin(K_2 + K_3 e) \quad \text{(Eq. 28.11)}$$

where $K_2 = \frac{\pi}{U}(e_p + e_{cp})$

[take $e_{cp} = 0.0002$, approximately]

$$K_3 = \frac{\pi}{U}$$

and e_{cu} = average of the e_{cu} values found in step 2 above (an approximation)

Step 5 (i) Referring to the left hand support section, the right hand support section and the maximum-bending-moment span section by suffixes 1, 3 and 2 in the parent cable case, respectively, and by suffixes $1'$, $3'$ and $2'$ in the transformed cable case, respectively, then H_1, H_2, H_3, H_1', H_2' and H_3' stand known from step 3. Read off e from the Hd versus e plot, corresponding to $Hd = H_1 d_1, H_2 d_2, \ldots$ and $H_3' d_3'$. Thus e_1, e_2, e_3, e_1', e_2' and e_3' are known.

(ii) Find the value of the terms $(1 + r)^2$, $(a - pr)^2$, $(c + qr)^2$, a^2 and c^2 from the relations:
$(1 + r) = d_1'/d_1$, $(a - pr) = d_2'/d_1$,
$(c + qr) = d_3'/d_1$
$a = d_2/d_1$, and $c = d_3/d_1$ respectively

Step 6 Evaluate X from Eq. (28.16).

Step 7 Then $(1 - X) \cdot 100$ represents the percentage change in "ultimate load carrying capacity below that of the parent cable case". [If $(1 - X) \cdot 100$ turns out to be positive, it means a decrease in carrying capacity compared with parent cable case.] This is so if self weight is negligible. Otherwise $\left(\frac{\bar{W} - \bar{W}'}{\bar{W}}\right)$ may be evaluated from Eq. (28.19) noting that in it $(1 - X)$ represents $\left(\frac{\bar{M} - \bar{M}'}{\bar{M}}\right)$ (Eq. 28.14).

It has to be noted that in case of an ordinary two-span beam, only two terms in the numerator and in the denominator in the above expression for X exist. This is so because, for instance if the span under consideration be the right hand one, then $M_3 = 0$ in Fig. 28.3, and so, therefore in Eqs. (28.12) and (28.13) too. Similarly if the span under consideration be the left hand one, then $M_1 = 0$ in Fig. 28.3, and so, therefore, in Eqs. (28.12) and (28.13) too.

CHAPTER 29

Simplified Method for the Analysis of Torsional Moment as an Effect of a Horizontally Curved Multispan Continuous Deck*

Synopsis

A simplified method of calculation for torsional effects on a bridge structure is presented.[1] Analysis and calculations are shown for torsional moments as an effect of loading on a horizontally curved, multispan, continuous bridge. Special attention is given to the analysis for torsional moment resulting from applied longitudinal bending moments (which include parasitic† prestressing moments but not primary prestress moments). A method of application is given in a step-by-step form with full interpretation of the problem, and a numerical example is solved in the same step-by-step form to illustrate the method.[1]

29.1 INTRODUCTION

Structures for highway interchanges of the type shown in Fig. 29.1 (see art plate) are usually curved in plan. The superelevation often varies, and in most cases the vertical alignment is curved. An important requirement for such structures is that the construction depth of the deck should be as shallow as possible. A number of roadways cross one another at points in close proximity and the vertical distances between them must include the construction depth for each structure. Deep deck systems result in increased excavation or fill and increase the overall size of the interchange.

Further requirements are that piers should be as few and as small as possible, and that they should be placed to suit the roadways beneath the structure rather than to conform to any simple structural requirements.

Cast-in-place post-tensioned concrete structures can be made to meet all of the foregoing requirements very economically and with results that are aesthetically pleasing. To develop the full potential of the material, however, it is necessary to use a more sophisticated structural configuration and to carry out a more complete analysis. The concept involved is one of *total design*, since advantage is taken of the latent structural strength which is not normally utilized.

Cross-Sections

Figure 29.2 shows a number of different types of cross-sections which have been used. The sections using round voids are cast in one operation whereas sections similar to Fig. 29.2(d) usually require construction joints and are more liable to shrinkage cracks. This factor and the more complex formwork go against the use of box sections except for long spans.

Section (a) has been used for spans from 125–150 ft (38–46 m). Sections (b) and (c) have been found useful for a wide range of spans from 70–150 ft (21–46 m), and section (d) is suitable for spans in excess of 150 ft. The width of the cross-sections does not often exceed 50 ft (15 m). Roadway widths greater than 50 ft are usually carried on two or more separate structures.

Structural Capacity

It is common practice to utilize sections such as those described only as a means of spanning in the longitudinal direction. Some of the additional structural capacities may, however, be identified as follows:

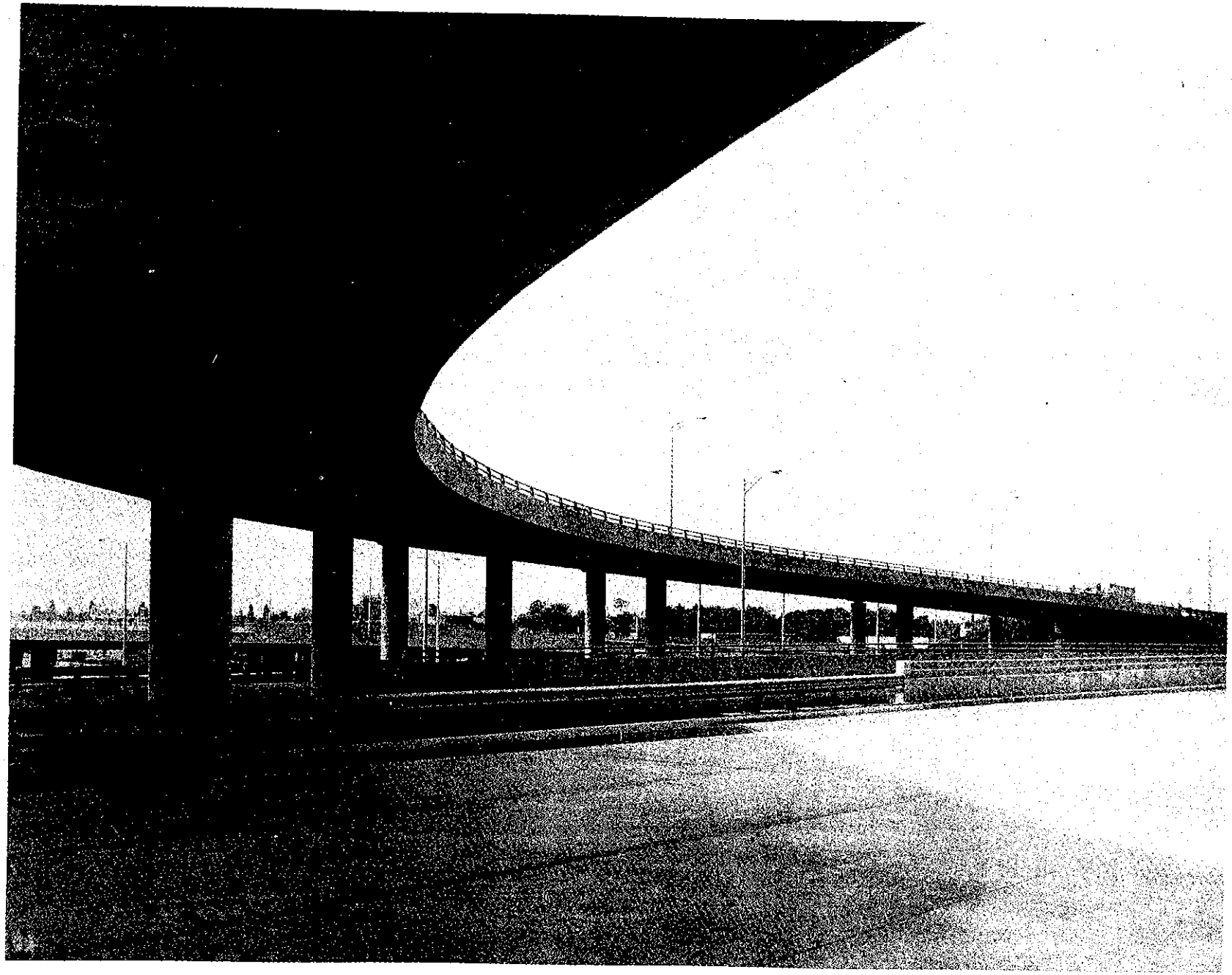
(a) *Transverse flexural strength:* All of the sections shown have the ability to resist transverse bending due to vertical loads. With the section depths and widths commonly used, a single column at the center of the section produces transverse bending moments which are within the capacity of the deck section using normal reinforcement.

(b) *Lateral flexural strength:* All sections are able to resist very large bending moments due to lateral forces. For bridges of normal length it is usually not necessary to resist lateral forces at piers. The abutments can be designed to provide adequate lateral restraint at negligible expense.

(c) *Torsional strength about longitudinal axis:* The sections shown in Fig. 29.2 have a very large resistance to twisting. Except for very long bridges, it is sufficient to resist twisting only at the abutments. Wide piers, piers

* The author wishes to thank Witecki, Bassi, Lin and Richardson, and acknowledges with grateful thanks the material taken from their papers as enlisted in the references 1 and 2 of this chapter.

† Only the parasitic prestress effect is akin to an (externally) applied load, not the primary prestress effect.



A curved continuous prestressed concrete bridge at the Spadina-Interchange, Toronto, Canada

Fig. 29.1 *Curved bridges in highway interchange*

integral with the deck, or piers made up of many columns are seldom necessary.

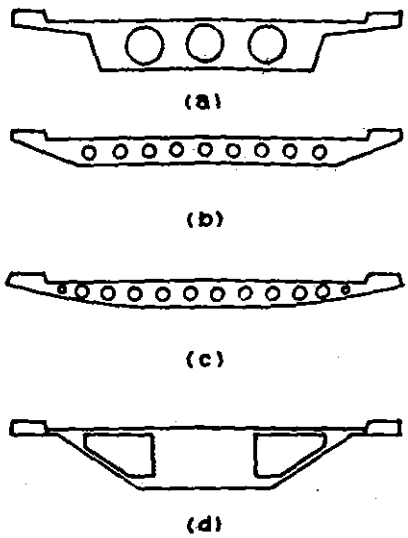


Fig. 29.2 *Types of bridge cross-sections used in Ontario*

Utilization of the structural capacities described above can lead to considerable economy, particularly in the reduction of the number and size of the columns. Initially this was done only when space limitations left no alternative, but the improved appearance and reduced costs that resulted have led to a general policy of reducing substructures to a minimum.

Large transverse bending moments occur particularly when only one column is used at each pier. Approximate calculations of transverse flexural deflection however are usually sufficient to demonstrate that the distribution of longitudinal bending stress is not significantly affected by transverse bending. Nearly all structures of the type considered can be designed longitudinally as beams, and transverse bending can be considered separately.

Twisting of the deck of a straight bridge can affect the transverse distribution of longitudinal bending stresses, but the loading required to produce such twisting is seldom critical for longitudinal flexure, and torsion can be considered separately. For curved bridges, however, longitudinal bending does affect the magnitude of torsional stresses very significantly and, if the curvature is sharp, torsion can increase longitudinal bending stresses to a limited extent.

29.2 LONGITUDINAL ANALYSIS

The bridge shown in Fig. 29.3 was built with a curvature of 30 degrees. The effect of the curvature on the longitudinal bending moments was investigated analytically, and the results for curvature from 0 deg. to 60 deg. are shown in Fig. 29.4. It can be seen that the bending moment at the center of the bridge is not appreciably affected by curvature and that the bending moment at the pier for 60 deg. curvature, radius 96 ft (30 m), is only 3% greater than for an equivalent straight bridge. The curves shown are for deal load. This is encouraging for the present analysis.

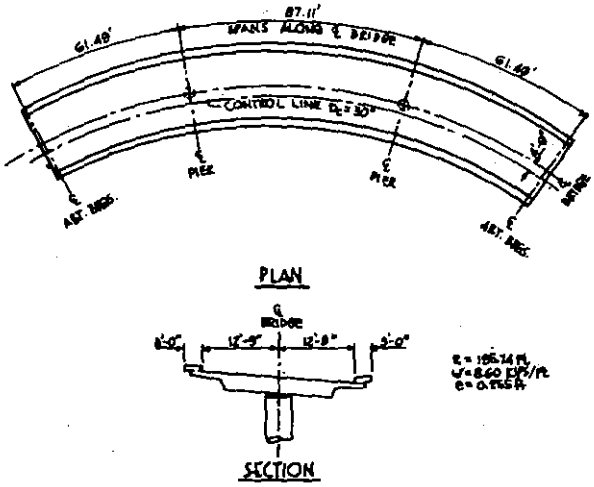


Fig. 29.3 *Dimensions of 30 degrees curvature bridge* (* 'Degree' of a curve is the angle subtended at the center by a 100 ft. length of arc.)*

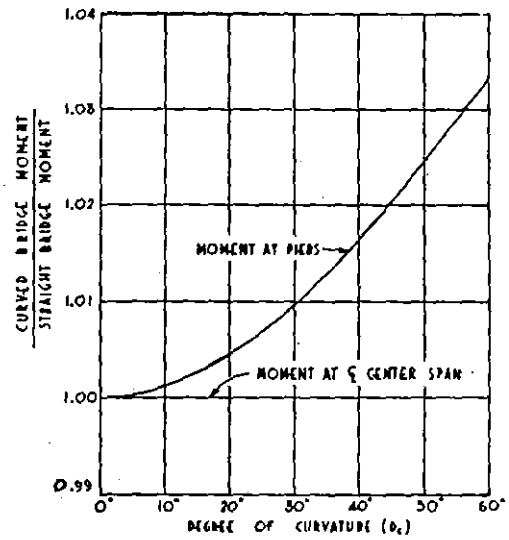


Fig. 29.4 *Effect of bridge curvature on longitudinal dead load moments*

A curvature of 60 deg. is far beyond the limit of normal highway design. Other variables are involved in the above comparison such as type of loading, width-to-span ratio, and

elastic properties; but investigation of a large number of cases has shown that for normal design purposes it may be assumed that the longitudinal bending moments, shears, and reactions are not significantly affected by curvature, and above about 300 ft. radius it can be safely assumed that in a curved-in-plan superstructure the values of longitudinal bending moments, shears and reactions are almost same as in a straight-in-plan case whose spans arc equal to arc spans of the former.

29.3 TORQUE ANALYSIS

An approximate method is available for the calculation of torque in curved bridges.[1] It is restricted to curved beams which have constant or nearly constant elastic properties. It is felt that the method is quite adequate for all normal bridge-design problems provided there are no drastic changes in cross-section properties. The procedure obtains torison values by calculating the shear due to a hypothetical distributed torque loading.

Consider a small element of length ds of a curved beam as shown in Fig. 29.5.

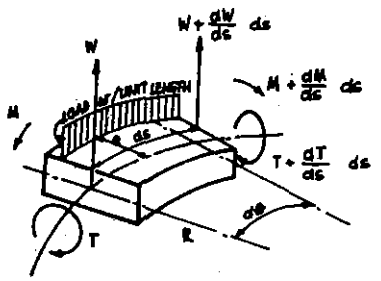


Fig. 29.5 *Torque analysis element for curved beam*

where t = applied torque per unit length at a section, = (w.e. + t')

R = radius of element ds at center-line of section

t' = any locally applied torque at a section (distributed over a dispersion width or uniformly distributed)

T = torsional moment created as a result of applied M, w and t'.

M = moment (externally applied, including parasitic moment due to non-concordant prestress)

W = shear

w = uniformly distributed load per unit length (self dead load)

$e = \dfrac{\gamma I_{yy}}{R \cdot w}$ eccentricity of the center of gravity of the uniformly distributed self dead load, measured from the center-line of the section

γ = density of material

I_{yy} = moment of inertia about vertical axis through centroid of section

a = distance from center-line of torque span to point under consideration

torque span = length of structure between points of torsional restraints (single bearing support has no torsional restraint)

Assuming $\cos d\theta = 1$ and $\sin d\theta = d\theta = ds/R$, the equilibrium equations for the element ds are,

$$\frac{dW}{ds} = -w \text{ or } W = \int -w ds \tag{29.1}$$

$$\frac{dT}{ds} = \frac{M}{R} + t \text{ or } T = \int \left(\frac{M}{R} + t\right) ds \tag{29.2}$$

$$\frac{dM}{ds} = W - \frac{T}{R} \text{ or } M = \int \left(W - \frac{T}{R}\right) ds \tag{29.3}$$

For long torque spans and/or sharply curved beams, Eq. (29.2) can be modified as follows to improve the accuracy of the approximate method (see Fig. 29.6):

$$T = \int \left[\frac{M}{R} + t\cos(a/R)\right] ds \tag{29.4}$$

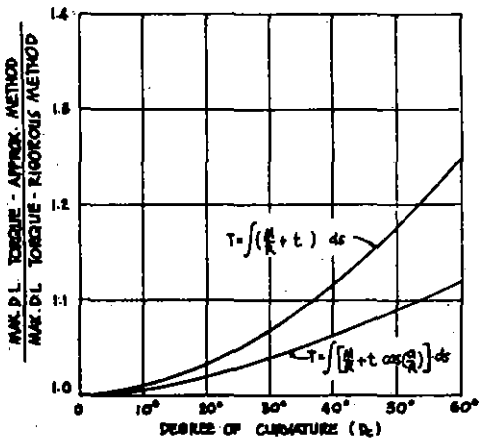


Fig. 29.6 *Effect of bridge curvature on accuracy of approximate method of torque analysis*

From Eq. (29.4) it is evident that the torque at any point along a curved beam is numerically equal to the shear (computed for the *torque span*) due to the application of a hypothetical load of $[(M/R + t\cos(a/R)]$ along the beam.

The magnitude of the torsional stiffness does not appear in the method. It is obvious that the torsional stiffness could affect the actual torque values. Using the rigorous method,

the effect of torsional stiffness on torque was investigated for a wide range of values. Figure 29.7 is a graphic presentation of the results and shows that, for a very wide range of torsional-stiffness values, the maximum torque (dead load is again used as an example) at the abutment is not appreciably affected. For low values of torsional stiffness, the torque decreases rapidly with stiffness to zero. Cross-sections of the type shown in Fig. 29.2 have torsional-stiffness values in the range in which torque is almost independent of stiffness.

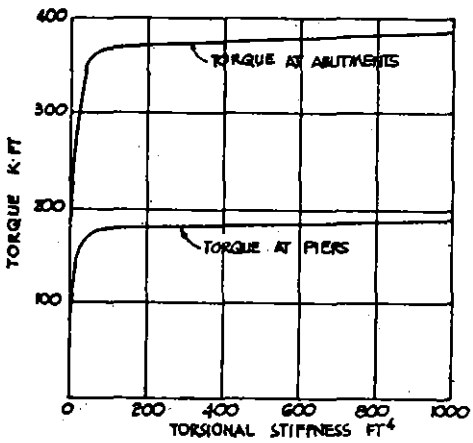


Fig. 29.7 *Effect of torsional stiffness on dead-load torques*

Application

The formula mentioned in Eq. (29.2) can now be applied to calculate the torsional moments as an effect of a horizontally curved multispan continuous bridge.

Step 1 Determine the geometry of the bridge.

Step 2 Establish the deck cross-section and compute the dead load, superimposed dead load, live load and any other anticipated load.

Step 3 Define uniformly distributed torque loading, t (it may have non-uniform values at certain sections).

Step 4 'Straighten out' the bridge deck to its 'full developed length' and compute bending moments for applied loadings, including prestress parasitic moments, not prestress primary moments. ('Straight' bridge means one with spans in length equal to 'arc spans' of curved bridge.)

Step 5 Add all of the computed bending moments and divide the result by radius R to obtain distributed torque loading M/R along the centerline of the bridge.

Step 6 Add the results of step 3 and step 5, at corresponding sections, to obtain the total distributed torque loading, $M/R+t$, along the centerline of the bridge (geometrical center line, not centroid of mass).

Step 7 Establish supports capable of restraining the torsional moment and consider the developed length of deck between these supports as a series of simply supported beams—torque spans.

Step 8 Apply the result of step 6 as a distributed load on the previously defined simply supported beams and compute simple beam reactions and shear diagram for each of the beams. These reactions are the concentrated torsional moments developed at the ends of the beams, whereas the ordinates of the shear diagram represent the torsional moment at any given section.

The total effect of torsion on support restraining the torque is the algebraic sum of the torsional moment at the two adjoining ends of the assumed simple beams, above that support.

Step 9 Compute stresses due to torsional moment at the desired sections along the deck; design the necessary reinforcement for (combined shear and) torsion.

NOTE Only stresses due to the torsional moment have been considered here. Stresses due to longitudinal bending moment, transverse bending moment, shear (principal tension) have to be considered separately.

Also see Annexure at chapter-end for more information.

Numerical Example

The structure considered here is a curved, three-span, continuous bridge with prestressed, post-tensioned, concrete superstructure as shown in Figs. 29.8, 29.9 and 29.10. The superstructure is of solid concrete and has a constant cross-section throughout (Fig. 29.10). It is supported by two abutments and two intermediate single bearing columns (Fig. 29.9). Bearing at the columns are rubber 'pot' bearings allowing free horizontal movement and rotation of the superstructure. Bearings at the abutments are considered to be capable of restraining the torsional moment.

In order to shorten the calculations in the example that follows, only the torsional moment as an effect of dead load and live load is considered, omitting the effect of prestress.

Step 1 The geometry of the bridge is shown in plan, in Fig. 29.8. The bridge is on a 13-deg. curve and the radius of curvature at the centerline of the bridge is R = 440.737 ft.

Step 2 Establish the deck cross-section and compute the dead load, superimposed dead load, live load, etc. (Fig. 29.11).

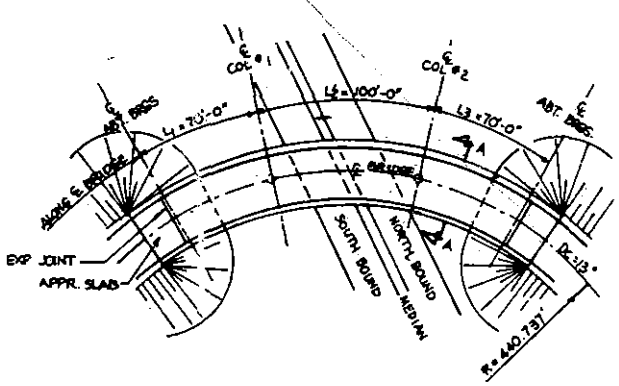


Fig. 29.8 *Plan*

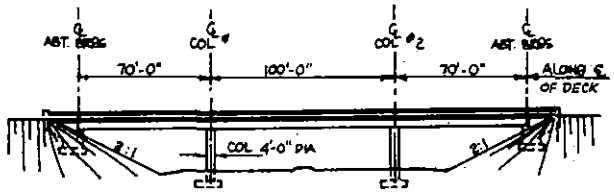


Fig. 29.9 *Elevation*

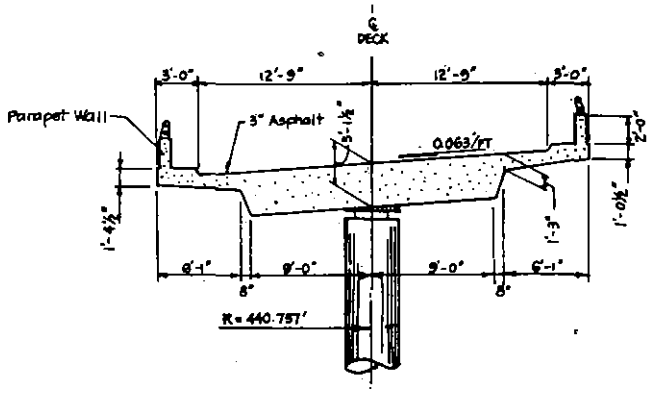


Fig. 29.10 *Cross-section A – A*

Location of CG of deck

Distance: CG from top of deck $Y_t = 1.28$ ft

CG from bottom of deck $Y_b = 1.84$ ft

Dead load

Area of deck cross-section = 73.796 sq. ft

Weight of deck,

including curbs, ω = 73.796× 0.150

= 11.07 k/ft

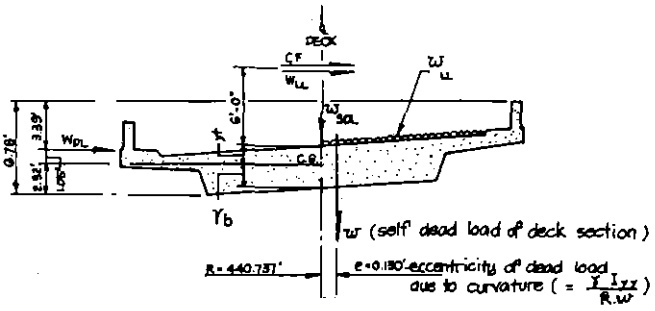


Fig. 29.11 *Cross-section showing all applied loading*

Superimposed dead load

Parapet walls = 0.833 × 2.000 × 0.150 × 2 = 0.50 k/ft

Asphalt (3-in.) = 25.50 × 0.250 × 0.150 = 0.96 k/ft

Total ω_{SDL} = 1.46 k/ft

Live load

Loading as per AASHTO HS20-44 including impact taken as max 30%.

(a) *Uniform vertical load*

Uniformly distributed load applied as shown in Fig. 29.11 to obtain maximum torsional effect. Concentrated load as per AASHTO specifications omitted in this example to simplify calculations.

$$\omega_{LL} = 0.064 \times 12.75 \times 1.30 = 1.06 \text{ k/ft}$$

(b) *Centrifugal force*

Maximum design speed (ramp) = 40 mph, as per AASHTO specifications.

$$C = 0.00117 \times 40^2 \times 13 = 24.34\% \text{ say, } 25\%.$$

$$CF = 0.25 \times 1.06 = 0.27 \text{ k/ft}$$

Wind

Loading as per AASHTO specifications:

(a) *Load on structure* acting at center of gravity of exposed area:

$$W_{DL} = 6.78 \times 0.050 = 0.34 \text{ k/ft}$$

(b) *Wind on live load* acting at point 6 ft 0-in. above deck:

$$W_{LL} = 0.100 \text{ k/ft}$$

Step 3 Determine distributed torque loading, t.

	Loading	*Moment*	
Self weight of deck	11.07 × 0.13	= 1.34	(= ωe)
Live load	1.06 × 12.75× 0.50	= 6.26	
Centrifugal force	0.27 × (6.00 + 1.28)	= 1.57	
Wind on structure	0.340× 1.05	= 0.36	(= t')
Wind on live load	0.100× (6.00 + 1.28)	= 0.73	
		Total: 10.26 kft/ft	

Step 4 'Straighten' out the bridge deck to its full developed length and compute dead load bending moment, superimposed dead load bending moment, live load bending moment, etc.

Steps 5 and 6

Tabulate the bending moments calculated in step 4 and compute the (total) distributed torque load, $M/R + t$, along the centerline of the bridge at corresponding sections (Table 29.1). Since the bridge is symmetrical, the calculations are limited to the point of symmetry at the centerline of the middle span (Fig. 29.12). These calculations are for every tenth section of each span.

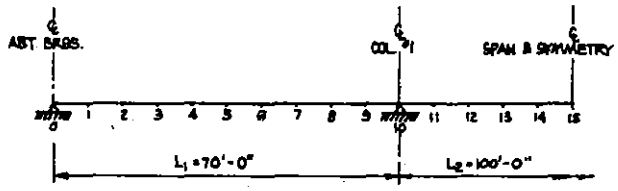


Fig. 29.12 *Developed length of bridge deck (half shown)*

Step 7 As mentioned before, the abutments, in the present case will be the only supports 'capable of restraining the torsional moment'. Therefore the full developed length of the deck between the abutments must be regarded as a single 'torque span'.

Step 8 Using the values of $M/R + t$ from Table 29.1 construct a distributed torque loading diagram as in Fig. 29.13.

Applying the above 'conjugate' loading (Fig. 29.13) on the previously defined 'torque span'

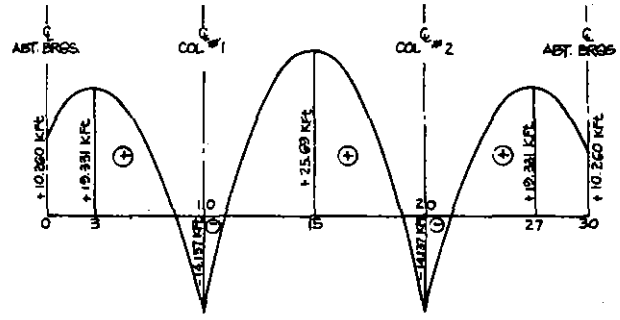


Fig. 29.13 *Distributed torque loading diagram*

(step 7), construct the shear diagram as for a simply supported beam (Fig. 29.14). This represents the final torsional moment diagram; where the ordinates are the final torsional at any given section.

The diagram, Fig. 29.14 shows ordinates at every tenth section of the 'torque span'.

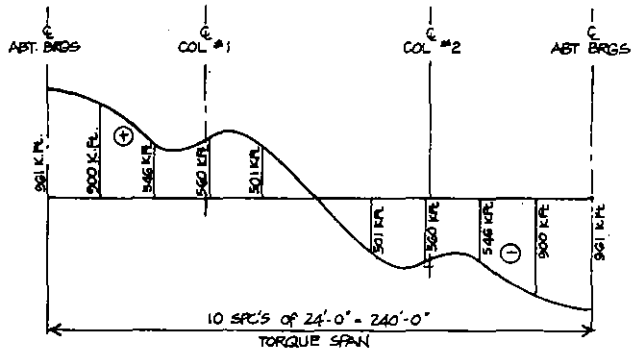


Fig. 29.14 *Final torsional moment diagram*

Table 29.1 *Computation of total distributed torque loading*

SECTION	Longitudinal bending moment $M(k \cdot ft)$					R	$\frac{M}{R}$	t	$\frac{M}{R} + t(kft/ft)$
	DL	SDL	LL	OTHER	TOTAL				
0	0	0	0		0		0		10.260
1	1646	217	156		2019	440.737 ft	4.580	10.260 kft/ft	14.840
2	2732	360	262		3354	440.737 ft	7.610	10.260 kft/ft	17.870
3	3257	429	312		3998	440.737 ft	9.071	10.260 kft/ft	19.331
4	3222	425	309		3956	440.737 ft	8.976	10.260 kft/ft	19.236
5	2626	346	251		3223	440.737 ft	7.313	10.260 kft/ft	17.573
6	1470	193	140		1803	440.737 ft	4.091	10.260 kft/ft	14.351
7	− 247	− 33	− 24		− 304	440.737 ft	− 0.690	10.260 kft/ft	9.570
8	−2524	− 333	−242		− 3099	440.737 ft	− 7.031	10.260 kft/ft	3.229
9	−5361	− 707	−513		− 6581	440.737 ft	−14.932	10.260 kft/ft	− 4.672
10	−8759	−1155	−839		−10753	440.737 ft	−24.397	10.260 kft/ft	−14.137
11	−3612	− 476	−346		− 4434	440.737 ft	−10.060	10.260 kft/ft	0.200
12	392	52	37		481	440.737 ft	1.091	10.260 kft/ft	11.351
13	3252	428	311		3991	440.737 ft	9.055	10.260 kft/ft	19.315
14	4967	655	476		6098	440.737 ft	13.8355	10.260 kft/ft	24.095
15	5540	730	530		6800	440.737 ft	15.428	10.260 kft/ft	25.688

Step 9 Compute stresses due to the torsional moment at the desired sections along the deck and design the necessary reinforcement for (combined shear and) torsion. (This portion is outside the scope of the present chapter but has been dealt with separately in Ch. 24 in this book to which reference may be made for the reinforcement design.)

REFERENCES

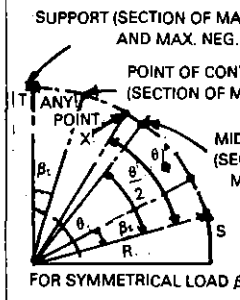
1. Witecki, A., "Simplified Method for the Analysis of Torsional Moment as an Effect of a Horizontally Curved, Multispan Continuous Bridge," *1st International Symposium on Concrete Bridge Design*, SP-23 American Concrete Institute, Detroit, 1968.
2. Bassi, K.G., WL Lin and BS Richardson, "Continuous Post-tensioned Torsionally Stiff Concrete Bridges", *1st International Symposium on Concrete Bridge Design*, SP-23, American Concrete Institute, Detroit, 1968.

ANNEXURE

Rapid Analysis of Circularly-Curved-in-Plan Girders*

BOW GIRDERS (CIRCULAR ARCS)

APPLICABLE TO (i) Beams forming a complete circular system with supports spaced uniformly, and with identically load on each span.
(ii) Beam of single-span with ends rigidly fixed.



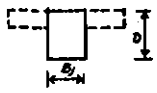
Section	Bending Moment	Twisting Moment	Shear Force
Support	UK_wR or $\frac{C_2G + C}{C_1G + 1} K_wR$	SK_wR or $\frac{C_4G + C_5}{C_1G + 1} K_wR$	QK_w
Point of Contra-flexure	ZERO	VK_wR or $\frac{C_6G + C_7}{C_1G + 1} K_wR$	NK_w
Mid-span	YK_wR or $\frac{C_8G + C_9}{C_1G+1} K_wR$	ZERO	ZERO

		Uniformly Distributed Load (K_w = wR) w lb. per Foot							Central Point Load P.(K_w = P)	
ANGLE θ		30°	45°	60°	72°	90°	120°	180°	90°	180°
NO. OF SUPPORTS		12	8	6	5	4	3	(2)	4	(2)
C_1		—	—	10.4	7.5	4.5	2.4	—	4.5	—
SUPPORT	U	0.02 Approx.	0.053	—	—	—	—	1.00	—	0.5
	C_2	—	—	0.99	1.00	1.00	1.11	—	0.97	—
	C	—	—	0.12	0.14	0.24	0.45	—	0.25	—
	S	NEGLIGIBLE		—	—	—	—	0.30	—	0.18
	C_4			0.025	0.03	0.03	0.27	—	0.04	—
	C_5			0.02	0.02	0.03	0.09	—	0.04	—
	V	0.001 Approx.	0.004	—	—	—	—	0.12	—	0.09
	C_6	—	—	0.08	0.10	0.13	0.16	—	0.17	—
	C_7	—	—	ZERO	0.01	0.02	0.03	—	0.26	—
MID-SPAN	Y	0.01 Approx.	0.023	—	—	—	—	ZERO	—	0.318
	C_8	—	—	0.40	0.38	0.32	0.11	—	0.19	—
	C_9	—	—	0.10	0.08	0.05	0.002	—	ZERO	—
SHEAR	Q	0.28	0.39	0.53	0.63	0.79	1.05	1.57	0.5	0.5
	N	0.20	0.22	0.26	0.35	0.40	0.49	0.66	0.22	0.18
Angle β		6°	10°	15°	18°	22°	36°	52°	$25\frac{1}{2}$°	$57\frac{1}{2}$°

$$G = \frac{\text{Torsional Rigidity}}{\text{Flexural Rigidity}}$$

G is approximately equal for rectangular, TEE, and ELL Sections with the same ratio of D:B.

D/B	1.0	1.5	2.0	2.5	3.0
G	1.12	0.70	0.47	0.33	0.22



* Refer to: Schulz M., and M. Chedraui, "Tables for circularly curved horizontal beams with symmetrical uniform loads", *Journal of the American Concrete Institute*, vol. 28, no. 11, May 1957, pp. 1033–1040 and Spyropoulos, P.J., "Circularly curved beams transversely loaded", *Journal of the American Concrete Institute*, vol. 60, no. 10, October 1963, pp. 1457–1469.

Numerical Example

A bow girder in 18 in. deep and 18 in. wide, has a radius of 12 ft, and subtends an angle of 90 deg. The ends are rigidly fixed and the total load is 45,000 lb uniformly distributed. Find the maximum negative bending moment, the maximum positive bending moment, and the maximum twisting moment. Thus θ = 90 deg; R = 12 ft;

$$G = 1.12; w = \frac{45,000}{\pi \times 2 \times 12 \times 0.25} = 2,390 \text{ lb per ft};$$

$K_w = 2{,}390 \times 12 = 28{,}700$ lb

According to the table given above:

The maximum negative bending moment occurs at the support, and the coefficients are $C_1 = 4.5$; $C_2 = 1.00$; and $C = 0.24$. The maximum negative bending moment

$$= \frac{(1.00 \times 1.12) + 0.24}{(4.5 \times 1.12) + 1} \times 28{,}700 \times 12 = 77{,}400 \text{ ft-lb}$$

The maximum positive bending moment occurs at midspan; $C_8 = 0.32$; $C_9 = 0.05$. The maximum positive bending moment

$$= \frac{(0.32 \times 1.12) + 0.05}{6.04} \times 28{,}700 \times 12 = 23{,}200 \text{ ft-lb}$$

The maximum twisting moment occurs at the point of contraflexure (β = 22 deg.) $C_6 = 0.13$; $C_7 = 0.02$. The maximum twisting moment is

$$= \frac{(0.13 \times 1.12) + 0.02}{6.04} \times 28{,}700 \times 12 = 9{,}360 \text{ ft-lb}$$

CHAPTER 30

Temperature Stresses in Concrete Bridge Decks —Simple Design Method

Synopsis

After briefly reviewing the background of the problem and clearly pinpointing the three causes that build up the thermal stresses, the phenomenon of distribution of temperature through the deck-depth is examined and idealisation of the non-linear differential thermal gradient suggested. The possible thermal crackpatterns are discussed. Some useful observations in relation to thermal stresses, including their effect on ultimate load carrying capacity of the member, their cyclic nature as distinct from the phenomenon of fatigue and their possible order of magnitude, are reported. Finally, suitable recommendations are made and typical numerical examples are presented clearly in a manner that would suit hand computation in a practising design office.

30.1 INTRODUCTION

Thermal effects have come in for close scrutiny in recent years. In the past the effect of non-linearity of temperature distribution across the deck-depth was not well understood so that, where thermal effect was considered (e.g., in an 'arch' or in a 'portal'), it was essentially the effect of rise or fall in the overall mean-temperature of the body over a long period of time.

As structures became more complex and the distribution of temperature through the deck-depth, particularly its nonlinear profile, began to be understood better, need for a closer look into thermal effects arose.

It is now realised that, in certain types of structures, the combined effects of 'change in the body-temperature', the 'nonlinear distribution of thermal gradient through the deck-depth' (and in case of a portal, across the abutment leg-thickness) and the 'continuity effect' owing to the constraints offered to free hogging (or sagging) desire of the structure (under unequal extreme fibre temperatures) by the intermediate and fixed supports (if any), can create flexural stress that can match the dead and live load stresses.

30.2 THE THREE CAUSES

(i) Effect of Change (Rise or Fall) in the 'Mean' Temperature of the Body of the Deck

For the purpose of this effect, it is assumed that the temperature of the entire body of deck has one 'mean' value at any instant of time and that this 'body-mean-temperature' rises or falls over a long period of time, thereby wanting the structure to 'heave'. If the structure is free to permit this 'heave' i.e., is free to expand or contract (e.g., simply supported beam or a continuous beam), this causes no thermal stresses. However, if the structure is unable to permit such a heave (e.g., arch, frame, fixed beam), i.e., offers constraint to its desire to heave, moments etc. are then caused, which create stresses (**thermal stress Type 1**). These moments can be evaluated by the usual methods of theory of elasticity. (See Annexure at chapter-end.)

(ii) The Effect of 'Nonlinear' Distribution of Temperature across the Deck-Depth

If the top surface of the concrete deck is hotter than its soffit surface (temperatures T_1 and T_2 in Fig. 30.1), the ordinate T of the thermal gradient at any intermediate depth, as measured from the base line $a - b$, has been discovered to follow a nonlinear variations as shown in Fig. 30.1.

Figure 30.2 indicates the build-up of the total thermal gradient. Its uniform part ⓐ, at the instant of consideration, is akin to the 'body-mean-temperature', the effect of change in which over a long period of time, is already taken care of earlier. However, the variable part ⓑ, better called the 'differential thermal gradient', would heat each fibre to a different degree, the variation being nonlinear. If the fibres were free to each other (i.e., unrestrained) then they could happily accept the corresponding non-linear thermal strains αt (α being the coefficient of expansion/contraction). But since their deformations must follow a linear law (plane sections must remain plane), they will not accept these non-linearly related strains, and the difference between the final 'linear' strain gradient ⓓ and the 'unrestrained' strain gradient ⓒ (i.e., $\epsilon_0 - y\theta - \alpha t$) will represent the

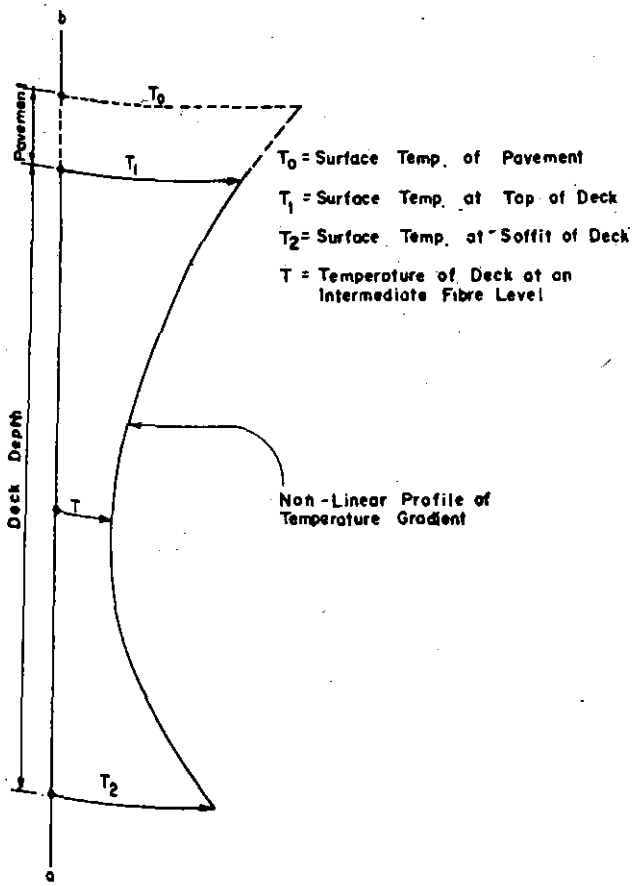


Fig. 30.1 *Total thermal gradient*

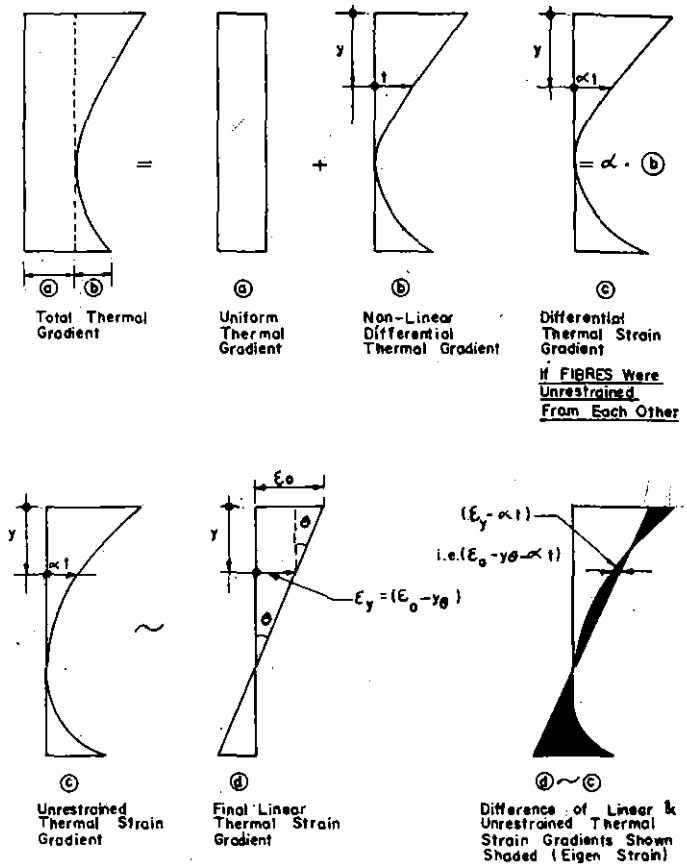


Fig. 30.2

uneven 'internal disturbance'. Its strain effect may be called the Eigenstrain and its stress effect may be called the Eigenstress, both of which would be zero if only the thermal gradient were linear (which it is not). This Eigenstress (and the Eigenstrain), as can be seen, is purely an internal entity, not associated with any support reactions.

Both its 'force' (at any section) and its 'moment' about any fibre in that section must sum up to zero (i.e., self-equilibriating).

As shown in Fig. 30.2, the ordinate of the shaded diagram ⓓ-ⓒ, at y below top, represents the Eigenstrain at the fibre, i.e.,

$$\text{eigenstrain}: \epsilon_{ei} = \epsilon_0 - y\theta - \alpha t \qquad (30.1)$$

and $\therefore$

$$\text{eigenstress}: f_{ei} = E_C \cdot \epsilon_{ei} = E_C(\epsilon_0 - y\theta - \alpha t) \qquad (30.2)$$

where E_C is the modulus of elasticity of concrete. If A be the section area of this fibre, then:

$$\text{since } \Sigma A f_{ei} = 0 \ \therefore E_C \Sigma A(\epsilon_0 - y\theta - \alpha t) = 0$$

and since $\Sigma[(A f_{ei})y] = 0 \ \therefore E_C \Sigma A(\epsilon_0 - y\theta - \alpha t)y = 0$

From these equations it follows that:

$$\epsilon_0 \Sigma A - \theta \Sigma A y = \alpha \Sigma A t \qquad (30.3)$$

and

$$\epsilon_0 \Sigma A y - \theta \Sigma A y^2 = \alpha \Sigma A y t \qquad (30.4)$$

Thus, in any given deck section, subjected to a known differential temperature gradient, the section can be broken into rectangular elements, the terms ΣA, ΣAy, ΣAy^2, ΣAt and ΣAyt quickly evaluated and a mere substitution in Eqs. (30.3) and (30.4) will yield the values of extreme fibre strain ϵ_0 (at $y = 0$) and the strain gradient θ, whereafter Eigenstress at any fibre can be readily evaluated from Eq. (30.2) **(thermal stress Type 2).**

Eigenstress, on its own, may be small or significant, depending on,

- depth of section
- thickness and colour of pavement
- wind speed
- orientation of bridge and incidence of sun rays
- ambient temperature
- material properties,

— thermal conductivity
— specific heat
— thermal diffusivity
— coefficient of thermal expansion and contraction
— coefficient of absorptivity
— coefficient of surface-heat-transfer

- surface temperature
- shape of thermal gradient.

The distribution of Eigenstress, not being linear, when added to the thermal 'Continuity' stress (see ahead), may show significant stresses not only at extreme fibres but also at intermediate fibres (e.g., mid height portion of webs) which are heavily loaded under shear. This can produce longitudinal cracks in webs.

(iii) Effect of Intermediate-Support Restraint on the Free Hogging (or Sagging) Desire of the Structure caused by Unequal Extreme Fibre Temperatures—the 'Continuity Effect'

In a beam-type deck, the difference of temperature between the extreme surfaces causes hogging (or sagging) of the beam.

If the beam is simply supported, it merely hogs (or sags) as its supports do not prevent rotation. This free deformation is not a 'moment-induced' deformation, but merely a 'strain-induced' deformation, and no moment is caused, Fig. 30.3.

However, if the beam is continuous, its aforementioned desire to freely hog (or freely sag) will be 'constrained' at the intermediate supports (presence of dead load reactions will prevent it from lifting up and presence of supports will prevent it from going down at these supports). This 'continuity' effect sets up moments that cause additional stresses **(thermal stress Type 3)** called 'continuity' stresses.

These 'continuity moments may simply be computed by superposition of 'free curvature moments M' and the 'distributed moments' due to equal and opposite moments M applied at the two ends of the continuous beam necessary to restore zero moment condition at the two ends. This is shown diagrammatically in Fig. 30.4 and solutions of continuity moments and reactions in some typical continuous beams are indicated in Fig. 30.5.

While here the discussion is restricted only to beams the equivalent of the aforementioned 'continuity moments' in the longitudinal frames (i.e., single or multi-bay portals) would be the 'frame moments' set up due to constraints to free hogging (or sagging) because of rigid joints, and can be analysed by the usual classic methods of elastic analysis. (See Annexure at chapter-end.)

Review

A review of three causes under subhead 30.2 above clearly indicates that the thermal stresses are caused:

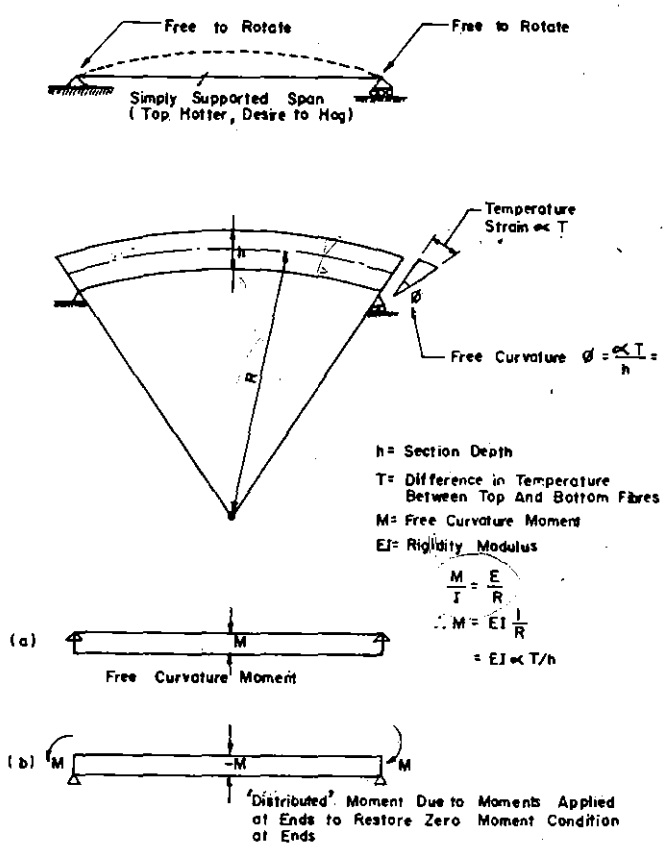


Fig. 30.3 *Net moment = (a) + (b) = zero, in the simple span*

— in a simply supported deck ... only by one cause i.e., 'Eigenstress'.
— in a continuous beam deck ... by two causes i.e., 'Eigenstress' and 'Continuity effect'.
— in a frame deck ... by all three causes i.e., 'Eigenstress', 'Continuity effect' and 'Change in body-mean-temperature'.

30.3 PREDICTING TEMPERATURE DISTRIBUTION THROUGH DECK-DEPTH (FOR EVALUATING EIGENSTRESS)

Theoretical Background

The thermal response of an isotropic solid with a discrete boundary in contract with air, Fig. 30.6, subjected to variations in ambient temperature θ_a and radiation q, is governed by the general Fourier Conduction Equation (Reference 5 at the chapter-end):

$$\frac{k}{\rho c}\left[\frac{\partial^2\theta}{\partial x^2}+\frac{\partial^2\theta}{\partial y^2}+\frac{\partial^2\theta}{\partial z^2}\right]=\frac{\partial\theta}{\partial t} \qquad (30.5)$$

with the boundary condition

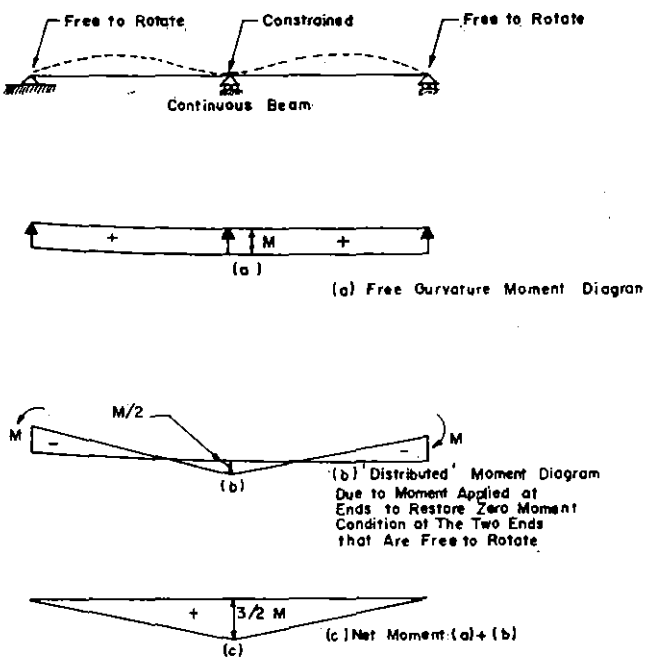


Fig. 30.4

$$k\frac{\partial\theta}{\partial_n} + q + H(\theta_a - \theta_0) = 0$$

where k = conductivity of the solid

ρ = its density

c = its specific heat

θ_0 = boundary temperature of the solid

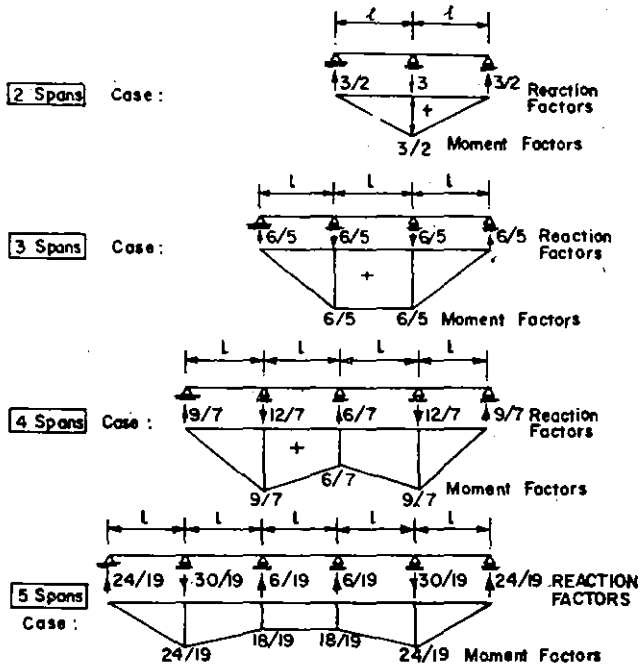
H = boundary heat transfer coefficient (depending mainly on wind speed)

n = the direction normal to the boundary

q, θ_a, and H are complex functions of time, and therefore a direct mathematical solution of Eq. (30.5) is almost impossible. Use of 3-dimensional (x, v, and z directions) finite element technique is appropriate. However, for Bridge decks, the problem can easily be reduced to a 1-dimensional heat flow problem since in most bridge decks the temperature variation along longitudinal (x) and breadth (z) axes is of little significance—only in the depth (y) direction it matters most. Hence Eq. (30.5) can be reduced to

$$\frac{k}{\rho c}\frac{\partial^2\theta}{\partial y^2} = \frac{\partial\theta}{\partial t} \qquad (30.7)$$

which may be solved along with Eq. (30.6) by finite difference method. To analyse the thermal response by such method in a given deck section, it may be necessary to analyse a few representative locations within the section and then combine the total results for the section. Finally



REACTION = (Reaction Factor)·M/l

MOMENT = (Moment Factor)·M

where M = free-curvature moment

= $EI \propto T/h$ (see Fig. 30.3)

Fig. 30.5 *Examples of continuity moments and reactions for various cases of prismatic continuous beams of equal spans due to temperature difference 'T' between top and bottom (top hotter).*

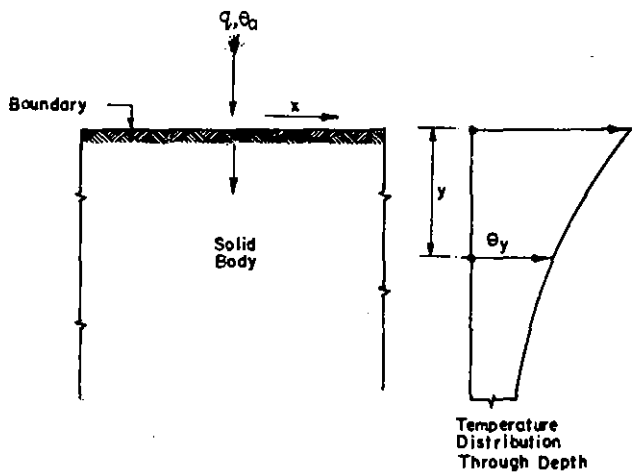


Fig. 30.6

a critical temperature distribution through the deck-depth may be established.

Computer Analysis

The Research Report 23–3F (Project 3–5–74–23) by the 'Center for Highway Research' at the University of Texas at Austin, USA, reports of the TSAP (Temperature and Stress Analysis Program) computer program developed there sometime back (1977–78). This program first predict the temperatures at various 'points' at 'various sections', based on certain relevant input data and a one-or-two-dimensional heat flow model, and then uses it and predicts thermally induced stress assuming the 'structure' as a one dimensional (i.e., linear elastic) beam.

Subsequently they have also developed SHELL-8 program which basically uses the temperature input from TSAP and then analyses thermal stresses assuming the deck to be a 2-dimensional grillage with 6 degrees of freedom at each 'node'.

Such programs are obviously powerful tools but unfortunately are beyond the reach of most designers, who therefore need a more straight-forward approach, a reasonably idealised temperature distribution (depicted in Fig. 30.2b) and a workman-like solution to the problem. (See Recommendations and Numerical Examples ahead.)

Idealisation of the (Differential) Thermal Design Gradient

It has been observed that out of the 'differential thermal gradient', shown in Fig. 30.2b, majority of the top surface temperature cools off in about top 25 cm depth of deck. The rate of cooling thereafter slows down considerably. The bottom surface temperature, up the depth, cools relatively much slower than the top surface temperature.

Owing to complex variations possible among the various dependant factors listed earlier, it is very difficult to generalise the shape of this Design Gradient.

However, until more refinement and accuracy is possible, the (differential) thermal Design Gradient (shown in Fig. 30.2b) may be assumed to be of the idealised shape put forth in the Recommendations under 30.6 ahead.

30.4 THERMAL CRACK PATTERN

Although temperature differential is reversible (though not with same intensity) so that there is the tendency to alternately hog and sag, the worst effect in a continuous beam deck generally occurs when top is hotter. This causes a tendency to hog, which invites downward reactions (restraints) at intermediate supports and consequent upward reactions at abutments, and hence a sagging type Bending Moment.

This, in turn, can lead to (thermal) cracking in the bottom of the central span in a 3-span continuous beam and around central pier in 2-span continuous beam and increase in reactions at Abutments.

Cracks, parallel to length of the bridge, may also occur in the bottom to top slab due to sagging moments set up in it owing to downward restraints exerted by its intermediate web supports against its desire to hog up under higher surface temperature.

Horizontal cracks may also occur in the webs when significant horizontal thermal gradient exists through webs. This happens in thick webs (thickness generally greater than 30 cm).

Unless sections are thin, considerable temperature differentials can be set up and cracks will be induced by thermal stresses. It is a good idea then to permit partial prestressing (rather than go for full prestress) i.e., permit tensile crack under working load in combination with sufficient small diameter reinforcement bars at close pitch in tension zones, and rely on the reduction in flexural rigidity due to thermal cracking to reduce the 'continuity' moments. (Unfortunately for various other but valid reasons, partial prestressing is not yet allowed in many codes.) It is true that upon thermal cracking and consequent reduction in stiffness (EI), the 'continuity' stresses (which are caused in the first place because of restraint to deformation at supports as the member tries to hog or sag) reduce.

Provision of additional small diameter bars at close spacing in crack-critical zones does not avoid thermal cracks, it only distributes them better and reduces their widths. But one of the simplest means to avoid thermal cracking is to avoid thick webs and slabs in hollow boxes.

Owing to Poisson's effect (that 'compression in a given direction causes tension perpendicular to it') there is already some tension in the transverse direction because of longitudinal compressive stresses, hence cracks may easily occur in combination with temperature in thick webs and slabs.

It has to be remembered that in certain hot places the surface of a box-deck on a hot day can heat up to about 70 to 80°C with the air inside the box reaching about 40 to 50°C. During the following night, while the temperature outside the box can fall rapidly, the warm air inside can maintain its temperature for a relatively longer period. The resulting temperature gradients between outside and inside create bending moments in the transverse direction, with a tendency for longitudinal cracks in webs and slabs. These moments can be large where webs and bottom slabs are thicker (e.g., near supports) than where they are thinner (e.g., in span area). Figure 30.7 (after **Leonhardt**) shows transverse bending moments in two box sections with different web and soffit-slab thicknesses under a 20°C temper-

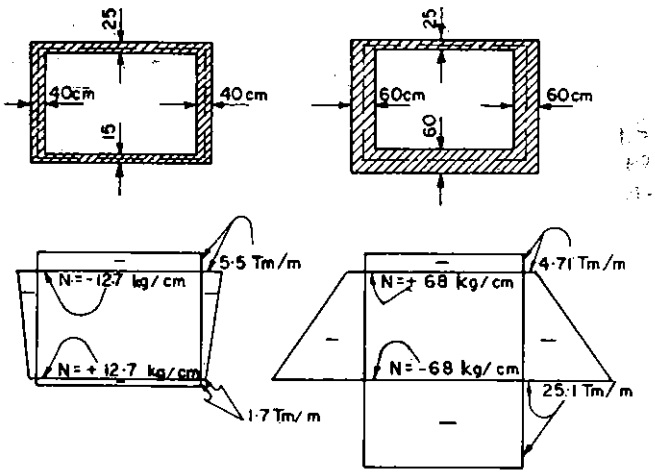


Fig. 30.7 *Transverse bending moments in hollow box sections due to temperature gradient of 20° Centigrades between inside and outside of box*
Left: Typical section in span
Right: Typical section near intermediate supports

ature difference between inside and outside (calculated by the usual methods of theory of elasticity; same as the continuity moments shown earlier; only the frame is in transverse direction). The results are revealing and show how high the thermal moments due to 'Continuity effect' can be.

30.5 THERMAL STRESSES

Possible Order of Magnitude

In the various Research Reports of the 'Center for Highway Research' at the University of Texas at Austin, USA (also see under Computer Analysis earlier) it has been shown that the total 'temperature induced tensile stresses in prestressed concrete slab type decks' and in composite decks comprised of 'prestressed beams and reinforced concrete slab', can be of the order of up to 60 and 80% respectively, of the modulus of rupture of concrete.

On the other hand, in composite decks consisting of 'steel beams and reinforced concrete slab', the total 'temperature induced tensile stresses' can reach about 10% of the total stress due to dead and live loads—but this 10% stress is well within the extra permitted under combination with temperature.

In certain box section prestressed concrete decks (45 cm webs), well detailed otherwise but poorly detailed against the effect of possible difference in temperature between inside and outside (sun beating on the outside while the atmosphere inside the box still thawing from winter conditions), the resultant tensile stresses far exceeded the modulus of rupture of concrete and longitudinal unsightly cracks developed in the webs.

'Eigenstresses' of 25 to 35 kg/cm^2 compression at top and bottom with 15 to 25kg/cm^2 tension in mid height region are not uncommon. In addition, reversible 'continuity stress' (resulting from the 'continuity effect' in continuous beams and frames) of 10 to 15 kg/cm^2 (compressive or tensile) at top and bottom, varying linearly in between it also not uncommon.

Hence it is important to estimate these stresses and ensure that the resultant stresses, in combination with those from other loads and forces, are safe.

Character

As pointed out in the earlier referred Research Reports, in general, Highway Bridges are subject to 'cyclic temperature induced stresses' at the rate of 30 cycles per month (365 cycles a year) and the frequency of one cycle is 24 hours. Furthermore the daily fluctuations of the thermal stresses are not necessarily at their maximum. Hence this cyclic character should not be compared to the classical fatigue phenomenon in which several hundred cycles per minute are normally applied.

Effect on Ultimate Load Carrying Capacity

At working load, thermal effect may be provided for as described earlier but its effect on ultimate load carrying capacity of the structure is usually small (Reference 5). This can be readily realised from Fig. 30.8. After considering the factored values of Dead and Live Loads, for a given thermal effect T, it is clear that its effect is relatively small owing to the drastic flattening of the moment-deformation curve in the yield zone. Temperature effect can even be disregarded in studying the ultimate strength of the structure.

Some Useful Observations

- Largest eigenstress and continuity stress can be induced in a concrete bridge deck, of given geometry, during summer time (when the solar radiation is maximum), particularly if the outer webs are protected by the shade of the overhangs.
- Eigenstresses increase with increase in depth of section, and are least in thin slabs.
- Continuity stresses reduce considerably due to thermal cracking.
- Unfavourable stresses will result also when the daily range of the ambient temperature is large and wind speed minimum.
- Higher wind speed reduces thermal inequalities in general and consequently reduces both the eigenstresses as well as the 'continuity stresses'. Hence bridges in exposed locations, as opposed to the bridges in sheltered locations, have lesser thermal stress problems.

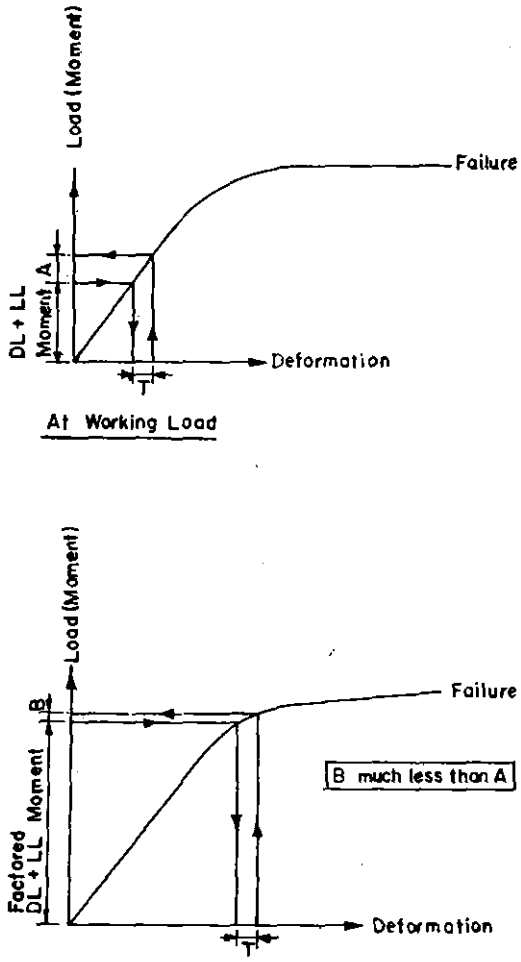


Fig. 30.8

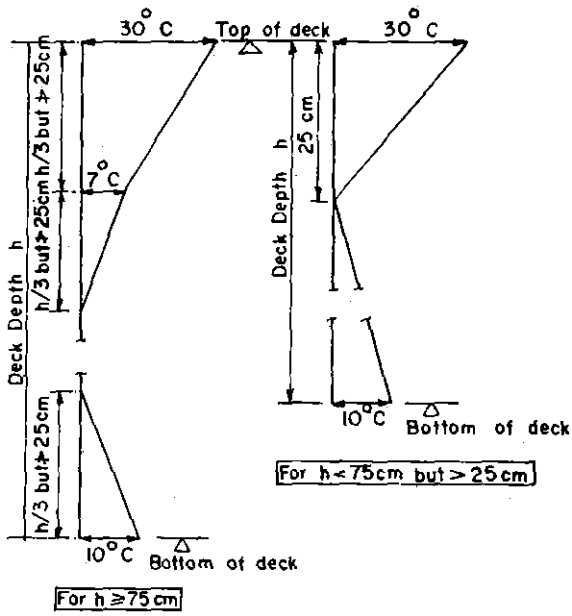
- High 'tensile' stresses due to temperature cannot practically be avoided unless the section thicknesses are reduced (and) or expensive prestress is introduced. Otherwise, provision of small diameter reinforcement bars at close spacing (10–12 cm) in crack-critical zones is advisable in order to avoid wide cracks and instead create finer cracks, better distributed.
- Thermal effect is not very significant in assessing the ultimate strength (load capacity) of a structure since it does not reduce the ultimate carrying capacity very much.
- Asphaltic wearing course, i.e., a 'black' pavement, accentuates thermal stresses in the deck since a black cover absorbs more heat.
- An increase in pavement thickness reduces thermal stresses.
- 'Asphalt' has higher absorptivity and emissivity coefficients (0.90 and 0.92, respectively) compared to grey 'concrete' surfaces (0.50 and 0.88, respectively). Hence Asphaltic wearing course will transmit higher temperature to the deck. However, after the concrete wearing course has slightly aged and become dirty and black, it too will transmit higher temperature to the deck.
- Surface Condition of a steel girder in a composite deck can create very high (even more than 50°C) difference of temperature between steel and concrete if its surface is grey black oxidised (absorptivity 0.95) compared with a mere 14°C if its surface is white (absorptivity 0.15).

30.6 RECOMMENDATIONS

Until more dependable data are collected, the bridges in essentially hot climates, where shade temperatures reach 50 to 55°C, may be designed for the following thermal effects:

a) 'The effect of a 50°C overall rise (or overall fall) in body-mean-temperature (i.e. ± 25°C range)', *using half the instantaneous modulus of elasticity of concrete.* (Its effect is felt in arches, frames and fixed beams, not in simply supported and continuous beams which are not restrained from movement. However, this effect is required in them for estimating movements at bearings and expansion-joints.)

b) 'Eigenstresses resulting from the non-linearity of the temperature distribution across the deck depth' as shown in Fig. 30.9, *adopting the instantaneous modulus of elasticity of concrete.* The effect of re-radiation (soffit hotter than top) may be ignored owing to the predominant nature of temperature.



Note Eigenstress may be ignored when h < 25 cm

Fig. 30.9 *Recommended non-linear differential thermal gradients*

Same as above but across the thickness of abutment-leg in a frame, with front hotter.

c) 'Continuity stresses' resulting from the 'continuity moments' caused by the restraint to free hogging (or sagging) desire of the structure under a reversible 'lump' temperature difference of $T = \pm 12°C$ between the extreme fibres, *using the instantaneous modulus of elasticity of concrete.*

NOTE: Field observations of actual long term variations in the body-mean-temperatures and of the distribution of temperature across the deck-depth of different types of deck-sections, should be undertaken in order to refine the above recommendations. In this connection Reference 2 is useful. This work may be undertaken by National Road Research Institutes in collaboration with the relevant Ministry Authorities.

30.7 NUMERICAL EXAMPLES

A) Simply Supported Concrete Deck

Estimate the thermal stresses in a simply supported beam-and-slab concrete deck (instantaneous $E_c = 2.4 \times 10^5$ kg/cm^2). Representative 'beam' section is as shown in Fig. 30.10. Follow the Recommendations stated earlier in 30.6.

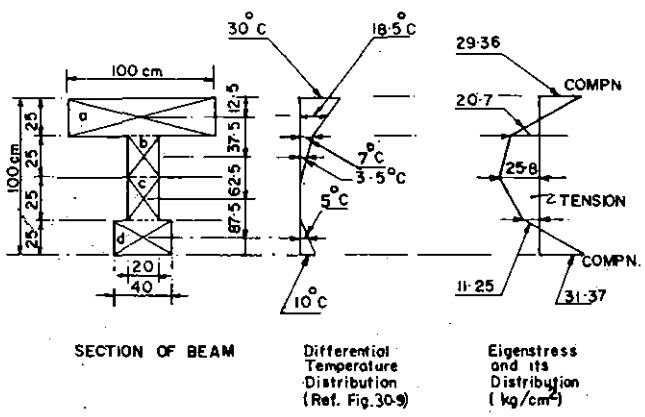


Fig. 30.10

Solution

Step 1 Stresses due to variation in body-mean-temperature are zero, since the deck is free to move longitudinally.

Step 2 Eigenstresses will exist since distribution of temperature across deck-dept is non-linear. These are estimated as shown in Table 30.1.

Refer to Eqs. (30.3) and (30.4) and Fig. 30.10.

Substituting in Eqs. (30.3) and (30.4) and taking $\alpha = 1.17 \times 10^{-5}/°C$:

$$\epsilon_0 \cdot (0.45) - \theta(0.16875) = 1.17 \times 10^{-5}(5.3)$$
$$\epsilon_0(0.16875) - \theta(0.10703) = 1.17 \times 10^{-5}(1.08125)$$

solving these, we get $\epsilon_0 = 2.2868 \times 10^{-4}$ and $\theta = 2.4235 \times 10^{-4}$ rad./m.

Hence eigenstress can be evaluated from Eq. (30.2) tabularly as shown in Table 30.2, taking:

$E_c = 2.4 \times 10^5$ kg./cm^2 $\quad \theta = 2.4235 \times 10^{-4}$ rad./m, $\varepsilon_0 = 2.2868 \times 10^{-4}$, and $\alpha = 1.17 \times 10^{-5}/°C$

The above calculated eigenstress values, f_{ei}, have been plotted in Fig. 30.10 for a pictorial view.

Step 3 Stresses due 'continuity effect' are zero, deck being simply supported and hence free to hog or sag.

Hence, total of the thermal stresses in this case (a simple span deck) is only the eigenstress as calculated in Step 2 above, shown in Fig. 30.10.

B) Continuous-Beam Concrete Deck

Estimate the thermal stresses in a prismatic 2-span continuous beam-and-slab concrete deck (instantaneous $E_c = 2.4 \times 10^5$ kg/cm^2). Representative 'beam' section is same as used in the previous simply supported deck illustration, shown in Fig. 30.10. Follow the Recommendations stated earlier.

Solution

Step 1 Stresses due to variation in body-mean-temperature are zero since the deck is free to move longitudinally.

Table 30.1

ZONE	*A* (m^2)	*y* (m)	*Ay* (m^3)	*Ay*^{2} (m^4)	*t* (°*C*)	*At*	*Aty*
a	$1.0 \times 0.25 = 0.25$	0.125	0.03125	0.00391	18.5	4.625	0.57812
b	$0.20 \times 0.25 = 0.05$	0.375	0.01875	0.00703	3.5	0.175	0.06563
c	$0.20 \times 0.25 = 0.05$	0.625	0.03125	0.01953	0	0	0
d	$0.40 \times 0.25 = 0.10$	0.875	0.08750	0.07656	5.0	0.500	0.43750
sum	0.45		0.16875	0.10703		5.30	1.08125
	ΣA		ΣAy	ΣAy^2		ΣAt	ΣAty

Table 30.2

y (m)	$y\theta$	t $(°C)$	αt	$f_{ci} = E_c(\epsilon_0 - y\theta - \alpha t)$ kg/cm^2
0	0	30	3.51×10^{-4}	−29.36 (compr.)
0.25	0.606×10^{-4}	7	0.819×10^{-4}	20.70 (tension)
0.50	1.212×10^{-4}	0	0	25.80 (tension)
0.75	1.818×10^{-4}	0	0	11.25 (tension)
1.00	2.424×10^{-4}	10	1.17×10^{-4}	−31.37 (compr.)

Step 2 Eigenstresses will exist since distribution of temperature across deck-depth is non-linear. These are estimated exactly as illustrated in the previous example and since section assumed is the same, the eigenstresses are also the same. These then are as shown in Fig. 30.10.

Step 3 Stresses due to 'continuity effect' will exist in the present case since a continuous beam, despite desiring to hog up (or sag down) freely between its abutments because of difference in extreme fibre temperatures, is prevented from doing so because of the presence of intermediate supports, which compel the beam to hog (or sag) only in between the consecutive supports.

First the moments due to this 'continuity effect' may be worked out as explained earlier and in Fig. 30.4. This moment diagram, in the present case, is then as shown in Fig. 30.11, and the 'continuity stress' at any fibre at any section will be obtained by dividing the 'moment' at that section by the appropriate section modulus to the concerned fibre.

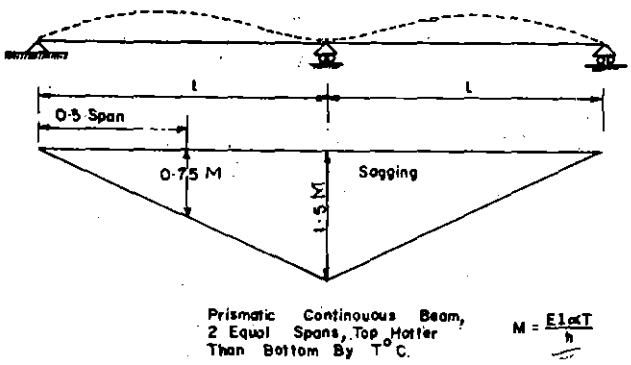


Fig. 30.11 *'Continuity' moment diagram*

As an example, let us find these stresses at the central support section (where second moment of area of the section is $I = 46 \times 10^5$ cm^4 and $y_t = 37.5$ cm, $y_b = 62.5$ cm)

Continuity moment $= 1.5EI\alpha T/h$

$$= \frac{1.5 \times (2.4 \times 10^5) \times (46 \times 10^5) \times (1.7 \times 10^{-5}) \times (\pm 12)}{100}$$

$= 23.25 \times 10^5$ kg.cm (reversible)

Continuity stresses—

$$\text{at top: } \pm \frac{23.25 \times 10^5}{46 \times 10^5} \times 37.5 = \pm 18.95 \text{ kg/cm}^2$$

$$\text{at bottom: } \pm \frac{23.25 \times 10^5}{46 \times 10^5} \times 62.5 = \pm 31.59 \text{ kg/cm}^2$$

(varying linearly in between)

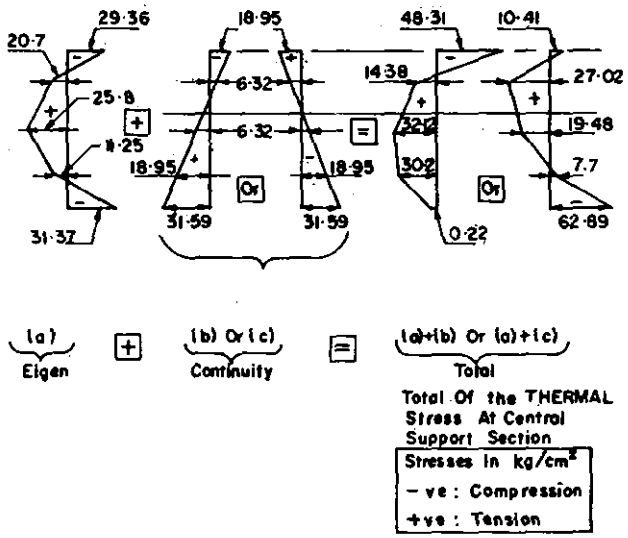


Fig. 30.12

Hence the total thermal stress configuration at the central support section, being the superposition of the above calculated eigenstresses and the continuity stresses, is as shown in Fig. 30.12. This may be checked at other critical sections too.

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1. Priestly, MJN, 'Thermal Gradients in Bridges—Some Considerations', *New Zealand Eng.*, Wellington, Vol. 27, No: 7, July, 1972.
2. Emerson, M., 'The calculation of distribution of temperature in Bridges', *UK Transport and Road Research Laboratory*, Report LR. 561, 1973.
3. Hunt, Bruce and Cook, 'Thermal calculations for Bridge Design', Proc., ASCE, Vol. 101, ST9, Sept. 1975
4. Hambly, EC 'Temperature distributions and stresses in Concrete Bridges', *The Structural Engineer*, Vol 56A, No: 5, May 1978.

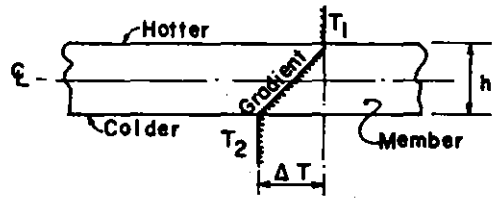
5. Priestly, MJN, 'Design of Concrete Bridges for temperature gradients', *ACI Journal*, May 1978.
6. Leonhardt, *Ribschaden an Betonbrucken—Ursachen und Abhilfe* (i.e. Crack damages at Concrete Bridges—cause and remedy), Beton und Stahlbetonbau, 2/1979.
7. El Badri and Amin Ghali, 'Nonlinear temperature distribution and its effects on bridges', IABSE Zurich, *Periodica* 3/1983, Aug. 1983.

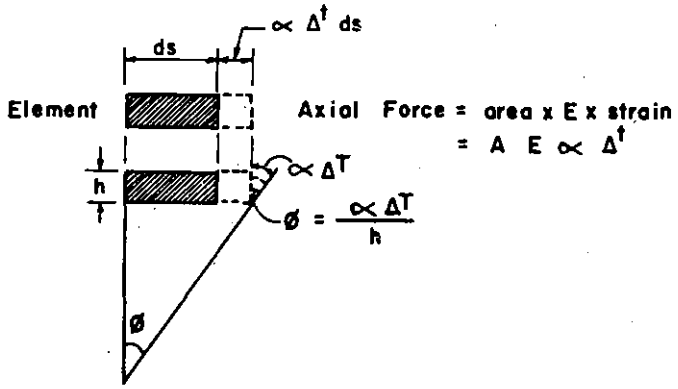
ANNEXURE

Estimation of the Effects of "Change in Body Mean Temperature" and "Unequal Extreme Fibre Temperatures" (by the Flexibility Method)

Assumptions:

- temperature varies linearly (note that the effect of non-linearity of temperature variation through section-depth is considered separately in Eigenstress calculation, described in the body of the chapter).
- no temperature 'discontinuity' between member surface and air.





$$\phi = \text{curvature} = \frac{1}{R}$$

$$\frac{M}{I} = \frac{E}{R}, \quad M = EI\phi$$

$$= EI\frac{\alpha \Delta T}{h}$$

$T_1, T_2 \ldots$ extreme fibre temperatures

$\Delta T = T_1 - T_2$ = difference in extreme fibre temperatures

$T_0 \ldots$ initial 'mean' temperature

$T_1 = T_0$ + maximum rise in temperature

$T_2 = T_0$ − maximum fall in temperature

$\Delta t = \left(\frac{T_1 + T_2}{2} - T_0\right)$ = change in body mean temperature

α = coefficient of thermal expansion/contraction of the material

h = section − depth

- axial force (thrust, tensile/compressive) = section area $A \times E \times \alpha \Delta t$
- bending moment $M = EI\alpha\Delta T/h$

These 'bending moment' $(EI\alpha\Delta T/h)$ and 'thrust' $(AE\alpha\Delta t)$ diagrams, when drawn along the structure, represent the 'applied load' moment and thrust diagrams on the statically made determinate structure. These are 'akin' to m_0 and n_0 diagrams explained in the Flexibility Method given earlier, but do not physically exist if the structure is made determinate (just as there is no m_0 diagram in case of differential settlement of supports in a continuous beam). Rest of the procedure is same as in the regular Flexibility Method. The following numerical example explains the process in a simple workman-like manner. However, thrust energy must be considered in evaluating the influence coefficients, viz.

$$V_{rs} = \int \frac{m_r m_s}{EI}\, ds + \int \frac{n_r n_s}{EA}\, ds$$

and

$$V_{ro} = \int \frac{m_r m_o}{EI}\, ds + \int \frac{n_r n_o}{EA}\, ds$$

The general compatibility equations of elastic stability remaining:

$$V_{ro} + V_{rs} p_s = 0 \quad \begin{cases} r \text{ ranging from 1 to } n, \\ s \text{ ranging from 1 to } n, \\ n \text{ being the number of statical-indeterminancies in the structure.} \end{cases}$$

Here, consequently,

$$V_{ro} = \int \frac{m_r(EI\alpha\Delta T/h)}{EI}\, ds + \int \frac{n_r(AE\alpha\Delta t)}{EA}\, ds$$

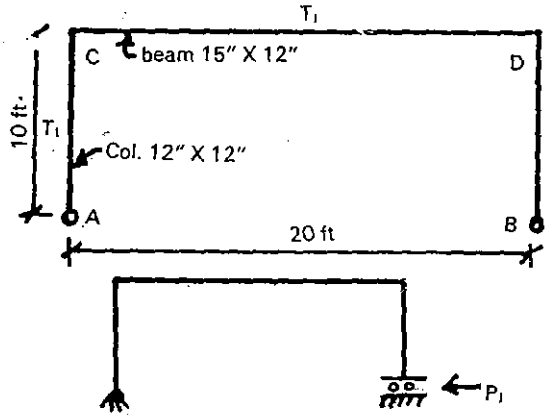
or simply, $V_{ro} = \alpha \int m_r(\Delta T/h)\, ds + \alpha \int n_r(\Delta t)\, ds$

Numerical Example

A two hinged portal in reinforced concrete,* 10ft. high, 20ft. span, T_1 on outsides and T_2 on insides, columns 12″ × 12″, transome 15″ × 12″,

$$T_1 = T_0 + 35°\text{F}, \quad T_2 = T_0 - 15°\text{F},$$
$$\Delta T = T_1 - T_2 = 50°\text{F}$$

* Actually in the case of structural concrete the Flexibility Analysis shown in this Example should infact be done separately for the $(\Delta T/h)$ case using instantaneous E (this being the case of "Temperature difference between extreme fibres") and then separately for the (Δt) case taking E as half of the instantaneous E value (this being the case of "change in body-mean-temperature"—a long term effect).



$$\Delta t = \frac{T_1 + T_2}{2} - T_0 = 10°\text{F}, \quad \alpha = \frac{1}{180,000} \text{ per°F}$$

$h_{\text{beam}} = h_b = 1.25\text{ft.}, \quad h_{\text{col.}} = h_c = 1.0ft,$

breadth $= 1.0$ft.

$E = 2 \times 10^6$ psi or 288×10^6 psf

(assumed constant for this example),

$$I_{\text{beam}} = I_b = \frac{1}{12} \times 1(1.25)^3 = 0.1628 \text{ ft}^4$$

$$I_{\text{col.}} = I_c = \frac{1}{12} \times 1(1)^3 = 0.0833 \text{ ft}^4$$

$\therefore I_b = 1.95 I_c$

$A_{\text{beam}} = A_b = 1.25 \times 1 = 1.25 \text{ ft}^2$

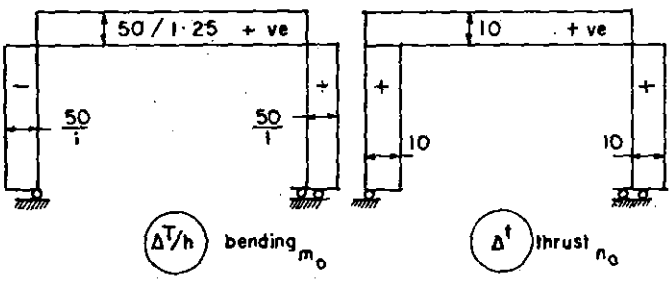
$A_{\text{col.}} = A_c = 1 \times 1 = 1.00 \text{ ft}^2$

$\therefore A_b = 1.25 A_c$

$\alpha_s = 3(M - N + 1) - R = 3(4 - 4 + 1) - 2 = 1 \ldots$

once statically indeterminate. (i.e. $n = 1$)

Introduce one release, a shear biaction p_1, at (say) right side foot:



$\Delta T/h$ and Δt diagrams are as shown. Due to unit p_1 the m_1 and n_1 diagrams are as shown.

$$V_{10} + V_{11} p_1 = 0, \quad \therefore p_1 = \frac{-V_{10}}{V_{11}}$$

$$V_{10} = \int \frac{m_1 \cdot (EI \propto \Delta T/h)}{EI} ds + \int \frac{n_1 \cdot (EA \propto \Delta t)}{EA} \cdot ds$$

$$= \alpha \left[\int m_1 \cdot \Delta^T/h \cdot ds + \int n_1 \cdot \Delta^t \cdot ds \right]$$

$$= \alpha \left[\left\{ 2 \times \frac{10}{6} \left(0 + 4 \times \frac{-10}{2} \times \frac{-50}{1} + (-10) \times \frac{-50}{1} \right) \right\} + \left\{ \frac{20}{6} \left(\frac{50}{1.25} \times 10 + 4 \times \frac{50}{1.25} \times 10 + \frac{50}{1.25} \times 10 \right) \right\} + \left\{ \frac{20}{6} (10 \times (-1) + 4 \times 10 \times (-1) + 10 \times (-1)) \right\} \right]$$

$$= +0.07111$$

$$V_{11} = \int \frac{m_1 m_1}{EI} ds + \int \frac{n_1 n_1}{EA} ds$$

$$= \left[\left\{ 2 \times \frac{10}{6EI_c} \left(0 + 4 \times \frac{-10}{2} \times \frac{-10}{2} + (-10)(-10) \right) + \frac{20}{6EI_b} (10 \times 10 + 4 \times 10 \times 10 + 10 \times 10) \right\} + \left\{ \frac{20}{6EA_b} (-1 \times -1 + 4 \times (-1)(-1) + (-1)(-1)) \right\} \right]$$

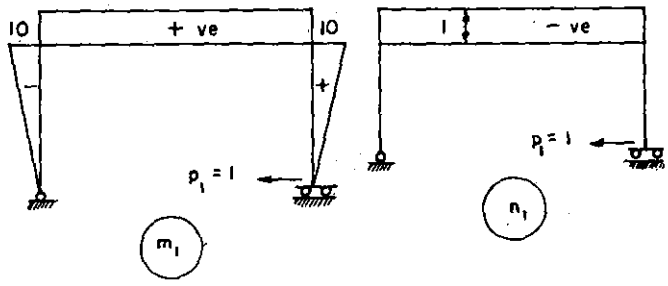
$$= \left(\frac{1692.3}{EI_c} + \frac{16}{EA_c} \right)$$

Substituting the values for E, I_c, A_c and finally the values of V_{10} and V_{11}, we finally get $p_1 = -1008$ lb

Hence, as a result of the above noted 'change in body mean temperature' and 'the temperature difference between extreme fibres':

moment at $A = m_1 p_1 = 0$, at $B = m_1 p_1 = 0$, at C in $AC = m_1 p_1 = (-10)(-1008) = 10,080$ lb.ft

at C and D in $CD = m_1 p_1 = 10 \times (-1008) = -10,080$ lb.ft.



at D in $BD = m_1 p_1 = 10 \times (-1008) = -10,080$ lb.ft.

thrust at $A = n_1 p_1 = 0$, at $B = n_1 p_1 = 0$, at C in AC and D in $BD = n_1 p_1 = 0$

at C and D in $CD = n_1 p_1 = (-1)(-1008) = 1008$ lbs

shear at any section in $AC = s_1 p_1 = \left(-\frac{10}{10} \right) (-1008) = 1008$ lbs

at any section in $CD = s_1 p_1 = 0 \times (-1008) = 0$

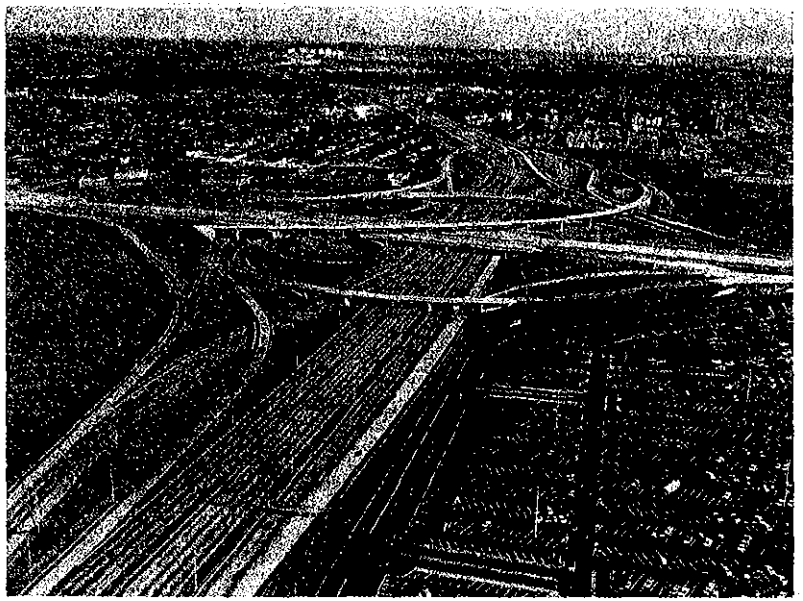


Fig. 31.1 *The Spadina interchange on Highway 401, Toronto*

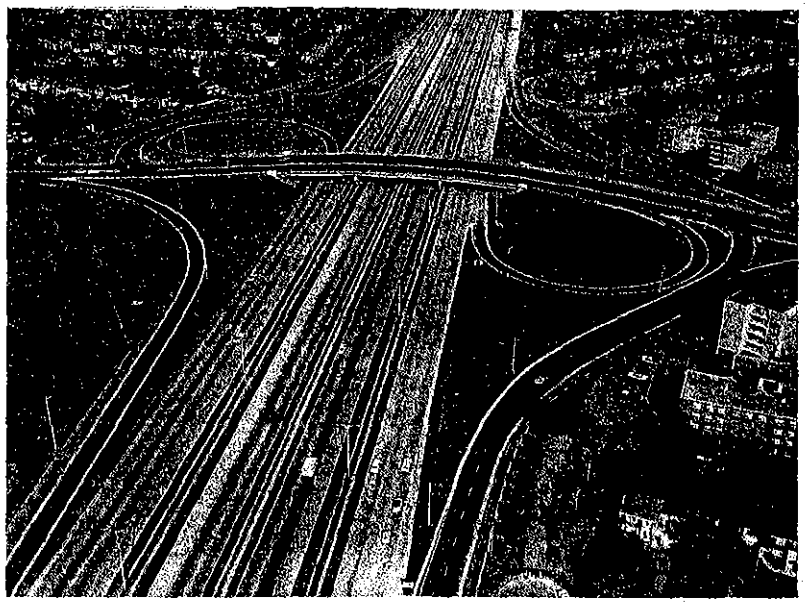


Fig. 31.2 *Intersection of Highway 401 and Avenue Road, Toronto*

Fig. 31.3 *The interchange at Highways 400 and 401, Toronto*

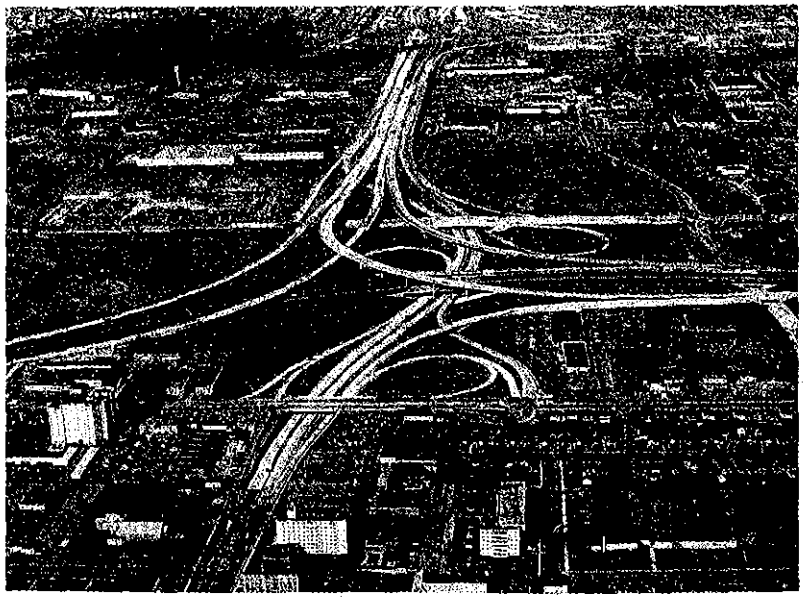


Fig. 31.4 *Highway 27 and Queen Elizabeth way interchange, Toronto*

(Photographs courtesy : Dept. Transptn. Commncn., Ontario)

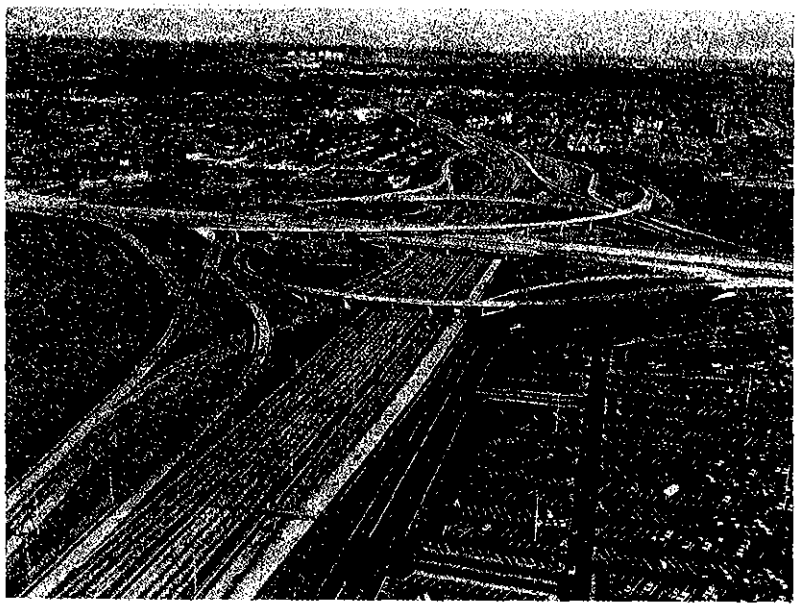


Fig. 31.1 *The Spadina interchange on Highway 401, Toronto*

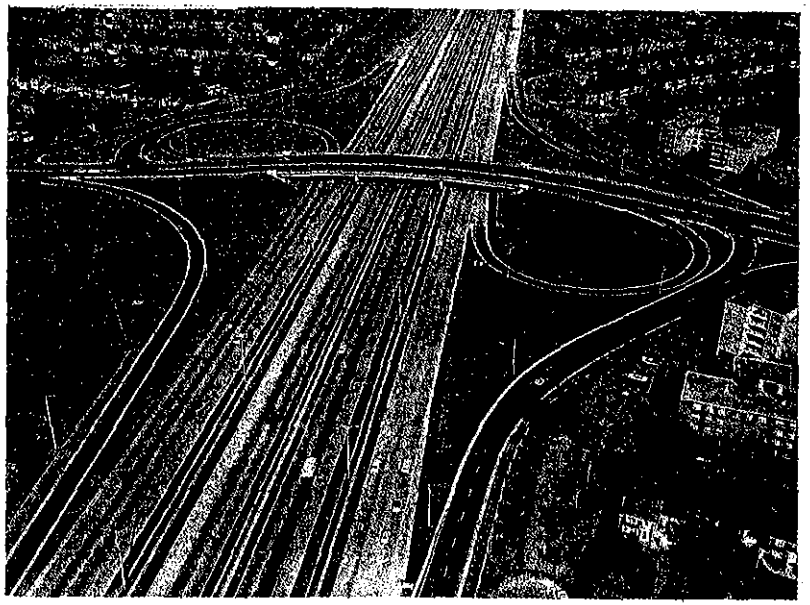


Fig. 31.2 *Intersection of Highway 401 and Avenue Road, Toronto*

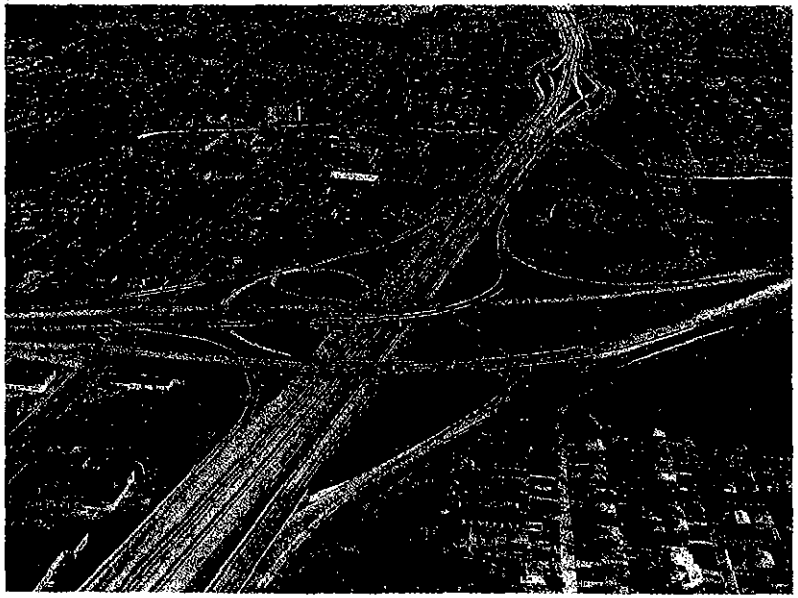


Fig. 31.3 *The interchange at Highways 400 and 401, Toronto*

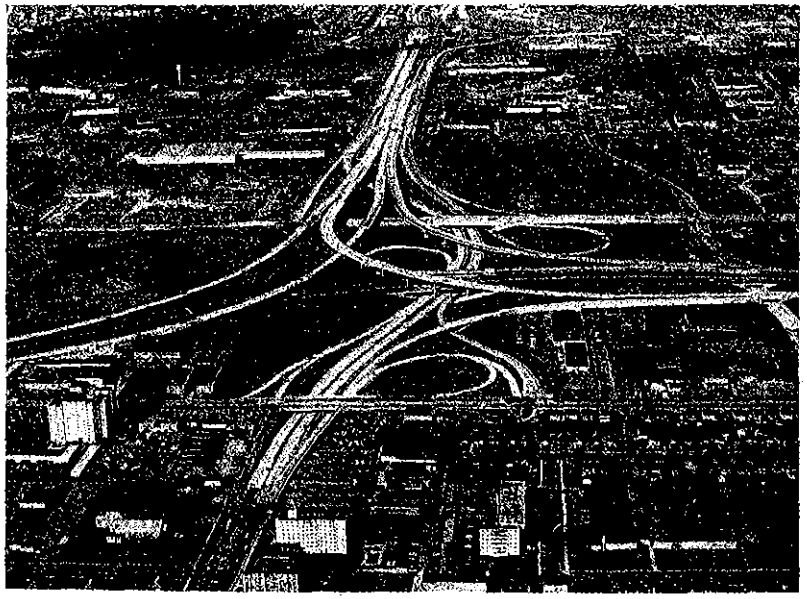


Fig. 31.4 *Highway 27 and Queen Elizabeth way interchange, Toronto*

(Photographs courtesy : Dept. Transptn. Commncn., Ontario)

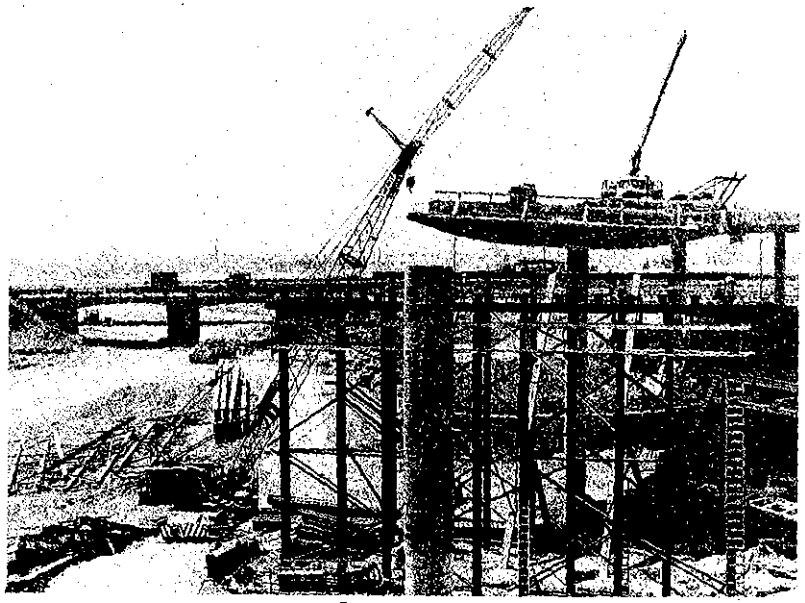


Fig. 31.5 *A bridge at Highway 27 and Queen Elizabeth way interchange, Toronto (under construction)*

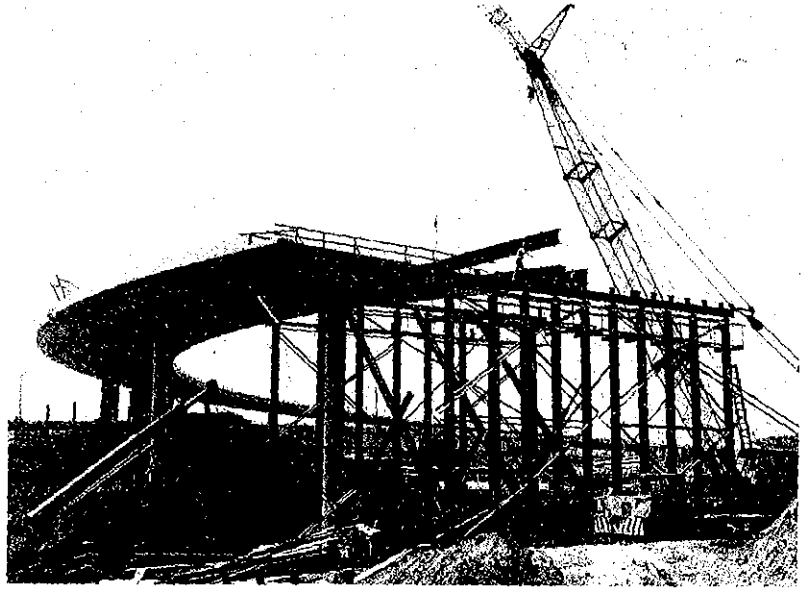


Fig. 31.6 *The steel frame of the bridge at Highway 27 and Queen Elizabeth way interchange, Toronto (temporary structure)*

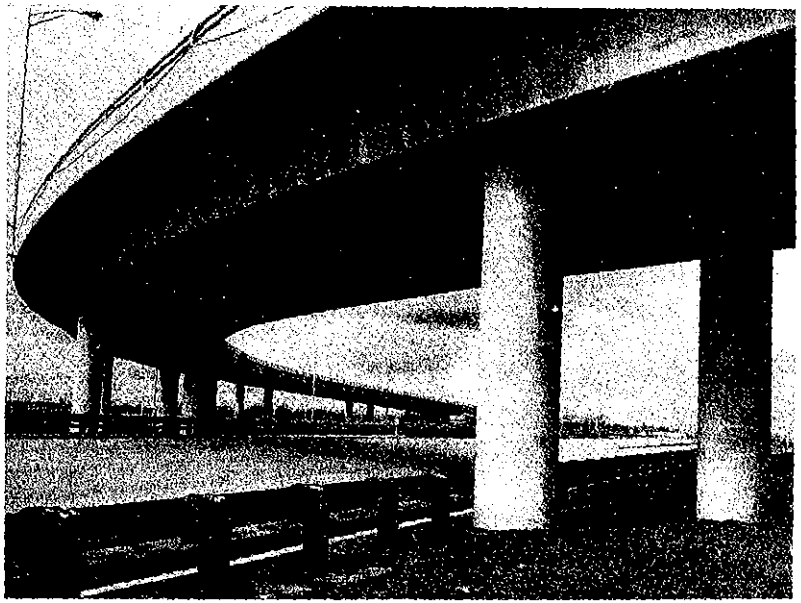


Fig. 31.7 *A bridge at junction of Highways 400 and 401, Toronto*

Fig. 31.8 *A bridge at the Spadina interchange of Highway 401, Toronto*

(Photographs courtesy : Dept. Transptn. Commncn., Ontario)

CHAPTER 31

Transverse Analysis of Some Typical Concrete Deck-Sections, i.e. Analysing a Deck Cross-section

31.1 BACKGROUND

With the development of modern highway networks, involving complex interchanges at their crossings, the bridge designer today is frequently confronted with the problem of bridging skewed and sloping gaps, which may, more often than not, have sweeping curves in plan. This factor, coupled with the stringent requirements for sight distance and clearance, often compels the designer to consider newer deck sections for meeting the conditions both aesthetically and economically. Needless to say, the designer can tackle the problem with the aid of the digital computer but for which, it would have been impossible to speedily analyse the combination of continuity, curve and skew.

The days are past when a highway would be realigned at a crossing in order to build a bridge according to the designer's conventional methods of analysis. Now the designer has to explore more refined techniques of calculation to fit in with the complicated geometry of the bridge alignment and the complex behaviour of the new types of deck sections. It would be almost unusual these days to think of bridging at highway intersection gap by the good old girder and slab section, except at right bridge crossings with almost no restrictions on the section depth, etc.

The present-day trend in Europe and in the American continent is to use box-sections. These sections enable hogging and sagging moments to be resisted efficiently, the spans being almost always continuous. Moreover, such sections are also sturdy enough to withstand torsion, and have more efficient load distribution characteristics. Figures 31.1 to 31.8 (art plates) illustrate some modern highway inter-section crossings using the latest types of prestressed concrete box-section bridges.

The current methods for the analysis of the transverse slab moments in bridge decks vary from the straight forward 'continuous beam on rigid supports' analysis to the 'plate' and 'grid' harmonic analyses, depending on the type of deck. For reasonably realistic answers by some of these methods, the use of a digital computer becomes essential, and this either leaves an average designer helpless, or he has to indulge in glorified guesswork.

It is the purpose here to try and derive the transverse slab moments by relatively easily applied and yet reasonably accurate procedures. The designer, nevertheless, will have to exercise his own judgement in particular cases. The results can only be as accurate as the validity of the assumptions that are made in deriving them. Given here are the methods of transverse analysis of five deck sections. Each of these methods of analysis is supplemented by an illustrative numerical example of an actual bridge.

Some Typical Deck Sections

Some of the more popular types of prestressed concrete deck sections are shown in Figs. 31.9 to 31.13. Figure 31.9 shows a twin-box-section bridge deck, interconnection being provided only by a relatively flexible top slap. This section looks slender and lends itself very efficiently to precast segmental construction. Figure 31.10 shows a multicell box-section with webs of equal thickness. Such a section is eminently suitable for in-situ construction, and has inherent strength against torsion and moment variation. It is generally made solid at the supports. Figure 31.11 represents a multicell box-section with a stub central web. It has all the advantages of the multicell box-section with webs of equal thickness and is particularly suited for bridges involving single column-pier supports. Figure 31.12 illustrates a two-cell box-section with solid central spine. This too has all the advantages of the multicell box-section with stub central web, and is suited for a bridge carrying less number of traffic lanes. Figure 31.13 shows a voided deck section. This type of bridge deck is particularly suitable where the depth of construction is severely restricted. This deck section may also be referred to as a voided slab deck. All voids need not be of the same size and even the same void need not be of a constant diameter along the length of the bridge. Void formers in hard cardboard, or thin sheet-metal are used for forming the voids in in-situ construction. Invariably such bridge decks are made solid at the piers and the abutments.

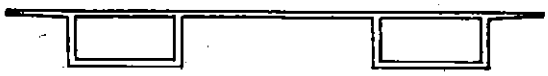


Fig. 31.9 *Twin-box-section bridge deck with interconnecting top slab*

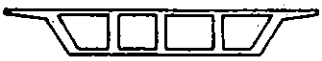


Fig. 31.10 *Multicell box-section with webs of equal thickness*

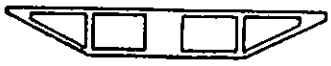


Fig. 31.11 *Multicell box-section with a stub central web*

Fig. 31.12 *Two-cell box-section with solid central spine*

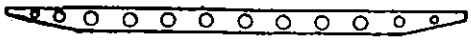


Fig. 31.13 *Voided deck-section*

CASE (A): Twin-box-section with an interconnecting top slab

Figure 31.14 is illustrative of a twin-box-section deck in which the boxes are interconnected only by a top slab. The two boxes can be loaded in different proportions by the applied live load, which itself may be variable along the length and width of the bridge. This can lead to heavy reversal of deformations in the deck slab, whereby the support and the span sections C and D may in fact be subjected to sagging and hogging bending moments, respectively. The cantilever section A, and the span section B may be analysed by conventional methods since they behave more like similar sections in a normal continuous beam on rigid supports, by virtue of the geometry of the deck section. For the critical sections C and D under applied live load, a more elaborate analysis is necessary because of the relative flexibility of the slab interconnecting the two boxes. A method is presented here for this analysis, in which the live load is placed for the maximum bending effect at section C. The procedure may be repeated for any other load position.

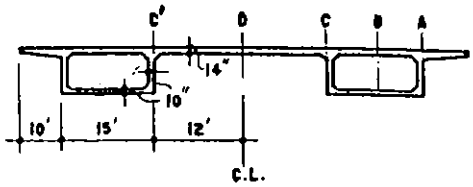


Fig. 31.14 *Twin-box-section deck with the boxes interconnected by the top slab*

• ***Analysis*** : Any particular distributed load may at any section be resolved into two loads, one of which is symmetrical and the other asymmetrical in the cross-section of the deck (Fig. 31.15). Under the symmetrical load, the system will deform symmetrically about the axis of symmetry of the cross-section, so that the shear there will be zero. Under the asymmetrical load, the system will deform asymmetrically about the axis of symmetry of the cross-section, so that the bending moment there is zero. If the system is 'cut' through the axis of symmetry, all along its length, it can be rendered statically determinate by introducing two redundant bi-actions q_x and M_x, the vertical shear and bending moment respectively. The third stress resultant, namely, the thrust bi-action, may be neglected here since the two boxes will not roll inwards or outwards laterally after introducing the 'cut'. The slab spanning in the transverse direction is assumed to be replaced by transverse beams of width dx, whose torsional rigidity is disregarded. The redundants q_x and M_x are functions of the longitudinal ordinate x and may be expressed as compound sine-curves as given below:

$$q_x = q_1 + q_2\left(\alpha \sin\frac{\pi x}{L} + \beta \sin\frac{3\pi x}{L}\right)$$

where $\alpha + \beta = 1$, L is the longitudinal span.

$$M_x = M_1 + M_2\left(\gamma \sin\frac{\pi x}{L} + \delta \sin\frac{3\pi x}{L}\right)$$

where $\gamma + \delta = 1$.

The two redundants may be evaluated from the two compatibility equations for continuity at the cut. In the case of symmetric loading, the slope at midspan of the slab is zero, while in the asymmetric case, the deflection at midspan of the slab is zero. In the symmetric case of loading, the rotations caused at the cut by the load and by M_x will sum up to zero, and in the asymmetric case of loading, the deflections caused at the cut by the load and by q_x will sum up to zero.

It should be noted that since the expressions for q_x and M_x themselves involve three unknowns each, there will in

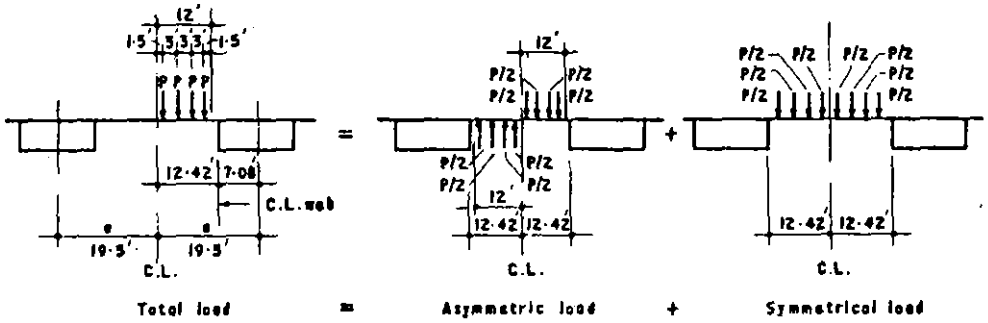


Fig. 31.15 *Total distributed load at a section resolved into asymmetrical and symmetrical loads*

fact be three unknowns in each of the two compatibility equations. These may, however, be solved by substituting for three values of x $\left(\text{say, } x = 0, x = \frac{L}{6} \text{ and } x = \frac{L}{2}\right)$ in each compatibility equation, thus getting three simultaneous equations for q_x and three for M_x. Solving these will completely define the two redundants.

• ***Wheel load as a mathematical function***: The live load under consideration here is the one according to the British highway bridge loading. This is placed on the bridge longitudinally as shown in Fig. 31.16. Transversely it is so placed on the bridge-deck as to create the maximum gap at the cut. It is assumed that a 45-ton HB axle of four wheels is dispersed transversely over a width of 12 ft (3.66 m). The applied live load may be expressed as a smooth continuous loading function in terms of Fourier series as

$$p(x) = \frac{2P}{L}\left(3.7398 \sin\frac{\pi x}{L} - 1.8942 \sin\frac{3\pi x}{L}\right)$$ on two boxes; details are given in Annexure 1.

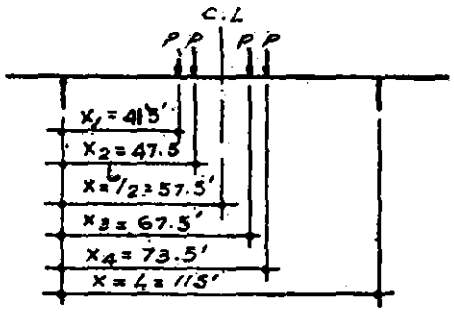


Fig. 31.16 *HB vehicle placed on bridge longitudinally*

• ***Gaps caused at the cut by various loads***: The total deflectional gap caused at the cut by the asymmetrical load, Fig. 31.17, comprises (a) simple vertical deflection of box beam, (b) deflection due to twist in the box beam, and (c) deflection due to local bending of the cantilever. The total deflectional gap caused at the cut by q_x, Fig. 31.18, consists of (d) simple vertical deflection of box beam, (e) deflection due to twist in the box beam and (f) deflection due to local bending of the cantilever. Hence one compatibility equation is formed as given below:

$$(a) + (b) + (c) = (d) + (e) + (f) \qquad (31.1)$$

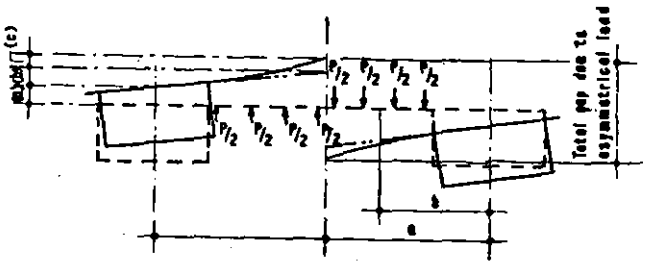


Fig. 31.17 *Total deflectional gap of the 'cut' due to asymmetrical load*

Fig. 31.18 *Total deflectional gap at the 'cut' due to q_x*

The total rotational gap at the cut by the symmetrical load, Fig. 31.19, comprises (g) twist rotation in the box beam, and (h) flexural rotation due to local bending of the cantilever. The total rotational gap caused at the cut by M_x, Fig. 31.20 consists of (j) twist rotation in the box beam, and (k) flexural rotation due to local bending of the cantilever.

A second compatibility equation is formed as follows:

$$(g) + (h) = (j) + (k) \qquad (31.2)$$

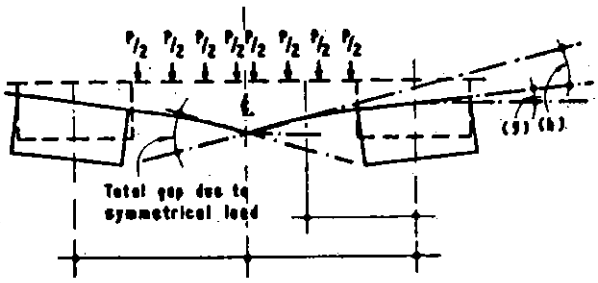


Fig. 31.19 *Total rotational gap at the 'cut' due to symmetrical load*

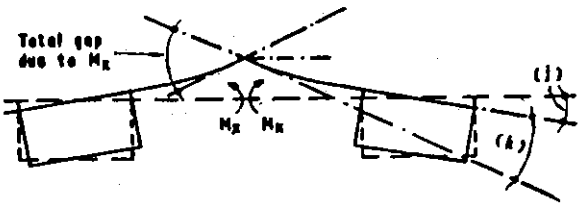


Fig. 31.20 *Total rotational gap at the 'cut' due to M_x*

• ***Formulae for various deflections and rotations***: Expressions for the various deflections and rotations, referred to as (a), (b), (c), (d), (e) and (f), and (g), (h), (j) and (k) respectively, are worked out in Annexure 2, and their final values are given as follows:

$$(a) = \frac{PL^3}{81\pi^4 EI_b}\phi_1 + \frac{PL}{15.39EI_b}\phi_2$$

$$(b) = \frac{PabL}{9\pi^2 GK}\phi_3$$

$$(c) = \frac{Pl^4}{96LEI_c}\phi_4$$

$$(d) = \frac{q_1}{67.13EI_b}\phi_6 + \frac{q_2L^4}{81\pi^4 EI_b}\phi_5 + \frac{(2\alpha - 1)L^2 q_2}{62.25EI_b}\phi_7$$

$$(e) = \frac{a^2 q_1}{2GK}\phi_8 + \frac{a^2 q_2 L^2}{9\pi^2 GK}\phi_9$$

$$(f) = \frac{q_1 l^3}{3EI_c} + \frac{q_2 l^3}{3EI_c}\phi_{10}$$

$$(g) = \frac{PbL}{9\pi^2 GK}\phi^3$$

$$(h) = \frac{Pl^3}{72LEI_c}\phi_4$$

$$(j) = \frac{M_1}{2GK}\phi_8 + \frac{M_2 L^2}{9\pi^2 GK}\phi_{11}$$

$$(k) = \frac{M_1 l}{EI_c} + \frac{M_2 l}{EI_c}\phi_{12}$$

where $\phi_1 = 302.8 \sin\frac{\pi x}{L} - 1.8942 \sin\frac{3\pi x}{L}$

$\phi_2 = x(x - L)$

$\phi_3 = 33.62 \sin\frac{\pi x}{L} - 1.8942 \sin\frac{3\pi x}{L}$

$\phi_4 = 3.7398 \sin\frac{\pi x}{L} - 1.8942 \sin\frac{3\pi x}{L}$

$\phi_5 = 81\alpha \sin\frac{\pi x}{L} + (1 - \alpha)\sin\frac{3\pi x}{L}$

$\phi_6 = x(2.8x^3 - 5.6Lx^2 + 1.8L^2 x + L^3)$

$\phi_7 = x(x - L)$

$\phi_8 = x(L - x)$

$\phi_9 = 9\alpha \sin\frac{\pi x}{L} + (1 - \alpha)\sin\frac{3\pi x}{L}$

$\phi_{10} = \alpha \sin\frac{\pi x}{L} + (1 - \alpha)\sin\frac{3\pi x}{L}$

$\phi_{11} = 9\gamma \sin\frac{\pi x}{L} + (1 - \gamma)\sin\frac{3\pi x}{L}$

$\phi_{12} = \gamma \sin\frac{\pi x}{L} + (1 - \gamma)\sin\frac{3\pi x}{L}$

a = distance from centre-line of box to centre-line of cut

b = distance from centre-line of box to centre-line of applied load

l = half of the clear distance between two boxes

L = longitudinal span of the deck

EI = flexural rigidity (suffix b for box beam and c for cantilever)

GK = torsional rigidity of a box beam

For the example under illustration

l = 12 ft

L = 115 ft

a = 19.5 ft

b = 13.5 ft

p = 45 tons

EI_b = 142000×10^6 lb ft^2 (per box beam)

EI_c = 57.15×10^6 lb ft^2/ft width (per cantilever)

GK = 144500×10^6 lb ft^2 per box beam, Annexure 3.

• ***Evaluation of various deflections and rotations***: Substituting for P, a, b, L, l, EI_b, EI_c, and GK, in the formulae derived earlier for various deflections and rotations,

$(a) = 10^{-10}\ (1367767.2957\phi_1 + 53061.9718\phi_2)$

$(b) = 10^{-10}\ (2377022.2420\phi_3)$

$(c) = 10^{-10}\ (33128380.6900\phi_4)$

$(d) = 10^{-10}\ [0.0010q_1\phi_6 + 1560.4488\ q_2\phi_5 + 14.9612\ (2\alpha - 1)q_2\phi_7]$

$(e) = 10^{-10}\ (13.1522 q_1\phi_8 + 0.2961\ q_2\phi_9)$
$(f) = 10^{-10}\ [100787.4015(q_1 + q_2\phi_{10})]$
$(g) = 10^{-10}\ (121898.00\ \phi_3)$
$(h) = 10^{-10}\ (3680000.00\ \phi_4)$
$(j) = 10^{-10}\ (0.0346\ M_1\phi_8 + 10.3015\ M_2\phi_{11})$
$(k) = 10^{-10}\ (2099.7375 M_1 + 2099.7375 M_2\phi_{12})$

Substituting these values in compatibility Eqs. (31.1) and (31.2)

$$q_1\Delta_1 + q_2\Delta_2 = 53061.9718\ \Delta_3 \qquad (31.3)$$
$$M_1\Delta_4 + M_2\Delta_5 = 121898.00\ \Delta_6 \qquad (31.4)$$

where $\Delta_1 = (0.001\phi_6 + 13.1522\phi_8 + 100787.4015)$
$\Delta_2 = [1560.4488\ \phi_5 + 14.9612\ (2\alpha - 1)\ \phi_7 + 0.296\ \phi_9 + 100787.4015\ \phi_{10}]$
$\Delta_3 = (25.776\ \phi_1 + \phi_2 + 44.797\phi_3 + 624.33\ \phi_4)$
$\Delta_4 = (0.0346\ \phi_8 + 2099.7375)$
$\Delta_5 = (10.3015\ \phi_{11} + 2099.7375\ \phi_{12})$
and $\Delta_6 = (\phi_3 + 30.21\ \phi_4)$

Equation (31.3) contains three unknowns, namely q_1, q_2 and α, and, therefore, it is essential to develop three simultaneous equations in order to solve for the three unknowns. These three simultaneous equations may be obtained by substituting for Δ_1, Δ_2 and Δ_3, in it at three locations of x, say at $x = 0$, $\frac{L}{6}$, and $\frac{L}{2}$. Equation (31.4) also contains three unknowns, namely, M_1, M_2 and γ, for evaluating which it is again essential to generate three simultaneous equations. This may be done by substituting for Δ_4, Δ_5 and Δ_6 at the same three locations of x as in the case of Eq. (31.3) as explained above.

• ***Evaluation of the various ϕ and Δ functions***: Values of various ϕ functions and Δ functions at $x = 0$, $x = \frac{L}{6}$ and $x = \frac{L}{2}$, as computed from their relevant formulae derived earlier, are calculated and are given in Table 31.1.

Solving simultaneously the three equations generated from Eq. (31.3) by substituting for Δ_1, Δ_2, and Δ_3 in it, at $x = 0$, $x = L/6$, and $x = L/2$:

$$q_1 = 0$$
$$q_2 = 1558.51 \text{ lb/ft width}$$
$$\alpha = 0.8905$$

Hence the redundant q_x is defined and may be expressed as $q_x = 1558.51\left(0.8905 \sin\frac{\pi x}{L} + 0.1095 \sin\frac{3\pi x}{L}\right)$

Similarly solving simultaneously the three equations generated from Eq. (31.4) by substituting in it for Δ_4, Δ_5, and Δ_6, at $x = 0$, $x = \frac{L}{6}$, and $x = \frac{L}{2}$:

Table 31.1 *Values of various ϕ and Δ functions*

Function	At $x = 0$	At $x = L/6$	At $x = L/2$
ϕ_1	0	149.5058	304.6942
ϕ_2	0	−1836.8055	−3306.2500
ϕ_3	0	14.9158	35.5142
ϕ_4	0	−0.0243	5.6340
ϕ_5	0	$39.5\alpha + 1$	$82\alpha - 1$
ϕ_6	0	33739133.2320	74332765.6250
ϕ_7	0	−1836.8055	−3306.2500
ϕ_8	0	1836.8055	3306.2500
ϕ_9	0	$3.5\alpha + 1$	$10\alpha - 1$
ϕ_{10}	0	$1 - 0.5\alpha$	$2\alpha - 1$
ϕ_{11}	0	$3.5\gamma + 1$	$10\gamma - 1$
ϕ_{12}	0	$1 - 0.5\gamma$	$2\gamma - 1$
Δ_1	100787.4015	not required	not required
Δ_2	0	$(-43716.5657\alpha + 129828.961)$	$(428465.5\alpha - 52882.679)$
Δ_3	0	2669.8679	9655.9525
Δ_4	2099.7375	not required	not required
Δ_5	0	$(2110.039 - 1013.814\gamma)$	$(-2110.039 + 4302.49\gamma)$
Δ_6	0	14.1817	205.7173

$$M_1 = 0$$
$$M_2 = 4735.40 \text{ lb ft/ft width}$$
$$\gamma = 1.7212$$

Hence the redundant M_x is defined and may be expressed as

$$M_x = 4735.40\left(1.7212 \sin\frac{\pi x}{L} - 0.7212 \sin\frac{3\pi x}{L}\right)$$

Now, with q_x and M_x determined, and the applied live load $p(x)$ already defined, the bending moment and shear at the critical section C at any location along the span can be worked out. For example, at the midspan location, *i.e.*, at $x = L/2$:

$$p(x) = \frac{2P}{L}\left(3.7398 \sin\frac{\pi x}{L} - 1.8942 \sin\frac{3\pi x}{L}\right)$$
$$= \frac{2 \times 45 \times 2240}{115}(3.7398 + 1.8942)$$
$$= 9865 \text{ lb per ft width}$$

i.e., as a uniformly distributed load $w = \frac{9865}{12}$

$$= 822 \text{ lb per ft run per ft width}$$
$$q_x = 1558.51\left(0.8905 \sin\frac{\pi x}{L} + 0.1095 \sin\frac{3\pi x}{L}\right)$$
$$= 1558.51(0.8905 - 0.1095)$$
$$= 1218 \text{ lb/ft width}$$
$$M_x = 4735.40\left(1.7212 \sin\frac{\pi x}{L} - 0.7212 \sin\frac{3\pi x}{L}\right)$$

$= 4735.40(1.7212 + 0.7212)$
$= 11550$ lbft./ft.width

Hence total bending moment at C, Fig. 31.21

$$= -\frac{wl^2}{2} + q_x l + M_x$$
$$= -\left(\frac{822 \times 12^2}{2}\right) + (1218 \times 12) + 11550$$
$$= -59200 + 14600 + 11550$$
$$= -33050 \text{ lb ft/ft width (hogging)}$$

Bending moment at C', Fig. 31.21,

$$= -q_x l + M_x$$
$$= -14600 + 11550$$
$$= -3050 \text{ lb ft/ft width, i.e., hogging}$$

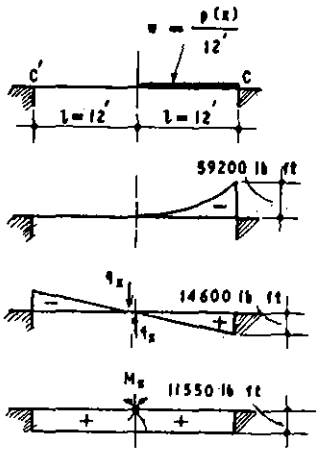


Fig. 31.21 *Bending moment distribution in-between boxes*

However, as the vehicle can be placed towards C' also, sections C and C' should both be designed for the bigger moment.

It is, however, necessary to be careful of the true bending moment at the critical section C, because for the vehicle placed in the longitudinal direction as shown in Fig. 31.16, physically there is no axle sitting at $x = L/2$ so that, in fact there are bending moment at C has very little effective contribution from $p(x)$; in other words, the bending moment at C

$$= q_x l + M_x$$
$$= 14600 + 11550$$
$$= 26150 \text{ lb ft/ft width, i.e., sagging}$$

Hence the critical sections C and C' in fact should be designed for a reversal of moments ranging from a hogging moment of 33,050 lb ft. to a sagging moment of 26,150 lb ft per ft width.

• ***Torsion in a box beam***: According to the general equation for torsion

$$\frac{T}{K} = \frac{G\theta}{L} \equiv G\frac{d\theta}{dx} \text{ so that } GK\frac{d\theta}{dx} = T$$

where G is modulus of rigidity, K the polar moment of inertia of the section, T the torsion, and θ the twist. Torsion in a box beam due to applied symmetric or asymmetric live load is given by:

$$T = GK\frac{d\theta}{dx}$$
$$= -\frac{Pb}{L}\left(-3.7398\frac{L}{\pi}\cos\frac{\pi x}{L} + 1.8942\frac{L}{3\pi}\cos\frac{3\pi x}{L}\right)$$

Details are given in Annexure 2. The maximum value of torsion occurs at $x = 0$ and at $x = L$, and is given by

$$-\frac{Pb}{L}\left(-3.7398\frac{L}{\pi} \sim 1.8942\frac{L}{3\pi}\right)$$

i.e., $-\dfrac{45 \times 2240 \times 13.5}{3\pi}(11.2194 \sim 1.8942)$
$= 1343000$ lb ft

Torsion in a box beam due to qx is given by

$$T = GK\frac{d\theta}{dx} = aq_2\left[\frac{\alpha L}{\pi}\cos\frac{\pi x}{L} + \frac{(1-\alpha)L}{3\pi} \times \cos\frac{3\pi x}{L}\right]$$

Details are given in Annexure 2. Its maximum value occurs at $x = 0$ and $x = L$, and is given by:

$$\frac{aq_2 L}{3\pi}[3\alpha + (1-\alpha)]$$

i.e., $\dfrac{19.5 \times 1558.5087 \times 115}{3\pi}[(2 \times 0.8905) + 1]$
$= 1,031,000$ lb ft

Torsion in a box beam due to M_x is given by

$$T = GK\frac{d\theta}{dx} = M_2\left[\frac{\gamma L}{\pi}\cos\frac{\pi x}{L} + \frac{(1-\gamma)L}{3\pi} \times \cos\frac{3\pi x}{L}\right]$$

This equation is similar to the one above; only aq_2 is replaced by M_2, and α by γ. Maximum value occurs at $x = 0$ and at $x = L$, and is given by

$$\frac{M_2 L}{3\pi}[3\gamma + (1-\gamma)]$$

i.e., $\dfrac{4735.40 \times 115}{3\pi}[(2 \times 1.7212) + 1]$
$= 256,000$ lb ft

At a support in a box beam, combining the torsional moments resulting from the symmetric load, the asymmetric load, q_x and M_x, the aggregate torsional reaction:

$= 1{,}343{,}000 \pm 1{,}343{,}000 \pm 1{,}031{,}000 + 256{,}000$

$= 1{,}399{,}000$ lb ft, and $-775{,}000$ lb ft respectively, in the two boxes.

• ***Transverse load distribution between the two boxes***: For determining the severest loading that a box has to take, it is sufficient to assume that the slab distributes the live load into the two boxes in accordance with the principle of simple statics, without considering the compatibility of deformations of the section as a whole. From the point of view of safety there can be no objection to this method of transverse load distribution. Such an approach can, however, result in the overdesign of the boxes, even if local overloading of the deck slab has been taken care of by analysing the moments by the compatibility of deformations method referred to earlier.

In order to estimate the reaction factors for the two boxes under a particular live load on the deck, it is sufficient to determine the total vertical deflection of each box, say, e_1 and e_2; then the corresponding reaction factors are $e_1/(e_1 + e_2)$ and $e_2/(e_1 + e_2)$. It is of interest to observe that, under HB loading, the reaction factor for the load-side box is found to be 1.195 on the basis of simple statics between the boxes as against 0.833 found on the basis of compatibility of deformations.

• ***Procedure for determining*** e_1 ***and*** e_2: The live load is placed in the most eccentric position on the deck section, and is split into symmetric and asymmetric parts. A 'cut' is introduced in the interconnecting slab, releasing the two redundants q_x and M_x, the former valid in the case of asymmetric loading and the latter in the case of symmetric loading. The compatibility equation is framed in the case of asymmetric load (which involves q_x), and q_x is determined as explained earlier. Since M_x does not contribute to the vertical deflection of the box beams, there is no need to frame the second compatibility equation.

With q_x known, the simple vertical deflection of each of the two boxes is determined, say at $x = L/2$, due to the asymmetric load, the symmetric load, and q_x, using the formulae similar to those given under (a) and (d) earlier. Then the algebraic addition of the relevant deflections gives e_1 and e_2.

Also refer to Chapter 37 for transverse distribution of Live Load in two-box girder deck where the boxes are essentially interconnected only by the common topslab and support and endcross girders.

CASE (B): Multicell box-section with equal-thickness webs

The analysis takes into account differential deflection of the constituent beams under the applied load, and derives the transverse slab moments on the basis of the strain energy dissipated in flexing the transverse slab resting on elastically sinking supports. Considering a typical continuous concrete box-section bridge as shown in Fig. 31.22, having spans L_m, m being = 1, 2, 3, ..., n, it is assumed that the bridge deck has n vertical or nearly vertical webs, as shown in Fig. 31.23(a) and (b). For the design of the transverse (top) slab the governing live load is assumed to be a wheeled truck load, because the transverse slab spans are short. Assuming that the deck is composed of n 'slab-web-slab' elements, Fig. 31.23(c), interconnected and sinking elastically, the moment of inertia (second moment of area) of the entire effective section is given by:

$$I = \phi \sum_{w=1}^{n} I_w \tag{31.5}$$

where I_w = the moment of inertia of the w^{th} 'slab-web-slab' component element, and ϕ = the reduction factor, because

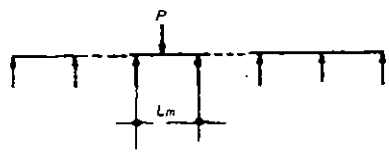


Fig. 31.22 *Typical continuous concrete box-section bridge*

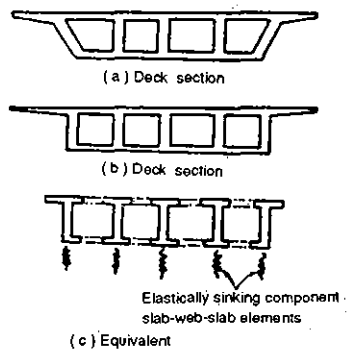


Fig. 31.23 *Deck section with vertical or nearly vertical webs, and equivalent slab-web, slab elements, interconnected and elastically sinking*

the moment of inertia of the whole may not be equal to the arithmetic sum of the moments of inertia of the component elements. In the case of bridge decks consisting of similar beams interconnected by a nominal number of diaphragms and a top slab, ϕ is very close to unity, and I is equal to nI_w.

Let i be the sum of the moments of inertia of the top and bottom transverse slabs over the effective dispersion width. The truck, with weight P, representing the live load, is placed on the span L_m of the continuous bridge, Fig. 31.22 so as to cause almost the greatest span deflection under it. This deflection is given by

$$\Delta = \frac{PL_m^3}{KEI} \tag{31.6}$$

Where K is a constant determined from the boundary conditions of the particular structural system, and EI is the sectional flexural rigidity. If R_w is the reaction caused in the wth component element, and the relative shear-distortion of these component elements is assumed on the basis of I_w being the section property equivalent to that afforded by the transverse interconnection by means of the slabs, then using Eq. (31.5), the wth element deflects by

$$\Delta_w = \frac{R_w L_m^3}{KEI_w}, \quad w \text{ being } = 1, 2, \ldots, n \tag{31.7}$$

If U be the strain energy stored in the system, then its partial derivative with respect to web reaction R_w gives the deflection in the wth element in the direction of R_w. In other words,

$$\frac{\partial U}{\partial R_w} = \Delta_w = \frac{R_w L_m^3}{KEI_w} \tag{31.8}$$

Releasing the strain energy stored in the system and equating the work done in so doing:

$$\frac{\partial U}{\partial R_w} = \frac{\partial}{\partial R_w} \int_O^S \frac{M^2}{2Ei}\, dx \tag{31.9}$$

$$= \int_O^S \frac{M}{Ei} \frac{\partial M}{\partial R_w}\, dx \tag{31.10}$$

Combining Eqs. (31.10) and (31.8)

$$\int_O^S \frac{M}{Ei} \frac{\partial M}{\partial R_w}\, dx = \frac{R_w L_m^3}{KEI_w} \tag{31.11}$$

Simplification yields

$$\int_O^S \frac{M}{i} \frac{\partial M}{\partial R_w}\, dx = \frac{1}{KI_w} L_m^3 R_w \tag{31.12}$$

where $w = 1, 2, 3, \ldots, n$.

The n unknowns, $R_1, R_2, \ldots, R_n$, can readily be evaluated by solving the n simultaneous Eqs. (31.12). Knowing these reactions in the elasticity sunk component support elements, the transverse bending moments in the slab can be easily written down.

• ***Application of the Method***: A two-span continuous bridge deck of four-cell box-section, as shown in Figs. 31.24 and 31.25 is considered. It is required to analyse the design moments in the transverse slab. Loads to be considered are:

live load — AASHTO H20-S16-44 truck
dead load — self and superimposed

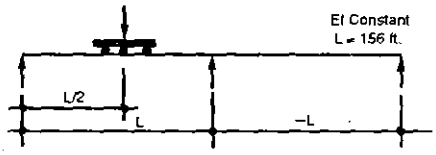


Fig. 31.24 *Two-span continuous bridge of four-cell box-section*

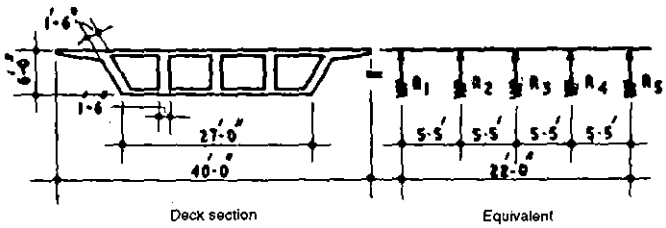


Fig. 31.25 *Deck section and equivalent elements*

Analysis for the case of live load is given below, and the analysis for the cases of dead load can be carried out in a similar manner.

From the relevant bridge calculations:

$EI =$ constant throughout
$I = 395.109$ ft^4
$i = 0.037$ ft^4
$L = 156$ ft
$n = 5$ (four cells)

Placing the truck as shown in Fig. 31.24, the maximum

deck deflection works out to $\Delta = \frac{3}{200}\frac{PL^3}{EI}$. This may be compared with Eq. (31.6).

All the webs being of equal thickness, and because of the overall symmetry of the section, it may be assumed that ϕ is equal to unity in Eq. (31.5), so that each

$I_w = \frac{I}{n} = \frac{I}{5}$. This may be compared with Eq. (31.5)

Hence, $\Delta_w = \frac{3}{200}\frac{R_w L^3}{EI_w}$

where w = 1, 2, 3, 4, and 5. This may be compared with Eq. (31.7).

Now, for example, if the truck is placed symmetrically on the deck cross-section, as shown in Fig. 31.26, then R_1 will equal R_5, R_2 will equal R_4, leaving only three unknowns, namely, R_1, R_2 and R_3, to be found. In fact, only R_1 and R_2 need to be found, because R_3 will then follow from the equilibrium condition between the applied load and the reactions $R_1 R_2$ and R_3. Using Eq. (31.12) the required two simultaneous equations for R_1 and R_2 are:

$$\int_A^E \frac{M}{i}\frac{\partial M}{\partial R_1}\,dx = \frac{3}{200} \times \frac{5}{I} L^3 R_1 \qquad (31.13)$$

and

$$\int_A^E \frac{M}{i}\frac{\partial M}{\partial R_2}\,dx = \frac{3}{200} \times \frac{5}{I} L^3 R_2 \qquad (31.14)$$

Owing to the symmetry about the centre, Fig. 31.26, the integration need be performed only from A to C, and the result doubled. Equations (31.13) and (31.14) may then be reduced as given below:

$$2\int_A^C \frac{M}{i}\frac{\partial M}{\partial R_1}\,dx = \frac{3}{200} \times \frac{5}{I} L^3 R_1 \qquad (31.15)$$

and

$$2\int_A^C \frac{M}{i}\frac{\partial M}{\partial R_2}\,dx = \frac{3}{200} \times \frac{5}{I} L^3 R_2 \qquad (31.16)$$

Noting that i is constant, further simplification yields

$$\int_A^C M\frac{\partial M}{\partial R_1}\,dx = 0.0375\frac{i}{I} L^3 R_1 \qquad (31.17)$$

and

$$\int_A^C M\frac{\partial M}{\partial R_2}\,dx = 0.0375\frac{i}{I} L^3 R_2 \qquad (31.18)$$

From Fig. 31.26, it is clear that the limit A to C comprises the sub-limits A to B, B to Y, Y to Z and Z to C. Considering these separately:

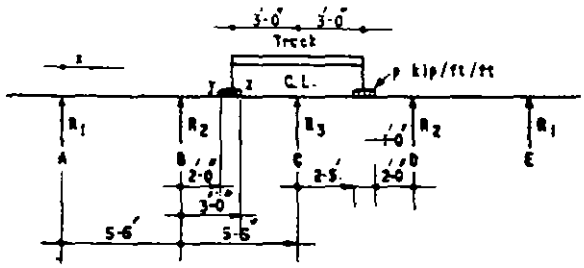


Fig. 31.26 *AASHTO truck placed symmetrically on deck section*

(i) From A to B
$M = R_1 x$, where x varies from 0 to 5.5 ft.

$$\frac{\partial M}{\partial R_1} = x, \text{ and } \frac{\partial M}{\partial R_2} = 0$$

(ii) From B to Y
$M = R_1(5.5 + x) + R_2 x$, where x varies from 0 to 2 ft.

$$\frac{\partial M}{\partial R_1} = 5.5 + x, \text{ and } \frac{\partial M}{\partial R_2} = x$$

(iii) From Y to Z
$M = R_1(5.5 + x) + R_2 x - \frac{p(x-2)^2}{2}$
where x varies from 2-3 ft

$$\frac{\partial M}{\partial R_1} = 5.5 + x, \text{ and } \frac{\partial M}{\partial R_2} = x$$

(iv) From Z to C
$M = R_1(5.5 + x) + R_2 x - p(x - 2.5)$, where x varies from 3–5.5 ft.

$$\frac{\partial M}{\partial R_1} = 5.5 + x, \text{ and } \frac{\partial M}{\partial R_2} = x$$

Substituting these values in Eqs. (31.17) and (31.18), integrating, and finally solving for R_1 and R_2, it is found that

$$R_1 = 49p \text{ lb/ft.}$$
$$R_2 = 483p \text{ lb/ft.}$$

The pressure intensity p under the heaviest wheel load of the truck may be found by dividing the load including impact factor by the dimensions of its dispersion, parallel and perpendicular to the bridge. In the present case these dimensions may be assumed as follows:

(i) dispersion dimension parallel to bridge: based on the longitudinal dispersion in plan, this may be taken to be equal to the span of the transverse slab when the

load is placed at mid-span i.e., 5.5 ft; according to the AASHTO specifications, the upper limit is 7 ft.

(ii) dispersion dimension perpendicular to bridge: based on the wheel contact width plus 45° dispersion up to the upper layer of reinforcement in the slab; this may be taken as equal to 1.0ft in the present case.

Then $p = \dfrac{16,000 \times 1.30 \text{ impact factor}}{1000 \times (5.5 \text{ ft} \times 1.0 \text{ ft})} = 3.782$ kip/ft./ft

Therefore $R_1 = -49 \times 3.782 = -185$ kip/ft

and $R_2 = 483 \times 3.782 = 1,825$ kip/ft

The bending moments in the slab can be easily obtained from Fig. 31.26, as follows:

At middle of span AB:

$$\text{Bending moment} = R_1 \times \frac{5.5}{2} = -185 \times 2.75 \times 1000 = -509 \text{ lb ft/ft}$$

At support B:

$$\text{Bending moment} = R_1 \times 5.5 = 185 \times 5.5 \times 1000 = -1,018 \text{ lb ft/ft}$$

At middle of span BC:

$$\text{Bending moment} = R_1\left(5.5 + \frac{5.5}{2}\right) + R_2 \times \frac{5.5}{2} - (0.75)^2\,\frac{p}{2}$$
$$= -185 \times 8.25 + 1,825 \times 2.75 - \frac{3782}{2}(0.75)^2$$
$$= 2,429 \text{ lb ft/ft}$$

At support C:

$$\text{Bending moment} = R_1(5.5 + 5.5) + R_2 \times 5.5 - (p \times 1) \times 3$$
$$= -185 \times 11.0 + 1,825 \times 5.5 - 3,782 \times 3$$
$$= -3,340 \text{ lb ft/ft.}$$

CASE (C): Multicell box-section with stub central web

Figure 31.27 represents a multicell box-section with stub central web. For its transverse analysis it is assumed that the Vierendeel type frames on either side of the centre-line of the deck are rigidly attached to the stub central web. The statically indeterminate system with various loads (self-weights and live load trucks together with any cantilever bending moment) is shown in Fig. 31.28. $I_{12}, I_{13}, \ldots$ and I_{56} represent second moments of area (moments of inertia) of members 1-2, 2-3, ..., and 5-6, respectively. The frame being six times statically indeterminate, it is rendered

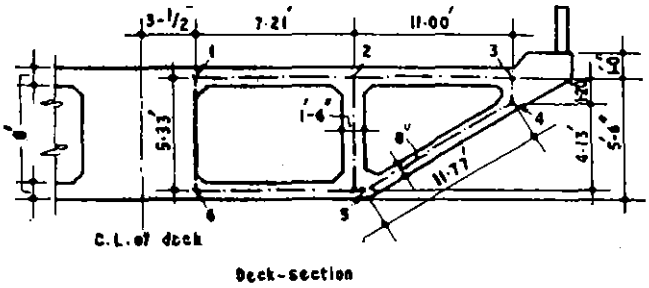


Fig. 31.27 *Multicell box-section with stub central web*

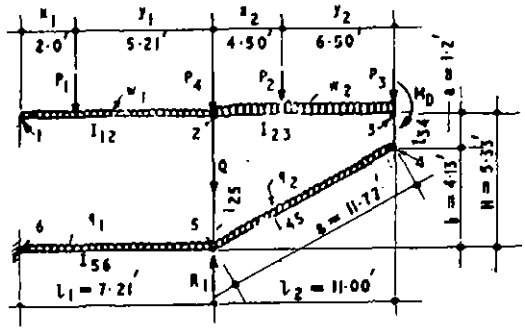


Fig. 31.28 *Statically indeterminate system with various loads*

statically determinate by introducing two cuts in it as shown in Fig. 31.29. The six stress resultants released are p_1, p_2, p_3, p_4, p_5, and p_6. These can be evaluated from the following compatibility equations once the influence coefficients δ_{rs} and δ_{ro} are evaluated (r = 1 to 6, s = 1 to 6):

$$\delta_{10} + \delta_{11}p_1 + \delta_{12}p_2 + \delta_{13}p_3 + \delta_{14}p_4 + \delta_{15}p_5 + \delta_{16}p_6 = 0 \quad (31.19)$$

$$\delta_{20} + \delta_{21}p_1 + \delta_{22}p_2 + \delta_{23}p_3 + \delta_{24}p_4 + \delta_{25}p_5 + \delta_{26}p_6 = 0 \quad (31.20)$$

$$\delta_{30} + \delta_{31}p_1 + \delta_{32}p_2 + \delta_{33}p_3 + \delta_{34}p_4 + \delta_{35}p_5 + \delta_{36}p_6 = 0 \quad (31.21)$$

$$\delta_{40} + \delta_{41}p_1 + \delta_{42}p_2 + \delta_{43}p_3 + \delta_{44}p_4 + \delta_{45}p_5 + \delta_{46}p_6 = 0 \quad (31.22)$$

$$\delta_{50} + \delta_{51}p_1 + \delta_{52}p_2 + \delta_{53}p_3 + \delta_{54}p_4 + \delta_{55}p_5 + \delta_{56}p_6 = 0 \quad (31.23)$$

$$\delta_{60} + \delta_{61}p_1 + \delta_{62}p_2 + \delta_{63}p_3 + \delta_{64}p_4 + \delta_{65}p_5 + \delta_{66}p_6 = 0 \quad (31.24)$$

where $\delta_{10} = (\delta_1 P_1 + \delta_1 P_2 + \delta_1 P_3 + \delta_1 P_4) + (\delta_1 w_1 + \delta_1 w_2) + (\delta_1 q_1 + \delta_1 q_2) + \delta_1 Q + \delta_1 M_D + \delta_1 R_1$

It may be noted that $\delta_{rs} = \int \dfrac{m_r m_s}{EI}\, ds$

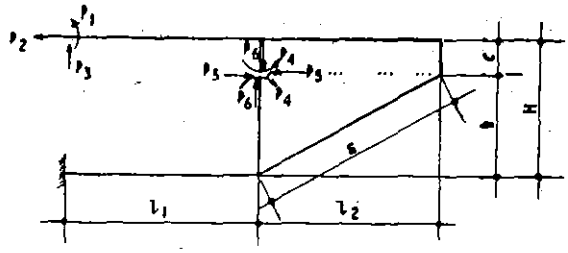


Fig. 31.29 *System made statically determinate by introducing two cuts*

$$= \int \frac{m_r m_s k}{E}\, ds,$$

where $k = \dfrac{1}{I}$

$$\delta_{ro} = \int \frac{m_r m_o}{EI}\, ds$$

$$= \int \frac{m_r m_o k}{E}\, ds,$$

where $k = \dfrac{1}{I}$

m_o = bending moment diagram on the determinate structure due to applied loads P_1, P_2, P_3, P_4, w_1, w_2, q_1, q_2, Q, M_D and R_1.

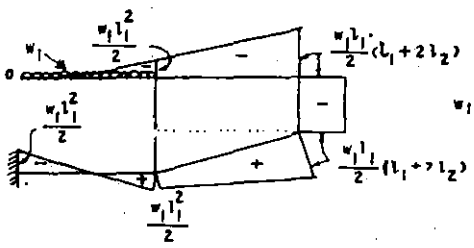


Fig. 31.30 *Bending moment diagram due to* w_1

Figures 31.30 to 30.40 represent the bending moment diagrams on the determinate structure due to various applied loads and moments, and Figs. 31.41 to 31.46 represent the bending moment diagrams on the determinate structure due to unit values of the six redundants p_1 to p_6.

- **Calculation of the various influence coefficients**

The influence coefficients involved in Eq. (31.19) are:

$$\delta_{11} = k_{12}l_1 + k_{23}l_2 + k_{34}a + k_{45}S + k_{56}l_1$$

$$\delta_{12} = 0.50\, k_{34}a^2 + 0.50\, k_{45}S(H + a) + k_{56}l_1 H$$

$$\delta_{13} = -0.50\, k_{12}l_1^2 - 0.50\, k_{23}l_2(2l_1 + l_2) - k_{34}a \times (l_1 + l_2) - 0.50\, k_{45}S(2l_1 + l_2) - 0.50\, k_{56}l_1^2$$

$$\delta_{14} = k_{23}l_2 + k_{34}\, a + k_{45}\, S$$

$$\delta_{15} = -k_{23}l_2 a - 0.50\, k_{34}a^2 + 0.50\, k_{45}\, Sb$$

$$\delta_{16} = 0.50\, k_{23}l_2^2 + k_{34}al_2 + 0.50\, k_{45}\, Sl_2$$

$$\delta_1 w_1 = 0.166\, k_{12}w_1 l_1^3 + 0.50\, k_{23}l_2 w_1\, (l_1^2 + l_1 l_2) + 0.50\, k_{34}w_1 l_1 a(l_1 + 2l_2) + 0.50\, k_{45}\, Sw_1 \times (l_1^2 + l_1 l_2)$$

$$\delta_1 w_2 = 0.166\, k_{23}w_2 l_2^2 + 0.50\, k_{34}w_2 l_3^2 a - 0.50\, k_{56}l_1 w_2(l_2^2 + l_1 l_2)$$

$$\delta_1 q_1 = 0.166\, k_{56}q_1 l_1^3$$

$$\delta_1 q_2 = -0.166\, k_{45}q_2 l_2^2 S - 0.50\, k_{56}l_1 q_2(l_2^2 + l_1 l_2)$$

$$\delta_1 Q = 0.50\, \frac{Qa}{H} l_2(k_{23}l_2 + 2k_{34}a + k_{45}S) - 0.50\, k_{56}Ql_1^2$$

$$\delta_1 P_1 = 0.50\, k_{12}P_1 y_1 + 0.50\, k_{23}P_1 l_2(2y_1 + l_2) + k_{34}P_1 a(y_1 + l_2) + 0.50\, k_{45}P_1 S(2y_1 + l_2) + 0.50\, k_{56}P_1 y_1^2 - 0.50\, k_{56}P_1 x_1^2$$

$$\delta_1 P_2 = 0.50\, k_{23}P_2 y_2^2 + k_{34}P_2 y_2 a + 0.50\, k_{45}P_2 \times y_2^2 \frac{S}{l_2} - 0.50\, k_{45}P_2 x_2^2 \frac{S}{l_2} - 0.50\, k_{56}P_2 l_1 \times (2x_2 + l_1)$$

$$\delta_1 P_3 = -0.50\, k_{45}P_3 l_2 S - 0.50\, k_{56}P_3 l_1(l_1 + 2l_2)$$

$$\delta_1 P_4 = 0.50\, P_4 l_2(k_{23}l_2 + 2k_{34}a + k_{45}S) - 0.50\, k_{56}P_4 l_1^2$$

$$\delta_1 R_1 = 0.50\, k_{56}R_1 l_1^2$$

$$\delta_1 M_D = -k_{34}M_D a - k_{45}M_D S - k_{56}M_d l_1$$

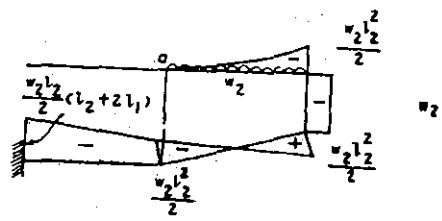


Fig. 31.31 *Bending moment diagram due to* w_2

The influence coefficients involved in Eq. (31.20):

$$\delta_{21} = 0.50\, k_{34}a^2 + 0.50\, k_{45}S(H + a) + k_{56}l_1 H$$

$$\delta_{22} = 0.333\ k_{34}a^3 + 0.333\ k_{45}(H^2 + Ha + a^2)S + k_{56}H^2 l_1$$

$$\delta_{23} = -0.50\ k_{34}a^2(l_1 + l_2) - 0.166\ k_{45}S [H(3l_1 + l_2) + a(3l_1 + 2l_2)] - 0.50\ k_{56}Hl_1^2$$

$$\delta_{24} = 0.50\ k_{34}a^2 + 0.50\ k_{45}S(H + a)$$

$$\delta_{25} = -0.166\ k_{34}a^3 + 0.166\ k_{45}bS(2H + a)$$

$$\delta_{26} = 0.50\ k_{34}a^2 l_2 + 0.166\ k_{45}l_2 S(H + 2a)$$

$$\delta_2 w_1 = 0.25\ k_{34}a^2 w_1 l_1(l_1 + 2l_2) + 0.083\ k_{45}w_1 l_1 \times S[3l_1(a + H) + 2l_2(2a + H)]$$

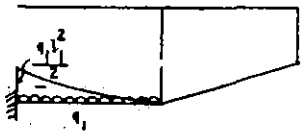


Fig. 31.32 *Bending moment diagram due to* q_1

$$\delta_2 w_2 = 0.25\ k_{34}w_2 l_2^2 a_2 - 0.083\ k_{45}w_2 l_2^2\ S(H - a) - 0.50\ k_{56}w_2 l_1 l_2 H(l_1 + l_2)$$

$$\delta_2 q_1 = -0.166\ k_{56}q_1 l_1^3 H$$

$$\delta_2 q_2 = -0.042\ k_{45}q_2 l_2^2 S(3H + a) - 0.50\ k_{56}q_2 l_2 \times (l_1 + l_2)Hl_1$$

$$\delta_2 Q = 0.50\ k_{34}Qa^3\frac{l_2}{H} + 0.166\ k_{45}QaS\frac{l_2}{H} \times (2a + H) - 0.50\ k_{56}Ql_1^2 H$$

$$\delta_2 P_1 = 0.50\ k_{34}P_1 a^2(y_1 + l_2) + 0.166\ k_{45}P_1 S \times [3y_1(H + a) + l_2(H + 2a)] + 0.50\ k_{56} \times P_1 y_1^2 H - 0.50\ k_{56}P_1 x_1^2 H$$

$$\delta_2 P_2 = 0.50\ k_{34}P_2 a^2 y_2 - 0.166\ k_{45}P_2 x_2^2 \times \left[3H - \frac{x_2}{l_2}(H - a)\right]\frac{S}{l_2} + 0.166\ k_{45}P_2 y_2^2 \times \left[3a + \frac{y_2}{l_2}(H - a)\right]\frac{S}{l_2} - 0.50\ k_{56}P_2 l_1 H \times (2x_2 + l_1)$$

$$\delta_2 P_3 = -0.166\ k_{45}P_3 l_2 S(2H + a) - 0.50\ k_{56}P_3 l_1 H(l_1 + 2l_2)$$

$$\delta_2 P_4 = 0.50\ k_{34}P_4 a^2 l_2 + 0.166\ k_{45}P_4 S l_2 \times (2a + H) - 0.50\ k_{56}P_4 l_1^2\ H$$

$$\delta_2 R_1 = 0.50\ k_{56}R_1 l_1^2 H$$

$$\delta_2 M_D = -0.50\ k_{45}M_D S(H + a) - k_{56}M_D H l_1 - 0.50\ k_{34}M_D a^2$$

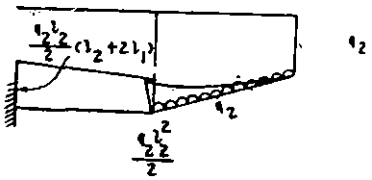


Fig. 31.33 *Bending moment diagram due to* q_2

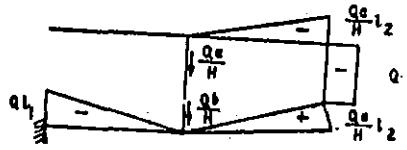
The influence coefficients involved in Eq. (31.21):

$$\delta_{31} = -0.50\ k_{12}l_1^2 - 0.50\ k_{23}l_2(2l_1 + l_2) - k_{34}a(l_1 + l_2) - 0.50\ k_{45}S(2l_1 + l_2) - 0.50\ k_{56}l_1^2$$

$$\delta_{32} = -0.50\ k_{34}a^2(l_1 + l_2) - 0.166\ k_{45}S[H(3l_1 + l_2) + a(3l_1 + 2l_2)] - 0.50\ k_{56}Hl_1^2$$

$$\delta_{33} = 0.333\ k_{12}l_1^3 + 0.333\ k_{23}l_2(3l_1^2 + 3l_1 l_2 + l_2^2) + k_{34}a(l_1 + l_2)^2 + 0.333\ k_{45}S \times (3l_2 + 3l_1 l_2 + l_2^2) + 0.333\ k_{56}l_1^3$$

$$\delta_{34} = -0.50\ k_{23}l_2(2l_1 + l_2) - k_{34}a(l_1 + l_2) - 0.50\ k_{45}S(2l_1 + l_2))$$



Portion of Q force above cut $= \dfrac{Qa}{H}$ Total $= Q$

Portion of Q force above cut $= \dfrac{Qb}{H}$

Fig. 31.34 *Bending moment diagram due to* Q

$$\delta_{35} = 0.50\ k_{23}l_2 a(2l_1 + l_2) + 0.50\ k_{34}a^2 \times (l_1 + l_2) - 0.166\ k_{45}bS(3l_1 + l_2)$$

$$\delta_{36} = -0.166\ k_{23}l_2^2(3l_1 + 2l_2) - k_{34}l_2 a(l_1 + l_2)$$

$$-0.166\ k_{45}l_2S(3l_1+2l_2)$$

$$\delta_3 w_1 = -0.125k_{12}w_1l_1^4 - 0.166\ k_{23}w_1l_1l_2 \times(3l_1^2+4.5l_1l_2+2l_2^2) - 0.50\ k_{34}w_1l_1a \times(l_1^2+3l_1l_2+2l_2^2) - 0.166\ k_{45}w_1l_1S \times(3l_1^2+4.5l_1l_2+2l_2^2) - 0.083\ k_{56}w_1l_1^4$$

$$\delta_3 w_2 = -0.042\ k_{23}w_2l_2^3(4l_1+3l_2) -0.50\ k_{34}w_2l_2^2a(l_1+l_2) -0.083\ k_{45}w_2l_2^3S +0.166\ k_{56}w_2l_1^2l_2(1.5l_2+l_1)$$

$$\delta_3 q_1 = 0.042\ k_{56}q_1l_1^4$$

$$\delta_3 q_2 = 0.042\ k_{45}q_2l_2^2S(4l_1+l_2) +0.166\ k_{56}q_2l_1^2l_2(1.5l_2+l_1)$$

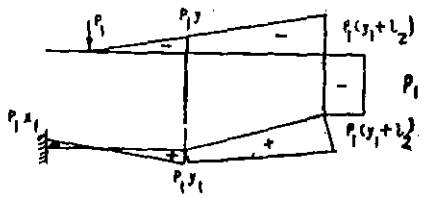


Fig. 31.35 *Bending moment diagram due to* P_1

$$\delta_3 Q = -0.166\ k_{23}\ Qa\frac{l_2^2}{H}(3l_1+2l_2) - k_{34}Qa^2 \times\frac{l_2}{H}(l_1+l_2) - 0.166\ k_{45}Qal_2\frac{S}{H}\times (3l_1+2l_2) + 0.166\ k_{56}\ Ql_1^3$$

$$\delta_3 P_1 = -0.166\ k_{12}P_1y_1^2(3l_1-y_1) - 0.166 \times k_{23}P_1l_2[3y_1(2l_1+l_2)+l_2(3l_1+2l_2)] -k_{34}P_1a(y_1+l_2)(l_1+l_2) - 0.166\ k_{45} \times P_1S[3y_1(2l_1+l_2)+l_2(3l_1+2l_2)] -0.166\ k_{56}P_1l_1^2(2y_1-x_1)$$

$$\delta_3 P_2 = -0.166\ k_{23}P_2y_2^2(3l_1+2l_2+x_2) -k_{34}P_2y_2a(l_1+l_2) + 0.166\ k_{45}P_2x_2^2 \times(3l_1+x_2)\frac{S}{l_2} - 0.166\ k_{45}P_2y_2^2\ (3l_1+2l_2 +x_2)\frac{S}{l_2} + 0.166\ k_{56}P_2l_1^2\ (3x_2+l_1)$$

$$\delta_3 P_3 = 0.166\ k_{45}P_3l_2S(3l_1+l_2) + 0.166\ k_{56} \times P_3l_1^2(l_1+3l_2)$$

$$\delta_3 P_4 = -0.166\ k_{23}P_4l_2^2(3l_1+2l_2) -k_{34}P_4al_2(l_1+l_2) - 0.166\ k_{45}P_4l_2S \times(3l_1+2l_2) + 0.166\ k_{56}P_4l_1^3$$

$$\delta_3 R_1 = -0.166\ k_{56}R_1l_1^3$$

$$\delta_2 M_D = k_{34}M_Da(l_1+l_2) + 0.50\ k_{45}M_DS \times(2l_1+l_2) + 0.50\ k_{56}M_Dl_1^2$$

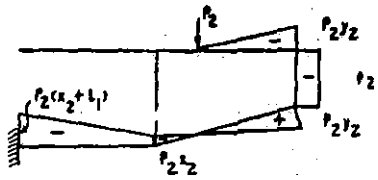


Fig. 31.36 *Bending moment diagram due to* P_2

The influence coefficients involved in Eq. (31.22):

$$\delta_{41} = k_{23}l_2 + K_{34}a + k_{45}S$$

$$\delta_{42} = 0.50k_{34}a^2 + 0.50\ k_{45}S(H+a)$$

$$\delta_{43} = -0.50\ k_{23}l_2(2l_1+l_2) - k_{34}a(l_1+l_2) -0.50\ k_{45}S(2l_1+l_2)$$

$$\delta_{44} = k_{33}l_2 + k_{34}a + k_{45}S + k_{25}H$$

$$\delta_{45} = -k_{23}l_2a - 0.50\ k_{34}a^2 + 0.50\ k_{45}bS -0.50\ k_{25}a^2 + 0.50\ k_{25}b^2$$

$$\delta_{46} = 0.50\ k_{23}l_2^2 + k_{34}l_2a + 0.50k_{45}l_2S$$

$$\delta_4 w_1 = 0.50\ k_{23}l_2w_1(l_1^2+l_1l_2) + 0.50\ k_{34}w_1l_1a \times(l_12l_2) + 0.50\ k_{45}Sw_1(l_1^2+l_1l_2)$$

$$\delta_4 w_2 = 0.166\ k_{23}w_2l_2^2 + 0.50\ k_{34}w_2l_2^2a$$

$$\delta_4 q_1 = 0$$

$$\delta_4 q_2 = -0.166\ k_{45}q_2l_2^2S$$

$$\delta_4 Q = 0.50\ Qa\frac{l_2}{H}(k_{23}l_2 + 2k_{34}a + k_{45}S)$$

$$\delta_4 P_1 = 0.50\ k_{23}P_1l_2(2y_1+l_2) + k_{34}P_1a(y_1+l_2) +0.50\ k_{45}P_1S(2y_1+l_2)$$

$$\delta_4 P_2 = 0.50\ k_{23}P_2y_2^2 + k_{34}P_2y_2a + 0.50\ k_{45}P_2 \times y_2^2\ \frac{S}{l_2} - 0.50\ k_{45}P_2x_2^2\frac{S}{l_2}$$

$$\delta_4 P_3 = -0.50\ k_{45} P_3 l_2 S$$

$$\delta_4 P_4 = -0.50\ P_4 l_2 (k_{23} l_2 + 2k_{34} a + k_{45} S)$$

$$\delta_4 R_1 = 0$$

$$\delta_4 M_D = -k_{34} M_D a - k_{45} M_D S$$

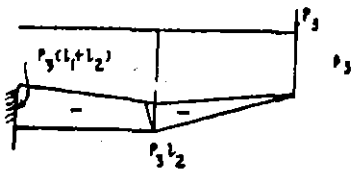


Fig. 31.37 *Bending moment diagram due to P_3*

The influence coefficients involved in Eq. (31.23):

$$\delta_{51} = -k_{23} l_2 a - 0.50\ k_{34} a^2 + 0.50\ k_{45} Sb$$

$$\delta_{52} = -0.166\ k_{34} a^3 + 0.166\ k_{45} bS(2H + a)$$

$$\delta_{53} = 0.50\ k_{23} l_2 a(2l_1 + l_2) + 0.50\ k_{34} a^2 (l_1 + l_2) - 0.166\ k_{45} bS(3l_1 + l_2)$$

$$\delta_{54} = k_{23} l_1 a - 0.50\ k_{34} a^2 + 0.50\ k_{45} bS - 0.50\ k_{25} a^2 + 0.50\ k_{25} b^2$$

$$\delta_{55} = k_{23} l_2 a^2 + 0.333\ k_{34} a^3 + 0.333\ k_{45} b^2 S + 0.333\ k_{25} a^3 + 0.333\ k_{25} b^3$$

$$\delta_{56} = -0.50\ k_{23} l_2^2 a - 0.50\ k_{34} l_2 a^2 + 0.166 \times k_{45} l_2 bS$$

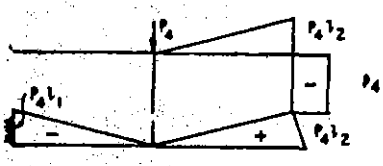


Fig. 31.38 *Bending moment diagram due to P_4*

$$\delta_5 w_1 = -0.50\ k_{23} l_2 a w_1 l_1 (l_1 + l_2) - 0.25\ k_{34} w_1 l_1 \times a^2 (l_1 + 2l_2) + 0.166\ k_{45} w_1 l_1 bS(1.5l_1 + l_2)$$

$$\delta_5 w_2 = -0.166\ k_{23} w_2 w_2 l_2^3 a - 0.25\ k_{34} w_2 l_2^2 a^2 - 0.083\ k_{45} w_2 l_2^2 bS$$

$$\delta_5 q_1 = 0$$

$$\delta_5 q_2 = -0.125\ k_{45} q_2 l_2^2 bS$$

$$\delta_5 Q = -0.50\ k_{23} Q\ a^2 \frac{l_2^2}{H} - 0.50\ k_{34}\ Qa^3 \frac{l_2}{H} + 0.166\ k_{45}\ Qabs\ \frac{l_2}{H}$$

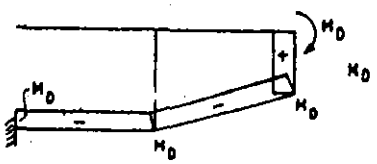


Fig. 31.39 *Bending moment diagram due to M_D*

$$\delta_5 P_1 = -0.50\ k_{23} P_1 l_2 a(2y_1 + l_2) - 0.50\ k_{34} P_1 a^2 (y_1 + l_2) + 0.166\ k_{45} \times P_1 bS(3y_1 + l_2)$$

$$\delta_5 P_2 = -0.50\ k_{23} P_2 y_2^2 a - 0.50\ k_{34} P_2 y_2 a^2 + 0.166\ k_{45} P_2 bS(y_2 - 2x_2)$$

$$\delta_5 P_3 = -0.333\ k_{45} P_3 l_2 bS$$

$$\delta_5 P_4 = -0.50\ k_{23} P_4 a l_2^2 - 0.50\ k_{34} P_4 a^2 l_2 + 0.166\ k_{45} P_4 bSl_2$$

$$\delta_5 R_1 = 0$$

$$\delta_5 M_D = 0.50\ k_{34} M_D a^2 - 0.50\ k_{45} M_D bS$$

The influence coefficients involved in Eq. (31.24):

$$\delta_{61} = 0.50\ k_{23} l_2^2 + k_{34} a l_2 + 0.50\ k_{45} Sl_2$$

$$\delta_{62} = 0.50\ k_{34}\ a^2\ l_2 + 0.166\ k_{45}\ l_2\ S(H + 2a)$$

$$\delta_{63} = -0.166\ k_{23} l_2^2\ (3l_1 + 2l_2) - k_{34} l_2 a \times (l_1 + l_2) - 0.166\ k_{45}\ l_2 S(3l_1 + 2l_2)$$

$$\delta_{64} = 0.50\ k_{23}\ l_2^2 + k_{34} l_2 a + 0.50\ k_{45}\ l_2 S$$

$$\delta_{65} = -0.50\ k_{23}\ l_2^2\ a - 0.50\ k_{34}\ l_2 a^2 + 0.166\ k_{45}\ l_2 bS$$

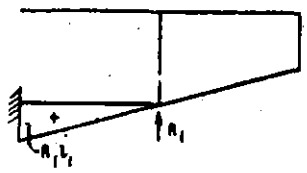


Fig. 31.40 *Bending moment diagram due to R_1*

$$\delta_{66} = 0.333\ k_{23} l_2^3 + k_{34} l_2^2 a + 0.333\ k_{45} l_2^2 S$$

$$\delta_6 w_1 = 0.166\ k_{23} w_1 l_1 l_2^2\ (1.5l_1 + 2l_2) + 0.50\ k_{34}\ w_1 l_1 l_2\ a(l_1 + 2l_2) + 0.166\ k_{45} \times w_1 l_1 l_2 S(1.5l_1 + 2l_2)$$

$$\delta_6 w_2 = 0.125\ k_{23}\ w_2\ l_2 + 0.50\ k_{34}\ w_2 l_2^3 a + 0.083 k_{45} w_2\ l_2^3 S$$

$$\delta_6 q_1 = 0$$

$$\delta_6 q_2 = -0.042\ k_{45}\ q_2 l_2^3 S$$

$$\delta_6 Q = 0.333\ k_{23} Qa \frac{l_2^3}{H} + k_{34}\ Qa^2 \frac{l_2^2}{H} + 0.333\ k_{45}\ QaS \frac{l_2^2}{H}$$

$$\delta_6 P_1 = 0.166\ k_{23}\ P_1 l_2^2\ (3y_1 + 2l_2) + k_{34} P_1 l_2 a \times (y_1 + l_2) + 0.166\ k_{45}\ P_1 l_2 S(3y_1 + 2l_2)$$

$$\delta_6 P_2 = 0.166\ k_{23}\ P_2 y_2^2 (2l_2 + x_2) + k_{34} P_2 l_2 y_2 a + 0.166\ k_{45}\ P_2 l_2 S(2y_2 - x_2)$$

$$\delta_6 P_3 = -0.166\ k_{45}\ P_3\ l_2^2 S$$

$$\delta_6 P_4 = 0.333\ k_{23}\ P_4\ l_2^3 + k_{34}\ P_4 a l_2^2 + 0.333\ k_{45}\ P_4\ S l_2^2$$

$$\delta_6 R_1 = 0$$

$$\delta_6 M_D = -k_{34}\ M_D l_2 a - 0.50\ k_{45}\ M_D\ l_2 S$$

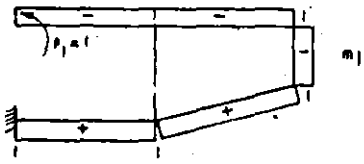


Fig. 31.41 *Bending moment diagram due to* m_1

In the design example under illustration, on one-foot width basis:

$w_1 = w_2 =$	138 lb/ft	
$q_1 =$	100 lb/ft	
$q_2 =$	108 lb/ft	
$P_1 = P_2 =$	3,400 lb	
$P_3 =$	4,680 lb	
$P_4 =$	3,460 lb	
$Q =$	1,350 lb	
$R_1 =$	3,640 lb	(upward force due to differential cable push in transverse section)
$M_D =$	2,030 lbft./ft.	

$$I_{12} = \frac{1}{12}\% \left(\frac{8}{12}\right)^2 = 0.025\ \text{ft}^4$$

so that $k_{12} = \dfrac{l}{I_{12}} = 40.0\ \text{ft}^{-4}$

Similarly $k_{23} = 40\ \text{ft}^{-4}$

$k_{34} = 0.77\ \text{ft}^{-4}$

$k_{45} = 40.0\ \text{ft}^{-4}$

$k_{56} = 40.0\ \text{ft}^{-4}$

$k_{25} = 3.57\ \text{ft}^{-4}$

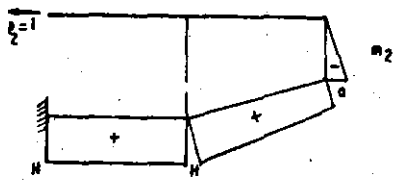


Fig. 31.42 *Bending moment diagram due to* m_2

Substituting the above in kip and ft. units in the formulae for the various coefficients determined earlier, the values obtained are given below for Eq. (31.19)

$\delta_{11} =$	1488.524,	$\delta_1 q_2 =$	− 4140.758,
$\delta_{12} =$	3074.886,	$\delta_1 Q =$	122.075,
$\delta_{13} =$	− 13672.45 ,	$\delta_1 P_1 =$	36636.36,
$\delta_{14} =$	911.724,	$\delta_1 P_2 =$	− 3453.30,
$\delta_{15} =$	443.648,	$\delta_1 P_3 =$	− 31830.936,
$\delta_{16} =$	5019.564,	$\delta_1 P_4 =$	13770.39,
$\delta_1 w_1 =$	8608.044,	$\delta_1 R_1 =$	− 3794.84,
$\delta_1 w_2 =$	− 2758.75,	$\delta M_D =$	− 1543.05,
$\delta_1 q_1 =$	− 248.85,		

Therefore, $\delta_{10} = 11366.38$

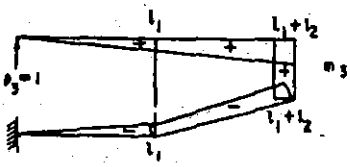


Fig. 31.43 *Bending moment diagram due to* m_3

For Eq. (31.20)

$\delta_{21} = \delta_{12}$		$\delta_2 q_2 =$	− 21069.11,
$\delta_{22} =$	13876.125,	$\delta_2 Q =$	− 5459.396,
$\delta_{23} =$	− 23235.55,	$\delta_2 P_1 =$	58135.289,
$\delta_{24} =$	1537.72,	$\delta_2 P_2 =$	− 43178.00,
$\delta_{25} =$	3827.84,	$\delta_2 P_3 =$	− 152784.27,
$\delta_{26} =$	6651.419,	$\delta_2 P_4 =$	3840.344,
$\delta_2 w_1 =$	12111.63,	$\delta_2 R_1 =$	− 20226.49,

$\delta_2 w_2 = -23936.04, \quad \delta M_D = -\ 6242.03,$
$\delta_2 q_1 = -\ 1326.48,$
Therefore, $\delta_{20} = 200134.56$

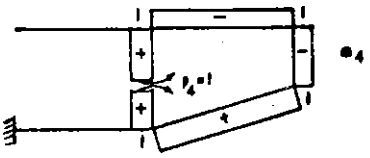


Fig. 31.44 *Bending moment diagram due to* m_4

For Eq. (31.21)

$\delta_{31} = \delta_{13}, \quad \delta_3 q_2 = 20017.40,$
$\delta_{32} = \delta_{23}, \quad \delta_3 Q = -\ 18750.979,$
$\delta_{33} = 166453.06, \quad \delta_3 P_1 = -471823.37,$
$\delta_{34} = -\ 11593.086, \quad \delta_3 P_2 = -\ 58314.67,$
$\delta_{35} = -\ 3811.04, \quad \delta_3 P_3 = 196236.37,$
$\delta_{36} = -\ 72747.04, \quad \delta_3 P_4 = -243093.85,$
$\delta_3 w_1 = -116901.28, \quad \delta_3 R_1 = 9083.78$
$\delta_3 w_2 = -\ 13977.04, \quad \delta_3 M_D = 14291.951,$
$\delta_3 q_1 = 453.88,$

Therefore, $\delta_{30} = -682777.81$
For Eq. (31.22):

$\delta_{41} = \delta_{14}, \quad \delta_{42} = \delta_{24}, \quad \delta_{43} = \delta_{34},$
$\delta_{44} = 930.75, \quad \delta_{45} = 471.52, \quad \delta_{46} = 5019.56,$
$\delta_4 w_1 = 8264.83, \quad \delta_4 w_2 = 1227.33, \quad \delta_4 q_1 = 0$
$\delta_4 q_2 = -1021.13, \delta_4 Q = 1525.65, \quad \delta_4 q_1 = 33216.80$
$\delta_4 P_2 = 4494.14, \quad \delta_4 P_3 = 12118.39, \quad \delta_4 P_4 = 17367.69,$
$\delta_4 R_1 = 0, \quad \delta_4 M_D = -957.60,$

Therefore, $\delta_{40} = 51992.32$

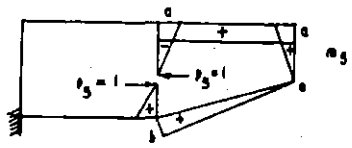


Fig. 31.45 *Bending moment diagram due to* m_5

For Eq. (31.23)

$\delta_{51} = \delta_{15}, \quad \delta_{52} = \delta_{25}, \quad \delta_{53} = \delta_{35}, \delta_{54} = \delta_{45},$
$\delta_{55} = 3394.00, \quad \delta_{56} = 640.38, \quad \delta_5 w_1 = 2214.55.$
$\delta_5 w_2 = -4163.00, \delta_5 q_1 = 0, \delta_5 q_2 = -3176.18,$
$\delta_5 Q = 194.639, \quad \delta_5 P_1 = 9967.185,$
$\delta_5 P_2 = -6203.40, \quad \delta_5 P_3 = -33332.82,$
$\delta_5 P_4 = -6518.459, \quad \delta_5 R_1 = 0, \quad \delta_5 M_D = -1972.44,$

Therefore, $\delta_{50} = -42989.93$
For Eq. (31.24)

$\delta_{61} = \delta_{16}, \delta_{62} = \delta_{26}, \delta_{63} = \delta_{36}, \quad \delta_{64} = \delta_{46},$
$\delta_{65} = \delta_{56}, \delta_{66} = 36810.66, \delta_6 w_1 = 54449.06,$
$\delta_6 w_2 = 17361.91, \quad \delta_6 q_1 = 0, \quad \delta_6 q_2 = -2835.52,$
$\delta_6 Q = 11188.233, \quad \delta_6 P_1 = 213343.16,$
$\delta_6 P_2 = 50402.21, \quad \delta_6 P_3 = -44256.37,$
$\delta_6 P_4 = 127364.912, \quad \delta_6 R_1 = 0, \quad \delta_6 M_D = -5277.14,$

Therefore, $\delta_{60} = -421740.45$

Substituting these values of influence coefficients in compatibility Eqs. (31.19) to (31.24) gives

$$1488.52\, p_1 + 3074.89\, p_2 - 13672.45\, p_3 + 911.72\, p_4 + 433.65\, p_5 + 5019.56\, p_6 + 11366.4 = 0$$

$$3074.89\, p_1 + 13876.13\, p_2 - 23235.55\, p_3 + 1537.72\, p_4 + 3827.84\, p_5 + 6651.42\, p_6 - 200134.6 = 0$$

$$-13672.45\, p_1 - 23235.55\, p_2 + 166453.06\, p_3 - 11593.08\, p_4 - 3811.04\, p_5 - 72747.04\, p_6 - 682777.8 = 0$$

$$911.72\, p_1 + 1537.72\, p_2 - 11593.08\, p_3 + 930.75\, p_4 + 471.52\, p_5 + 5019.56\, p_6 + 51992.3 = 0$$

$$443.65\, p_1 + 3827.84\, p_2 - 3811.04\, p_3 + 471.52 p_4 + 3394.00\, p_5 + 640.38\, p_6 - 42989.9 = 0$$

$$5019.56\, p_1 + 6651.42\, p_2 - 72747.04\, p_3 + 5019.56\, p_4 + 640.38\, p_5 36810.66\, p_6 + 421740.5 = 0$$

Solving these six equations simultaneously gives

$$\left.\begin{aligned} p_1 &= 42.68 \\ p_2 &= 26.84 \\ p_3 &= 12.15 \\ p_4 &= 14.95 \\ p_5 &= -11.62 \\ p_6 &= 0.046 \end{aligned}\right\} \text{(kip and ft. units)}$$

With the six redundants p_1, p_2, p_3, p_4, p_5, and p_6 thus determined, the bending moment at any point can be readily calculated from the following formula:

Bending moment at any section
$= m_0 + m_1 p_1 + m_2 p_2 + m_3 p_3 + m_4 p_4 + m_5 p_5 + m_6 p_6$
where m_0 = sum of the ordinates of the bending moment diagrams due to applied load and moments

Thus the bending moment, say at point 1, Fig. 31.12:
$= 0 + (-1.00) p_1 + 0$
$= (-1.00) \times 42.68 = 42.680$ lb ft/ft width (hogging)

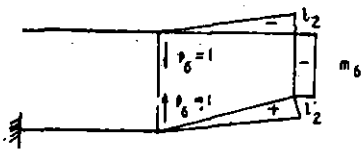


Fig. 31.46 *Bending moment diagram due to m_6*

Similarly, thrust n and shear s can also be readily calculated at any point from the formulae:

$n = n_0 + n_1 p_1 + n_2 p_2 + n_3 p_3 + n_4 p_4 + n_5 p_5 + n_6 p_6$

$s = s_0 + s_1 p_1 + s_2 p_2 + s_3 p_3 + s_4 p_4 + s_5 p_5 + s_6 p_6$

Thus thrust, say at point 1, Fig. 31.27:

$= 0 + (-1.00)p_2 + 0$

$= (-1.00) \times 26.84 = -26{,}840$ lb ft/ft width (tension)

and shear at point 1, Fig. 31.27:

$= 0 + (1.00)p_3 + 0$

$= (1.00) \times 12.15 = 12{,}150$ lb/ft width

Similarly, the bending moment, thrust and shear force can be determined right round the frame, and the analysis completed.

CASE (D): Two-cell box-section with solid central spine

Figure 31.47 represents a two-cell box-section with a solid central spine. For its transverse analysis it is assumed that the triangulated frame on either side of the centreline of the deck is rigidly attached to the solid central spine. The statically indeterminate system with various loads (self-weights and live load trucks, together with any cantilever bending moment), is shown in Fig. 31.48. I_1, I_2 and I_3 represent second moments of area (moments of inertia) of the three members of the frame in Fig. 31.48. The frame being three times statically indeterminate, is rendered determinate by introducing one cut in it as shown in Fig. 31.49. The three stress resultants released are p_1, p_2, and p_3. These can be evaluated from the following compatibility equations once the influence coefficients δ_{rs} and δ_{ro} are evaluated ($r = 1$ to 3, $s = 1$ to 3):

$$\delta_{10} + \delta_{11}\, p_1 + \delta_{12}\, p_2 + \delta_{13}\, p_3 = 0 \qquad (31.25)$$

$$\delta_{20} + \delta_{21}\, p_1 + \delta_{22}\, p_2 + \delta_{23}\, p_3 = 0 \qquad (31.26)$$

$$\delta_{30} + \delta_{31}\, p_1 + \delta_{32}\, p_2 + \delta_{33}\, p_3 = 0 \qquad (31.27)$$

where $\delta_{10} = \delta_1 w + \delta_1 q + (\delta_1 P_1 + \delta_1 P_2) + \delta_1 M_D$, and so on. w, q, P_1 and P_2 are the applied loads, and M_D the applied moment.

It may be noted that

$$\delta_{rs} = \int \frac{m_r m_o}{EI}\, ds = \int \frac{m_r m_s k}{E}\, ds, \text{ where } k = \frac{1}{I}$$

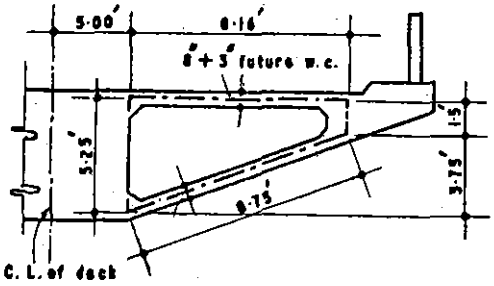


Fig. 31.47 *Two-cell box-section with solid central spine*

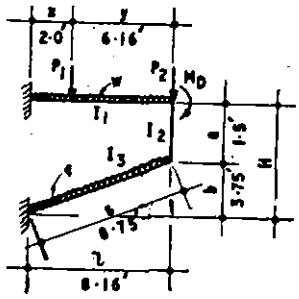


Fig. 31.48 *Statically indeterminate system with various loads*

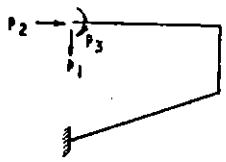


Fig. 31.49 *Statically determinate system*

$$\delta_{ro} = \int \frac{m_r m_o}{EI}\, ds = \int \frac{m_r m_o k}{E}\, ds, \text{ where } k = \frac{1}{I}$$

m_o = bending moment diagram on the determinate structure due to applied loads and moments.

Figures 31.50 to 31.54 represent the bending moment diagrams on the determinate structure due to various applied loads and moments, and Figs. 31.55 to 31.57 represent the bending moment diagrams on the determinate structure due to unit values of the three redudants p_1, p_2 and p_3.

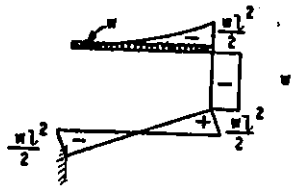


Fig. 31.50 *Bending moment diagram due to w*

- **Calculation of various influence coefficients**

~ *Influence coefficients involved in Eq (31.25)*

$$\delta_{11} = 0.333\ k_1 l^3 + k_2\ al^2 + 0.333\ k_3 l^2 s$$
$$\delta_{12} = -0.500\ k_2 a^2 l - 0.166\ k_3\ ls(H + 2a)$$
$$\delta_{13} = -0.500\ k_1 l^2 + k_2 la - 0.500\ k_3 ls$$
$$\delta_1 w = 0.125\ k_1\ wl^4 + 0.500\ k_2\ wl^3 a + 0.083\ k_3 wl^3 s$$
$$\delta_1 q = -0.042\ k_3\ ql^3 s$$
$$\delta_1 P_1 = 0.166\ k_1\ P_1 ly^2 \left(3 - \frac{y}{l}\right) + k_2 P_1\ lya + 0.166\ k_3\ P_1 y^2 s \left(3 - \frac{y}{l}\right) + k_3 P_1 s \frac{x^3}{l}$$
$$\delta_1 P_2 = -0.166\ k_3\ P_2 l^2 s$$
$$\delta_1 M_D = -0.500\ M_D l(2\ k_2 a + k_3 s)$$

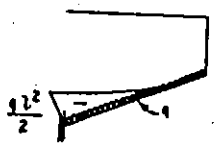


Fig. 31.51 *Bending moment diagram due to q*

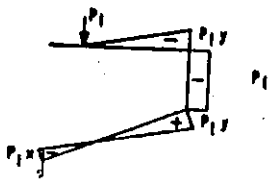


Fig. 31.52 *Bending moment diagram due to p_1*

· *Influence coefficients involved in Eq. (31.26)*

$$\delta_{21} = -0.50\ k_2 a^2 l - 0.166\ k_3 ls(H + 2a)$$
$$\delta_{22} = 0.333\ k_2 a^3 + 0.333\ k_3 s(H^2 + Ha + a^2)$$
$$\delta_{23} = 0.500\ k_2 a^2 + 0.500\ k_3 s(H + a)$$
$$\delta_2 w = -0.250\ k_2 wl^2 a^2 + 0.083\ k_3 wl^2 s(H - a)$$
$$\delta_2 q = 0.043\ k_3 ql^2 s(3H + a)$$
$$\delta_2 P_1 = -0.500\ k_2 P_1 a^2 y + 0.166\ k_3 P_1 x^2 \times \left[3H - \frac{x}{l}(H - a)\right] \frac{s}{l} - 0.166\ k_3 P_1 y^2 \times \left[3a + \frac{y}{l}(H - a)\right] \frac{s}{l}$$
$$\delta_2 P_2 = 0.166\ k_3 P_2 ls(2H + a)$$
$$\delta_2 M_D = 0.500\ k_2 M_D a^2 + 0.500\ k_3\ M_D s\ (H + a)$$

~ *Influence coefficients involved in Eq. 31.27*

$$\delta_{31} = -0.500\ k_1 l^2 - k_2 la - 0.500\ k_3 ls$$
$$\delta_{32} = 0.500\ k_2 a^2 + 0.500\ k_3 s(H + a)$$

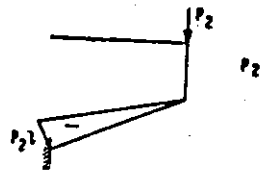


Fig. 31.53 *Bending moment diagram due to p_2*

$$\delta_{33} = k_1 l + k_2 a + k_3 s$$
$$\delta_3 w = -0.166\ k_1\ wl^3 - 0.500\ k_2\ wl^2 a$$
$$\delta_3 q = 0.166\ k_3 ql^2 s$$
$$\delta_3 P_1 = -0.500\ P_1 y(k_1 y + 2k_2 a) - 0.500 k_3 P_1 \times (y^2 - x^2)\frac{s}{l}$$
$$\delta_3 P_2 = 0.500\ k_3 P_2 ls$$
$$\delta_3 M_D = k_2 M_D a + k_3 M_D s$$

In the design example under illustration on 1-ft width basis:

w= 137 lb/ft
q= 110 lb/ft
P_1= 2,670 lb
P_2= 4,440 lb
M_D= 2,390 lb ft

$$I_1 = \left(\frac{1}{12}\right) \times 1 \times \left(\frac{8}{12}\right)^3 = 0.025\ \text{ft}^4$$

Hence, $k_1 = \dfrac{1}{I_1} = 40\ \text{ft}^{-4}$

Similarly, $k_2 = 0.633\ \text{ft}^{-4}$

$k_3 = 40.0\ \text{ft}^{-4}$

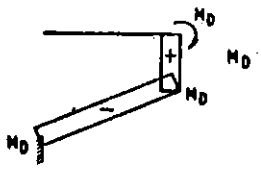


Fig. 31.54 *Bending moment diagram due to M_D*

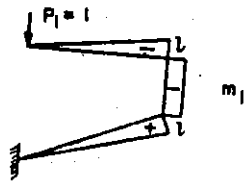


Fig. 31.55 *Bending moment diagram due to m*

Substituting the above in kip and ft. units in the formulae for the various influence coefficients derived earlier, the values obtained are given below:

For Eq. (31.25):

$\delta_{11} = 15061.04, \quad \delta_1 q = -879.80,$
$\delta_{12} = -3917.10, \quad \delta_1 P_1 = 24750.07,$
$\delta_{12} = -2767.46, \quad \delta_1 P_2 = -17176.92,$
$\delta_1 w = 5234.85, \quad \delta_1 M_D = -3431.37,$

therefore, $\delta_{10} = 8496.33$

For Eq. (31.26):

$\delta_{21} = \delta_{12}, \quad \delta_2 q = 1901.51,$
$\delta_{22} = 4389.70, \quad \delta_2 P_1 = -4172.31,$
$\delta_{23} = 1181.97, \quad \delta_2 P_2 = 25259.84,$
$\delta_2 w = 990.85, \quad \delta_2 M_D = 2824.92,$

therefore, $\delta_{20} = 26804.81$

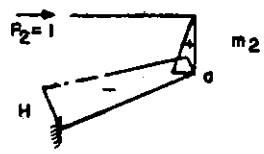


Fig. 31.56 *Bending moment diagram due to m_2*

For Eq. (31.27):

$\delta_{31} = \delta_{13}, \quad \delta_3 q = 425.38,$
$\delta_{32} = \delta_{23}, \quad \delta_3 P_1 = -3985.67,$
$\delta_{33} = 677.35 \quad \delta_3 P_2 = 6340.32,$
$\delta_3 w = -498.76, \quad \delta_3 M_D = 838.77,$

therefore, $\delta_{30} = 3119.99$

Substituting these values of the influence coefficients in the three compatibility Eqs. (31.25), (31.26) and (31.27):

$$15061.04p_1 - 3917.10p_2 - 2767.46p_3 + 8496.33 = 0$$
$$-3917.10p_1 + 4389.70p_2 + 1181.97p_3 + 26804.81 = 0$$
$$-2767.46p_1 + 1181.97p_2 + 677.35p_3 + 3119.99 = 0$$

Solving these three equations simultaneously:

$$\left.\begin{aligned} p_1 &= -3.848 \\ p_2 &= -7.696 \\ p_3 &= -6.630 \end{aligned}\right\} \quad \text{(kip and ft. units)}$$

With the three redundants p_1, p_2 and p_3 determined, the bending moment at any point may now be readily calculated from the following:

Bending moment at any section $= m_0 + m_1 p_1 + m_2 p_2 + m_3\, p_3$ where m_0 = sum of the ordinates of the bending moment diagrams due to applied loads and moments.

Thus the bending moment, say at the junction between the top slab and the central spine,

$= 0 + (1.00)p_3 = 1.00 \times (-6.630)$
$= 6{,}630$ lb ft per ft width (hogging)

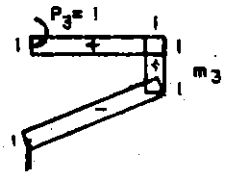


Fig. 31.57 *Bending moment diagram due to m_3*

Similarly, the thrust at this section:

$= n_0 + n_1 p_1 + n_2 p_2 + n_3 p_3$
$= 0 + (1.00)p_2 = 1.00 \times (-7.696)$
$= 7{,}696$ lb ft per ft width (tension)

The shear at this section

$= s_0 + s_1 p_1 + s_2 p_2 + s_3 p_3$
$= 0 + (-1.00)p_1 = (-1.00) \times (-3.8484)$
$= 3{,}848$ lb per ft width.

Similarly, the bending moment, thrust and shear can be calculated at any other point in the frame and the analysis completed.

CASE (E) Voided deck section

Figure 31.58 represents a section through a voided slab deck. For the transverse analysis, such a deck section may be assumed as comparising of two elastic cantilevers protruding on either side of centre-line of the deck, on rigidly holding the other.

Considering one of these cantilevers, if M and S represent the total bending moment and shearing force at the root of this cantilever per foot of its width, and if w and l represent the equivalent uniformly distributed load and the equivalent cantilever span respectively, such that

$$\frac{wl^2}{2} \equiv M$$

and

$$wl \equiv S$$

then, rearrangement of terms gives

$$S\frac{l}{2} = M$$

or

$$l = \frac{2M}{S} \qquad (31.28)$$

and $$w = \frac{S^2}{2M} \qquad (31.29)$$

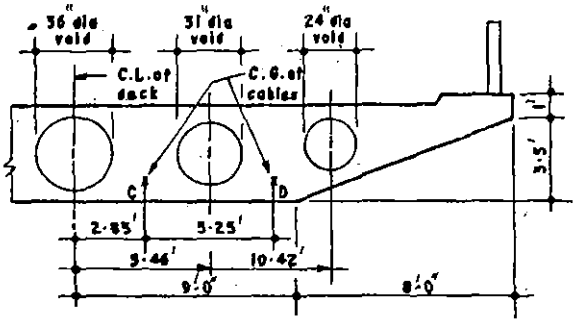


Fig. 31.58 *Voided slab deck*

It is further assumed that the longitudinal prestressing cables support the cross-section of the deck all along their length in a manner somewhat akin to the cables in a suspension bridge. The longitudinal prestressing cables are assumed to provide the supporting reactions in proportion to the product of their number and drape in a particular span. The bending moment and shearing force at any point in the transverse section of the deck can then be calculated from statics.

In the design example under illustration, per foot-width basis, the bending moment M and shearing force S at the root of the cantilever are as follows:

Load	*M*	*S*
Self and superimposed deadload	58,800 lb ft	7,315 lb
Live Load	31,500 lb ft	4,930 lb
TOTAL	90,300 lb ft	12,245 lb

Therefore, for the equivalent loaded cantilever

$$l = \frac{2M}{S} = \frac{2 \times 90.3}{12.245} = 14.72 \text{ ft}$$

$$w = \frac{S^2}{2M} = \frac{12.245^2}{2 \times 90.3} = 832 \text{ lb/ft}$$

Figure 31.59 represents the transverse section of the deck in terms of the equivalent loaded cantilevers, described above, in which A, B, C and D are the concealed supports provided by the longitudinal prestresssing cables. In the present case there are four 24/0.6 in Freyssinet multistrands at A, B, C and D, each. All the four cable groups are similarly profiled so that they all have the same drapes. Thus each of the four supports provides a reaction of $\frac{w(2l)}{4}$ i.e. $\frac{0.832(2 \times 14.72)}{4} = 6{,}120$ lb

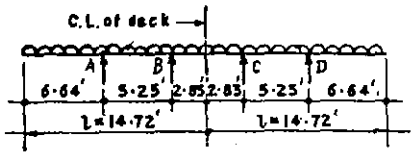


Fig. 31.59 *Equivalent elastic cantilevers of length l*

Hence the bending moment, say at the centre-line of the deck

$$= 6.12(2.83 + 5.25 + 2.83) - 0.832 \times \frac{(14.72)^2}{2}$$

$$= -23{,}200 \text{ lb ft per ft width (hogging).}$$

Thus the bending moment may be determined at any point in the transverse section of the deck.

CASE (F): Analysis of 'Deck-slab' in ordinary 'Beam-&-slab' type of deck and ordinary 'Slab' type of deck

This may be done by various simplified/emperical methods, e.g., those given in the AASHTO (and A.C.I.) bridge design specifications (which are summarised below for a ready reference), those suggested by Westergaard as early as 1930 (!), those suggested in the Indian Roads Congress Specifications for bridge design (which calculate the effective-width of wheel load dispersion by yet other emperical formulae (!) and require the use of 'influence-line', etc. Rüsch and Hergenroader have produced 'influence-surfaces, for analysing moments in skew slabs. Subsequently, Püchher also has produced 'influence surfaces' for computing moments in deck slabs under wheel loads. Each approach has its own limitations. However, the below described AASHTO-ACI approach is good enough for most cases, and is extremely quick to apply:

Design of Deck Slab—Empirical Method

- **Limitations**

The method described here applies to monolithic concrete deck slabs carrying highway loads. It should not be used where the skew angle exceeds about 20 degrees. The skew angle is defined as the horizontal angle between the abutment and the perpendicular to the bridge centre line.

For skews greater than 20 degrees, the slab design should be based on model tests, studies of highway skew slab bridges done by other agencies, a rational analysis based on principles of the theory of plates, or other acceptable methods (e.g. plane-grid finite element).

The reinforcement at the obtuse corners should receive special consideration.

- **One-way Slab**

The design of one-way slabs should be based on the analysis of a strip of unit width at right angles to the supports

considered as a rectangular beam. Bending moment per unit width of slab due to a standard truck load, should be calculated according to empirical formulae given below, unless more exact methods are used. In the following

M_l = Live load moment per foot width of slab, in foot-pounds

l_n = Clear span length of slab, in feet

E = Effective width of slab resisting a wheel or other concentrated load, in feet

P_l = Load on one rear wheel of truck, in pounds

(a) Main reinforcement perpendicular to traffic (Spans 2-24 feet, inclusive). The live load moment for simple spans should be determined by the following formula (impact not included):

$$M_l = \frac{l_n + 2}{32} P_l$$

In slabs continuous over three or more supports, a continuity factor of 0.8 should be applied to the above formula for both positive and negative moments.

(b) Main reinforcement parallel to traffic: Longitudinally reinforced slabs should be designed for the appropriate truck or lane loading, which ever causes larger design moment. The effective width of slab resisting a wheel load should be estimated to be

$$E = 4 + 0.06\, l_n \text{ (in feet)}$$

but not to exceed 7 ft. The effective slab width resisting lane load should be taken as $2E$.

For simple spans, the maximum live load moment per foot width of slab, without impact, may be approximated by the following formulae:

Loading: In case of AASHTO Truck

Spans up to and including 50 feet:

$M_l = 900 l_n$ foot-pounds

spans 50–100 ft

$M_l = 1000(1.30 l_n - 20.0)$ foot-pounds.

Moments in continuous spans should be determined by a suitable analysis using the truck or appropriate lane loading.

Edge beams should be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb. The edge beam should be designed to resist a live load moment of 0.10 P_{ln} for simple spans. For continuous spans, 80% of the above calculated value should be used for both positive and negative moments, unless a greater reduction results from a more exact analysis.

• Two-way Slab

Two-way slabs are supported on all four sides by beams, girders or walls and reinforced in both directions. For rectangular slabs simply supported on all four sides, the proportion of the load carried by the short span of the slab may be estimated by the following equations:

For load uniformly distributed, $p = \dfrac{l_2^4}{l_1^4 + l_2^4}$

For load concentrated at centre, $p = \dfrac{l_2^3}{l_1^3 + l_2^3}$

where: p = proportion of load carried by short span

l_1 = length of short span of slab

l_2 = length of long span of slab.

where l_2 exceeds $1\frac{1}{2}$ times l_1, the entire load should be assumed to be carried by the short span.

For concentrated load, the effective slab width, E, for the load carried in either direction should be determined in accordance with (*a*) and (*b*) above.

Moments obtained should be used in designing the center half of the short and long spans. The reinforcement steel in the outer quarters of both short and long spans may be reduced 50 per cent.

For other supporting conditions at the edges, the formulae for p can be adjusted to account for the restraining effects the edges. At the ends of the bridge and at the intermediate points where the continuity of the slab is broken, the edges should be supported by diaphragms or other suitable means.

• Ribbed Slabs

A two-way system consisting of a slab with equally closely spaced ribs of similar size in two directions may be analyzed by the empirical method as an equivalent two-way slab. For two-way systems in which the ribs are not equally spaced or are of different size, an elastic analysis may be used in which the system is treated as an equivalent orthotropic plate. A model analysis may also be used.

• Cantilever Slabs

(a) *Truck loads:* Cantilever slabs may be designed to support truck wheel load independent of edge support along the end of the cantilever. The provisions given below include the effect of wheels on parallel elements.

(1) *Main reinforcement perpendicular to traffic:* The effective slab width, perpendicular to traffic, for each wheel load should be

$$E = 0.8\chi_1 + 3.75 \text{ (in feet)}$$

where χ_1 = distance in feet from load to point of support.

(2) *Main reinforcement parallel to traffic:* The effective slab width, parallel to traffic, for each wheel load, should be

$$E = 0.35\chi_1 + 3.2 \text{ (in feet) but not to exceed 7.0 feet}$$

(b) *Railing loads:* The effective width of slab resisting railing post loadings should be estimated as $E = 0.8\chi_2 + 3.75$ if no parapet is used and $E = 0.8\chi_2 + 5.0$ if a parapet is used. In the above expressions. χ_2 is the distance in feet from the center of the post to the point under investigation.

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ANNEXURES

ANNEXURE 1

Expressing the wheel-load as a mathematical function: In order to transform the HB loading into a smooth continuous loading function, the Fourier series is made use of. The equation for a point load P at $x = x_1$ in the span may be expressed as

$$p(x_1) = 2\frac{P}{L} \sum_{n=1,2,3,\ldots} \sin\frac{n\pi x_1}{L} \sin\frac{n\pi x}{L}$$

Considering only the first three terms in the series:

$$p(x_1) = \frac{2P}{L}\left(\sin\frac{\pi x_1}{L}\sin\frac{\pi x}{L} + \sin\frac{2\pi x_1}{L}\sin\frac{2\pi x}{L} + \sin\frac{3\pi x_1}{L}\sin\frac{3\pi x}{L}\right)$$

For all four point loads, the equation of equivalent continuous loading curve becomes

$$\begin{aligned} p(x) &= p(x_1) + p(x_2) + p(x_3) + p(x_4) \\ &= \frac{2P}{L}\left[\sin\frac{\pi x}{L}\left(\sin\frac{\pi x_1}{L} + \sin\frac{\pi x_2}{L} + \sin\frac{\pi x_3}{L} + \sin\frac{\pi x_4}{L}\right) + \sin\frac{2\pi x}{L}\left(\sin\frac{2\pi x_1}{L} + \sin\frac{2\pi x_2}{L} + \sin\frac{2\pi x_3}{L} + \sin\frac{2\pi x_4}{L}\right) + \sin\frac{3\pi x}{L}\left(\sin\frac{3\pi x_1}{L} + \sin\frac{3\pi x_2}{L} + \sin\frac{3\pi x_3}{L} + \sin\frac{3\pi x_4}{L}\right)\right] \end{aligned}$$

Referring to Fig. 31.16 it is noted that $x_1 = 41.5$ ft, $x_2 = 47.5$ ft, $x_3 = 67.5$ ft, and $x_4 = 73.5$ ft; substituting these values in the above equation:

$$p(x) = \frac{2P}{L}\left(3.7398\sin\frac{\pi x}{L} - 1.8942\sin\frac{3\pi x}{L}\right)$$

It may be noted that this is the load on the two boxes.

ANNEXURE 2

Derivation of formulae for various deflections and rotations

(a) Simple vertical deflection of a box beam due to asymmetric load: The intensity of loading on one box is $\left(\frac{1}{2}\right)p(x)$, Fig. 31.60 and is given by

$$\frac{P}{L}\left(3.7398\sin\frac{\pi x}{L} - 1.8942\sin\frac{3\pi x}{L}\right)$$

From the general equation of flexure

$$EI_b\frac{d^2y}{dx^2} = -M_x, (M_x \text{ being sagging moment})$$

Therefore, $EI_b\dfrac{d^4y}{dx^4} = -\dfrac{d^2M_x}{dx^2}$

The second differential of moment represents the first differential of shear, which represents the load intensity in magnitude.

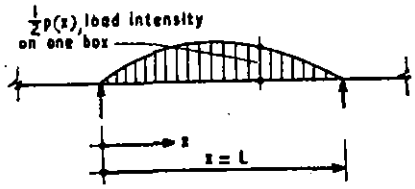


Fig. 31.60 *Variable downward loading along span*

Hence, $EI_b\dfrac{d^4y}{dx^4} = \dfrac{P}{L}\left(3.7398\sin\dfrac{\pi x}{L} - 1.8942\sin\dfrac{3\pi x}{L}\right)$

Integrating both sides four times in succession, and evaluating the four constants of integration from the four boundary conditions, at $x = 0$ and at $x = L$, each,

$EI_b\dfrac{d^2y}{dx^2} = 0.13PL$, (see Annexure 4)

and, $y = 0$

Finally it is found that

$$y = \frac{PL^3}{81\pi^4 EI_b}\phi_1 + \frac{PL}{15.39EI_b}\phi_2$$

where $y =$ deflection of box beam

$$\phi_1 = 302.8\sin\frac{\pi x}{L} - 1.8942\sin\frac{3\pi x}{L}$$

$$\phi_2 = x(x - L)$$

(b) Deflection at the 'cut' due to twist in the box-beam caused by asymmetric load, and (g) twist rotation in a box beam caused by symmetric load: The magnitude of twist rotation caused in a box beam by asymmetric as well as symmetric load is the same. The intensity of torque-loading on a box (Fig. 31.61),

$$\begin{aligned} &= \text{load} \times \text{arm} \\ &= \frac{1}{2}p(x)b \\ &= \frac{Pb}{L}\left(3.7398\sin\frac{\pi x}{L} - 1.8942\sin\frac{3\pi x}{L}\right) \end{aligned}$$

This represents $-\dfrac{dT}{dx}$ (just as first differential of shear represents the loading in magnitude). Assuming GK as constant and noting that

$$GK\frac{d\theta}{dx} = T, \text{ (as explained in the text)},$$

$$GK\frac{d^2\theta}{dx^2} = \frac{dT}{dx}$$

$$= -\frac{Pb}{L}\left(3.7398\sin\frac{\pi x}{L} - 1.8942\sin\frac{3\pi x}{L}\right)$$

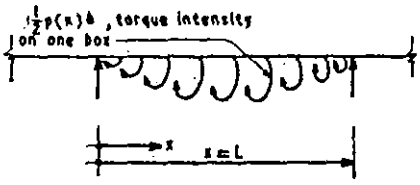


Fig. 31.61 *Variable torque loading along span*

Integrating both sides twice in succession, and evaluating the two constants of integration from the two boundary conditions, at $x = 0$ and at $x = L$, each, $\theta = 0$, finally it is seen that,

$$\theta = \frac{PbL}{9\pi^2 GK}\theta_3 \quad (= (\mathbf{g}))$$

where θ = twist rotation in the box beam under symmetric or asymmetric loading

$$\phi_3 = \left(33.62\sin\frac{\pi x}{L} - 1.8942\sin\frac{3\pi x}{L}\right)$$

Hence the deflection at the 'cut' due to twist in the box beam caused by asymmetric load

$$= \text{twist rotation} \times \text{arm}$$

$$= \theta a$$

$$= \frac{PabL}{9\pi^2 GK}\phi_3 \quad (= (\mathbf{b}))$$

(c) Deflection at the cut due to local bending of cantilever under asymmetric load: This is the case of a cantilever subjected to a uniformly distributed total load of $\frac{1}{2}p(x)$. Its intensity, based on 12-ft dispersion (explained in the text), is $\frac{1}{24}p(x)$ lb/ft/ft. Deflection at the free end of cantilever under this uniformly distributed load

$$= \frac{\frac{1}{24}p(x)l^4}{8EI_c}$$

where $p(x) = \dfrac{2P}{L}\left(3.7398\sin\dfrac{\pi x}{L} - 1.8942\sin\dfrac{3\pi x}{L}\right)$

Hence deflection

$$= \frac{Pl^4}{96EI_cL}\phi_4$$

where $\phi_4 = \left(3.7398\sin\dfrac{\pi x}{L} - 1.8942\sin\dfrac{3\pi x}{L}\right)$

(d) Simple vertical deflection of a box beam due to q_x: The intensity of loading (Fig. 31.62):

$$q_x = q_1 + q_2\left[\alpha\sin\frac{\pi x}{L} + (1-\alpha)\sin\frac{3\pi x}{L}\right]$$

Now $EI_b\dfrac{d^4y}{dx^4} = q_x$ (the loading)

$$= q_1 + q_2\left[\alpha\sin\frac{\pi x}{L} + (1-\alpha)\sin\frac{3\pi x}{L}\right]$$

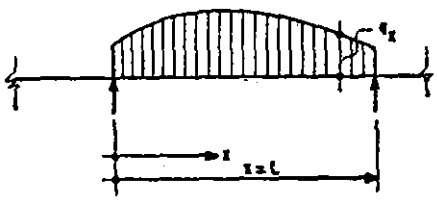


Fig. 31.62 *Variable load q_x along span*

Integrating both sides four times in succession, and evaluating the four constants of integration from the four boundary conditions, at $x = 0$ and at $x = L$, each,

$EI_b\dfrac{d^2y}{dx^2} = [q_1 + 0.6\ q_2(2\alpha - 1)] \times 0.0536\ L^2$ (see Annexure 4)

and $y = 0$.

Finally it is seen that

$$y = \frac{q_1}{67.13EI_b}\phi_6 + \frac{q_2L^4}{81\pi^4EI_b}\phi_5 + \frac{(2\alpha-1)L^2q_2}{62.25EI_b}\phi_7$$

where $\phi_6 = x(2.8x^3 - 5.6Lx^2 + 1.8L^2x + L^3)$

$$\phi_5 = \left[81\alpha\sin\frac{\pi x}{L} + (1-\alpha)\sin\frac{3\pi x}{L}\right]$$

$$\phi_7 = x(x - L)$$

(e) Deflection at the cut due to twist in the box beam caused by q_x: The intensity of torque-loading on a box

$$= \text{load} \times \text{arm} = q_x \cdot a$$

$$= aq_1 + aq_2\left[\alpha\sin\frac{\pi x}{L} + (1-\alpha)\sin\frac{3\pi x}{L}\right]$$

Following the same explanation as given under (*b*) earlier, for the present case

$$GK\frac{d^2\theta}{dx^2} = \frac{dT}{dx}$$
$$\equiv -q_x a$$

Substituting for q_x, integrating both sides of the above equation twice in succession, and then evaluating the two constants of integration from the two boundary conditions, at $x = 0$ and at $x = L$, $\theta = 0$, it is finally found that

$$\theta = \frac{aq_1}{2GK}\phi_8 + \frac{aq_2L^2}{9\pi^2GK}\phi_9$$

where θ = twist rotation in the box beam due to q_x,

$\phi_8 = x(L - x)$

$\phi_9 = x\left[9\alpha \sin\frac{\pi x}{L} + (1-\alpha)\sin\frac{3\pi x}{L}\right]$

Hence deflection at the 'cut' due to twist in the box beam caused by q_x

= twist rotation × arm

$= \theta_a$

$$= \frac{a^2q_1}{2GK}\phi_8 + \frac{a^2q_2L^2}{9\pi^3GK}\phi_9$$

(f) Deflection at the 'cut' due to local bending of cantilever under q_x: This is the case of a cantilever subjected to a point load q_x at its tip. Deflection at the tip is equal to $\frac{q_x l^3}{3EI_c}$

i.e. $\frac{q_1 l^3}{3EI_c} + \frac{q_2 l^3}{3EI_c}\left[\alpha \sin\frac{\pi x}{L} + (1-\alpha)\sin\frac{3\pi x}{L}\right]$

$= \frac{q_1 l^3}{3EI_c} + \frac{q_2 l^3}{3EI_c}\phi_{10}$

where $\phi_{10} = \alpha \sin\frac{\pi x}{L} + (1-\alpha)\sin\frac{3\pi x}{L}$

(h) Flexural rotation at the cut due to local bending of cantilever under symmetric load: This is the case of a cantilever subjected to a uniformly distributed total load of $\frac{1}{2}p(x)$, whose intensity is $\frac{1}{24}p(x)$ lb/ft/ft, as explained under (*c*) earlier. Flexural rotation at the free end of the cantilever under this uniformly distributed load

$$= \frac{\frac{1}{24}p(x)l^3}{6EI_c}$$

$= \frac{Pl^3}{72EI_cL}\phi_4$ {$p(x)$ and ϕ_4 have been defined already}.

(j) Twist rotation in a box-beam due to M_x.

$$M_x = M_1 + M_2\left[\gamma \sin\frac{\pi x}{L} + (1-\gamma)\sin\frac{3\pi x}{L}\right]$$

M_x being analogous to the product $q_x a$, in the formula for θ in (*e*) earlier, $q_x a$ is replaced by M_x thus:

$$\theta = \frac{M_1}{2GK}\phi_8 + \frac{M_2L^2}{9\pi^2GK}\phi_{11}$$

where θ = twist rotation in box-beam due to M_x

$\phi_8 = x(L - x)$

$\phi_{11} = 9\gamma \sin\frac{\pi x}{L} + (1-\gamma)\sin\frac{3\pi x}{L}$

(k) Flexural rotation at the 'cut' due to local bending of cantilever under M_x: This is the case of a cantilever subjected to a constant moment M_x along its length. Flexural rotation caused at the tip of the cantilever is given by

$\frac{M_1 l}{EI_c}$

i.e. $\frac{M_1 l}{EI_c} + \frac{M_2 l}{EI_c}\left[\gamma \sin\frac{\pi x}{L} + (1-\gamma)\sin\frac{3\pi x}{L}\right]$

or $\frac{M_1 l}{EI_c} + \frac{M_2 l}{EI_c}\phi_{12}$

where $\phi_{12} = \gamma \sin\frac{\pi x}{L} + (1-\gamma)\sin\frac{3\pi x}{L}$

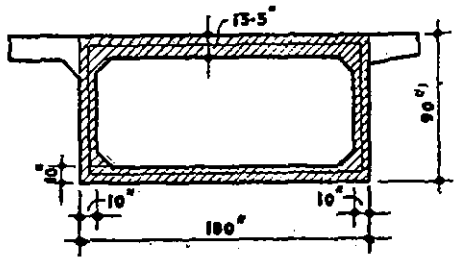


Fig. 31.63 *Torsional rigidity—box section*

ANNEXURE 3

Torsional rigidity of a box-beam: Timoshenko and Goodier in their book *Theory of Elasticity*, published by McGraw-Hill Book Company Inc. of New York, show that the twist θ in the case of a thin walled tube can be expressed as

$$\theta = \frac{T}{4A^2G}\Sigma\frac{s}{t}$$

where A is the mean of the areas enclosed by the outer and the inner boundaries of the cross-section of the tube, s is

the length, and t the thickness of the elements composing the cross-section of the tube.

In the present instance (Fig. 31.63)

$$A = \frac{(180 \times 90) + (160 \times 66.5)}{2}$$
$$= 13420 \text{ in}^2 \text{ or } 93.3 \text{ ft}^2$$
$$\Sigma\frac{s}{t} = \frac{170}{10} + \frac{170}{13.5} + \frac{78.25}{10} \times 2$$
$$= 45.25$$
$$G = \frac{E}{2(1 + \text{Poisson's ratio})}$$
$$= \frac{3 \times 10^6}{2(1 + 0.15)}$$
$$= 1.304 \times 10^6 \text{ lb/in}^2$$
$$\text{or } 187.8 \times 10^6 \text{ lb/ft}^2$$

$$\text{Therefore, } \theta = \frac{T \times 45.25}{4 \times 93.3^2 \times 187.8 \times 10^6}$$
$$= \frac{T}{14.45 \times 10^{10}}$$

so that, $\frac{T}{\theta} = 14.45 \times 10^{10}$ lb/ft^2

per unit length, $GK = \frac{T}{\theta}\left(\text{from}\frac{T}{K} = \frac{G\theta}{L}\right)$

Hence, $GK = 14.45 \times 10^{10}$ lb/ft^2

ANNEXURE 4

Bending moment in the box-beam at support sections due to asymmetric load (placed as in Fig. 31.16 and due to redundant q_x:

(i) due to asymmetric load placed longitudinally as in Fig. 31.16. This load, on one box-beam, comprises 4 axles of halved HB vehicle, Fig. 31.15. By reading under the load the ordinates of the bending moment influence diagram for an intermediate support ($x = 0$ or $x = L$) of a multispan continuous beam (assuming constant EI), the bending moment, $\left(i.e.\ EI \cdot \frac{d^2y}{dx^2}\right)$, is obtained. This comes to 0.13 PL (of the hogging type). This approach is approximate.

(ii) due to q_x:

$$q_x = q_1 + q_2\left[\alpha \sin\frac{\pi x}{L} + (1 - \alpha)\sin\frac{3\pi x}{L}\right]$$

Therefore, at $x = 0$ and at $x = L$, $qx = q_1$, and at $x = \frac{L}{2}$, $q_x = q_1 + q_2(2\alpha - 1)$. Note that q_1 distribution is rectangular, *i.e.*, it is a uniformly distributed load, q_2 distribution is sinusoidal, *i.e.*, it lies between a convex parabola and a triangle, which may, therefore, be approximated into an equivalent rectangular distribution whose average ordinate is 0.6 of the maximum, *i.e.* $0.6q_2(2\alpha - 1)$.

Thus, for the present consideration only, q_x may be approximated into a uniformly distributed load whose intensity is given by $q_1 + 0.6q_2(2\alpha - 1)$. Placing this load on the bending moment influence diagram for an intermediate support ($x = 0$ or $x = L$) of a multispan continuous beam (assuming constant EI), the bending moment $\left(i.e. EI \cdot \frac{d^2y}{dx^2}\right)$ is obtained. This comes to:

$0.0536\ L^2\{q_1 + 0.6q_2(2\alpha - 1)\}$ and is of hogging type.

CHAPTER 32

Expansion Joints for Bridge Decks

A deck movement joint is defined as a structural discontinuity between two elements, at least one of which is a deck element, designed to permit relative translation and/or rotation between the two. This definition includes all joints in the deck or between the deck and the abutments at which 'movement' can take place, but excludes construction joints. An expansion joint implicitly also refers to a contraction joint and hence it is more rational to call it a movement joint. However, in general, it is called an expansion joint merely because of colloquial usage of the name.

The width of the gap, the maximum linear expansion and contraction and rotation of the bridge structure at the gap are the predominant factors in the design. The design of the joint should allow free rotation and deflection or shifting of the structure at the edges without damage or inconvenience to the traffic. Structurally, the expansion joint has to be strong enough to withstand the knocking of the wheels of vehicles passing over the bridge, and the latter is the main cause for gradual damage and disintegration of joints.

The basic philosophy sometimes adopted is that 'minimum number of joints be used consistent with the need for articulation of the structure'. The approach to eliminate as many expansion joints as possible is taken because of the generally poor performance of deck joints. There are considerable benefits to be derived from eliminating joints entirely, where possible, as might be the case for curved structures or those which have flexibility in the support system, such as, with the slender or hinged columns. The structure acts like a bow with the superstructure fixed at both abutments. Movement at intermediate supports is accommodated by sliding bearings.

Where deck joints are required, the following two types should be generally excluded because of unsatisfactory service records: any joint which has been found prone to failure within a few years of installation, and joints placed directly in concrete without provision for armour at its edges with the roadway surface whereby they fail because of spalling of the concrete. This type of failure was common when it was the normal practice to install compression seals (see ahead) in direct contact with a concrete deck slab.

Deck joints divide into those which are designed to be sealed (waterproof) and those that are of open type (designed to permit the free flow of water through the joint). All joints must be designed so that none of the components fail at serviceability limit states and the sealed joints should remain watertight. Since deck joints do not directly affect the safety of a structure, some damage can be tolerated from conditions exceeding serviceability limit states since it would be uneconomical to design joints for ultimate limit states! Acceptable damage is considered to be that which can be repaired while the deck remains in service, even though one or more lanes may have to be closed to traffic to permit the repairs to be carried out.

32.1 SOURCES OF MOVEMENTS AND LOADS AT EXPANSION JOINTS

Movements in a structure result from either 'loading conditions' or a 'change in volume' of the components of the structure. The magnitude of the movement must be calculated for each structure since it depends upon the nature of the structure and its response to static and dynamic forces. The most common sources of loads on and movements in deck joints are direct impact of wheels, temperature, and those caused by soil conditions which result in the movement of supports. Conversely, some joints, particularly compression seals or those which accommodate movement by the shear deformation of an elastomer, can impose some forces on the structure which may have to be included in the design of the structure. This characteristic of compression seals and certain proprietary joints may prevent substitution of these joints for those which do not exert forces on the structure—unless the forces are not significant.

A checklist, which has been found to be a convenient method of ensuring that all factors affecting the magnitude of movements and loads have been considered in design, is as follows:

(a) Properties of the materials in the structure, including coefficient of thermal expansion, modulus of elasticity, Poisson's ratio, creep and shrinkage strains
(b) Effective temperature variation in the structure
(c) Sizes and properties of the structural members
(d) Method and sequence of construction

(e) Tilt, settlement and movement of supports
(f) Construction tolerances
(g) Static and dynamic response of the structure
(h) Interaction of the forces to which the structure may be subjected, including dead and live loading; wind, earthquake and earth pressures
(i) Structural restraints, if any

Joints which are considered 'fixed' may be fixed against translation, but not rotation. Flexural deflection of a member results in rotation at the joint, and failure to accommodate this rotation has been a common cause of leakage and deterioration at 'fixed' joints.

The 'plan-geometry' of a bridge superstructure is an important factor in determining the movements which take place at different points in the deck structure. Bridges which are straight and narrow in plan will generally exhibit movements along their centreline in response to changes in temperature. Bridges which are curved in plan or with skew joints have two components of movement at deck joints, a longitudinal component along the centreline of the superstructure and a transverse component perpendicular to the longitudinal centreline as shown in Figs. 32.1 and 32.2. The transverse component can cause serious problems with some types of expansion devices, for example, 'finger' joints (sometimes called 'tooth' or 'comb' joints). Binding up of the joint can result in large forces on the joints, bearings and abutments with corresponding reactions at intermediate piers. (Skewing the fingers in the finger joints can be costly and cumbersome.)

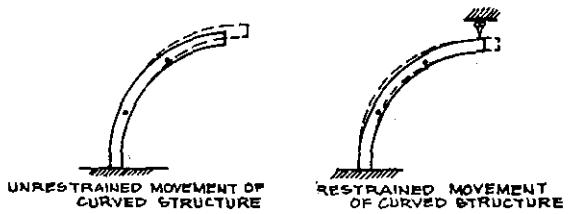


Fig. 32.1 *Response of curved structures*

To determine the actual movement of a bridge super-structure, at all the points of attachment of the superstructure, the position of the zero movement point must be established. The position of this point depends upon the geometry of the superstructure, the location of the piers and bearings and their relative shear stiffnesses. For straight structures, the zero movement point is generally located on the longitudinal centreline of symmetry as shown in Fig. 32.3. In curved structures the zero movement point will generally not be located on the longitudinal centreline and may even fall outside the plan of the superstructure. Movements are always towards and away from the zero movement point as shown in Fig. 32.4. The calculation of the location of the zero movement point can be quite involved for complex structures. Detailed explanations for determining the location of this point have been presented in two separate chapters in this book.

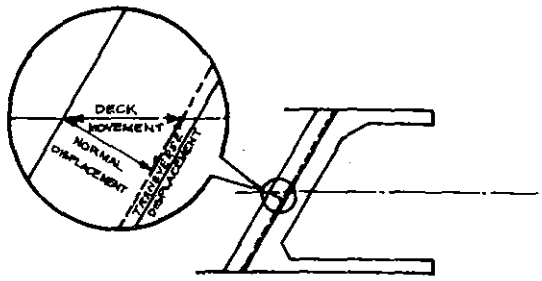


Fig. 32.2 *Normal and transverse displacements across a skewed joint*

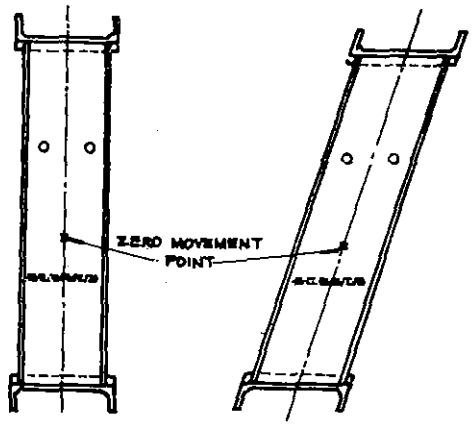


Fig. 32.3 *(a) 'Right' superstructure (b) 'Skewed' superstructure. Zero movement point in straight and skewed superstructures*

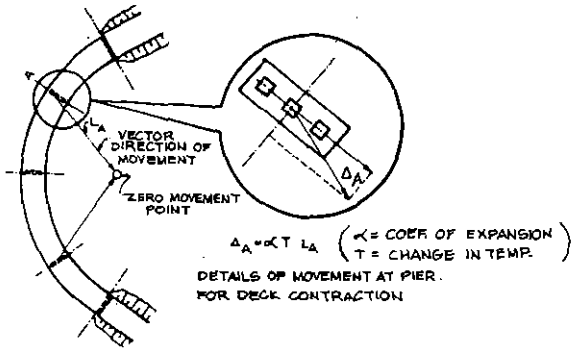


Fig. 32.4 *Zero movement point in curved structures*

32.2 FUNCTIONAL REQUIREMENTS OF EXPANSION JOINTS

The first step towards a performance specification is a statement of the required results. Four such requirements for deck joints are:

(a) The components of the joint must accommodate the translation and rotation of the structure at the joint.
(b) The joint must provide for the passage of traffic across the joint without significantly impairing the riding quality of the pavement surface. It must not be a safety hazard to road users, including pedestrians and cyclists where they are permitted on the bridge.
(c) The joint shall be almost maintenance-free for the design-life of the structure.
(d) Deck joints must be constructed and detailed such that they prevent damage to the deck, bearings, piers and abutments from water, deicing chemicals and other foreign materials, either by preventing water from passing through the joint or by permitting water to drain through the joint in such a manner that damage to the deck, bearings, pier or abutment, does not occur. The selection of an 'open' type joint (which allows water to percolate through) will normally require particular attention to the details of components of the structure other than the joint itself.

The maximum permissible gap in a joint at the serviceability limit states is generally limited to 100 mm for deck joints with a 'single' opening, and to 80 mm for each opening in deck joints with 'multiple' openings. The gap is measured perpendicular to the length of the joint at the deck surface. These limits are based on experience and have been found to be the maximum values which can be permitted, consistent with maintaining an adequate quality of ride across the joint for user comfort and safety of vehicle loads. While improvements in ride-quality result from keeping the maximum gap as small as possible, this requires the use of more multiple opening joints which cost considerably more than joints with single openings. Since 'incompressibles' do get trapped in many types of sealed joint, the minimum gap should be not less than 20 mm in order to prevent the transfer of forces through the debris in the joint. Consequently, limits on the minimum gap also increase the maximum gap for the same total movement capacity.

Open Type Joints

Under certain conditions, as open type of joint can provide a good enough, effective and an economic solution. Satisfactory performance depends upon the provision of an effective deck drainage system, control of flow of surface drainage through the joint and disposal of the run-off from the site. The success of open joints in the past has been limited by the damage (to the portion of the structure immediately beneath the joint) from debris and run-off, especially in areas where deicing salts are used. Wet dirt often accumulates on the flanges of steel beams or around bearings, and leads to corrosion. Chlorides penetrate concrete surfaces, causing the spalling of concrete and the corrosion of embedded reinforcement. The amount of deterioration can be minimized by removal of the accumulated debris by routine washing (followed by spot repainting in steel structures) but this has rarely been done in practice.

Sealed Type Joints

Sealed type of joints are required to seal the surface of the deck, including curbs, sidewalks, medians and, where necessary, even the barrier or parapet walls, against the seepage of moisture and other contaminants through the joint. It is also a requirement that the geometry of the joint be such as to prevent the accumulation of water and roadway debris in the joint. Metal to metal sliding surfaces have not performed satisfactorily, largely because of the effects of vibration, the accumulation of debris and the fact that they cannot be sealed effectively. The presence of debris and, in the winter months, possibly ice, has been found to restrict the movement capabilities of joints, displacing thin membrane seals. Where the movement of the joint is accommodated through the change in geometry of a thin 'gland' or 'membrane strip', these should not be in direct contact with vehicles in order to prevent their failures from wear and puncture by the trapped incompressibles.

Some of the proprietary joints, claimed to be 'self-cleansing', have been found to be no less vulnerable to damage by accumulated debris than other types of joints.

Deck joints having elastomeric components in direct contact with the deck surface, do not provide adequate support for the adjacent edges of the deck wearing surface. Nosings of epoxy mortar or similar material are sometimes used between the edges of the deck surface and the joint to try and prevent spalling of the concrete edge of the deck.

The chief factors in determining whether to use an open or a sealed joint are the cost and the location of the site. The total cost of each system must be compared and, for open joints, this includes the cost of disposing of surface run-off. The total costs of sealed joints may include the cost of providing for additional forces on the structure, and an increase in maintenance costs and frequency of inspection.

In general, open joints are well suited to secondary highways where little sand and salt is supplied during winter

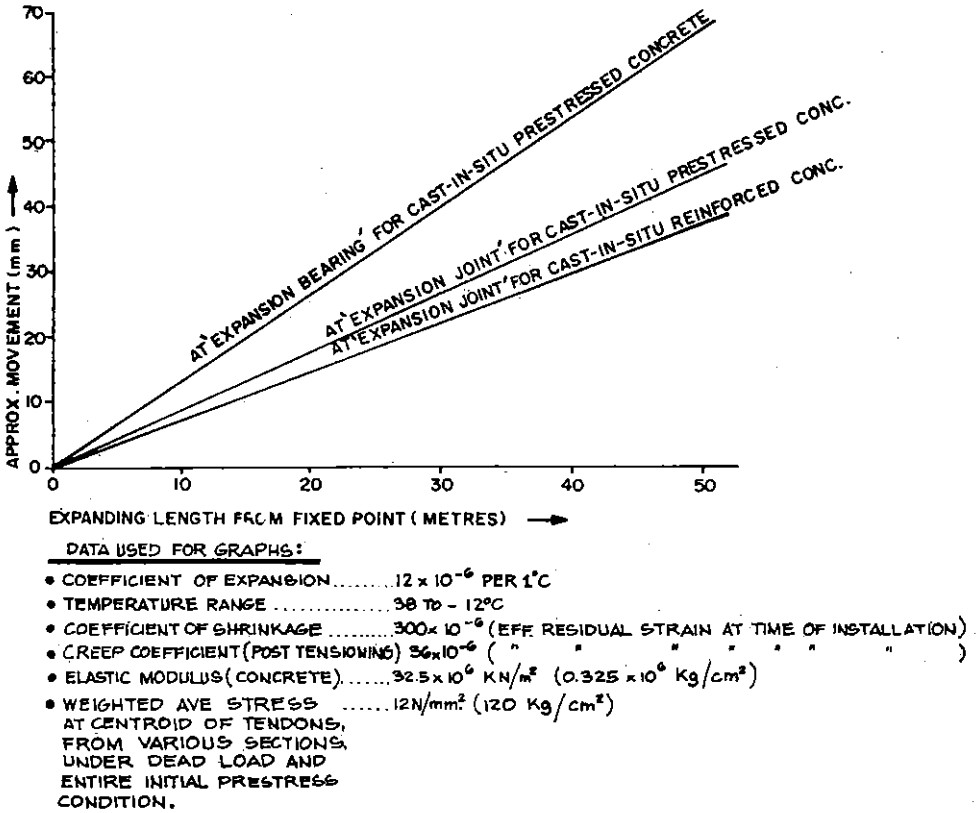


Fig. 32.5

maintenance operations. Sealed joints tend to be more suited to urban areas where the costs of providing for the surface run-off can be high.

Movements at Joints — A Relative Comparison

As explained earlier, the movements to be catered for at (bearings and) expansion joints arise from the following causes:

(i) Thermal expansion and contraction of the superstructure and in certain cases even of substructure
(ii) Shrinkage of concrete
(iii) Creep in the concrete
(iv) Elastic shortening under prestress
(v) Displacement of the structure under load or any other action.

Because expansion joints are installed at a late stage in construction, therefore some of these movements will already at least partly have taken place. Hence, less total movement has to be catered for at expansion joints than in bearings at the same location.

Elastic shortening under prestress is normally assessed on the mean stress induced in the deck section by the prestress and dead loads.

In most bridges, displacements of the structure under load, produce very minor movements.

Figure 32.5 shows an approximate comparison of the total movements at an expansion joint in cases of cast-in-situ reinforced concrete deck and prestressed concrete deck, for various expanding lengths. (Various assumptions made are given in the figure itself.) Also shown is the movement at the expansion bearing (for various expanding lengths) in case of a cast-in-situ prestressed concrete deck. (Note that the movement at the bearing is more than at the expansion joint for the same deck since the latter is installed much after the deck is cast on top of the bearing and, therefore, suffers movements only due to temperature and parts of concrete shrinkage and creep).

Early Types of Expansion Joints

The structural steel 'angles and plate' type, shown in Fig. 32.6, is one of the oldest types of expansion joints. It has never performed satisfactorily, primarily because of its inefficient (even though cheaper) detail that lends itself as an easy prey both to poor installation as well as to poor response to wheel-knocking and rotational deformation.

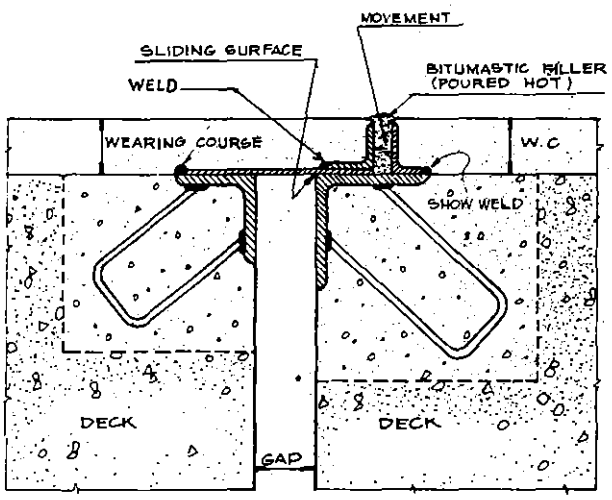


Fig. 32.6 *Angle and plate joint (Ancient)*

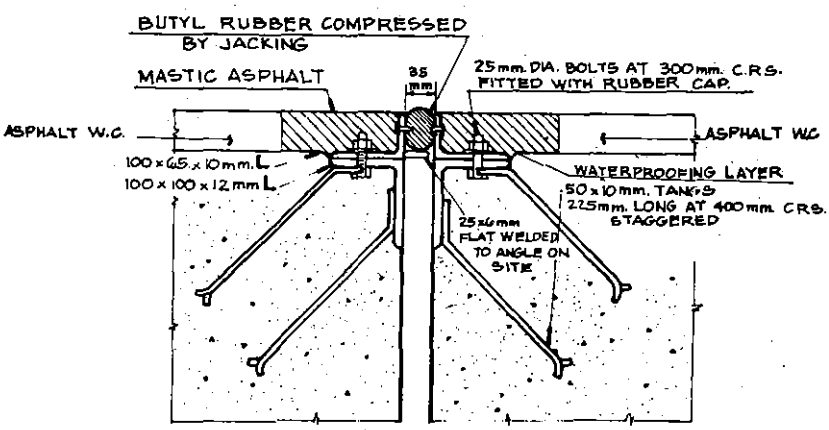


Fig. 32.7 *Angle joint sealed with butyl rubber strip (Ancient)*

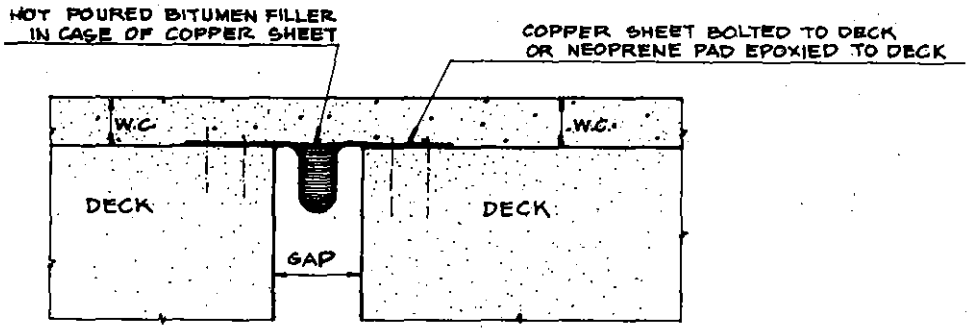


Fig. 32.8 *Bellow-type joint*

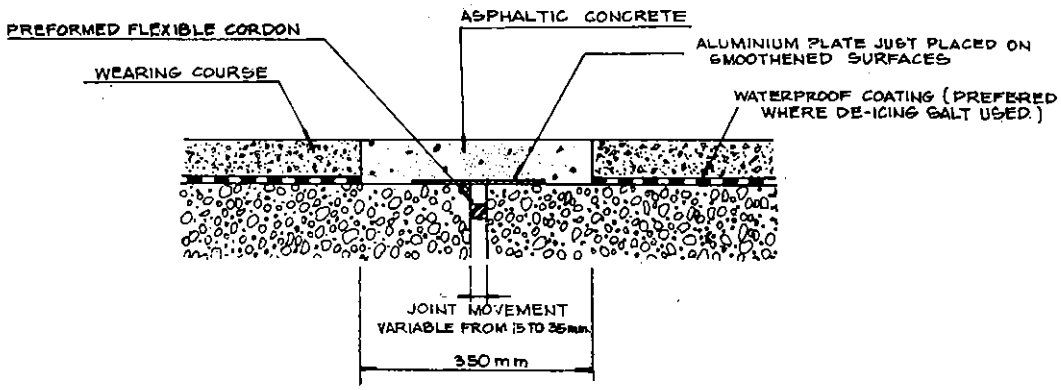


Fig. 32.9 *Buried plate type joint*

Figure 32.7 shows a slightly less inefficient cousin of the same type of joint. Figures 32.8 and 32.9 respectively show the Bellow-type and the Buried-plate-type joints, which permit a continuous wearing surface, thus permitting a better ride.

All these joints can basically cater only for small to medium amounts of movements (say 15-50 mm).

Elastomeric (Neoprene) Compressed Seals

There are two basic types of extruded elastomeric seals. One is the 'open-cell compression-seal' dependent upon its ability to maintain pressure on the joint side-walls, with varying degrees of stress. The other type consists of 'strip-seals' and 'box-configuration seals' using locking lugs or ears placed in compression within the cavity of a retainer (Figs. 32.10 and 32.11).

The former seals may be designated 'compression seals' and the latter addressed as either 'strip seals' or 'box seals' for whatever case applies, as will be seen in the figures ahead.

These types of seals are necessarily dependent upon the material they are made of. This material is what makes 'preformed' elastomeric seals superior to all other types of seals and sealants.

Generically, the basic material is polychloroprene, otherwise known as Neoprene. Polychloroprene was selected because of its agreeable physical attributes.

Since the first uses of these seals were oriented to the transportation field, they had to resist deterioration from exposure to weather, sunlight, oils, chemicals, heat, abrasion, and impact, while still remaining elastic during their expected life cycle.

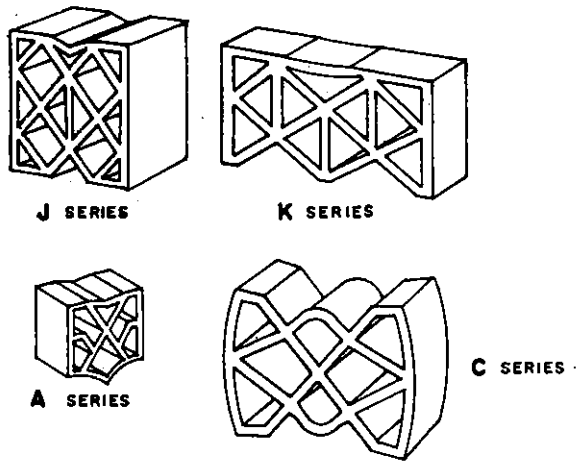


Fig. 32.10 *Standard compression seals (Glands)*

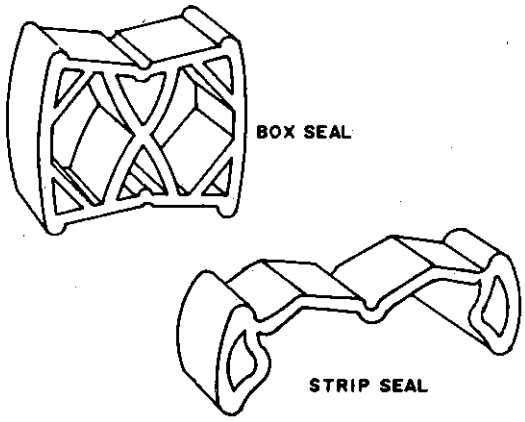


Fig. 32.11 *Lock in compression seals (Glands)*

32.3 COMPRESSION SEALS

Compression seals derive their name from the fact that they

Table 32.1 *Typical Compression Seal Data*

	Style No.	*W*	*H*	*W* min	*MIW*	*W* max	*M*	*CH*	*G* min
SERIES — J	J-175	1.3/4	2	0.76	1.02	1.48	0.72	2.3/16	0.56
		(44.45)	(50.80)	(19.30)	(25.91)	(37.59)	(18.27)	(55.56)	(14.22)
	J-200	2	2.1/16	0.92	1.19	1.70	0.78	2.15/32	0.52
		(50.80)	(52.39)	(23.37)	(30.27)	(43.18)	(19.81)	(62.71)	(13.21)
	J-225	2.1/4	2.5/8	1.06	1.36	1.91	0.85	2.7/8	0.56
		(57.15)	(66.68)	(26.92)	(34.54)	(48.51)	(21.59)	(73.03)	(14.72)
	J-250	2.1/2	2.3/4	1.15	1.49	2.12	0.97	3.1/8	0.65
		(63.50)	(69.85)	(29.21)	(37.85)	(53.85)	(24.64)	(79.38)	(16.51)
	J-300	3	3	1.30	1.74	2.55	1.25	3.11/16	0.80
		(76.20)	(76.20)	(33.02)	(44.20)	(64.77)	(31.75)	(93.66)	(20.32)
	J-325	3.1/4	3.1/8	1.43	1.90	2.76	1.33	3.13/16	0.81
		(82.55)	(79.38)	(36.32)	(48.26)	(70.10)	(33.78)	(96.84)	(20.57)
	J-350	3.1/2	3.1/2	1.47	2.01	2.97	1.50	4.1/8	0.85
		(88.90)	(88.90)	(37.34)	(51.05)	(75.44)	(38.10)	(104.78)	(21.59)
	J-400	4	4.23/32	1.77	2.35	3.40	1.63	5.1/4	1.02
		(101.60)	(119.86)	(44.96)	(59.69)	(86.36)	(41.40)	(133.35)	(25.91)
	J-450	4.1/2	4.1/2	1.82	2.52	3.82	2.00	5.11/32	1.07
		(114.30)	(114.30)	(46.23)	(64.01)	(97.03)	(50.80)	(135.73)	(27.18)
	J-500	5	5.5/16	1.75	2.64	4.25	2.50	5.7/8	1.00
		(127.00)	(134.94)	(44.45)	(67.06)	(107.95)	(63.50)	(149.22)	(25.4)
	J-600	6	5.3/4	2.03	3.12	5.10	3.07	6.5/8	1.28
		(152.40)	(146.05)	(51.56)	(79.25)	(129.54)	(77.98)	(168.27)	(32.51)
SERIES — K	K-300	3	2.19/32	1.25	1.71	2.55	1.30	3.02	0.62
		(76.20)	(65.88)	(31.75)	(43.43)	(64.77)	(33.02)	(76.71)	(15.75)
	K-350	3.1/2	2.1/2	1.25	1.86	2.97	1.72	3.0	0.82
		(88.90)	(63.50)	(31.75)	(47.24)	(75.44)	(43.69)	(76.20)	(15.75)
	K-400	4	2.1/2	1.63	2.26	3.40	1.77	3.10	0.88
		(101.60)	(63.50)	(41.40)	(57.40)	(86.36)	(44.96)	(78.74)	(22.35)
	K-500	5	3.1/4	2.00	2.80	4.25	2.25	3.62	1.25
		(127.00)	(82.55)	(50.80)	(71.12)	(107.95)	(57.15)	(91.95)	(31.75)
	K-600	6	4	2.25	3.27	5.10	2.85	4.37	1.50
		(152.40)	(101.60)	(57.15)	(83.06)	(129.54)	(72.39)	(111.0)	(38.10)

NOTE JW–The recommended 'joint width at the time of installation' is considered the same as MIW or larger. When ambient temperatures are over 85° F the compression seal becomes difficult to install.

KEY: W — Seal Width—Nominal Uncompressed
H — Seal Height—Nominal Uncompressed
W min — Seal Width—Compressed at the Highest Anticipated Temp.
MIW — Minimum Installation Width
W max — Seal Width—at Lowest Anticipated Temp.
M — Total Seal Movement Capability (W max minus W min)
JD — Minimum Recommended Joint Depth for Seal
CH — Compressed Height of Seal
G min — Clear gap between 'nitches' or 'support ears' supporting the Compn. Seal.

Dimensions in inches (mm)

are compressed when inserted into the joint-opening, and remain in the state of compression during all movement phases of the joint.

They are generally designated by size according to their width. A movement rating is usually established, for each width seal, along with other pertinent data, by the manufacturer. These critical dimensions and data are available in table form published by the various manufacturers (Table 32.1. typical).

Naturally, the seal to be used, in any particular case, must have a nominal width which is greater than the largest expected gap opening. Therefore, the 'working dimensions' of compression seals vary generally from 85% of nominal width—when the joint is fully open, to approximately 40-50% of nominal width—when the joint-gap is at its minimum dimension, (Fig. 32.12). Figure 32.13 shows a compression seal with armoured structural steel edges.

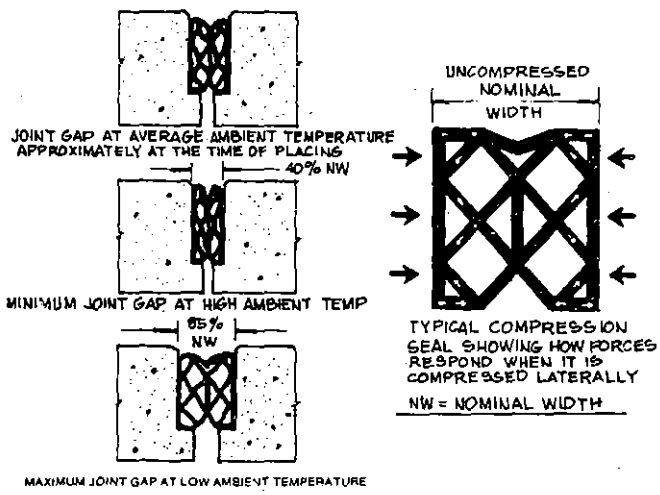


Fig. 32.12 *Compression seals are always in stressed state. They maintain watertightness and keep out incompressibles by maintaining sidewall pressure on the joint interfaces*

In order to design an expansion joint, the engineer would first determine the joint movements that will take place due to temperature, creep, shrinkage, elastic shortening, etc. After this anticipated movement is ascertained, the joint geometry can easily be determined. Simply stated, the joint geometry/configuration for the entire movement cycle should accommodate the physical characteristics of the compression seal selected (i.e., physical properties of a seal exhibiting a movement rating equal to or greater than the expected joint movement).

After the designer has obtained the joint movement, he can utilize the manufacturers charts for getting the maximum and minimum gap openings as well as other data for the seal selected. Knowing the ambient temperature range, he can then proceed with calculations for the joint geometry.

Satisfactory performance of compression seals is judged upon by their ability to maintain sidewall-contact with the joint-gap interfaces at all times, thereby creating a watertight seal and preventing incompressibles from accumulating in the joint or penetrating downward along the sidewalls.

Therefore, several physical properties of a compression seal have been identified which serve to provide data to ascertain the expected performance capability of the elastomer and, hence, of the seal itself. These various parameters have been incorporated into specifications, and certain test procedures have been developed for quality assurance.

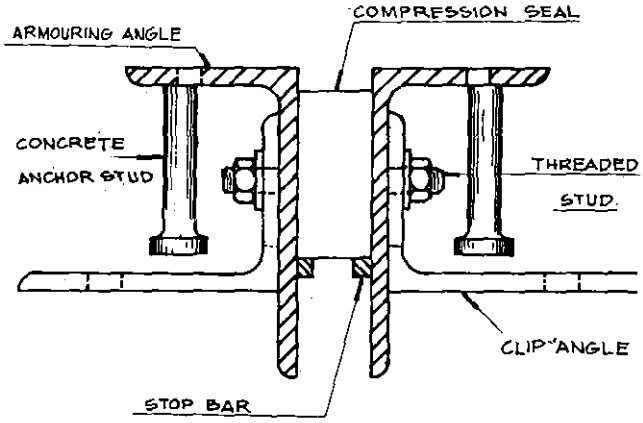


Fig. 32.13 *Cross-section of armoured joint with compression seal*

Physical Properties of Compression Seal (see Table 32.2)

- **Tensile Strength and Elongation**

After it has been established that a rubber composition has desirable characteristics and the product has been placed in production, tensile strength and elongation measurements are excellent control tools. They are sensitive to manufacturing mishaps and tell the rubber technologists whether or not the product has been properly vulcanized and the ingredients thoroughly blended. However, it is good to keep in mind that tensile strength and elongation cannot be used in design calculations, and they bear little relation to a rubber product's ability to perform its function.

It is often argued that these factors, while not in themselves significant, are nonetheless an index to general all round quality. This simply is not so, with exceptions at the extremes of the tensile strength range. For example, compounds below 1,000 psi

Table 32.2 *Physical Properties of Preformed Elastomeric Joint Seals*

Property		Requirements	ASTM Method
• Tensile Strength, min. psi		2000	D 412
• Elongation at break, min. per cent		250	D 412
• Hardness, Type A Durometer:			D 2240
—Lock in Seals		60 ± 7	(modified)
—Stdd. Compression Seals		55 ± 5	
• Compression set, 70 hr. @ 212° F, max.		40%	D 395 Method B (modified)
• Oven aging, 70 hr. @ 212° F:			
—Tensile strength loss max		20%	
—Elongation loss max.		20%	D 573
—Harness, Type A Durometer (count 'points' can easily vary, see explanation in text ahead)		0 to + 10	
• Ozone resistance: 20% strain, 300 pphm, in air 70 hr at 104° F (wiped with toluence to remove surface contamination)		No cracks	D 1149
• Low temperature recovery, 72 hr at 14° F, 50% deflection, min.	Used for Standard Compression Seals Only	88%	D-2628-69 SECTION 7
• Low temperature recovery, 22 hr at —20° F, min. 50% deflection		83%	D-2628-69 SECTION 7
• High temperature recovery, 70 hr. at 212° F, min. 50% deflection		85%	D-2628-69 SECTION 7

(70 kg/cm^2) in tensile strength are usually rather poor in most mechanical properties while those with tensile strength over 3,000 psi (210 kg/cm^2) usually have good properties. On the other hand, for those in the middle range say from 1,500 to 3,000 psi (105 to 210 kg/cm^2), where the great majority of rubber products fall, a correlation is at best haphazard between tensile strength and such properties as resilience, abrasion resistance, compression set, and flexlife.

- **Hardness**

Hardness, as applied to rubber products, is defined as the relative resistance of the surface to indentation usually taken by a device called a durometer. There are two problems encountered with the validity of this test:

(i) The measured values are unreliable because of mechanical considerations. It is very common to obtain a five-point variation in readings made by people using different durometer gauges on the same place of rubber; in some cases, these differences may be as high as 10 points.

(ii) The characteristic that is measured rarely bears any relation to the ability of a rubber product to function properly.

The point of this discussion is to alert those who are responsible for quality assurance that minor deviations of this specification are not criteria enough for rejection.

- **Compression Set**

The compression set values are helpful in that a rubber composition can be eliminated from consideration for sealing applications if it falls in the distinctly high range of set. This is one more test to examine quality control of the rubber matrix.

- **Oven Aging and Ozone Resistance**

The oven aging test was designed for predicting natural aging in an accelerated time frame. Ozone attack is evident by the formation of a few deep cracks or myriad small cracks, usually occurring at right angles to the direction of applied stress. It is a common phenomenon since most compression seals are under stress and ozone is universally present in the atmosphere. Ozone is generated from oxygen in the air and by Ultra-violet light from the sun. (Higher altitudes have higher ozone content.)

However, ozone attack has nearly ceased to be a problem since all neoprene compounds, used for premolded elastomeric sealers, have additives to arrest this problem. The performance test for ozone resistance is normally not required although the specification must state this requirement to ensure its continued observance.

- **Recovery Test**

The recovery test is a limiting measure of 'performance predictability'.

The test at 14°F will determine crystallization tendencies of an elastomer, and the high and low temperature recovery values will provide data in predicting field performance under similar

environmental conditions. The high temperature test also suffices for an accelerated aging test in determining long range performance.

The seal's ability to 'recover', after compression, to its nominal width, under all ambient temperature conditions in the field, is probably the single most important factor affecting its performance.

Web-Configuration of Compression Seals

One of the factors in the seal design is the internal web configuration of the seal which is often ignored by users and always neglected in specifications.

Certain geometric web configurations can produce excellent laboratory results, but are often misleading. For example, a configuration utilizing webs only in the horizontal and vertical planes (series of squares) will create a seal with high initial pressure and lower ultimate pressures, a trait most desirable in a compression seal. But, in the field, where the seal must be installed by forcing it into the joint-crevice (not by merely placing it between a set of plattens), this box-like configuration becomes 'unstable'. This phenomenon parallels that of a truss designed similarly with squares only, which would more easily permit lateral forces to cause a failure in the direction of force (Fig. 32.14).

In the case of a compression seal similarly designed, where a force is applied to the top surface in order to squeeze the seal in the joint, lateral collapse takes place causing the seal to roll or twist in the joint, resulting in poor sidewall contact, thereby allowing the incompressibles and moisture to penetrate and eventually migrate downward, ultimately leading to failure (Fig. 32.14).

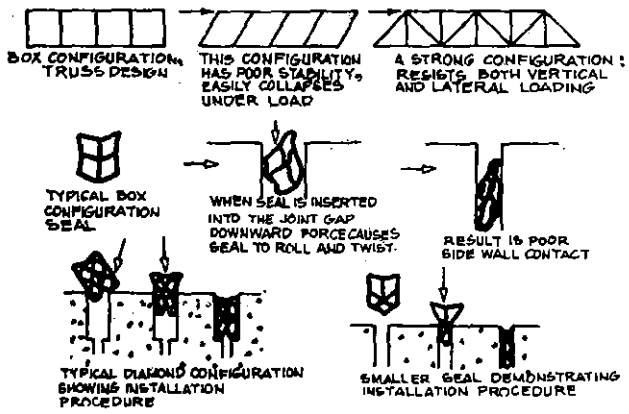


Fig. 32.14

Compression Seal Installation

It is normal practice to include, in tabular form (Table 32.2 and Fig. 32.12), the temperature range on the contract plans. Factors, such as creep, shrinkage, elastic-shortening and the sequence of construction, need to be taken into account in calculating the installation settings. The effective or installation temperature may be taken as the mean shade air temperature for the 48 hours prior to joint installation for concrete structures, and for the previous 24 hours for composite structures with a concrete deck slab and steel beams. The difference reflects the fact that concrete is a poor thermal conductor and changes in the effective temperature occur more slowly in concrete structures than in steel structures. More sophisticated methods are rarely necessary since the temperature will change even during the time taken to install the joint. There is also a danger that more complex procedures will not be understood by the contractor's personnel and the inspectors. Where the joints are to be installed in concrete, temporary connections (if any) must be released as soon as the concrete is placed to prevent distress in the concrete from the movement of the structure.

Field observations have shown a lower incidence of damage in joints where concrete end-dams are at least 400 mm wide since a wider concrete surface appears to reduce the dynamic load on the joint. A detail which has been found satisfactory is illustrated in Fig. 32.15 for the case where a compression seal is used. A similar end-dam detail can be used with other types of joint. The steel angles between the end-dam and the bituminous concrete are necessary to prevent spalling of the concrete. Vent-holes, approximately 5 mm diameter at 250 mm centres, are required in the top of all angles to prevent air entrapment when placing the concrete. A minimum angle thickness of 12 mm is desirable. The (minimum) 200 mm gap beneath the joint facilitates inspection and improves ventilation so that the ballast wall and the ends of the beams are less susceptible to damage if leakage should occur.

Installation of compression seal is normally a relatively easy task if the correct seal for given joint opening is specified and other simple procedures are adhered to (Figs. 32.14, 32.16 and 32.17).

The proper joint gap-opening is, without question, important to the ease for installing compression seal. Therefore, manufacturers normally indicate the minimum gap-openings permissible for field installation, in their product tables. It is preferred to design the joint geometry so that at the time of installation the joint interstice is greater than the recommended minimum.

Even so, it would be difficult to insert a typical compression seal into a joint-gap without the aid of a lubricant. Elastomeric material has a high coefficient of

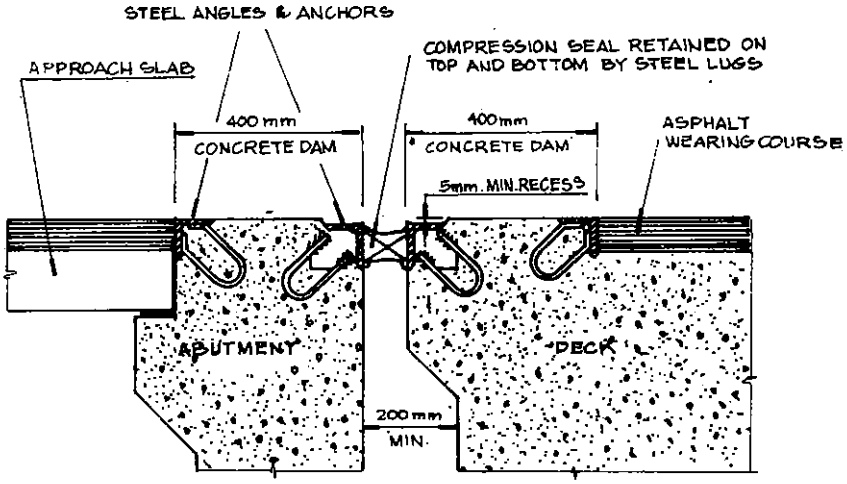


Fig. 32.15 *Typical detail for installation of a deck joint assembly*

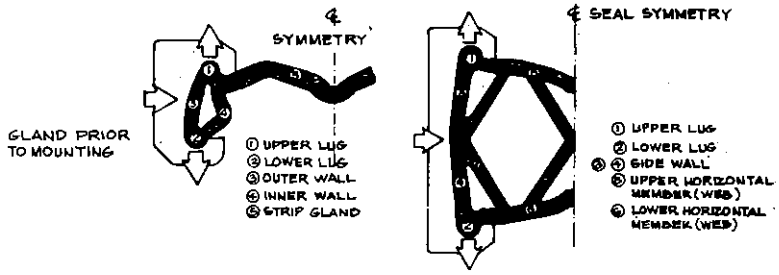


Fig. 32.16

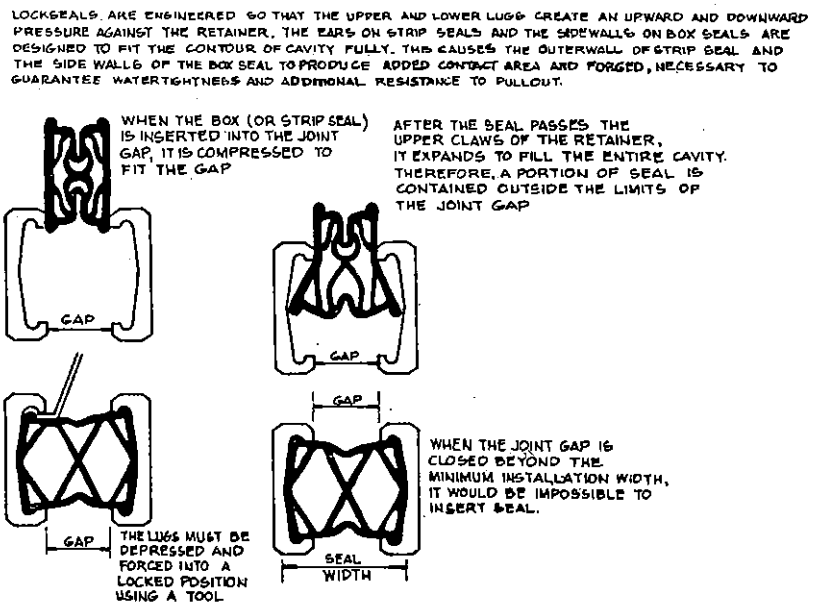


Fig. 32.17

friction, and under field conditions, where the joint-gap is predetermined by design, it would sometimes be impossible to insert the seal into it unless a lubricant is utilized.

Oil soap was the first lubricant to be used on the earliest installations. Since that time, it has been determined that an adhesive, in a single component, system, containing a high degree of lubricity when applied to the joint interfaces would give better performance.

There are, basically, two types of lubricant-adhesives currently in use today. One is used where compression seals are inserted directly into joints with concrete faces and the other is used for installing seals in structures having armoured (metal faced) joints.

The first type is used primarily for contraction joints and small expansion joints in pavements and slabs. It consists of a one component polychloroprene compound having a solids content of 24% ± 3% by weight. The low solids content permits the use of a spray applicator which is a necessity in cases where miles of seal are placed. This adhesive primes the joint faces and seals small holes and imperfections in concrete surfaces.

Another type, used for larger expansion joints and in conjunction with steel and other metal faced joints, consists of a one part moisture curing polyurethane, an aromatic hydrocarbon solvent mixture, with a solids content of approximately 72% ± 3% by weight.

The larger the seal the larger the cell openings or cavities. In order for the rubber to be able to span these greater

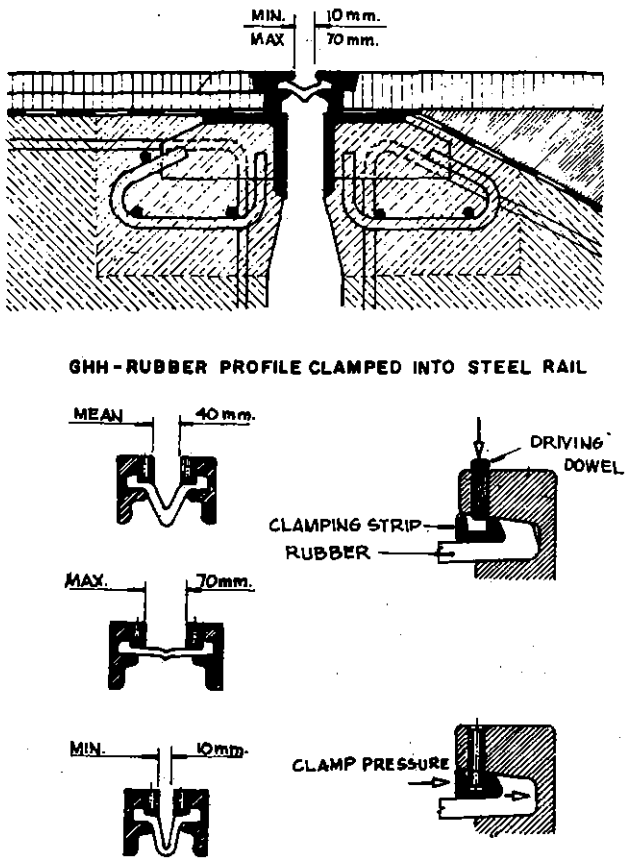


Fig. 32.18

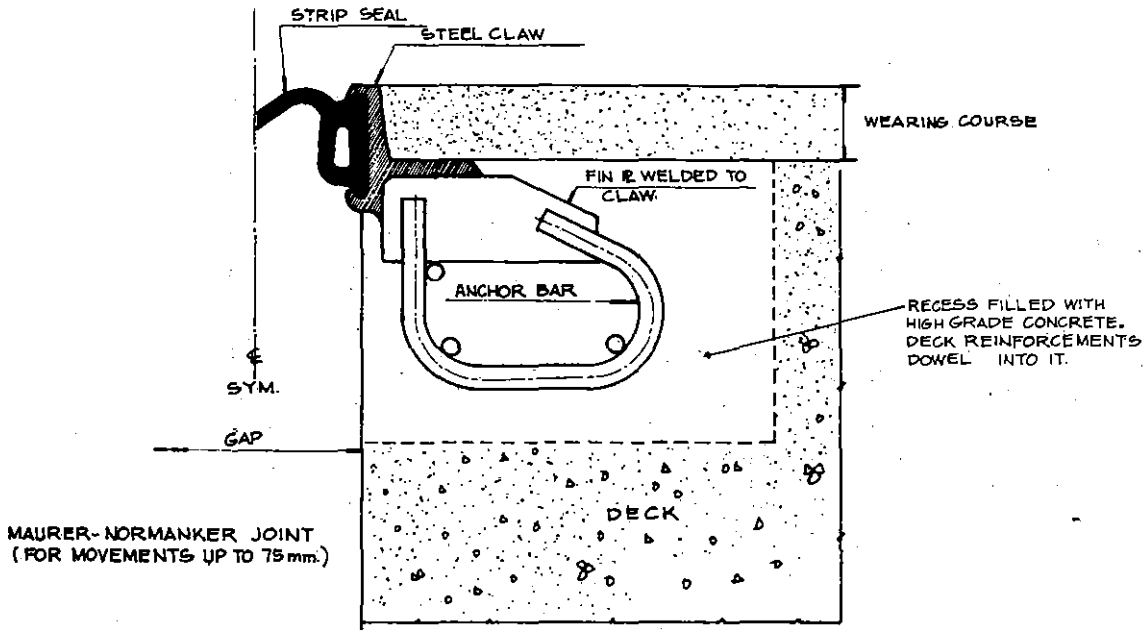


Fig. 32.19

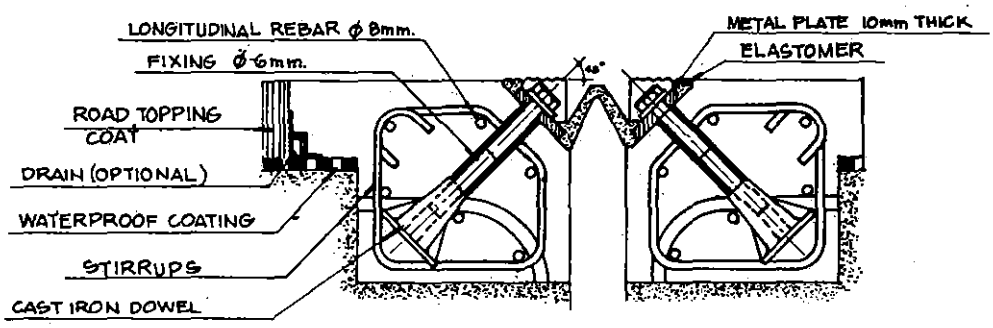


Fig. 32.20

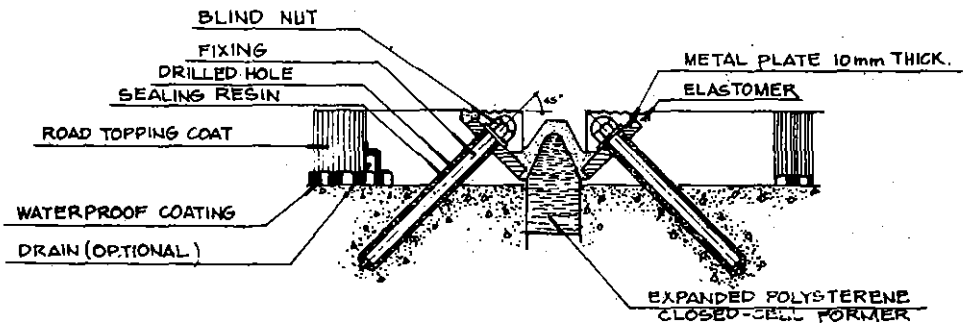


Fig. 32.21

open dimensions, the webs must be constructed thicker and heavier for increased strength. Due to the physical characteristics of rubber, this increase in mass is not strictly an arithmetical ratio to size of seal.

Therefore, because of these factors, and the maximum allowable joint-gap openings, other alternatives were needed for movements larger than about 50 mm and for joints on skews which tended to magnify these limiting features.

Among these alternatives are the 'mechanically secured' elastomeric seals which are a form of compression seal. (Figs. 32.18 to 32.38). These seals derive their waterproofing characteristics from the fact they are placed in a state of compression at the point where moisture or the incompressibles can enter. This stress is contained within the locking device in the case of 'strip-seals'.

The uniqueness of compression type strip-seals is the complete elimination of ancillary component parts such as plates or clamps. They are self-contained with no components to rust or wear, that could cause failure.

A derivation of the strip-seal is the box-type 'lock-seal' which works on the same principles as the former. Their main advantage is that they have secondary webs providing additional strength, and their ability for self-cleansing of the incompressibles is superior to that of strip-seals. (This claim is not entirely true, though.)

Another desirable aspect of these types of compression seals is that they afford resistance to tension forces. Strip seals can be stretched up to 100% of their width before pulling out and box seals stretch approximately 25% beyond their normal movement range. While the stretch force will permit the expansion joint to overtravel in the open direction, it should never be used in the operating range.

The Freyssinet joints shown in Figs. 32.20 and 32.21, composed of an elastomeric membrane stiffened by two lateral metallic plates, can take movements up to 65 mm.

The joints may be installed in the traditional manner, in rebates (recesses) in a rapid economical manner, within the thickness of the road wearing course.

Wabo Strip Seal (Figs 32.22 and 32.23)

This joint system consists of a clawlocked neoprene strip between two steel shapes forming a sealed joint. A preformed 'hinge' controls folding movements of the seal element.

Versatile in design, this system can be prefabricated ready for placement, or field assembled. There are several series of seal elements to offer solutions to a wide range of field applications.

The S series seals (Fig. 32.23), which mate with the Type A steel extrusion (Fig. 32.22), meet typical job requirements with movement ratings up to 5" (125 mm).

The SE-series, also capable to movements up to 5" (125 mm) has a typical usage in rehab (rehabilitative) work where a low height replacement system is required. The type

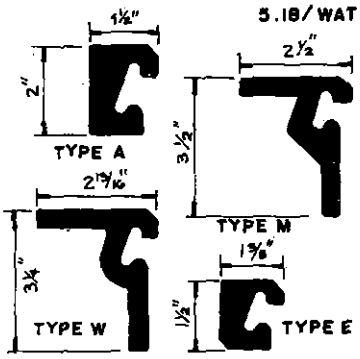


Fig. 32.22

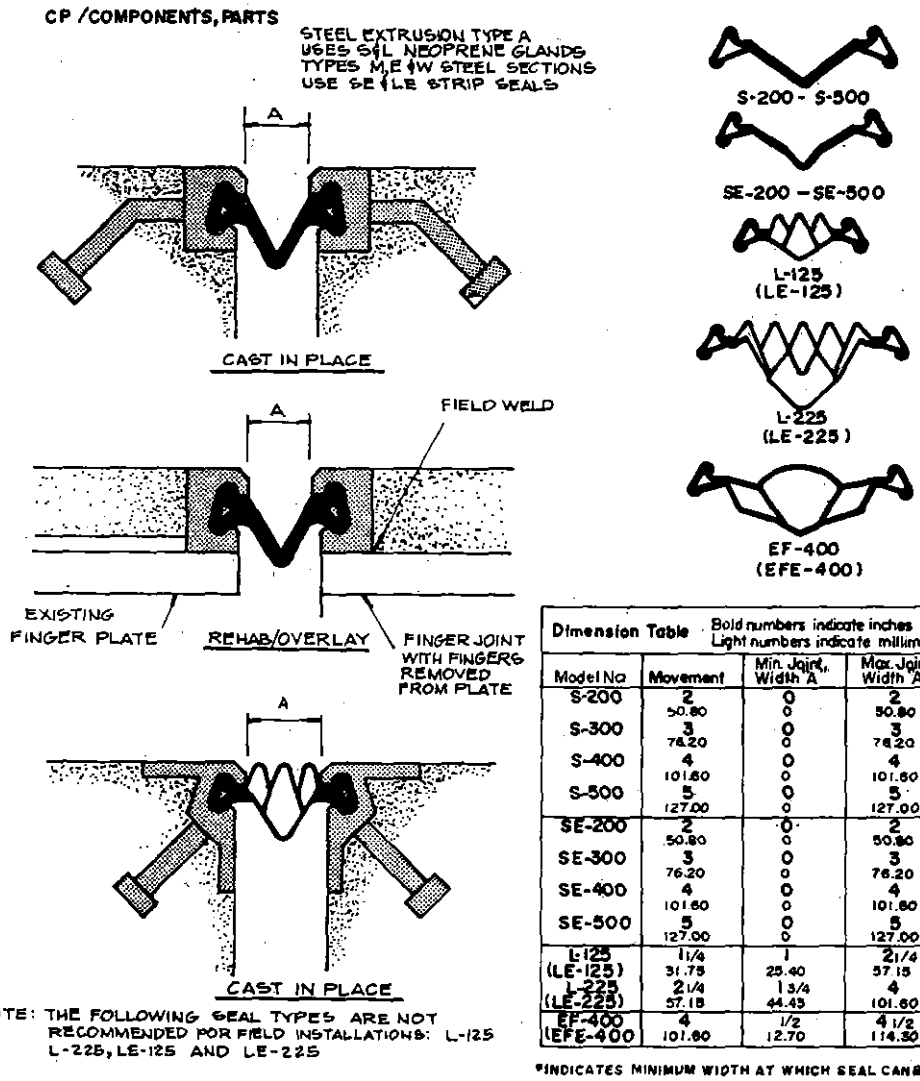
M, E and W steel extrusions (Fig. 32.22) are used with this series of seals.

For applications where an even traffic surface must be maintained, the L (LE) and EF (EFE) series seals (Fig. 32.23) provide the answer for movements up to 4" (100 mm).

Due to the mechanically-locked feature of the Strip Seal products, this system is excellent in applications where differential slab movements are anticipated.

32.4 LARGE MOVEMENT EXPANSION JOINTS

Basically the "ELASTOMER-BASED" joints exploit the



Dimension Table	Bold numbers indicate inches Light numbers indicate millimeters			
Model No	Movement	Min. Joint Width A	Max. Joint Width A	Installation Width A*
S-200	2 50.80	0 0	2 50.80	1 1/2 38.10
S-300	3 76.20	0 0	3 76.20	1 1/2 38.10
S-400	4 101.60	0 0	4 101.60	1 1/2 38.10
S-500	5 127.00	0 0	5 127.00	2 50.80
SE-200	2 50.80	0 0	2 50.80	1 1/2 38.10
SE-300	3 76.20	0 0	3 76.20	1 1/2 38.10
SE-400	4 101.60	0 0	4 101.60	1 1/2 38.10
SE-500	5 127.00	0 0	5 127.00	1 1/2 38.10
L-125 (LE-125)	1 1/4 31.75	1 25.40	2 1/4 57.15	1 3/4 44.45
L-225 (LE-225)	2 1/4 57.15	1 3/4 44.45	4 101.60	2 3/4 69.85
EF-400 (EFE-400)	4 101.60	1/2 12.70	4 1/2 114.30	2 1/4 57.15

*INDICATES MINIMUM WIDTH AT WHICH SEAL CAN BE INSTALLED

Fig. 32.23

elastic deformation of an elastomeric 'gland' in order to allow relative movements at the joint gap.

They can cover a wide range of movements by 'parallel coupling' of several basic units (distribution of the movement) (Figs. 32.24 to 32.38)

By the late 1960s several devices were introduced on the market which utilized compression seals-'in parallel', enabling engineers to use two or more seals, simultaneously, to obtain the desired larger movement rating. This system is referred to as a 'Modular Joint'

The modular expansion joint was a breakthrough in engineering design of movement joints, providing an alternative to the articulated joints, finger joints, and slider-plate assemblies.

Fassbinder proposed a watertight modular joint consisting of a deformable closing element made up of flat steel plates and foamed plastic intermediate layers forming a composite unit. Later, a tubular preformed elastomeric seal was utilized in place of the foamed plastic (Fig. 32.24).

Subsequent to this modular expansion joint, 'heavy duty' compression seals were placed in parallel between structural shapes resting on support bars. This particular concept, with modifications, served well for almost a decade (Fig. 32.25).

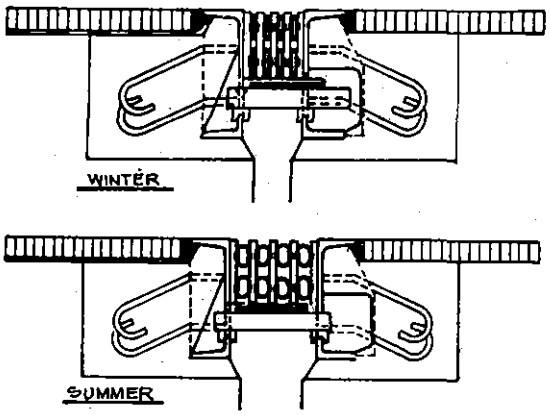


Fig. 32.24 *The Fassbinder modular expansion joint*

The latest models of modular expansion joint systems are using the 'locked in seal' principles simply because of their advantages, including their greater operating range. These newer models have been made heavier, stronger, more durable, and are practically maintenance free.

The modular expansion joints have gone through a period of growth much the same as any new product might. There were problems that had to be overcome (seal's performance and the mechanical aspects).

The early joints were noisy due to flexural rebound, they leaked because of pressure decay, some seals opened more

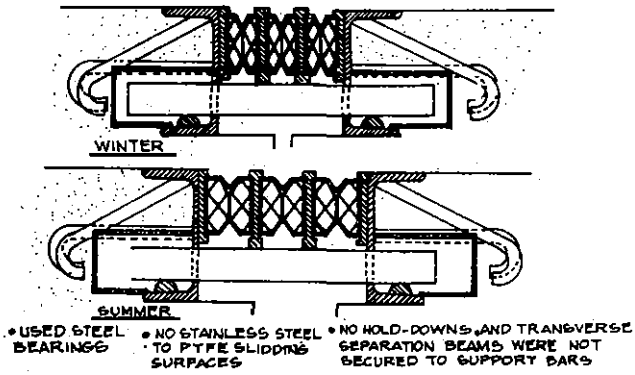


Fig. 32.25 *Early American modular expansion joints*

than others causing seal failures, and surfaces in contact with one another induced unwanted forces. Happily the present state of art has overcome many of these problems.

In case of very large movements the modular joints require supporting by a rather complex mechanical system (Figs. 32.25 to 32.34).

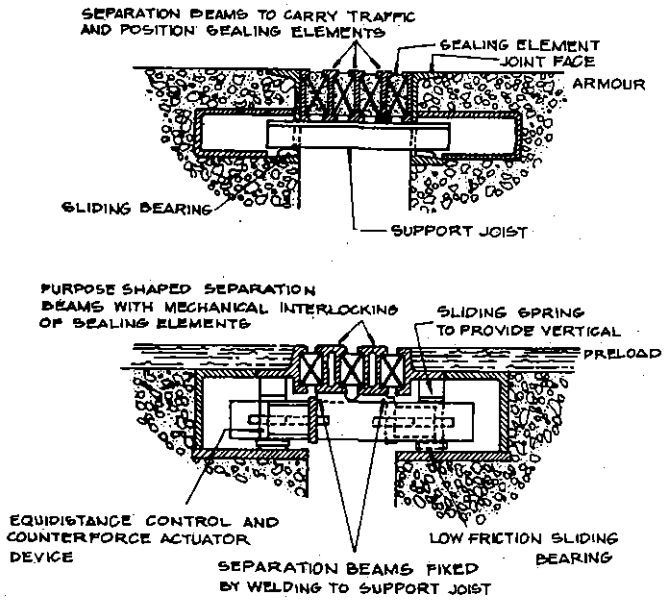


Fig. 32.26

All newer types of modular joints, with good performance records, have transverse components attached to the support bars, to prevent flexural rebound; the support bars are held down with some form of pressure device; contact surfaces are usually stainless steel to PTFE (Teflon); bearings for support bars are polyurethane; and some form

of equalization device or even a spring is utilized for maintaining equidistant spacing of the elastomeric elements.

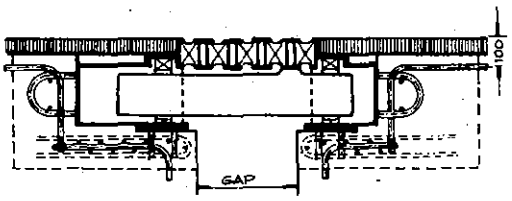


Fig. 32.27

The basic principles of modular joints have not changed radically over the years. They achieve a sealed waterproofed expansion joint, with large movement capability and low maintenance characteristics.

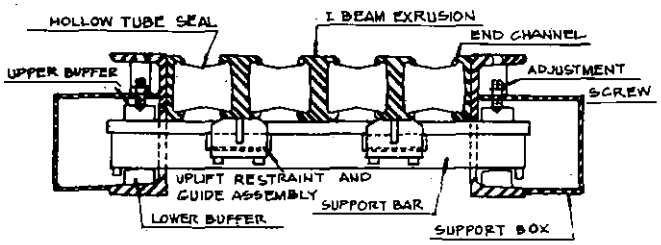


Fig. 32.28 *Cross-section of ACME-BETA joint*

The new 'locked in compression seals' have proved extremely reliable with respect to performance in both waterproofing and material endurance over the years.

The technologically improved materials and designs of appurtenant components have increased their maintenance-free service-life, providing quiet, functional and durable

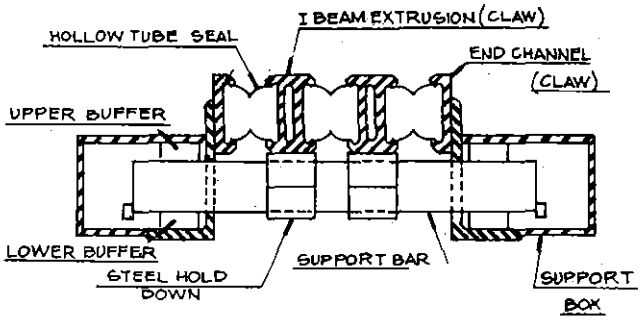


Fig. 32.30 *Cross-section of WABO-BETA joint*

(modular) expansion joints.

Wabolum Joint (Fig. 32.32) (by Watson-Bowman Group)

This is a modern aluminium modular joint system, providing a watertight seal through use of continuous neoprene sealing strips. Joint movements to 32" can be accommodated—through the addition of aluminium and neoprene standard components (Fig. 32.32).

Aluminium grade 6061-T6(ASTM) is employed to meet structural requirements necessary for heavy traffic loadings and to facilitate ease in field assembly due to the metal's lightweight characteristics.

Standard sections can be made and welded to handle a variety of skew, parapet and curb conditions.

This is manufactured by the Watson-Bowman-Acme in the US.

'Wabo-Maurer' Systems (by Watson-Bowman-Maurer Combine)

See Fig. 32.34 and its table.

For heavily-trafficked bridge structures requiring large

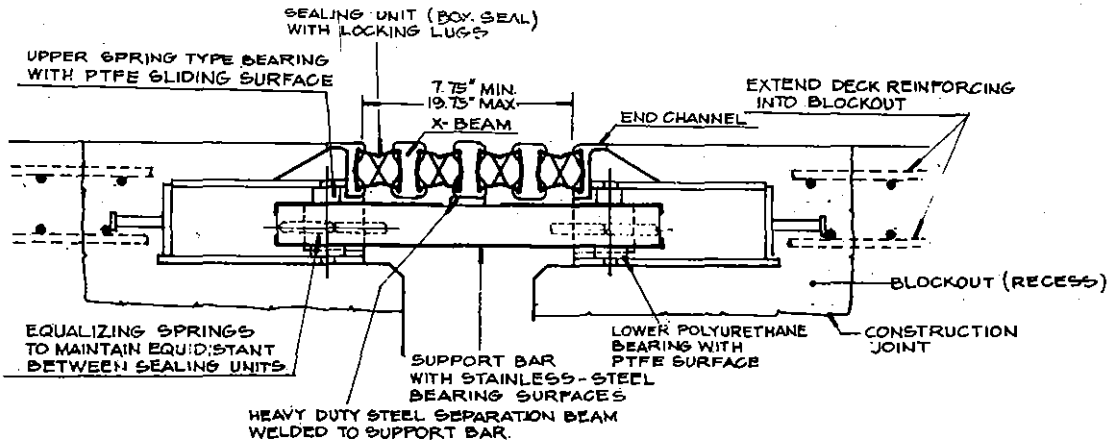


Fig. 32.29 *Newer type modular joint system with many improvements*

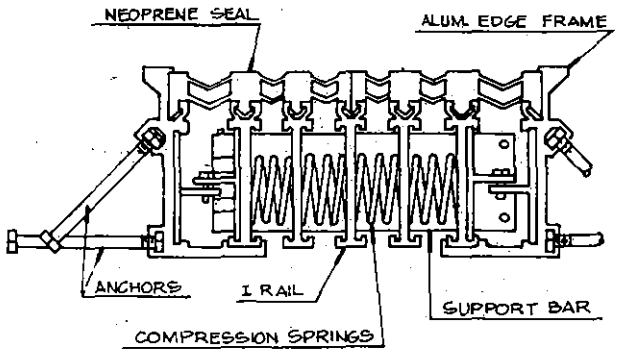


Fig. 32.31 *Cross-section of delastiflex D.L. joint*

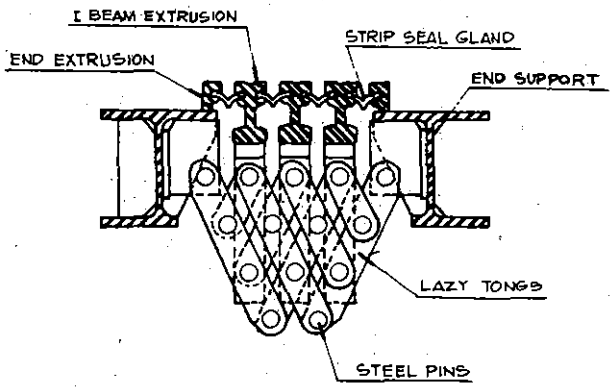


Fig. 32.33 *Cross-section of rheinstal "Lazy-tongs" type joint*

movement joints, this system provides a sealed joint through use of low-stress neoprene tubes mechanically locked between rigid extruded steel shapes. With movement capabilities to 26", this field proven system guarantees a long and maintenance-free expansion-joint life.

Other features of this system include:

- A 'unitized' construction as each extruded steel Support Beam is welded to an individual Support Bar, ensuring rigidity in resisting horizontal forces imparted from traffic.
- Urethane Bearings at the top and at the bottom of the Support Bar absorb impact loading with noise free action while securing against lifting forces.
- Teflon PTFE to Stainless Steel sliding surfaces for nearly friction-free support bar sliding action.
- Counterforce Mechanisms which neutralize internal forces, thus minimizing undesirable compressive forces exerted on deck edges.
- Heavy-duty Edge Member and Anchor Unit, which minimize edge member deflection while transmitting the forces into structural concrete.
- Wabo-Maurer Systems described here are manufactured according to US Patents 3604322 and 3626822 and Canadian Patents 852961 and 882450.

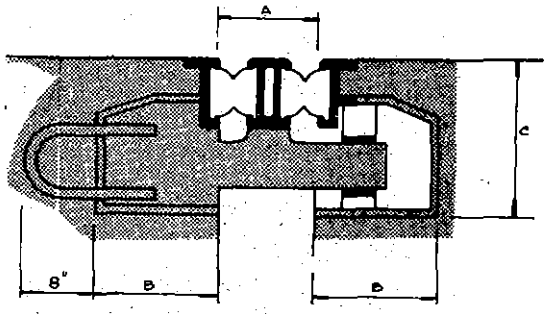


Fig. 32.34

NOTE: See Dimension Table on next page.

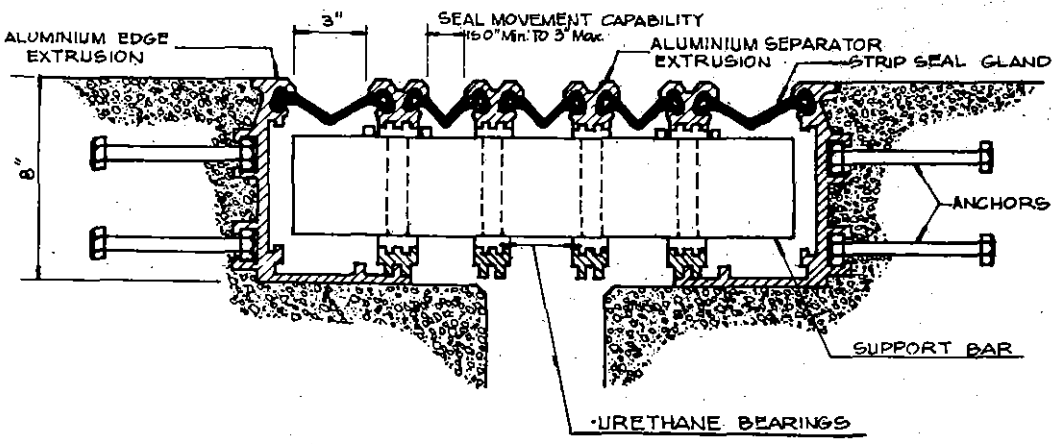


Fig. 32.32

Dimension Table (Fig. 32.34) Unbracketed numbers indicate inches, bracketed numbers indicate millimetres

Model No.	Movement	Weight	Dimensions A Min.	Dimensions A Max.	Dimensions B	Dimensions C
D-300	3.0	30 plf	0	3.0		
(One Seal)	(76.20)		(0)	(76.20)	—	—
D-600	6.0	75 plf	(3.0)	9.0	8.5	10.5
(Two Seals)	(152.40)		(76.20)	(228.60)	(215.90)	(266.75)
D-900	9.0	115 plf	6.0	15.0	11.5	10.5
(Three Seals)	(228.60)		(152.40)	(381.0)	(292.10)	(266.70)
D-1200	12.0	175 plf	9.0	21.0	14.5	10.5
(Four Seals)	(304.80)		(228.60)	(533.40)	(368.30)	(266.70)
D-1500	15.0	240 plf	12.0	27.0	17.5	10.5
(Five Seals)	(381.00)		(304.80)	(685.80)	(444.50)	(266.70)
D-1800	18.0	335 plf	15.0	33.0	20.5	10.5
(Six Seals)	(457.20)		(381.00)	(838.20)	(520.70)	(266.70)
D-2100	21.0	445 plf	18.0	39.0	23.5	10.5
(Seven Seals)	(533.40)		(457.20)	(990.60)	(596.90)	(266.70)
D-2400	24.0	590 plf	21.0	45.0	26.5	10.5
(Eight Seals)	(609.60)		(533.40)	(1143.00)	(673.10)	(266.70)
D-2700	27.0	765 plf	24.0	51.0	29.5	10.5
(Nine Seals)	(685.80)		(609.60)	(1295.40)	(749.30)	(266.70)
D-3000	30.0	880 plf	27.0	57.0	32.5	10.5
(Ten Seals)	(762.00)		(685.80)	(1447.80)	(825.50)	(266.70)

NOTE The above dimensions refer to systems using foreign-made steel extrusions. The dimensions vary slightly for those using American-made extrusions.

Waboflex-SR Joint (by Watson-Bowman Group)

See Figs. 32.35 to 32.38.

A moulded neoprene monolithic unit, Waboflex-SR is a rubber cushion expansion joint. Eight sizes enables the SR series to handle movements to 13". Moulded in standard 6 ft. lengths (except SR 13 which is 4 ft. long), this system requires a minimum of field labour to install in bridge decks, ramps, and roadways.

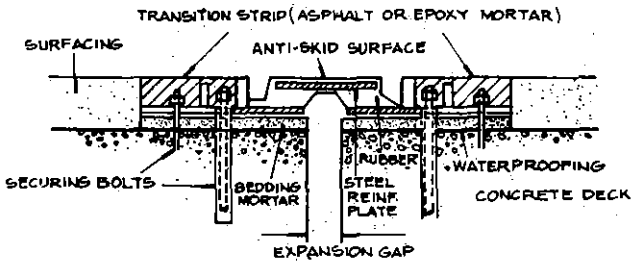


Fig. 32.35

Typical cross-section (SR2A-SR13) consists of steel angles and plates embedded in the neoprene moulded section, arranged so as to meet structural requirements. High strength extruded aluminium sections, affixed to the upper surface in the moulding process, are grooved to provide skid resistance as well as long life performance under heavy traffic loadings.

Further features include unitized, welded curb sections

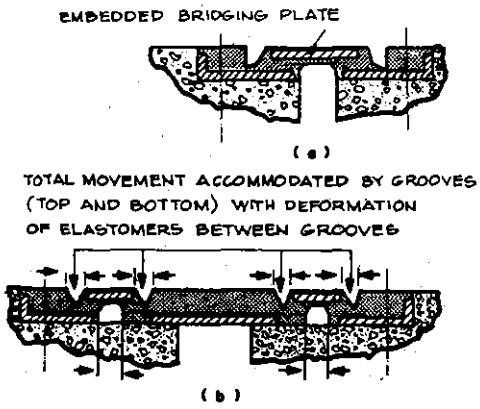


Fig. 32.36

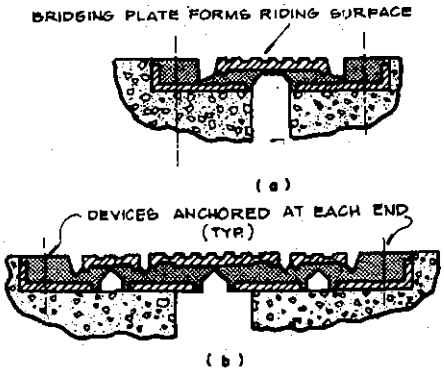
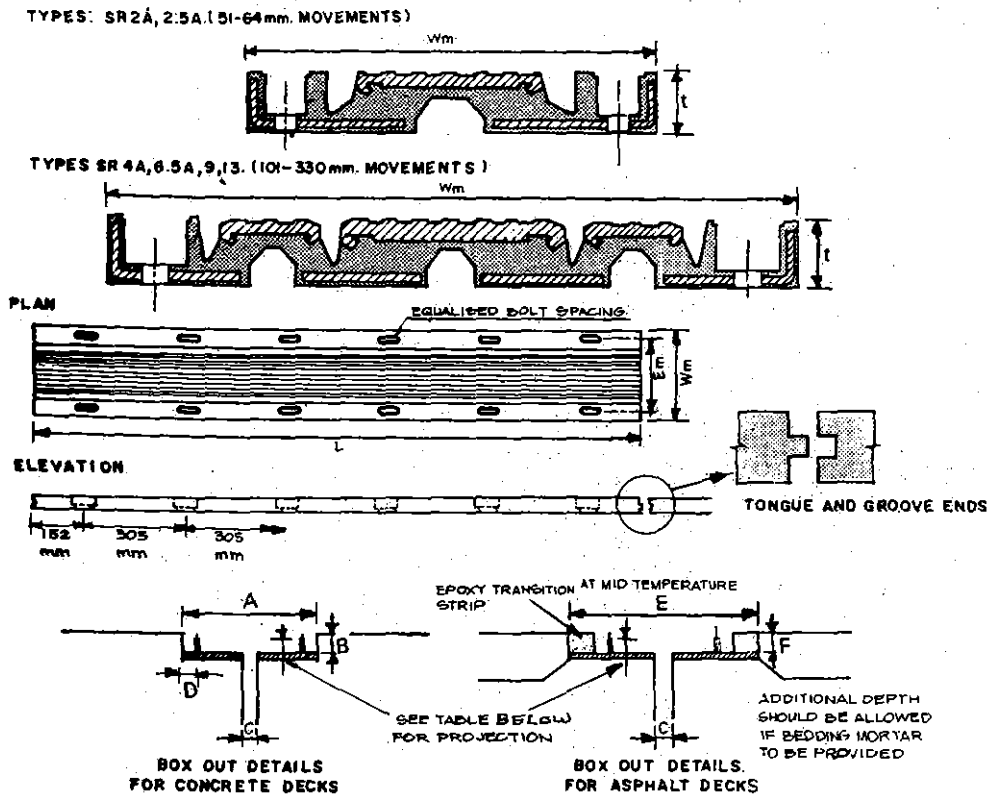


Fig. 32.37



WABOFLEX DIMENSIONS							BOX OUT DIMENSIONS							
							CONCRETE DECK						ASPHALT DECK	
Model No.	Total Movement	*L* Length	Wm Width	*t* Thickness	Em Bolt Centres	Wt of Unit	A		B	C		D	E	F
							Min.	Max	Depth	Min.	Max.	Bolt ℄	Width	Depth
	mm	mm	mm	mm	mm	kg	mm	mm	mm	mm	mm	mm	mm	mm
SR 2A	51	1829	270	40	213	40.83	251	302	44	25	76	32	370	44
SR 2.5A	64	1829	343	46	280	55.35	324	387	50	35	98	38	543	50
SR 4A	101	1829	578	54	499	104.80	540	641	58	25	127	46	778	58
SR 6.5A	165	1829	711	76	616	183.76	635	800	80	38	203	51	911	80
SR 9	229	1829	950	95	851	303.99	841	1070	99	44	273	52	1200	99
SR 13	330	1219	1397	127	1270	492.22	1238	1568	131	51	380	67	1697	131

NOTE "m" Suffix denotes dimensions at mean temp. when Waboflex in relaxed state

Fig. 32.38 *Physical properties of Waboflex SR expansion joints*

Table 32.3 *Mechanical Properties of Waboflex SR Expansion Joints*

Waboflex Type	SR2A	SR2.5A	SR 4A	SR 6.5A	SR 9	SR 13
Design Movement mm	51 kg/m	64 kg/m	101 kg/m	165 kg/m	229 kg/m	330 kg/m
Compressive force required in moving joint sections through the recommended movement range	2118	1328	738	1210	1845	2214
Tensile force required in moving joint sections through the recommended movement range	2214	1107	789	1299	2037	2362

Refer to Servicised Ltd., UK, for forces required in case of Skew Deck Joints.

Table 32.4 *Waboflex SR Fixing Specification*

Fixing Specification		WABOFLEX TYPE					
		SR 2A	SR 2.5A	SR 4A	SR 6.5A	SR 9	SR 13
Length of each unit mm		1,829	1,829	1,829	1,829	1,829	1,219
No. of fixings per unit		12	12	12	12	12	8
Studding*	Diameter mm	12	16	20	24	24	30
	Length* mm	140	170	200	230	235	245
Hole in Structural	Diameter mm	14	18	22	28	28	35
Concrete	Depth mm	110	135	160	185	185	185
Stud projection above bedding mm		25	30	35	40	45	55
Resin Capsule Diameter mm		12	16	20	24	24	30
Flat Washers	Outside Diameter mm	27	30	38	44	44	57
	Thickness mm	3.0	2.8	2.8	4.5	4.5	4.5
Spring lock washer thickness mm		3.0	3.5	4.0	4.0	4.0	6.0
Hex.Nut	Thickness mm	10.0	13.0	15.5	19.0	19.0	24.0
	Width across Flats mm	19.0	24.0	30.0	35.5	35.5	45.0
Torque Setting	kg m.	5.5	9.0	12.0	14.0	14.0	21.0
	ft lbs.	40	65	85	100	100	150
Bitu-Thene Mastic** Litres		0.92	0.92	0.92	0.92	0.92	0.61
Two-part synthetic rubber for tongue & groove joints		50 joints	34 joints	17 joints	10 joints	6 joints	3 joints
Uncured Rubber Strip	width mm	270	343	578	711	950	1397
For Butt Joints	Depth mm	40	46	54	76	95	127
Bolt hole sealant black verti seal † lb per unit		2.79	4.32	9.92	15.34	21.73	26.4
• Ancillary materials per metre length of joint							
PML Screed Primer kg		0.12	0.17	0.23	0.25	0.32	0.45
PML Epoxy Mortar QS †† litres (kg)		1.85 (3.7)	2.39 (4.78)	3.51 (7.02)	3.96 (7.92)	5.21 (10.42)	7.41 (14.82)
PML NS/FG Transition Mortar litres (kg)		4.0 (10.29)	9.2 (23.66)	10.8 (27.77)	15.2 (39.09)	23.75 (61.07)	38.1 (97.97)

• All studs, washers and hex nuts are supplied in mild steel to BS970 Part 1 220 MO7 having a tensile strength of 432–494 MN/m^2 and zinc plated to BS 3382 Part 2 1961 5–10 microns thick.

* Standard stud lengths assuming maximum 5 mm epoxy bedding mortar.

** Bitu-thene Mastic supplied in 3-litre units.

† Bolt hole sealant, black vertiseal supplied in 5lb. packs.

†† Assuming 5 mm thickness.

for leakproof sealing and moulded 'tongue and groove' connections between standard sections.

Waboflex-SR is an advanced design of elastomeric expansion jointing system developed for bridges, ramps and roadways. It is designed for long-life performance under heavy traffic. Latest improvements incorporated in Waboflex-SR provide stress reduction and "armoured" skid resistant wearing surfaces.

A skid resistant system is achieved with ribbed surface high strength aluminium 6061-T6 extrusions specially grooved for maximum friction. There are no large surface areas of rubber exposed to traffic and a continuous drainage groove assures rapid surface water run-off.

The armoured aluminium surface, ribs and impact edges protect the neoprene rubber from tyre abrasion.

The aluminium extrusions are 'T' locked, preventing delamination and resisting the buckling forces encountered in cycling. Steel angles and plates in A36 steel are embedded in the neoprene moulding to provide structural reinforcement.

These joints are compounded from	Du Pont neoprene to ASTM specifications
Tensile strength	D-412
Elongation	D-412
Hardness	D-2240
Compression set	D-395
Ozone resistance	D-1149
Over ageing	D-573
Low/High temperature Recovery	D-2628-67T

There is a significant reduction in the stress required to stretch or compress Waboflex SR Systems to the correct temperature-related setting.

It is recommended that Waboflex SR systems should be set 3-4 mm below wearing surface. Experience has shown that there is migration, erosion or flow of hot mix asphalt away from expansion joint systems leaving it exposed to traffic and general attrition.

Pre-Setting of Waboflex SR Joints—Calculations of Movements

The extent of movements in bridge deck expression joints should be based on the local highway authority's design requirements for the accumulative effects of temperature, prestress, creep and shrinkage, deflection of deck and rotation at the supports.

Figure 32.39 shows the pre-tension or precompression dimensions required for the Waboflex SR range to suit the Effective Bridge Deck Temperature at the time of installation and, therefore, the centreline to bolt offset. The theoretical Mean Temperature, at which the Waboflex SR is in a relaxed state, should be provided by the design engineer as part of the design specification.

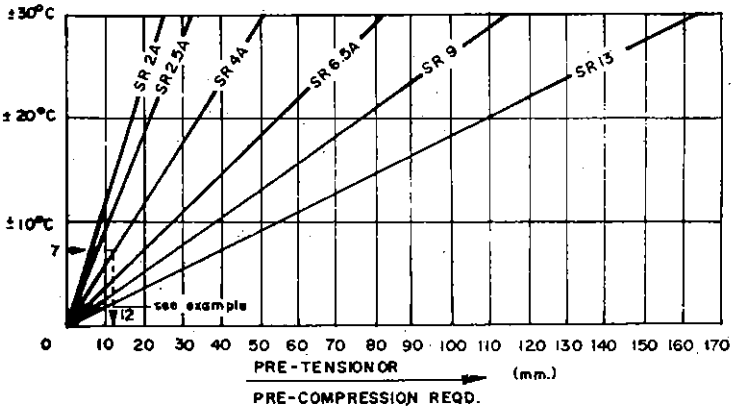


Fig. 32.39

Preset of Waboflex SR Range

It has been proved by extensive field testing that an approximate method of establishing the "Effective Bridge Deck Temperature", within a ± 3°C tolerance, without resorting to complex calculations and elaborate temperature gauges, is by measuring the 'Mean Shade Temperature' beneath the deck at 8.00 hours ± 1 hour and increasing this temperature by 3°C. (Refer to UK Department of the Environment TRRL 696 by Mary Emerson). Another way of establishing the movement of the bridge is by monitoring the structure over a period prior to installing the bridge expansion joint.

- EXAMPLE

Waboflex:

Theoretical Mean Deck Temperature given by Engineer = 20°C
Mean Shade Temperature at 08.00 hours = 10°C
Effective Bridge Deck Temperature = 10°C + 3°C = 13°C
Change from Theoretical to Effective-Bridge Deck Temperature: 20°–13°C = 7°C

From Fig. 32.39 it would be necessary to extend the Waboflex SR 4A by 12 mm (by using 'Companders' i.e. compression of extension clamps).

- Therefore, to obtain correct bolt offset from mean centreline for drilling studs:

Bolt centres when relaxed (Em, from Table in Fig. 32.38)	= 499 mm
plus pre-tension	= 12 mm
	511 mm
	÷2
∴ offset	= 255.5 mm

32.5 INSTALLATION OF WABOFLEX SR SYSTEMS

These instructions should be read in conjunction with working drawings prepared by the suppliers. Waboflex SR Expansion Joints can be easily installed by the main contractor in 'cutout' sections of surfacing in existing decks and specially prepared concrete strips of new decks without resorting to specialist subcontractors. The application procedure consists of drilling fixing holes at predetermined positions, jacking the sections together to ensure water-tightness and bolting them down. Ideally, a site survey should be carried out after installation of the road surfacing and pavement lines to obtain an accurate profile of the bridge expansion joints on which the working drawings are based:

Tools Required

Heavy Duty Disc Cutter
Concrete Scabbler
Electric Wire Brush
Drill-Hammer/Rotary type with pillar drilling rig
Air Compressor for cleaning holes
Drive-Unit for installing studding
Small Jack
Ratchet, Wrench and Sockets, speed brace with 100 short extension
Torque Wrench
'Companders'—width adjusting clamps to extend or compress Waboflex to suit ambient temperature (available on hire from suppliers)
'T' Paddle and Mastic Gun with Follower Plate
Remaining ancillaries are found as normal site equipment and materials.

Materials Required:

* Screed Primer
* Bedding Mortar
Stud anchors and Resin capsules
Bitu-thene Mastic
Prefabricated Kerb Sections
Straight Waboflex Sections
Heavy Duty Flat Washers
Spring Lock Washers
Nuts
Cleaning and Priming Solutions
Two-part Liquid Cold Rubber Polymer
* Transition Strip Mortar
Bolt Hole Sealant (Black Vertiseal)
Uncured Rubber Strips for butt joints
Tool Cleaner

* These materials are available from the suppliers.

- **Procedure: (Steps 1 to 10, Figs. 32.40 to 32.49):** Determine mean centreline of expansion joint by establishing setting-out points at extremity of carriageway, taking into account any irregularities produced during concreting of the deck. Using chart shown in Fig. 32.38 mark width of box-out symmetrically about centre line with a chalked string line.

Remove the road surfacing materials over the specified

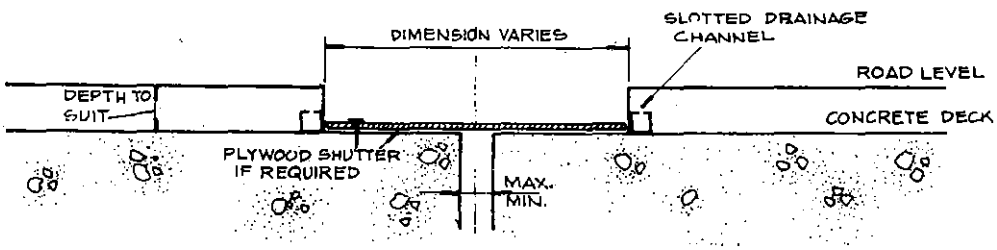


Fig. 32.40 *(Step 1)*

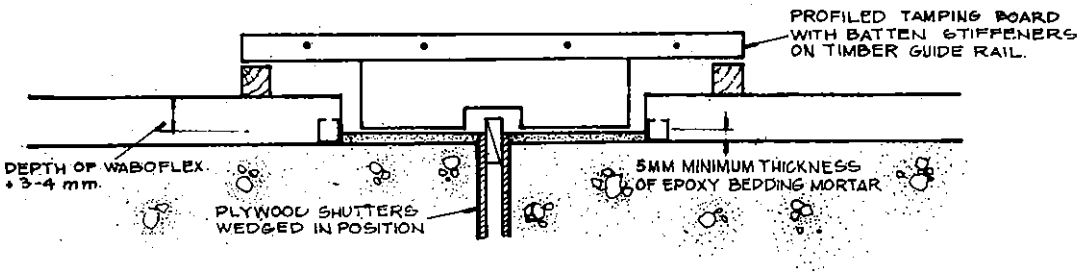


Fig. 32.41 *(Step 2)*

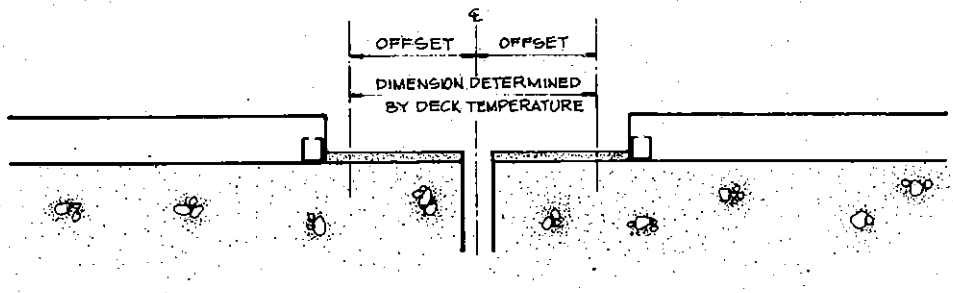


Fig. 32.42 *(Step 3)*

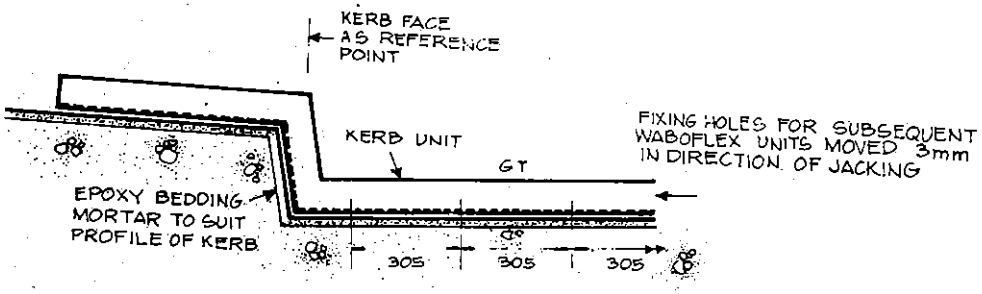


Fig. 32.43 *(Step 4)*

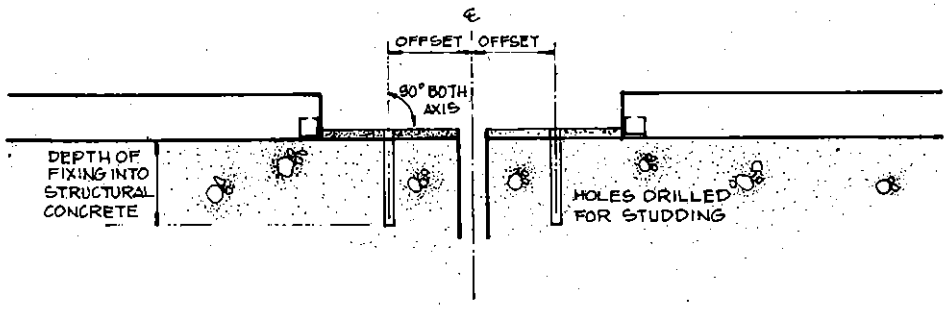


Fig. 32.44 *(Step 5)*

width, measured symmetrically about the centre line of the expansion joint gap. If the road surfacing has not been laid, a piece of plywood can be fixed to the concrete deck to allow easy removal of the surfacing after laying has been completed. A heavy duty disc cutter should be used to cut through the surfacing materials to the level of the concrete deck and the material between the saw cuts broken out and cleared away.

The concrete surface within this area should be mechanically scabbled, in order to remove surface laitence and bituminous materials that may still be adhering to the concrete. Following this operation all lose material should be removed by brush and air line.

When required, the level of the concrete surface should

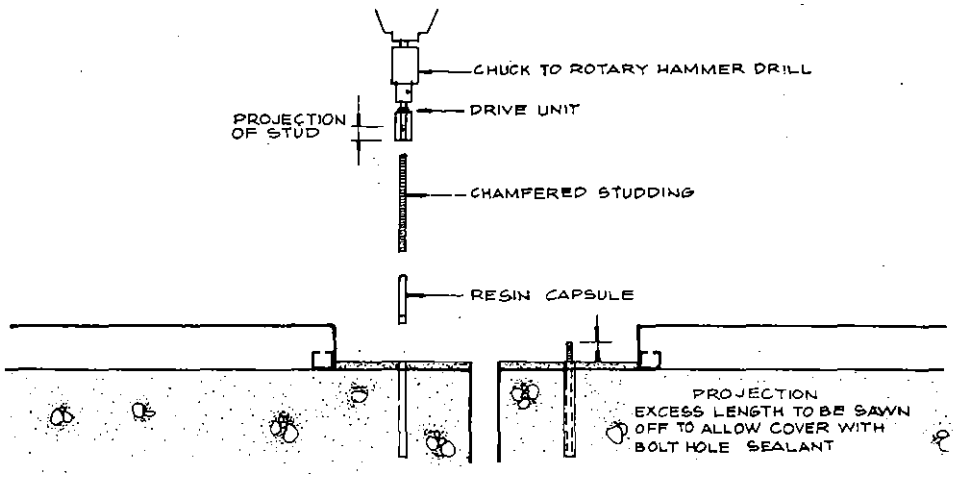


Fig. 32.45 *(Step 6)*

be made-up, using a bedding mortar, mixed and applied as specified by the manufacturer to ensure a true and even surface on which to bed the Waboflex to the correct level in relation to the existing road surfacing. This is achieved by having timber battens on each side of the joint acting as guide rails and using a profiled tamping board with stabilising batten. A steel float can be used to smooth off any tamping marks.

A longitudinal check should be made with a straight edge to ensure that the Waboflex will have an even seating.

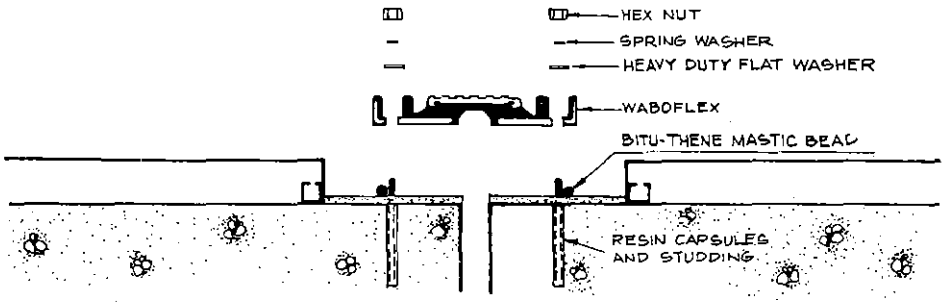


Fig. 32.46 *(Step 7)*

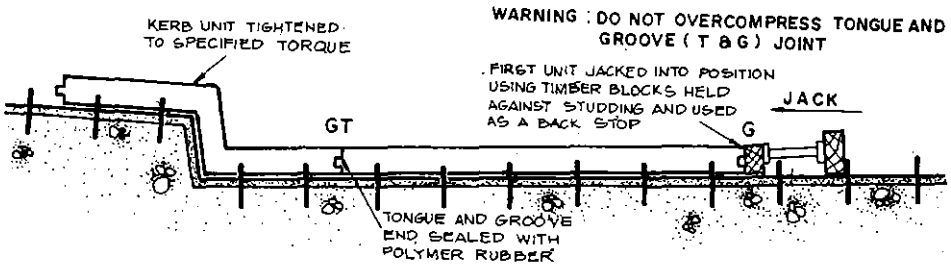


Fig. 32.47 *(Step 8)*

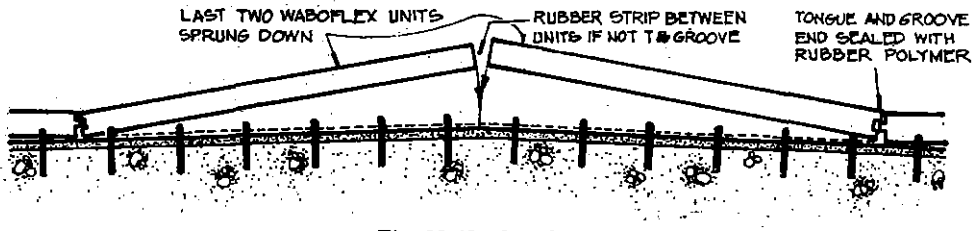


Fig. 32.48 *(Step 9)*

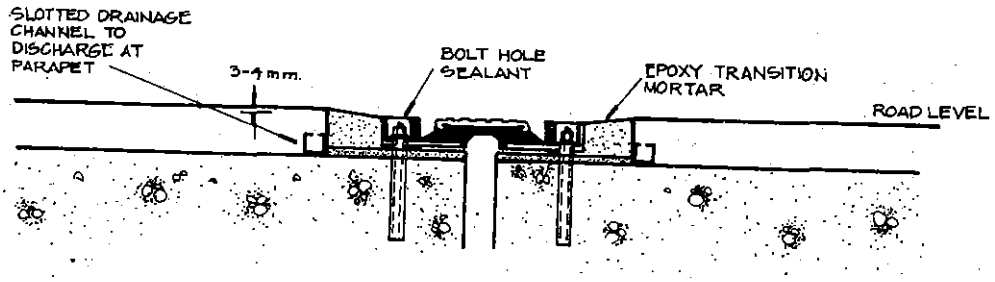


Fig. 32.49 *(Step 10)*

It is essential to use the kerb unit at the lowest level of the cross falls to act as a reference point for establishing the transverse setting out of the holes for the main deck.

'Longitudinal' means along Waboflex, parallel to joint. Transverse means 90° across the joint. The transverse centre line of the fixing studs should be set out symmetrically about the centre line of the expansion joint by marking the bedding mortar and projecting the setting-out across the carriageway, using a string line or theodolite when a string line is impractical or inaccurate.

The distance between fixing stud centre lines will be dependent on the relationship between actual bridge deck temperatures at the time of construction and at the time of fixing. An approximate method of determining the bridge deck temperature and stud hole centres is shown in the Example under Fig. 32.39. These dimensions should be calculated in conjunction with a representative of the Design Authority. In the majority of cases Waboflex will be installed in a 'relaxed state' without resorting to precompression or tension.

The longitudinal setting-out of the studs should commence by laying the lowest kerb unit in line with the kerb face. The sections of Waboflex can be used as a template to mark out the longitudinal positions of the fixing studs.

A rotary hammer-drill, with pillar drilling rig for accurate positioning and square alignment to the bedding surface, should be used to form holes to the diameter and depth specified in Table 32.4. In the absence of a rig, use a steel template as a guide to prevent the drill bit from wandering.

Where steel reinforcement is encountered, it will be necessary to employ a hollow core, water cooled diamond tipped drilling bit. Following the use of a diamond drill, the hole should be roughened using the rotary hammer drill.

Holes to be thoroughly blown free of loose materials and free water and the specified anchor capsule inserted with the filler cap at the bottom.

A drive unit adaptor is fitted to the rotary hammer drill to accommodate the studding and act as a depth check. The chamfered end is driven to the bottom of the hole, mixing the resin and hardener together. Care should be taken not to over mix, as this will cause the compound to extrude from the hole. The fixed studs should be left undisturbed for a period as specified. Should oversize holes be drilled in error, they should be filled with polyester grout with studs aligned vertically on both axes.

When the studding is firmly set, a continuous bead of Bitu-thene Mastic should be gunned or trowelled along the outside of the Waboflex stud line on a clean and dry surface.

Installation of the Waboflex should commence from the kerb, upstand or end-unit at the lowest level of the roadway. The width of the first unit may be 'compressed' or 'extended' to suit the joint-width, using companders, and fitted over the studding. One heavy duty flat washer, one lock washer, and a nut are tightened to the specified torque.

The adjacent unit is adjusted to width clamps as before and placed over the studding, ensuring that the tongue and grooved end faces 'correspond'. Before bringing these faces together the tongue and grooved ends should be treated with cleaning and priming solutions followed by a coating of cold-applied polymer-rubber into the groove. This unit is then jacked into position to close the units and form a tight fitting joint. The fixings are tightened down to the specified torque. Any excess compound extruded when units are brought together should be removed. This procedure is continued across the width of the bridge until the last two units are ready to position. This would occur at the crown of the road or one unit out from the highest kerb. When sawn

ends abut, inserts of rubber strips should be included prior to jacking the units together.

In order to achieve a tightly-fitting joint when the final piece is inserted, it will normally be necessary to measure the length of the closing piece required and cut to suit on site and spring the last two units down into position in compression. Careful cutting of these sections can be carried out with the equipment used for cutting the road surfacing. The temperature of the Waboflex should not be allowed to rise excessively (to avoid damage to the rubber).

The dimension for the closing pieces should not be taken until all the remaining units are in position and firmly fixed.

After the installation has been completed a minimum period of four hours should be allowed to elapse before checking the torque on the locating studs and, if required, these should be re-torqued to the specified figure.

After re-torquing, the stud hole recesses should be cleaned (without using solvents), and filled with bolt-hole sealant. The gap between the edges of the Waboflex and the adjacent road surfacing should then be filled with flexibilised epoxy mortar mixed and applied as specified. Ensure that this material is fully compacted and 'steel-float finished'.

- **Finger-type Expansion Joint**

The most silent expansion joint is still the 'finger type', in steel, which can be used for expansions of up to ± 400 mm (Fig. 32.50). The fingers should have a trapezoidal cross section so that no stones etc., can remain sticking between the fingers. Of course this joint is not watertight and it is necessary to drain the water which runs through between the fingers. It can be collected in a drain gutter which should be made of a fibre reinforced plastic membrane and should easily be approachable for inspection and cleaning (Fig. 32.51).

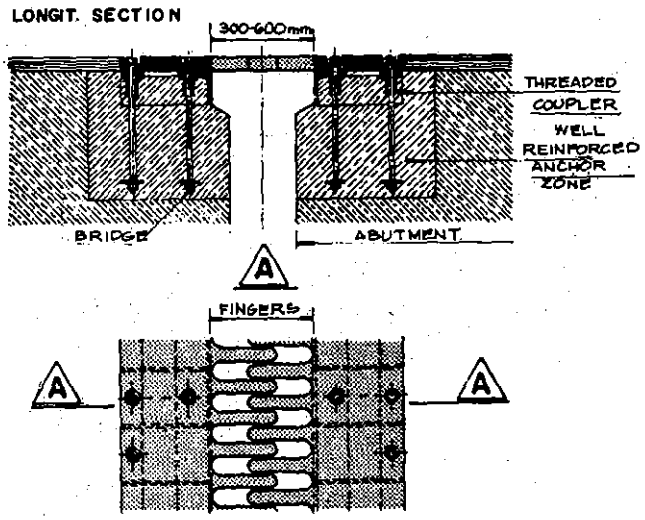


Fig. 32.50 *Finger type expansion joint*

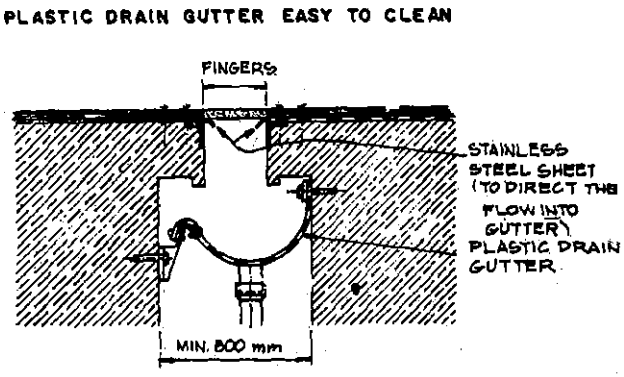


Fig. 32.51 *Gutter with plastic membrane*

- **Freyssinet's 'FT' and 'Monobloc' Expansion Joints**

See Fig. 32.52.

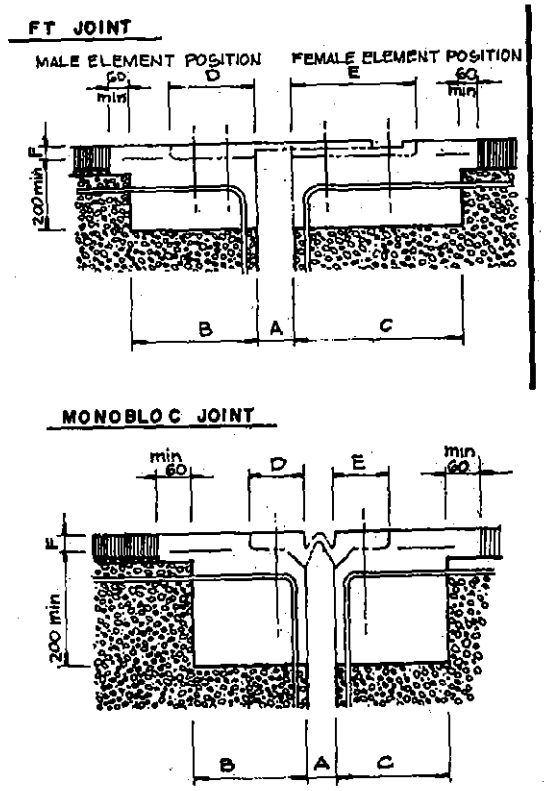


Fig. 32.52

(a) *THE FT JOINT:* The FT joint (0-600 mm movement) consists of two basic components: a male element and a female element, both made of a combination of STEEL and NEOPRENE. The front part of the male element slides back and forth over the female element (contraction and expansion). The front part of the male element generally has 'fingers' that move in the receiving grooves of the female element.

(b) *THE MONOBLOC JOINT*: The MONOBLOC joint (0–50 mm movement) consists of a moulded elastomer profile vulcanized between two identical steel or aluminium elements.

The advantages of these neat joints (as indeed in certain other proprietary types too) are:

- No thrust on the structure (unlike compressing an elastic section)
- No bracket fixings
- Flexibility
- Compact size
- Self-cleansing
- Easily adjusted to road level
- Retightenable
- Movement up to 600 mm, in 10 standard models, and
- Models for skew bridges up to 50 degrees (with or without fingers, as convenient).

(d) *KERB AND FOOTWAY JOINTS BY FREYSSINET*

- See Fig. 32.53.
- A complementary range of kerb and footway joints for each type of FT and MONOBLOC joint is available from FREYSSINET INTERNATIONAL. These sliding joints are also made of combination of STEEL and NEOPRENE. The kerb joint is always adapted to the kerb profile used for a particular structure.
- Footway joints are manufactured in 1 metre lengths with 10 fixing studs per linear metre.

(e) *INSTALLATION OF FREYSSINET EXPANSION JOINTS*

This is shown in Fig. 32.54 step and sequence wise:

Step 1 REBATES (recesses) IN THE STRUCTURAL CONCRETE

Rebates are formed across the entire width of the deck during the construction of the structure.

Step 2 PROTECTION OF REBATES AND SURFACING

The rebates are filled with loose gravel and covered with a thick steel plate. The waterproofing and black top are then laid continuous across the rebate.

Step 3 CUTTING OUT THE SURFACING AND CLEANING OF REBATES

The black top and waterproofing are sawn through on either side of the rebate. The surfacing, water-

(c) *RANGE AND DETAILS OF FREYSSINET 'FT' AND MONOBLOCK' JOINTS (see Fig. 32.52)**

Type	Total movement	A		B	C	D	E	F	Length of each element	Fixing Device			Weight
		min	max							ϕ	Number of fixings male el.	Number of fixings female el.	kg.
MONOBLOC 15	15	10	25	150	150	60	60	26	997	14	5	5	13
FT 25	25	15	40	200	200	100	100	27	747	14	5	5	30
FT 30 D*	30	10	40	200	200	100	130	22	997	16	5	5	22
MONOBLOC 30	30	15	45	200	200	105	105	28	997	16	5	5	23
FT 50	50	20	70	250	250	115	145	33	997	16	4	4	41
FT 50 D*	50	20	70	250	250	120	155	33	747	14	5	5	52
MONOBLOC 50	50	20	70	200	200	100	100	30	997	16	4	4	26
FT 75	75	20	95	300	300	135	190	39	997	16	5	3	53
FT 75 D*	75	20	95	300	300	135	190	39	997	16	5	3	50
FT 100	100	20	120	350	350	160	250	47	747	16	5	3	84
FT 150	150	20	170	350	450	220	360	50	747	16	8	8	151
FT 200	200	30	230	450	600	300	450	50	497	16	6	6	190
FT 250	250	50	300	500	700	330	560	57	497	16	7	6	250
FT 300	300	50	350	650	750	370	550	90	497	20	6	4	430
FT 400	400	50	450	650	850	370	650	90	497	20	6	4	470
FT 500	500	50	550	650	950	370	750	90	497	20	6	4	505
FT 600	600	50	650	650	1050	370	850	90	497	20	6	4	560

— All dimensions in mm.

— D* indicates absence of teeth (fingers)

TYPE	opening	A		B	C	D	E	Weight per 1 m kg.
		min	max					
JTFT 25	25	20	45	200	200	100	100	25
JTFT 30	30	20	50	200	200	100	100	25
JTFT 50	50	20	70	200	200	100	100	25
JTFT 75	75	20	95	200	200	120	100	28
JTFT 100	100	20	120	250	200	150	100	32
JTFT 150	150	20	170	300	200	200	100	39
JTFT 200	200	30	230	350	200	260	100	64
JTFT 250	250	50	300	400	250	310	150	86

All dimensions in mm (JTFT 300 to 600 also available from Freyssinet international).

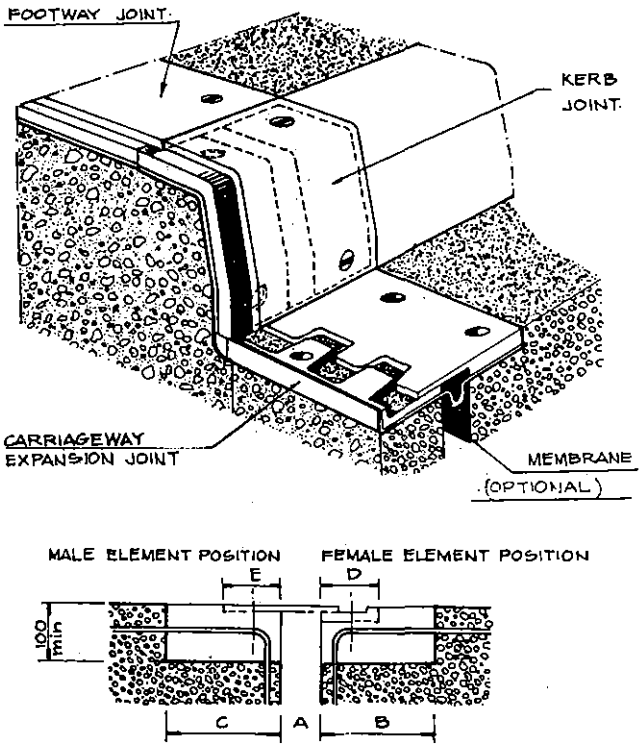


Fig. 32.53

proofing, and gravel are removed and the rebate is thoroughly cleaned.

Step 4 PLACING OF DRAIN (option) AND REINFORCING

A drain may be placed along the edge of the waterproofing cutaway. The rebate is reinforced with mild steel stirrups and continuity bars. A polystyrene and plywood void-former is then positioned.

Step 5 ADJUSTMENT AND CONCRETING

The joint, complete with fixings, placed in position, element by element. The embedding concrete which must be thoroughly vibrated, is poured into the rebate on either side of the void-former.

Step 6 PLACING THE WATERTIGHT MEMBRANE (option) AND FINAL TIGHTENING

When the concrete has set, a watertight membrane may be placed between the joint and it's seating. The final tightening is then carried out with a torque wrench.

Anchorage of Expansion Joints

Each side of the joint must be anchored to the deck (and abutment, as the case may be) so as to safety transfer all static and dynamic loads from the joint assembly to the structure. It is normal practice to design anchorages by neglecting any contribution from concrete under the joint. Field experience and laboratory testing (References 1 and 2) have shown that at least the following minimum anchorage requirements should be satisfied:

(a) Where the deck joint assembly is attached by reinforcing bars cast into concrete, the total cross-sectional area of such bars on each side of the joint should not be less than 1600 mm^2/m and the bar spacing be not more than 250 mm. Perforated plates giving an equivalent anchorage capacity may also be used. The anchorage system should be fixed to the main structural reinforcement. In the case of perforated plates or loops, this can be done by passing the deck transverse bars through these loops.

(b) Where the joint assembly is attached directly to the steel superstructure by welding or bolting, the welds or bolts should have an ultimate capacity of not less than 600 kn/m. The spacing between the bolts or between welds should not exceed 250 mm.

(c) Any other method of anchorage should have a capacity at least equal to (a) or (b) above.

Anchor bolts, set in concrete, have not been found terribly satisfactory as they tend to work loose under dynamic loading (Ref. 3). Loosening of the bolts has been observed with both friction type anchors and with bolts set in grout. Anchor bolts installed nearer the face of the joint than the first transverse reinforcing bar have also caused spalling of the concrete at the joint.

The steel structure of expansion joints must be strongly anchored to the deck slab, especially in case of concrete decks where securing by welding is generally not possible. There must be sufficient overlapping of the anchor bars

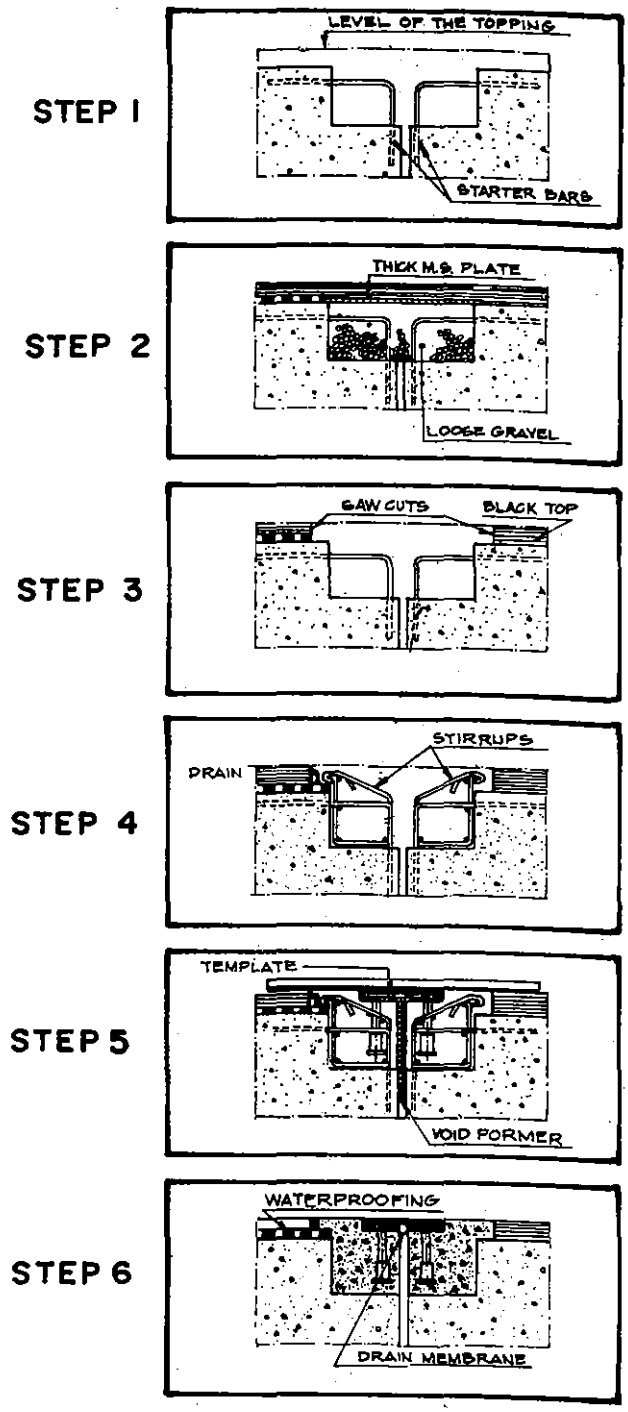


Fig. 32.54

with the reinforcement of the slab. Round ribbed reinforcing bars should be used and safely welded to the steel profiles (angles, fins, etc.).

The detailing of such reinforcement must be planned with attention to the 'placing' process. The gap must be well cleaned and wetted before the high strength concrete is placed and compacted in the recesses.

A word of general caution: Too often a proprietary joint is selected from a manufacturer's catalogue at the last moment, improperly installed by unskilled labour, and then not maintained well. Where a competing system is actually installed under an 'or approved equal' provision in the contract documents, the joint may bear little resemblance to that shown on the plans. Appropriate modifications in anchorage or blockout details may not always be attended to in advance. Consequently, it is not surprising that the performance of deck joints has often not been satisfactory.

Proprietary Joints

There are many organisations which manufacture these joints. While the products of many of them differ only slightly in actual details, one may not be able to say the same for their qualities. In selecting an appropriate joint the Engineer must carefully weigh the manufacturer's claims on performance against the actually required functions of rotation and translation and the required directions of the incumbent movements, the fixing details, the first-cost and the maintenance problems.

Some of the well known brands are: Maurer, Freyssinet-International, Thormajoint, Transflex, Waboflex,* Gutehoffnungshutte (GHH), and Demag. Strict adherance to quality controls and actual performance-record remain the governing yard-sticks for impartial selection of the actual product.

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* Servicised Ltd.

CHAPTER 33

Parapets and Railings for Highway Bridges

The requirement for a parapet to provide a safeguard against a vehicle which is out of control, plunging over the edge of a bridge, cannot be specified in terms of a static loading condition. The ability to absorb or redirect the energy of an errant vehicle is a function of the flexibility and constructional details of a parapet as much as on the nature and speed of the vehicle. Design regulations have, therefore, been based on the 'containment' requirements in terms of a specified weight of a vehicle and its approach angle and speed. The assessment of suitable parapet designs has become a matter of tests rather than design calculations.

It would be impracticable to stipulate that a parapet should be capable of containing 'any vehicle' travelling at 'any speed'. Requirements must be rationalized, and very few incidents have arisen in which vehicles have plunged through parapets, although there is inevitably much publicity in instances where this does occur with a consequent loss of life.

The selection of the type of parapet for a bridge is of fundamental importance to its appearance. In fact, for traffic users crossing a bridge, the parapet is likely to be the only indication that they are on a bridge structure. The fundamental choice is between a 'solid concrete parapet' usually surmounted by a single rail, and a more-open 'metal parapet'. Each can have visual merits depending on the general configuration of the bridge structure.

Other things being equal, if a three-span or four-span bridge is required over a motorway to carry a local road, with consequent light loading, it would seem inappropriate to introduce heavy concrete parapets onto a structure which would otherwise be slender.

Because it is very important to the finished appearance of a bridge, the parapet and its supporting upstand merit particular attention during detailing. The main potential hazard is weathering as a result of water staining. Even where the parapet is non-corrosive, such as where it is of aluminum, if water running off the parapet is allowed to run over the front face of the supporting upstand, this will lead to severe staining in time, which will have a disfiguring effect. The width of the supporting upstand, therefore, needs to be ample to accommodate the parapet post fixings and base plate, with a sufficient marging of width to ensure that the water drains into the bridge rather than over the front face.

Where a metal parapet is to be used, a choice must be made between steel, which will then require painting (not only in the course of construction but as a regular item of maintenance), and aluminium, which has gained widespread favour. Its colour is complementary to that of concrete, and the absence of any need for routine maintenance in the form of painting is a significant advantage.

33.1 DEFINITIONS

(a) **Safety Fence/Guard Rail**
A continuous metal beam intended to redirect an errant vehicel along the line of the fence. Guard Rail generally has wire ropes instead. These fences may be used where the road-embankment is less than 4 m high and is steeper than 1V to 4H.

(b) **Parapet**
A protective fence or wall at the edge of a bridge or similar structure.

(c) **Vehicle-Parapet**
A parapet designed to 'contain' vehicles on a structure from which pedestrians, animals and cyclists are excluded by Order.

(d) **Pedestrian-Parapet**
A parapet designed to safeguard pedestrians, but not intended to contain vehicles.

(e) **Vehicle-Pedestrian Parapet**
A parapet designed to contain vehicles and to safeguard pedestrians.

33.2 CLASSIFICATION OF HIGHWAY BRIDGE PARAPETS

While different countries may have separate designations and classifications for these, the British Deptt. of Environment, previously 'The UK Deptt. of Transport' (reference its Technical Memorandum No.: BE5) divided these parapets into six categories, P1 to P6. Of these, the P3 category has since been discontinued and the remaining five are briefly described in Table 33.1 ahead.

Table 33.1 *Application and Containment Requirements for Parapets*

Parapet Group Designation	Application	Containment for which Designed
P1	Vehicle-parapets for bridges carrying motorways or roads to motorway standards* (excluding motorway bridges over railways and high risk locations).	1.5 t vehicle at 113 km/hr. and 20° angle of impact
P2	Vehicle-pedestrian parapets for bridges carrying all purpose roads and for accommodation bridges (excluding bridges over railways and high risk locations).	1.5 t vehicle at 80 km/hr. and 20° angle of impact
P3	Parapet of this designation is no more in vogue.	
P4	*Pedestrian-parapets* for use on *footbridges* and bridges carrying bridleways (excluding bridges over railways).	
P5	*Parapets for use over railways* (excluding use on bridges at high risk railway locations):	
	i. On bridges carrying motorways or roads to motorway standards*	As for Group P1
	ii. On bridges carrying all purpose roads	As for Group P1
	iii. On footbridges	As for Group P4
P6	High Containment vehicle-parapets and vehicle-pedestrian parapets at high risk locations (excluding accommodation bridges).	30 t vehicle at 64 km/hr. and 20° angle of impact

* Roads from which pedestrians, animals, pedal-cycles and vehicles drawn by animals are excluded by order.

NOTE: In general. P2 type parapets are intended to have half the strength of P1 parapets (see Table 33.2 ahead). These strengths are approximately proportional to the square of the speed of the vehicle to be contained.

33.3 VARIOUS DETAILS

High Containment Parapets (Type P6)

High containment parapets are provided at certain high risk locations where the likelihood of parapet impact and consequent dangers resulting from parapet penetration are judged to be more than the hazards resulting from containment of errant vehicles and their redirection back into the traffic stream.

At certain locations, the nature of the area below the bridge may justify the High Containment parapet. Such a location could be when:

- the bridge is above a high speed railway line or motorway
- the bridge is above a busy railway line or motorway
- the bridge is close to an area occupied by people or by valuable installations or used for storage of hazardous materials
- the bridge is over any type of railway line or motorway but has 'inferior' horizontal or vertical alignment
- the bridge is part of a complex interchange where driver's error is more likely
- where road junctions are very close to the bridge or its approaches
- where the road arrangement and actual traffic have a record of accidents.

Materials

All parapets may be constructed of metal or reinforced concrete.

Height

The height of parapets is measured above the 'adjoining paved surface' and should not be less than the following:

1000 mm	...	for vehicle-parapets and vehicle-pedestrian parapets except as given ahead.
1500 mm	...	for accommodation bridges
1150 mm	...	for footbridges except over railways
1800 mm	...	for bridleways
1250 mm	or	
1500 mm	...	for all bridges over railways (and the height could be more)
1500 mm	...	for High Containment applications.

Shape/Form of Various Parapets (Figs. 33.1 to 33.4)

Reinforced concrete parapets or metal parapets with shaped plinths may be used for all vehicle-parapets and vehicle-pedestrian applications. P5 has same containment standard as for P1. (Reference may be made to the stated BE5 (or its latest version) for more details, as also of P4 and P5.)

Parapet type P6 may have the form same as that shown in Fig. 33.1 or Fig. 33.2 but should be of adequate section strength (see Table 33.3 ahead) and 1500 mm tall. (The minimum height of shaped reinforced concrete parapets is kept between 1250 and 1500 mm above the adjoining paved surface.) Special conditions at particular sites may require taller parapets; these cases should be considered on their individual merits and as a departure from the standards.

Standard Bridge Parapet and Metal Handrail for Expressway Traffic Conditions

See Fig. 33.5 and Fig. 33.6.

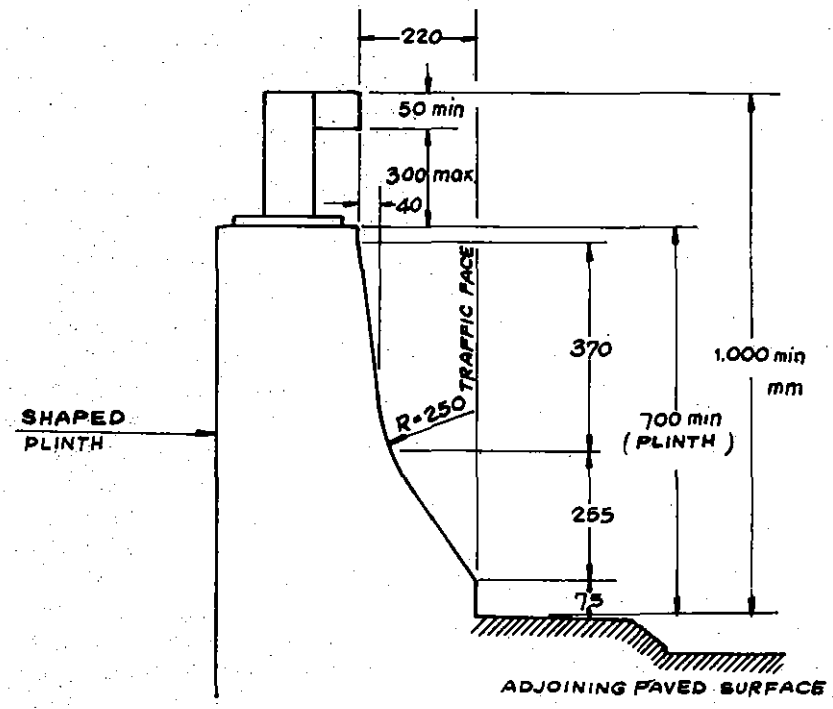


Fig. 33.1 *P1 'Vehicle-Parapet' with plinth 700 mm high or more for use in motorway under bridges*

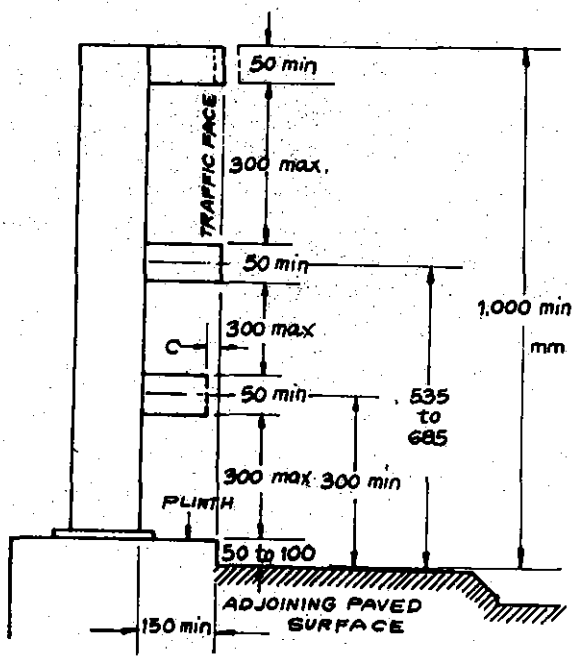


Fig. 33.2 *P1 'Vehicle-Parapet' with plinth less than 700 mm high for use on motorway under-bridges*

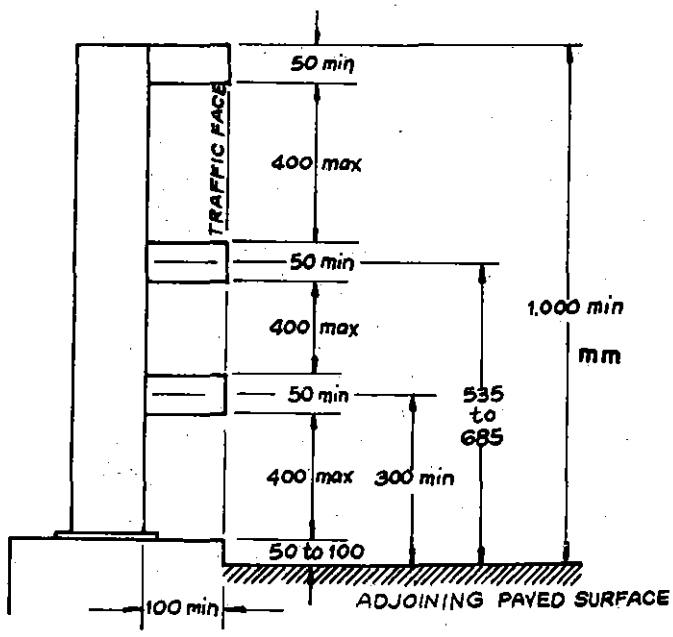


Fig. 33.3 *P2 'Vehicle-Pedestrian Parapet' for use on all purpose road bridges. The design speed being stated*

Notes for Parapet and Railing:

(i) All concrete shall be f'_c = 250 kg/cm^2 (28 days cylinderstrength)
(ii) All dimensions are in cm unless stated otherwise.
(iii) Reinforcing steel shall be grade 60 (F_y = 4200 kg/cm^2)
(iv) Concrete cover shall be 4 cm, unless otherwise shown.
(v) All exposed concrete edges shall provide 2 cm, chamfer.
(vi) Metal hand railing shall be approved by the Engineer.
(vii) Parapet shall be cast-in-situ.
(viii) All structural steel shall be galvanized and shall conform to AASHTO M-183.
(ix) All the exposed edges of structural steel shall be rounded.
(x) All connections of structural steel shall provide 6 mm fillet weld on all sides around.

NOTE: These details have demonstrated superior safety aspects for high-containment of Expressway Traffic.

Parapet used on Many US Highways

Figure 33.7 shows typical dimensions of a standard Parapet Kerb with metal hand rail as adopted by many authorities in the USA. It has demonstrated very satisfactory safety aspects for Expressway Traffic Conditions.

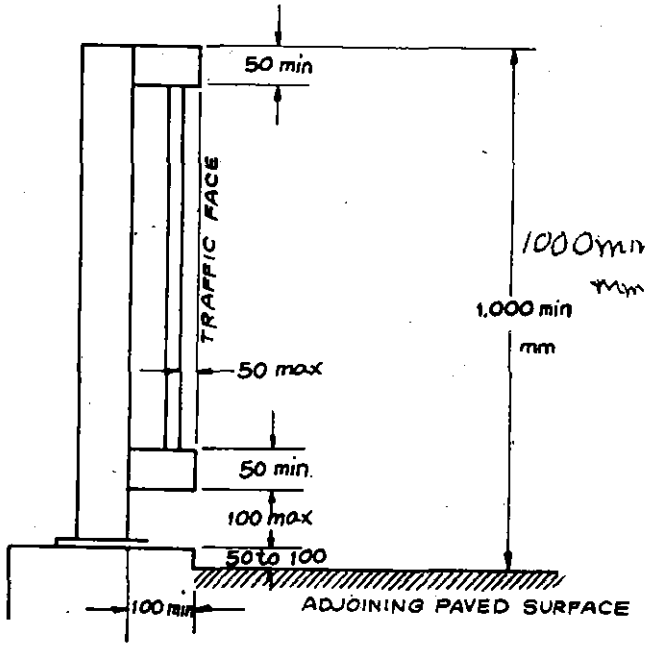


Fig. 33.4 *P2 'Vehicle-Pedestrian Parapet' for use in road bridges where speed is restricted to 48 km/hr*

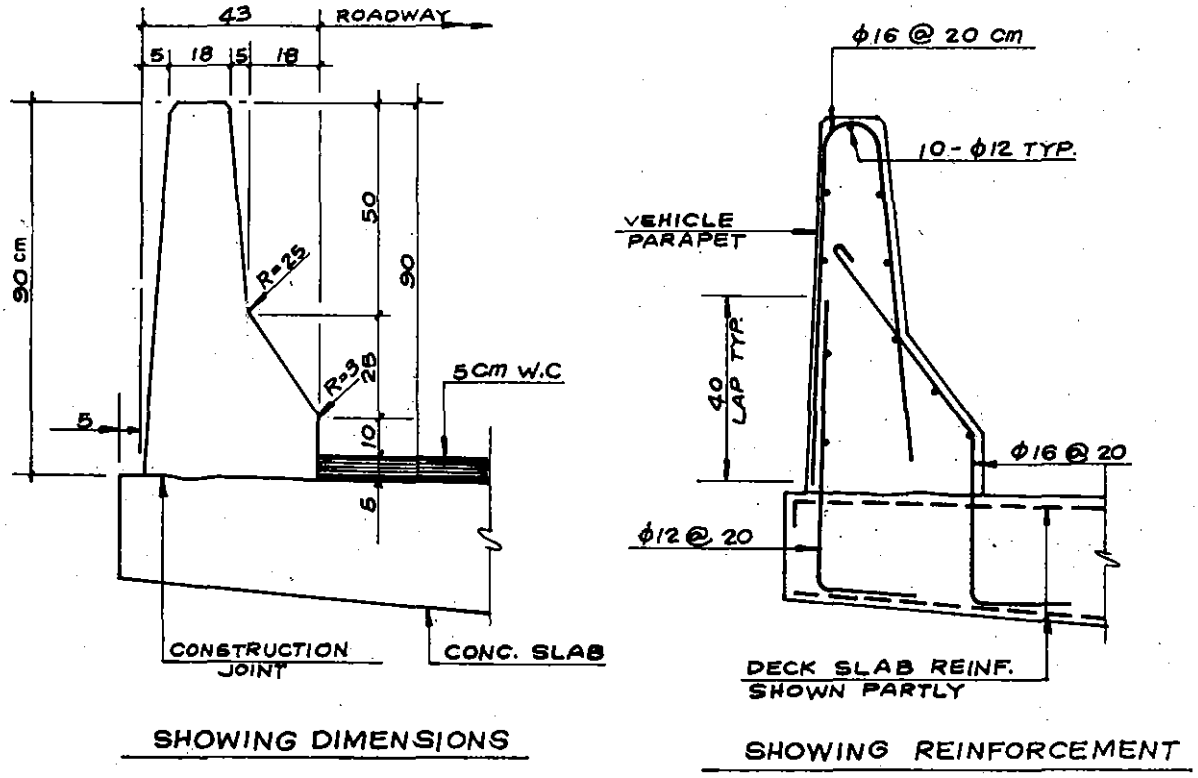


Fig. 33.5 *Vehicle-Parapet for bridge without sidewalk*

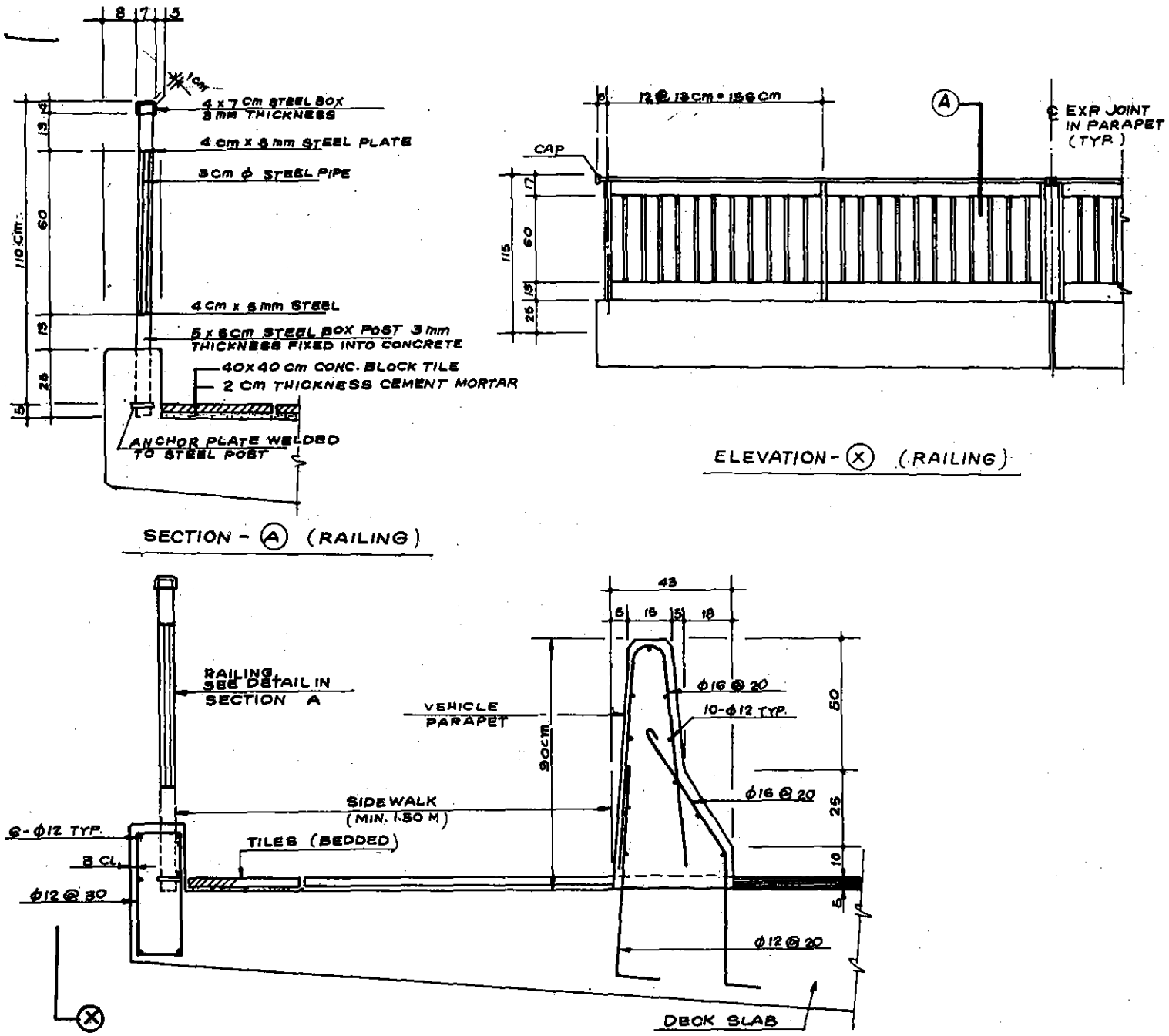


Fig. 33.6 *Vehicle-Parapet and railing for bridge with side-walk (slab reinforcement not shown.)*

An Almost Fool-proof Parapet for Expressway Traffic

There is no such thing as a fool-proof parapet. However, for an 'almost' fool-proof parapet system for Expressway traffic, it is suggested that a 300 mm unmountable RC kerb in place of the usual 230 mm standard kerb be provided along with the Fig. 33.8 arrangement.

It has been confirmed by the Road Research Laboratory in the UK (who have supervised dynamic testing of parapet details) that even a 230 mm kerb does not throw the errant vehicle up because the shock absorbers of the vehicle absorb the impact.

Requirements for Metal Members of Metallic Parapets

Reference may be made to the UK DOE Technical Memorandum BE5 (or its latest version) regarding the following requirements:

- elongation
- minimum thickness of metal
- protection against corrosion
- maximum distance between posts supporting the longitudinal members
- strength of the effective longitudinal members

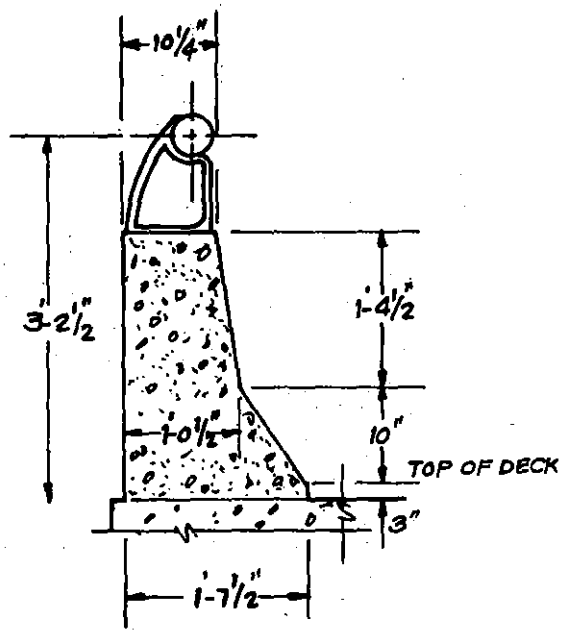


Fig. 33.7 *Vehicle-Parapet Kerb (reinforcement not shown) AASHTO/ACI*

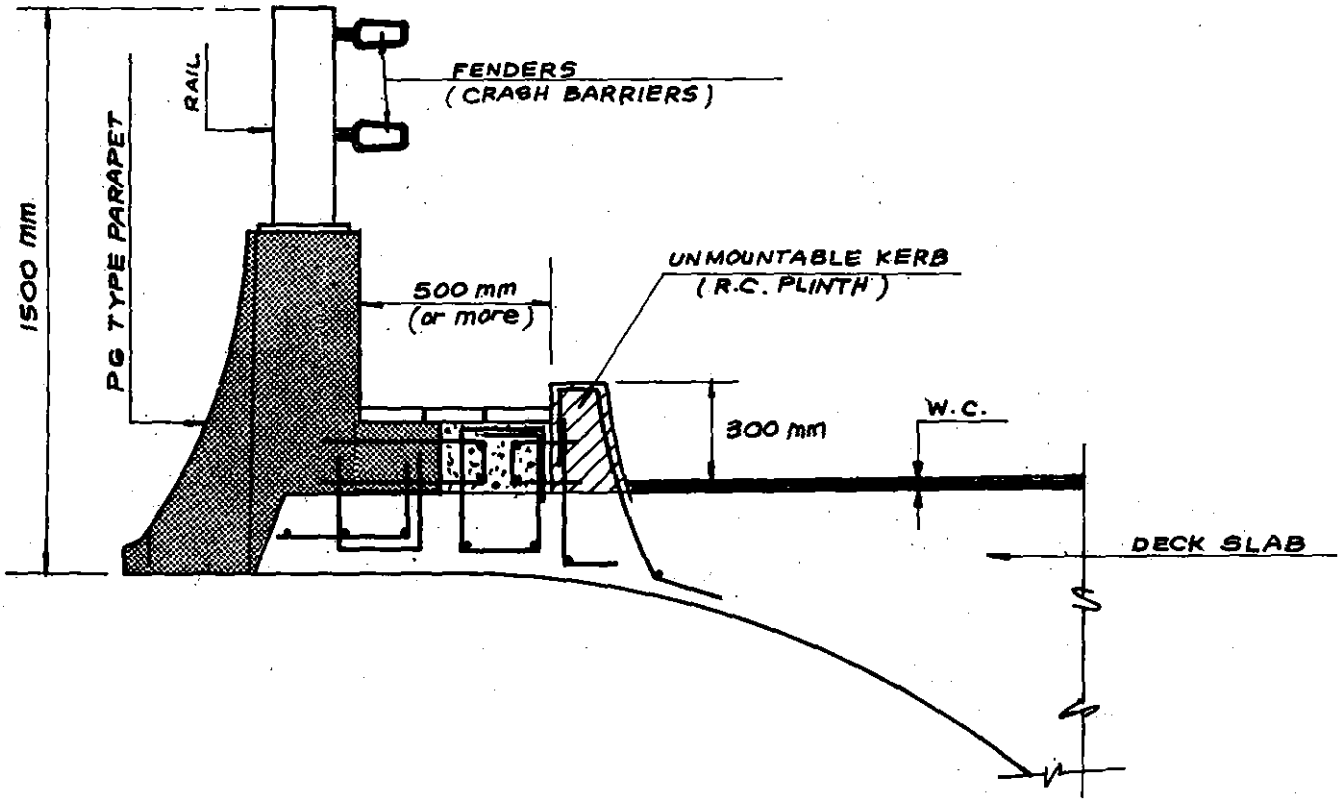


Fig. 33.8

- structural continuity of the longitudinal members
- strength of the supporting posts under transverse loading
- strength of supporting posts under longitudinal loading
- strength and descriptions of post-fixings
- details of fixing longitudinal members to supporting posts (strength criteria)

The steel design is essentially based on plastic approach.

Requirements for Reinforced Concrete Parapets and Plinths

Here too reference may be made to the above referred BE5 (or its latest version) on which the following are based:

(a) **Minimum Concrete Strength**

The concrete should have a minimum 28-day works 'cube' strength of 30 N/mm^2

(b) **Design Requirements for Reinforced Concrete In-situ and Precast Parapets (Groups P1, P2, P4 and P5)**

The 'vehicle' and 'vehicle-pedestrian' parapets shall be of the form shown in Fig. 33.9, except that where the permitted traffic speed does not exceed 48 km/hr the front face of the parapet may be vertical. For P4 parapets the front face of the parapets shall always be vertical.

Generally the shaping of the top of the parapet should be as shown in Fig. 33.9 to prevent anybody walking on top of these walls, but for parapets on bridges carrying motorways, or roads to motorway standards, from which pedestrians are excluded by order, such shaping is not essential.

For 'vehicle' and 'vehicle-pedestrian' parapets the minimum length of each of the 'precast' reinforced concrete panels shall be 1200 mm.

For the design requirements for these parapets see Table 33.2.

(c) **Design Requirements for Reinforced Concrete In-situ and Precast Plinths (Groups P1, P2, P4 and P5)**

The requirements given in (iii) and (iv) below apply only to 'plinths' for 'vehicle' and 'vehicle-pedestrian' parapets. Reference should be made to BE5 for the Group P4 plinth requirements.

(i) **Height**

A reinforced concrete plinth shall always be provided under a metal parapet. The plinth height for 'vehicle' and 'vehicle-pedestrian' parapets, measured above the adjoining paved surface, shall either be (1) not less than 50 mm and not greater than 100 mm, or (2) shall be at least 800 mm. Where the permitted traffic speed does not exceed 80 km/hr the above limitation (2) on plinth height may be reduced to 700 mm [see (iii)].

(ii) **Strength**

The strength of the plinth shall be sufficient to withstand the moment and shear developed by the fixing of the parapet posts.

(iii) **Plinth as an Effective Longitudinal Member**

A plinth may be considered as an effective longitudinal member provided its height is at least 700 mm above the adjoining paved surface, and that it can support a horizontal transverse force of "F/n" kN, applied as a point load to its top edge, in addition to the forces induced by the posts (F being the appropriate force given in Table II in BE5 and n the number of effective longitudinal members). If the derived ultimate moments of resistance at any point for the plinth are less than the

Table 33.2 *Design Requirements for Reinforced Concrete Parapets (P1, P2, P4 and P5)*

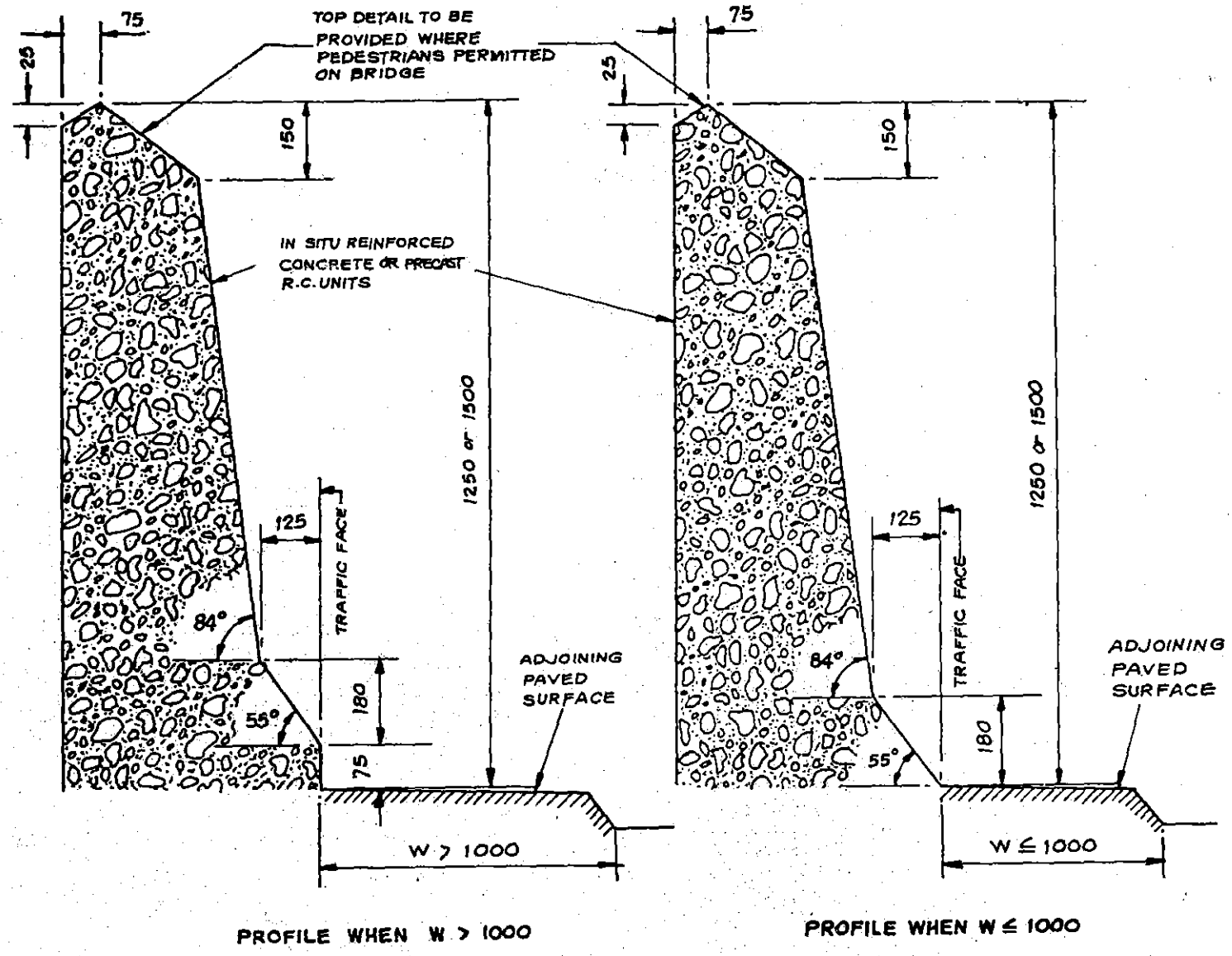
Item No.	Requirement	Parapet Group [see Note (iii)]	
		P1	P2
1.	Minimum ultimate moment of resistance, at the base of wall [see Note (i)] for bending in the vertical plane, with reinforcement adjacent to the traffic face [see Note (ii)].	25 kNm/m	12.5 kNm/m
2.	Minimum ultimate moment of resistance for bending in the horizontal plane, with reinforcement adjacent to outer face [see Note (ii)].	12.5 kNm/m	6.25 kNm/m
3.	Minimum ultimate moment of resistance of anchorages at the base of a precast reinforced concrete panel.	37.5 kNm/m	18.7 kNm/m
4.	Minimum ultimate transverse shear resistance at vertical joints between precast panels, or at vertical joints made between lengths of in-situ parapet.	65.5 kN/m of joint	32.5 kN/m of joint
5.	Minimum thickness of RC wall.	180 mm	150 mm

NOTES:

(i) The base of wall refers to horizontal sections of the parapet within 300 mm above the adjoining paved surface level. The minimum ultimate moment of resistance shall reduce linearly from the 'base of wall'' value to zero at top of the parapet.

(ii) In addition to the main reinforcement, in items 1 and 2 above, distribution steel equal to 50% of the main reinforcement shall be provided in the respective faces.

(iii) For Group P4 (pedestrian parapets) and Group P5 (parapets for bridges over railways) see BE5 for details.



NOTES:

1. W is the width of adjoining paved surface between Kerb and Traffic-face of Parapet.
2. Dimensions are in mm.
3. Sketches not to scale.

Fig. 33.9 *Shaped Concrete Parapets (see item 5 in Tables 33.2 and 33.3 ahead for wall thickness).*

Table 33.3 *Design Requirements for High-Containment In-situ Reinforced Concrete Parapets (P6)*

Item No.	Requirement	
1.	Minimum ultimate moment of resistance at base of wall [see Note (i)] for bending in the vertical plane with reinforcement adjacent to traffic-face [see Note (ii)]	
	• For end sections [see Note (iii)]	165 kNm/m
	• For intermediate sections [see Note (iii)].	125 kNm/m
2.	Minimum ultimate moment of resistance for bending in the horizontal plane, with reinforcement adjacent to outer face [see Note (ii)]	62.5 kNm/m
3.	Minimum ultimate horizontal transverse shear resistance	220 kN/m
4.	Minimum ultimate transverse shear load to be transferred at vertical joints made between lengths of in-situ parapets [see Note (iv)]	165 kN
5.	*Minimum thickness of RC wall*: at top	250 mm
	: at adjoining paved surface level	430 mm

NOTES:

(i) The base of wall refers to horizontal sections of the parapet within 300 mm above or below the adjoining paved surface level. The minimum ultimate moment of resistance shall reduce linearly from the 'base of wall' value to zero at top of the parapet.

(ii) In addition to the main reinforcement in items 1 and 2 above, distribution steel equal to 50% of the main reinforcement shall be provided in the respective faces.

(iii) For design purposes the parapet shall be divided into 'end-sections' extending a distance not greater than 3.0 m from ends of the parapet and 'intermediate-sections' extending along the remainder of the parapet.

(iv) The minimum ultimate transverse shear load may be reduced to 50 kN if the 3.0 m longitudinal lengths adjacent to and on either side of the joint are designed as 'end-sections'. The shear load shall be assumed to be uniformly distributed over the joint length.

appropriate moments given in Table 33.2 the values as specified in the table shall be used.

(iv) **Precast Sections**

Where a plinth is built of precast concrete sections, they shall be not less than 1200 mm long. The fixing moment of the section of the plinth to the deck shall be not less than the holding down moment of the post fixing, plus 1-1/2 times the moment induced by F/n where appropriate. If the derived ultimate fixing moment is less than the moment given in item 3 of Table 33.2, the value as specified in the table shall be used.

When a precast concrete plinth acts as an effective longitudinal member, the vertical joints shall be

Table 33.4 *Additional Design Requirements for High-Containment In-situ Reinforced Concrete Parapets (P6)*

Item No.	Additional Requirements	
1.(a)	Minimum ultimate moment of resistance, at level d (metres) below the adjoining paved surface level, for bending in the vertical plane with reinforcement adjacent to traffic face—	
	• For end sections } [see Note (i)]	$165 + 48.(d - 0.3)$ kNm/m
	• For intermediate sections } [see Note (i)]	$125 + 12.(d - 0.3)$ kNm/m

NOTE:

(i) For design purposes the parapet shall be divided into 'end-sections' extending a distance not greater than $3.0 + 3 \cdot (d - 0.3)$ metres from ends of the parapet and 'intermediate-sections' extending along the remainder of the parapet.

Table 33.5 *Design Requirements for High-Containment Precast Reinforced Concrete Parapets (P6)*

Loading No.	Requirement
	The minimum ultimate moment of resistance and horizontal transverse shear resistance of the precast panels [see Note (i)] shall be sufficient to withstand the more severe of the following ultimate design forces:
1.	A transverse force of 330 kN applied to each panel over a horizontal length of 1.5 m at a height of 0.85 m above the adjoining paved surface.
2.	A transverse force of 330 kN applied over a horizontal length of 3.0 m at the top of the parapet.

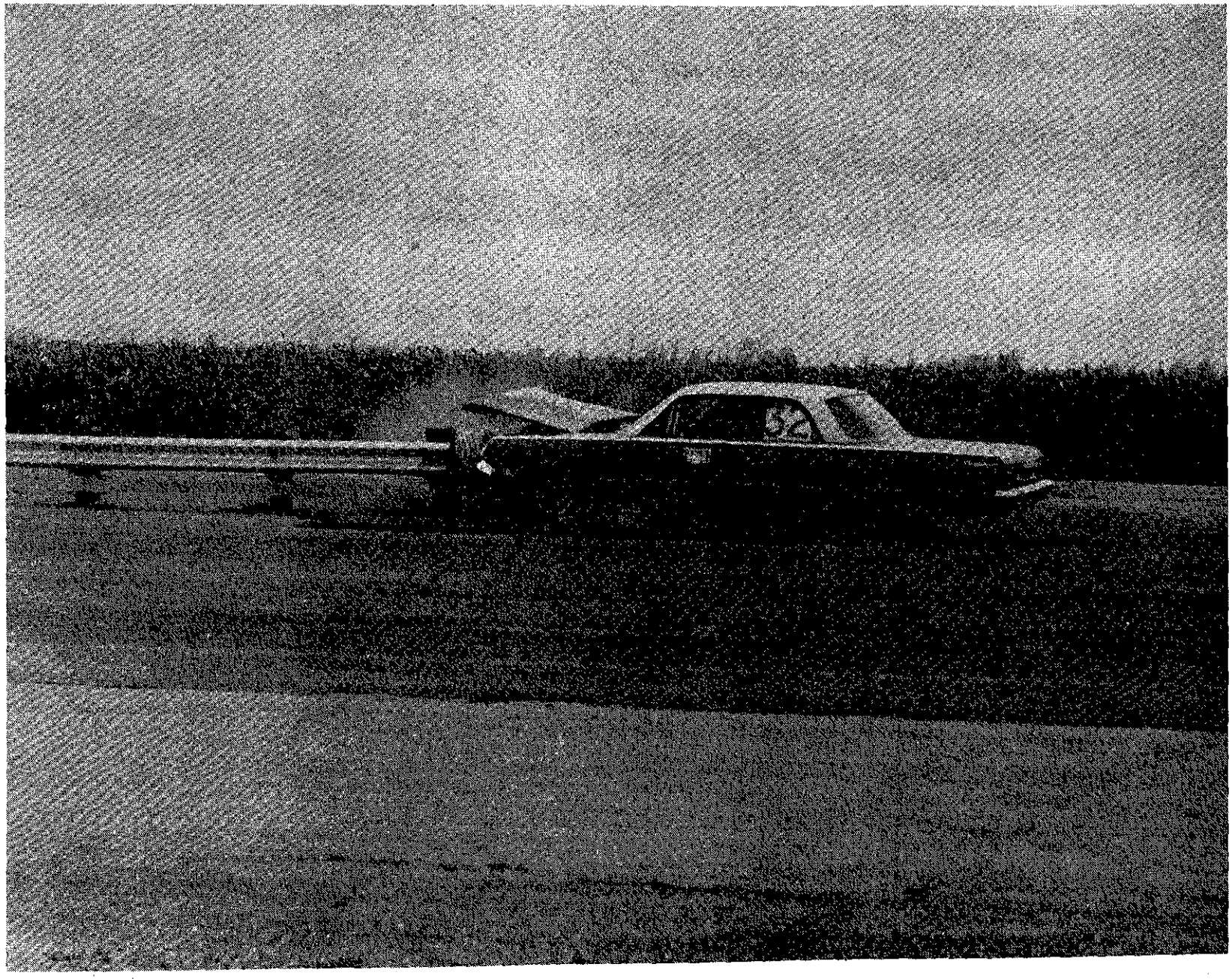
NOTES:

(i) If the ultimate moment and shear resistances derived from Table 33.5 are less than the values given in Tables 33.3 and 33.4 [items 1 and 1(a) end-sections, and item 2 and item 3], Tables 33.3 and 33.4 values shall be used.

(ii) The minimum ultimate moment of resistance and horizontal transverse shear resistance of the precast panel anchorages shall be 1.5 times the values calculated at the base of the precast panels.

(iii) The minimum ultimate transverse shear resistance at vertical joints between precast panels shall be 110 kN. The effect of shear transfer between adjacent panels shall be ignored when applying the above requirements.

(iv) The requirement for minimum thickness given under item 5 of Table 33.3 shall apply to precast panels also. The minimum length of panels shall be 1500 mm.



Vehicle parapet (crash-barrier) impact test

provided with shear keys or dowels which will transfer the appropriate load given in item 4 of Table 33.2.

(d) Design Requirements for Reinforced Concrete High-containment In-situ and Precast Parapets (Group P6)

(a) **In-situ Reinforced Concrete Parapets (P6)**
The design requirements for these parapets are given in Table 33.3. Parapets which are joined to the supporting structure at a level deeper than 300 mm below the adjoining paved surface level, shall, in addition, comply with the requirements of Table 33.4.

(b) **Precast Reinforced Concrete Parapets (P6)**
The design requirements for these parapets are given in Table 33.5.

Requirements for Dynamic Testing of Prototype Vehicle-Parapet (Reference BE5)

1. **Testing Arrangements**
The testing facilities and method shall be capable of demonstrating that the prototype parapet meets the appropriate 'containment standard' and other criteria given below.

The prototype parapet shall be not less than 30 metres long. Unless otherwise agreed, it shall be identical in every detail, including the capacity of its post fixings, to the production parapet which it represents. The impact vehicle shall be a roadworthy production model of approved type. The steering should be in good condition and the bodywork reasonably free from corrosion. Howsoever the vehicle is guided to the impact point, it shall be unrestrained at and after the time of impact.

The angle of impact shall always be 20° to the line of the parapet.

2. **Containment Standards**

P1 (application as described in Table 33.1)	1.5 t at 113 km/hr.
P2 (application as described in Table 33.1)	1.5 t at 80 km/hr.

3. **Criteria for Acceptance**

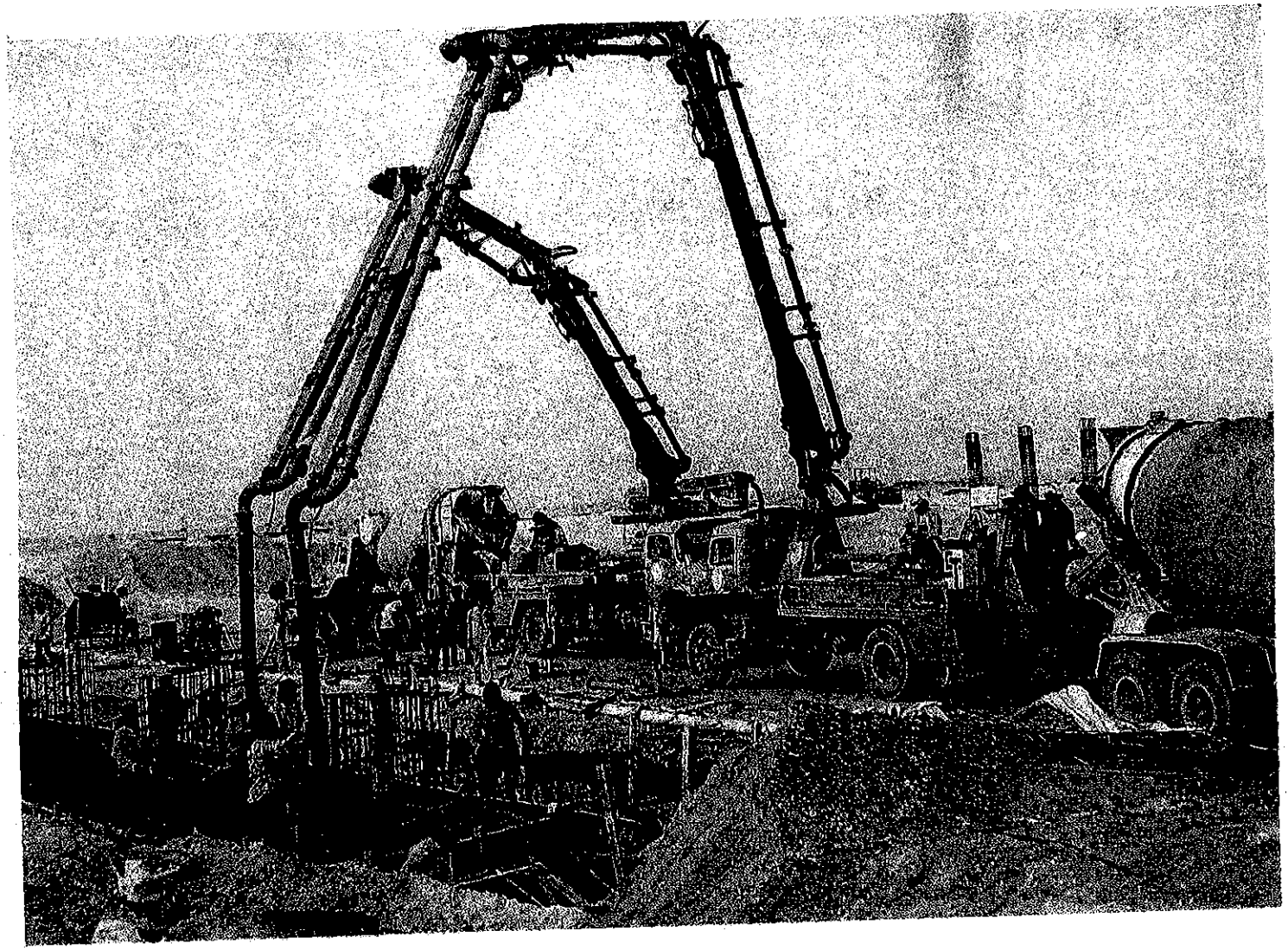
(i) The vehicle shall be contained.
(ii) No part of the parapet shall become detached.
(iii) The transverse displacement of the parapet shall not exceed 800 mm.
(iv) Examination of the concrete base under the baseplate of a metal parapet after testing should show no visible signs of damage or cracking.
(v) No part of the vehicle shall be arrested abruptly by impact with a post.
(vi) The test vehicle shall not roll over.
(vii) The car shall preferably be steered back to the line of the parapet but if steerback is not achieved, it shall be redirected at not more than 12° to the line of the parapet.

CHAPTER 34

Construction Techniques

This subject has already been dealt with in quite some detail in the author's other book: "*Concrete Bridge Practice: Construction, Maintenance and Rehabilitation*" to which reference may be made. Given below for ready reference are the heads under which this subject has been discussed in it.

- **CONSTRUCTION TECHNIQUES**
  - **Techniques for Constructing 'Substructure':**
    - for footings, piles and caissons
    - stability of floating caissons
    - floating the well curb from shore
    - cofferdams (false steining)
    - floating pile-driving plant
    - support by ground-anchors
    - site preparation
    - excavation methods
    - stability of slopes (in different strata)
    - some unusual pier construction techniques (examples of actual bridges)
  - **Construction of Reinforced-Earth Walls and Abutments**
  - **Techniques for Constructing 'Superstructure':**
    - General
      - Crane erection, Jacking or counter-weighing, Cantilever erection, metal erection—trusses, false-work piers, cable erection.
    - mobile crane (floating-type, gantry-type, crawler-type)
    - use of service platform, temporary trestle, erection span with launching-nose,
    - casting beams vertically and then pivotting down
    - casting beams along embankment and then swivelling them into position
    - free cantilever method for segmental Construction:
      - various details
      - cast in-situ vs. precast segmental methods
    - span by span construction method
    - progressive placement method
    - incremental-launching (push-out) method
    - loads applied to segmentally constructed deck during construction.
    - two additional techniques for constructing continuous p.s.c. decks on tall piers.
    - temporary structures (i.e. Construction Apparatus):
      - general
      - casting yards (various details)
      - launching girders and trusses (various methods)
      - movement of precast elements (various phases)
      - movement of erection equipment
      - working of a typical Launching Girder for launching precast p.s.c. beams (stage-wise).
      - crane systems (Derrick Trolleys, Guy Derrick Cranes, Tower Cranes, Portal Gantries, etc.)
    - manufacture of precast segments:
      - long-line casting,
      - short-line casting,
      - handling, temporary assembly,
    - placing precast segments:
      - independent lifting equipment
      - beam and winch method
      - launching girder method
    - notable construction details of some major bridges:
      - two p.s.c. bowstring decks (U.K. and France)
      - open spandrel arches for a bridge cast vertically (South Africa)
      - Vijle Fjord Bridge (Segmental)
      - Houston Ship Channel Bridge
      - Medway Bridge (U.K.)
      - Paris Belt-way Bridge (upstream)
      - Oleron Viaduct (France)
      - Linn Cove Viaduct (U.S.A.)
      - Eastern Scheldt Bridge (Rotterdam)
      - Pasco-Kennewick Bridge (U.S.A)
      - Gladsville Arch Bridge (Australia)
      - Pontchartrain Bridge
      - Caracas Arches (Venezuela)



Laying foundation using pumped concrete

CHAPTER 35

Construction Considerations

This subject has already been dealt with in quite some detail in the author's other book: *Concrete Bridge Practice: Construction, Maintenance and Rehabilitation* to which reference may be made. Given below for ready reference are the heads under which this subject has been discussed in it.

CONSTRUCTION CONSIDERATIONS

- General
- Excavation and backfill:
  - — general
  - — preparation of channel
  - — depth of footings
  - — preparation of foundations (in various strata)
  - — sheeting, cofferdams, open cribs
  - — Caisson (well) foundations:
    - various types
    - important details
    - problems involved
    - temporary islands
    - sinking through various strata
    - sand blowing
    - artesian conditions
    - counteracting tilts and shifts
    - tackling obstacles met with in well-sinking
  - — backfilling, placing, compacting
  - — drainage
- Foundation Piles:
  - — general
  - — equipment
  - — construction and driving procedures (various details)
- Concrete . . . production and control:
  - — various details regarding the Aggregates, cement, water, admixtures, proportioning and finalising a mix and its workability; Batching, mixing, delivering, placing, vibrating and finishing of concrete.
- Control of Surface Evaporation and its Importance
- Joints (Construction, Expansion, and Cold)
- Deposition of concrete under water:
  - — various details pertinent to Tremie concreting
- Inspection and Testing 'for' Concrete:
  - — for batching, mixing, delivery, etc.
  - — for sampling and testing of materials
- Curing of Concrete:
  - — various details about membrane-curing, watercuring and steam-curing.
- Concreting in adverse weather conditions:
  - — in cold weather
  - — in hot weather
- Concrete exposed to sea water
- Falsework
- Forms (Shutters)
- Reinforcing Steel (various considerations)
- Falsework for structural steel work
- Bearings and Anchors under steel deck (relevant to composite construction)
- Structural Steel Expansion Joints and Roller Bearings
- Assembly and alignment of steel decks (relevant to composite construction)
- Connections in Steel Decks (relevant to composite construction) — various details.
- Painting of Structural Steel (relevant to composite construction).
- Inspection of Structural Steel (relevant to composite construction).
- Prestressed Concrete:
  - — various details more pertinent to prestressed concrete in addition to those given earlier for concrete.
- Prestressing:
  - — proprietary systems
  - — pretensioning components (considerations and various details)
  - — post-tensioning components (considerations and various details)
  - — Prestressing reinforcement (various considerations)
- Placement of reinforcement, inserts and bearings prior to casting prestressed concrete
  - — various details and considerations

- Prestressing the Structure:
  - some important considerations and precautions
- Precast-Segment Manufacture and Erection:
  - manufacture
  - erection of segments
- Epoxy bonding agents for precast segments
- Inspection of precast segment-joining procedure
- Epoxy bonding agent TESTS:
  - details of 7 recommended tests
- Grouting of cable ducts:
  - various considerations and details
- Load testing a Bridge Superstructure:
  - purpose
  - 'Test Load' and Computation of deflections (theoretically)
  - Load application and Testing
  - Assessment of Results
  - Precautions
  - Some Important Considerations
- Prestressed Rock Anchors:
  - various considerations and requirements
- Application of Sprayed Concrete and Mortar
  - various considerations
- Common problems associated with concrete:
  - workability
  - excessive salts in aggregates
  - cracking in finished concrete
  - settlement at abutments
  - settlement around culverts
  - settlement of falsework
- Considerations for making good concrete in Hot/Windy/Marine atmospheric conditions.
- Care for Structural Concrete:
  - general
  - corrosion of reinforcement
  - reaction between aggregate and cement
  - resistance to various chemicals/substances
  - resistance to sulphate attack
  - resistance to soft moorland and peaty waters
  - resistance to sewage
  - resistance to erosion and abrasion
  - resistance to fire
- Bridge Construction Execution: General preparation, mobilisation, and stores.
  - various items based on construction considerations.
- SEQUENCE of Road Construction, Typical EQUIPMENTS used, and their RATED CAPACITIES

CHAPTER 36

Cantilever Construction of Bridges

36.1 A MODERN CONSTRUCTION TECHNIQUE

It is no longer necessary to emphasize the advantages of prestressed concrete, the birth of which is linked with the name of Freyssinet. A modern bridge is a prestressed concrete bridge and among prestressed bridges those built by the cantilever method demonstrate the latest refinements of this construction technique.

Deep valleys are crossed without high and expensive scaffolding and rivers without the risk of having the scaffolding washed away by a flood.

36.2 VARIOUS DETAILS*

The Two Ways of Building in Cantilever

Cast In-situ Construction

The bridge is cast in-situ with sections 3 to 6 m long cantilevering symmetrically on both sides of the pier. Formwork is supported by a steel truss attached to the completed part of the bridge and which moves from one completed section to the next.

The sequence of operations consists of:

- Setting up and adjusting the steel truss and shuttering
- Placing reinforcement and prestressing ducts
- Concreting
- Threading the cables to be anchored in the section and stressing
- Releasing the shuttering and moving on to the next section

Normal construction time is one section nearly per week per formwork unit. This time can be reduced by using heat curing.

Construction with Precast Segments

Bridge segments are precast in a yard with special forms using the immediately preceding unit as end boxing. A perfectly matching joint is thus obtained.

Precast segments are placed by means of a mobile launching girder or, when access under the bridge is possible with barges or trucks by means of a crane or a mobile hoist located at the extremity of the cantilevers.

Joints are glued with epoxy resin. Segments are held temporarily by ties and then prestressed by cables threaded through empty ducts. The speed of placing is 4 to 8 segments per day.

The Bridge Type

Usually the bridges have a continuous superstructure which has now practically replaced the midspan hinge system used earlier. Hinges are complicated and, because of creep, the riding surfaces often develop a severe break of the profile at this point.

In continuous structures, expansion joints may be spaced very far apart. Continuous bridges up to 1000 m long have been carried out without intermediate joints and this is perhaps not a limit provided the bearing and pier design is correctly integrated into the total design of the structure. Free longitudinal movement of the bridge due to creep and temperature change is allowed for by placing the structure on elastomer or sliding (teflon) bearings. Pier flexibility can also be used to accommodate these movements and the superstructure fixed to the piers. In this case pier flexibility is obtained either by height or by using single or double thin plate walls thus reducing the flexural resistance.

Single span bridges with counterweight abutments have also been built.

A typical cross-section would be a 'box' girder of constant or variable depth.

Transverse prestressing of the top slab is not uncommon.

The Choice of Spans and the Construction System

The use of cast in-situ construction or precast segments depends on the size of the bridge and on the span lengths.

The field of application for bridges built in cantilever is very wide and covers mainly spans from 50 to 200 m. At present a new trend consists in using precast segments for smaller spans, say of about 35 m, for elevated roads or flyovers, when scaffolding must be avoided and existing road facilities maintained beneath the structure.

The cantilever method of construction is virtually the only one to be considered today for spans over 70 m, a field

* also see Chapter 18

in which it competes successfully with steel construction. For smaller spans it competes with other erection methods, in particular casting in-situ on falsework which is still economical if not too high.

The most economical span length when a free choice is possible depends on the cost of foundations and on the height of the piers. Experience shows that economical spans range from 70 to 110 m for average conditions of profile, piers and foundations. Superstructures of variable, depth are often used in this range, whereas constant depth is maintained for smaller spans.

For very large spans (over 110 m), since precast segments would be too heavy and, therefore, the equipment for handling them too expensive, the cast in-situ method of construction is generally used—unless the number of spans is large.

For conventional spans, the choice between casting in-situ and using precast segments depends essentially on the bridge surface area, due to the elevated cost of the precasting plant and placing facilities, as well as construction time. A figure of approximately 5000 m^2 can be given as a minimum for justifying precast construction. It must be noted that this cost is reduced when placing of segments can be carried out using standard available equipment (floating or truck canes) and when the weight of segments is small (shorter spans). Thus, this method has been used successfully for small spans of urban viaducts and flyovers.

The Desirable Prestressing System

For good detailing and easy construction, the prestressing system must present certain specific properties:

- Cables must be easily threaded into the ducts after completion of each pair of sections.
- Tendon force must not be too small, in order to avoid cable congestion at the top of the cross sections near the piers and not too large to avoid excessive stress concentrations near the anchorages.
- The void left in the duct around the cable must be large enough for good grouting.
- The most convenient final cable force depends on the size of the structure and should be chosen to respect the above criteria.

The frequently used cables are the 12 T 13 (12/0.5") strand cable and the 12 ϕ 8 mm wire cable (both Freyssinet) and similar sized cables of other prestressing systems like BBR, VSL, etc.

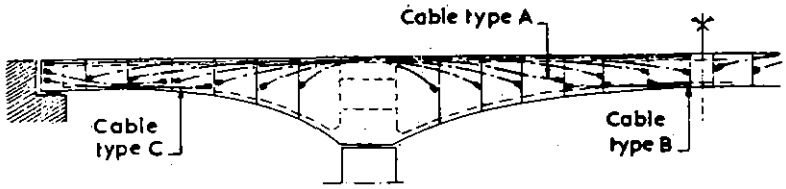
The Cost Advantage

The cost advantages of the cantilever construction technique have been demonstrated in successful tenders where this technique was in competition with other methods and other materials, in particular steel. When comparing with steel, maintenance costs have to be taken into account.

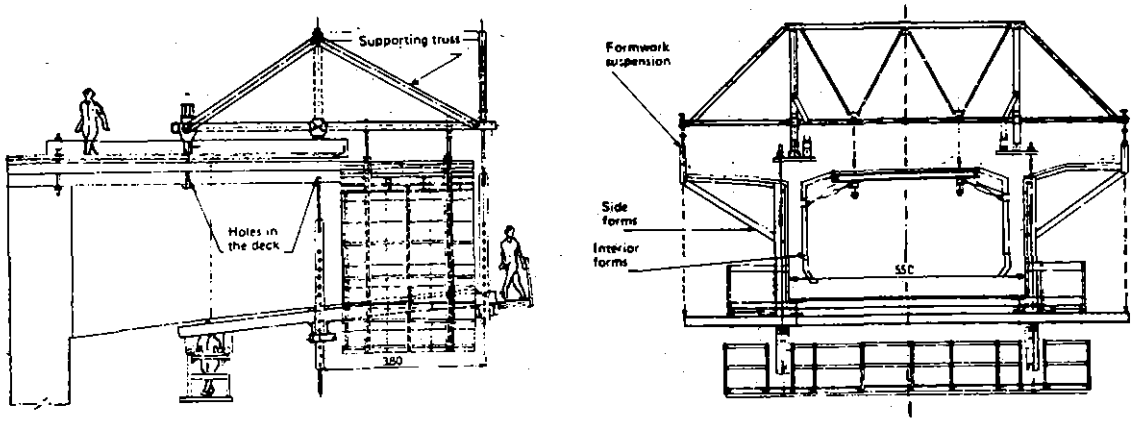
The rapid growth of this technique is the best proof of its value from the economic point of view.

Services Provided by Various Prestressing Agencies

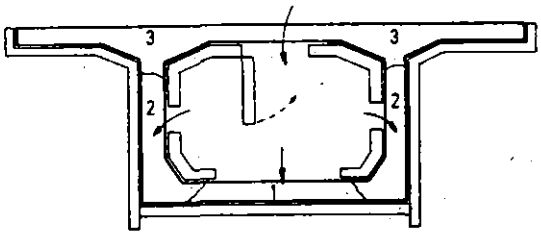
- Preliminary feasibility studies
- Technical assistance
- Supervision of design and construction
- Technical advice during design
- Computation using specially developed computer programs
- Supply of suitable prestressing material and equipment on a sale or hire basis
- Design of special equipment for cantilever construction: mobile falsework for cast-in-situ projects; precasting—cells, launching girders, etc . . . for precast segmental structures
- Supply of the above equipment including its commissioning
- Technical assistance on site including complete subcontracting of prestressing operations



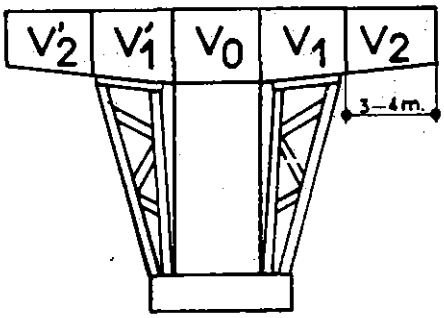
TYPICAL CABLE LAYOUT FOR CANTILEVER CONSTRUCTION



MOBILE FALSEWORK FOR CANTILEVER CONSTRUCTION



CONCRETING SEQUENCES FOR A TYPICAL SEGMENT



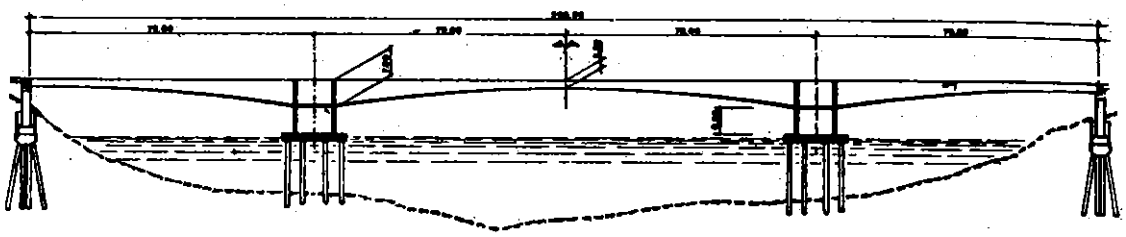
CASTING OF THE SEGMENTS NEAR A PIER

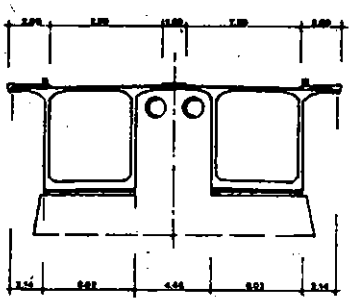
BRIDGE OVER THE SETUBAL LAGOON — ARGENTINA 1970

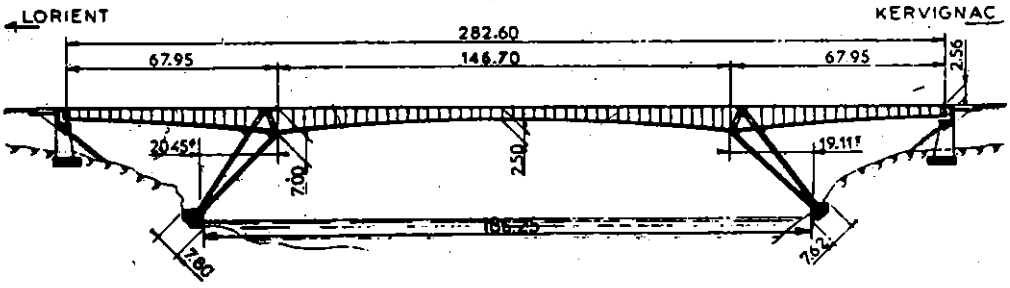
140 m main span fixed on twin wall piers.

Design — Europe Etudes

Contractor — Christiani and Nielsen



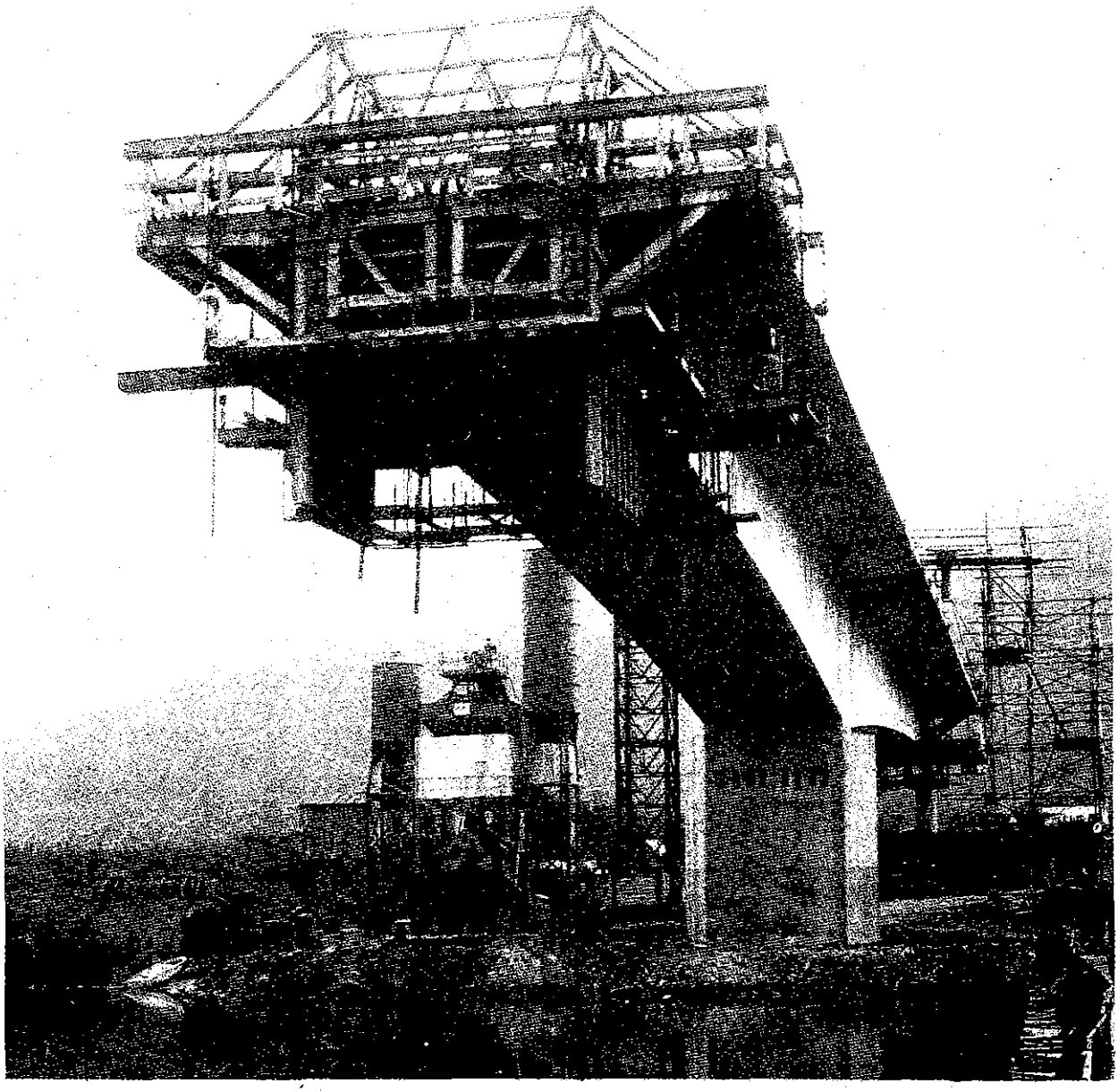




"BONHOMME" BRIDGE OVER THE BLAVET RIVER, MORBIHAN — FRANCE 1974

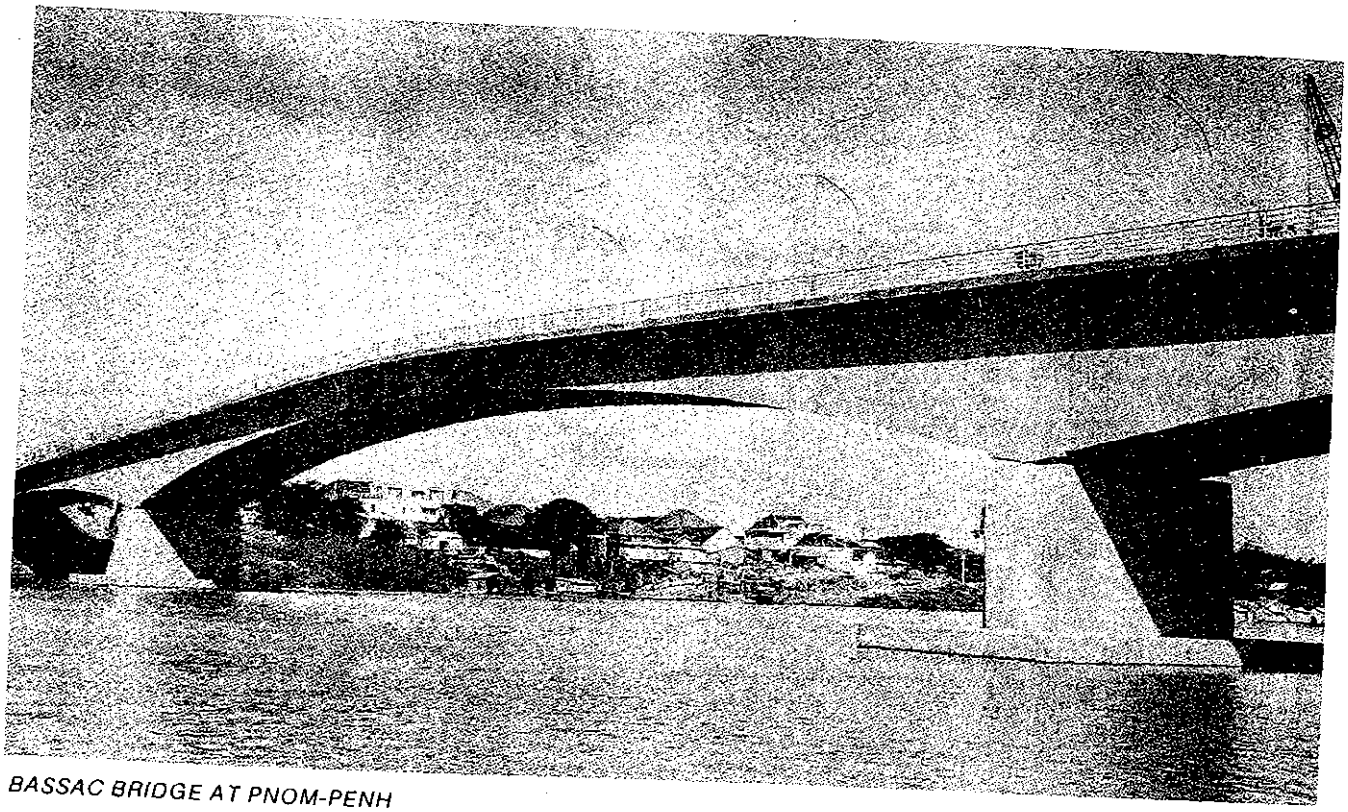
Racked legs of this 186m span portal frame bridge were built using steel tube falsework which served as temporary support during the cantilever construction. For adjusting the geometry of the bridge, flat jacks were placed under the legs and at mid span.

Design and Construction: Campenon Bernard



CAST IN-SITU CANTILEVER

BOUSSENS BRIDGE OVER THE GARONNE RIVER *FRANCE 1969*
Spans : 49 m. 96 m. 49 m.
Design : Europe Etudes
Contractor : J.C. Stribick et Fils



BASSAC BRIDGE AT PNOM-PENH

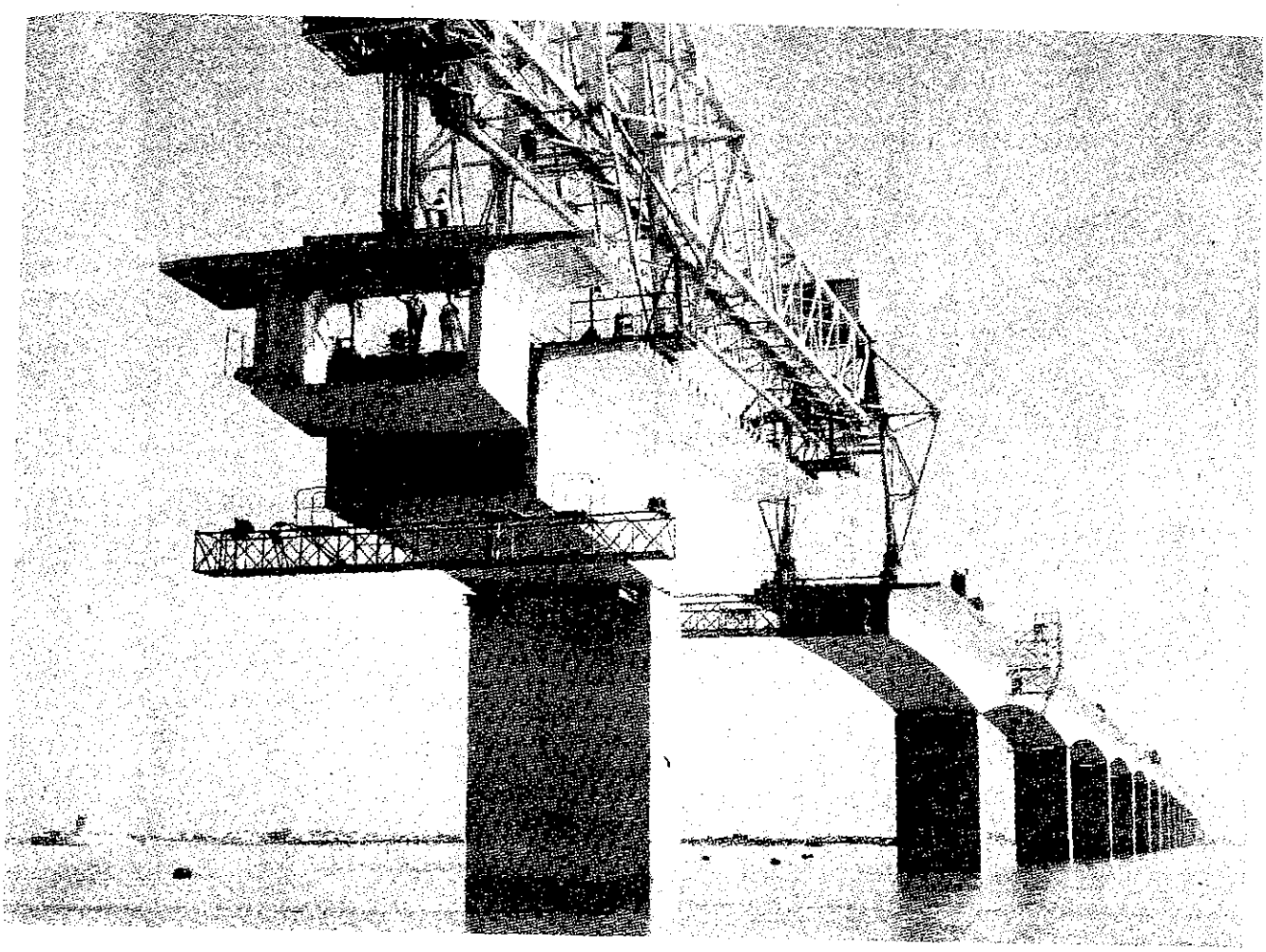
CAMBODIA 1966

Spans 75 m-120 m-75 m. Continuous single box section with depth varying from 2.55 m to 6.90 m.
Design and Construction : Compagnie Francaise de Dragages et de Travaux Publics.

CHANGHUNG BRIDGE OVER THE SHIUKULUANCHI RIVER

TAIWAN 1968

Single 120 m span hinged at the centre and fixed on counterweight type abutments.
Design : Japan Bridge and Structure Institute
Contractor : Kung Sing Eng. Corp.

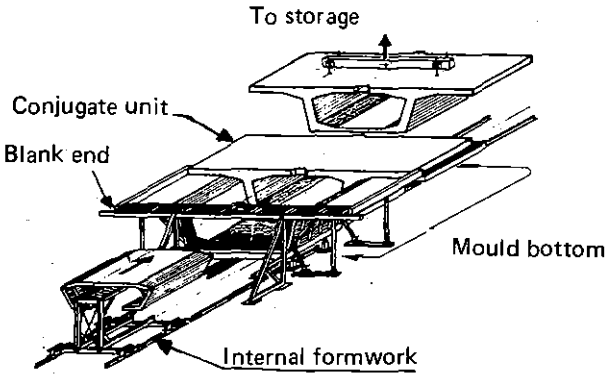


PRECAST SEGMENTAL CANTILEVER

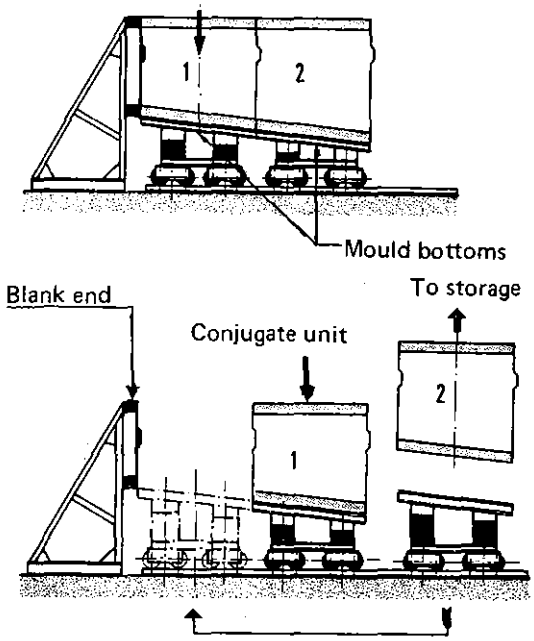
BRIDGE BETWEEN THE OLERON ISLAND AND THE CONTINENT *FRANCE 1966*

Longest bridge in France (2.862 m) 26 main spans of 79 m.
Design and Construction : Campenon Bernard

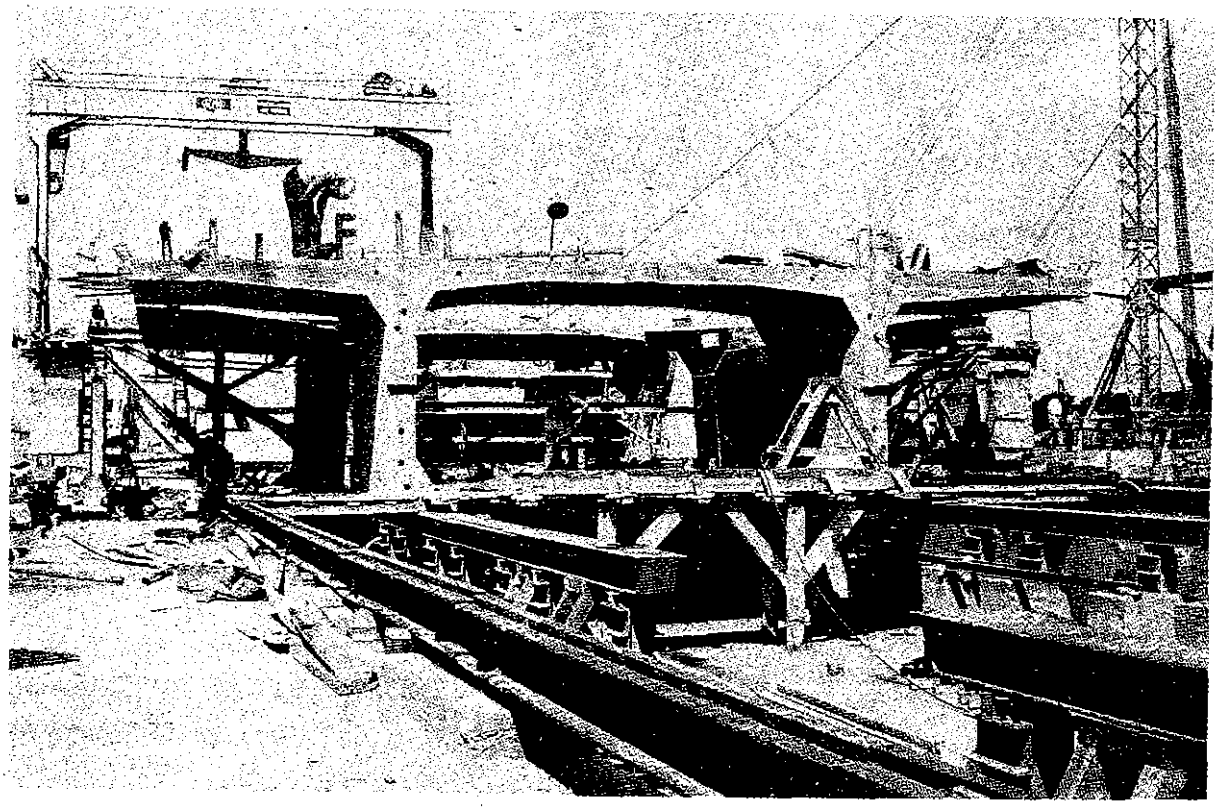
SCHEME OF A PRECASTING CELL

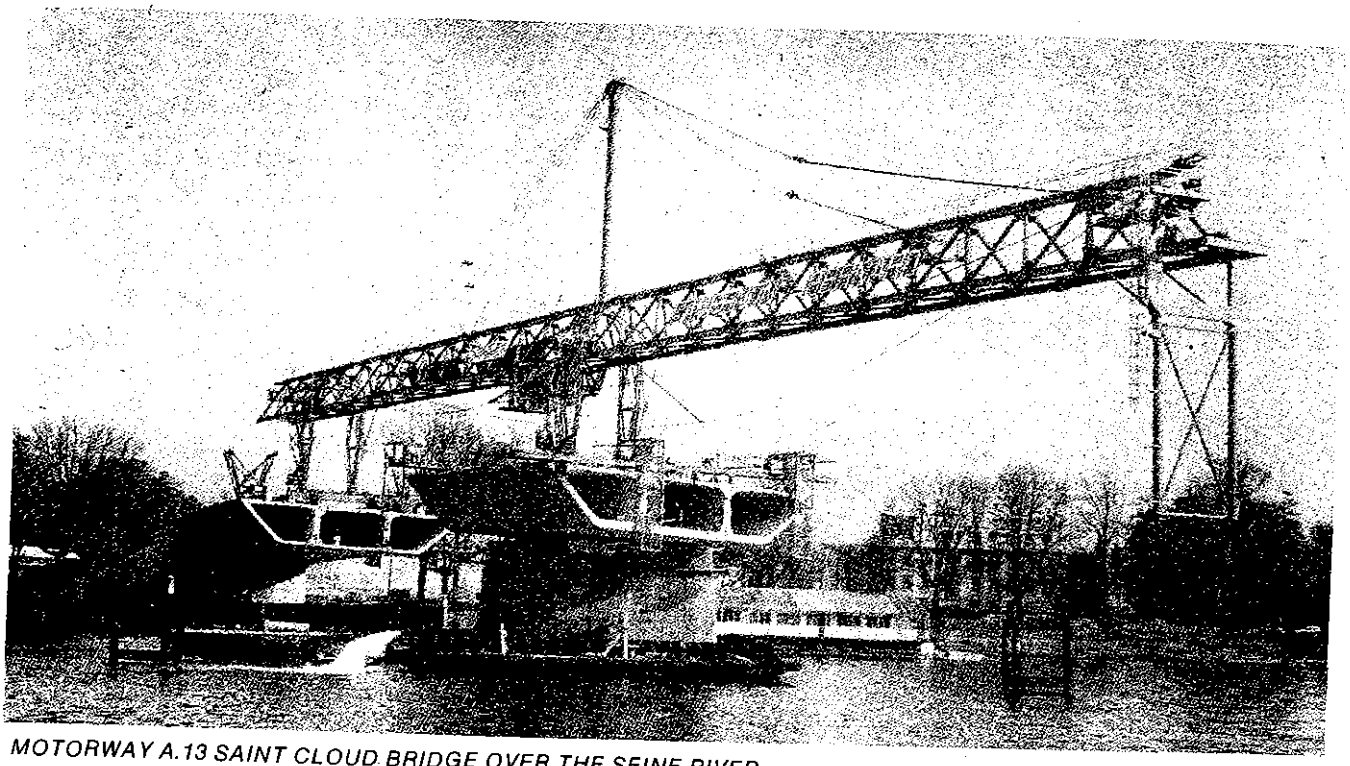


PERSPECTIVE VIEW



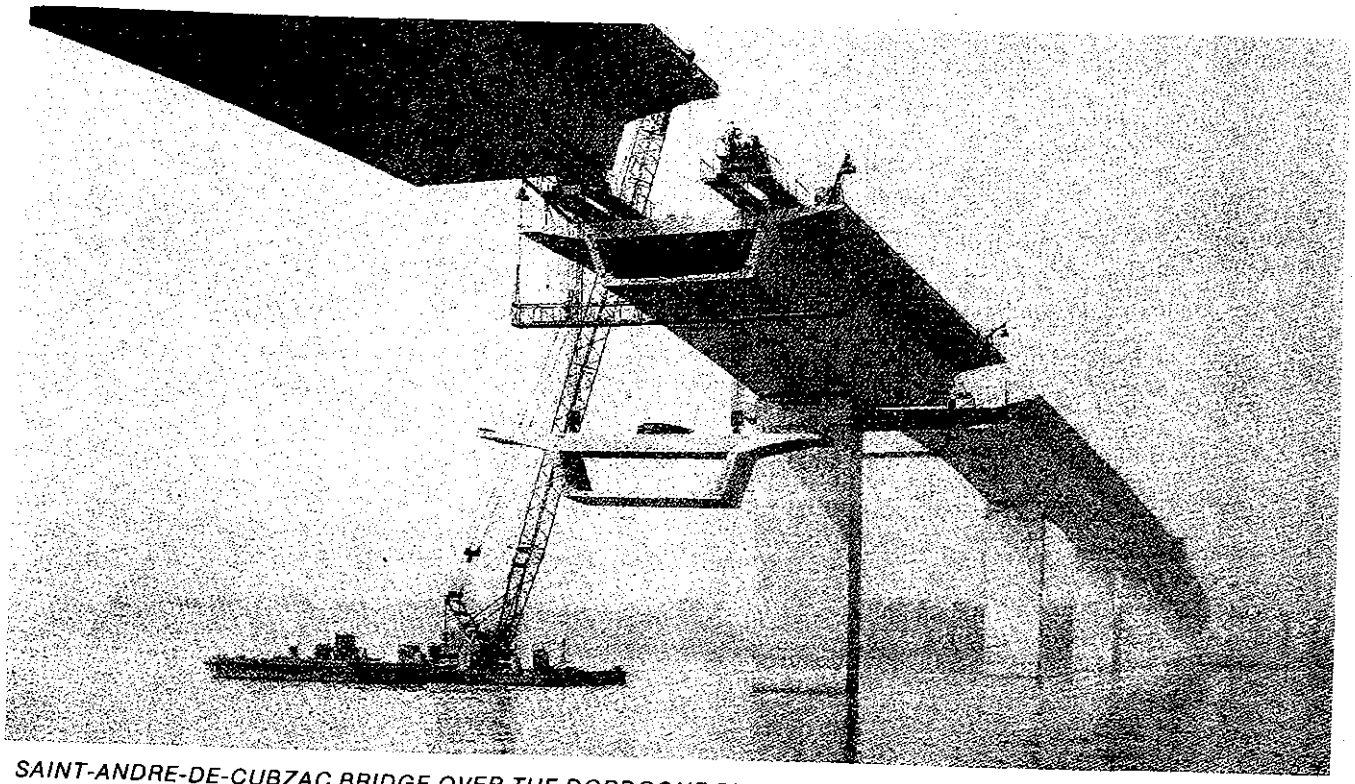
PRINCIPLE OF OPERATION





MOTORWAY A.13 SAINT CLOUD BRIDGE OVER THE SEINE RIVER *FRANCE 1973*

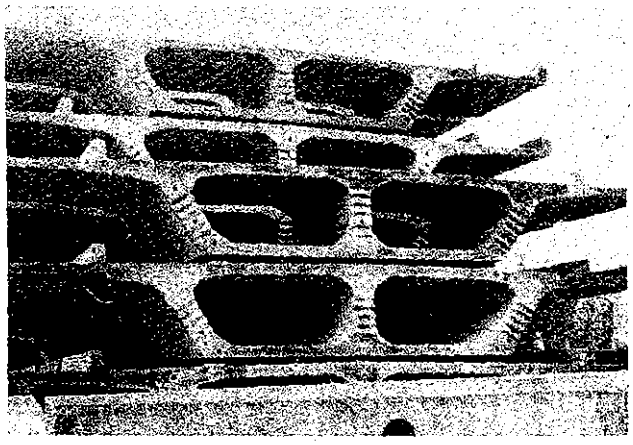
1,103 m long curved bridge and straight viaduct with spans of 46 m to 102 m. Exceptionally heavy precast segments (up to 130 tonnes), 20 m wide, placed by launching girder.
Design and Construction : Campenon Bernard



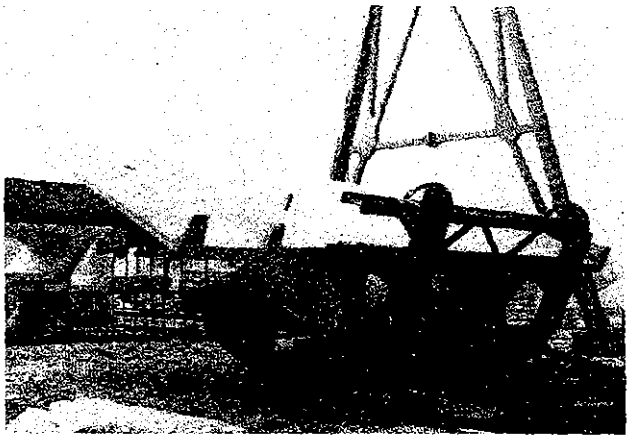
SAINT-ANDRE-DE-CUBZAC BRIDGE OVER THE DORDOGNE RIVER *FRANCE 1974*

1,278 m long structure with 37 m to 95 m spans. Segments placed by beam and winch lifting devices.
Design and Construction : Campenon Bernard

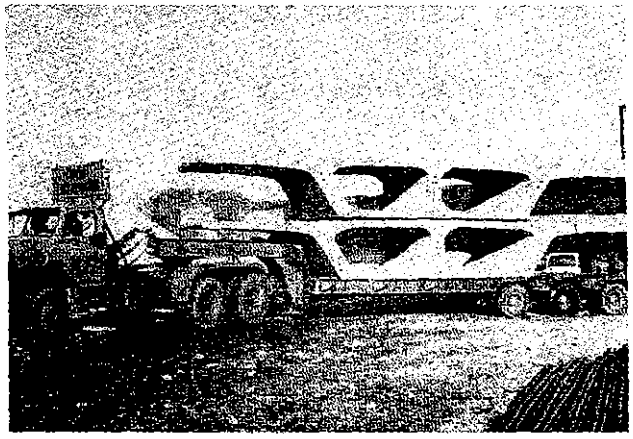
The construction of 146 overbridges for the Alpine Motorways project is an example of cantilever construction of small bridges using segments cast in a central plant, delivered on trucks, placed by crane and temporarily assembled by bars before threading the tendons. Total construction time is two weeks per bridge, foundations included !



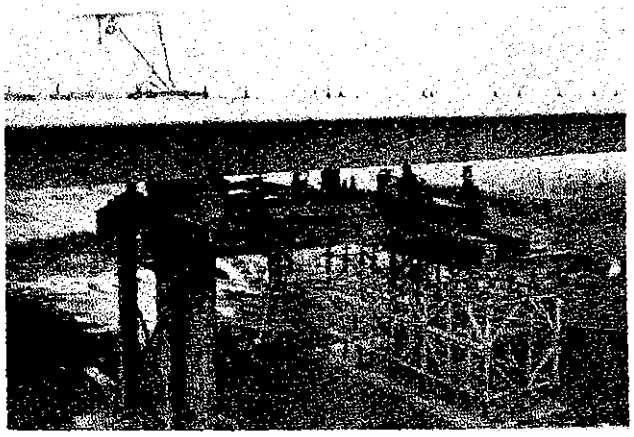
Storage of segments



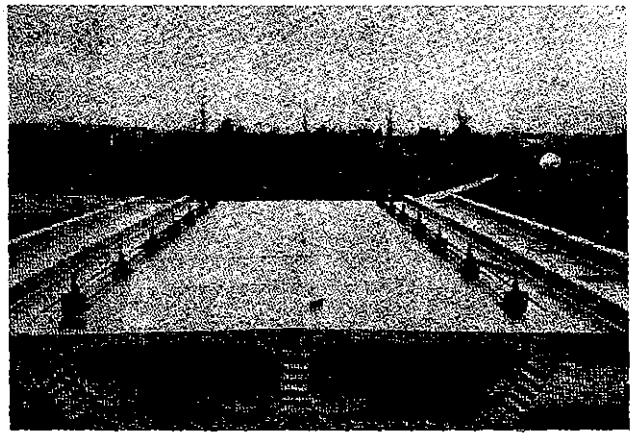
Turning a segment



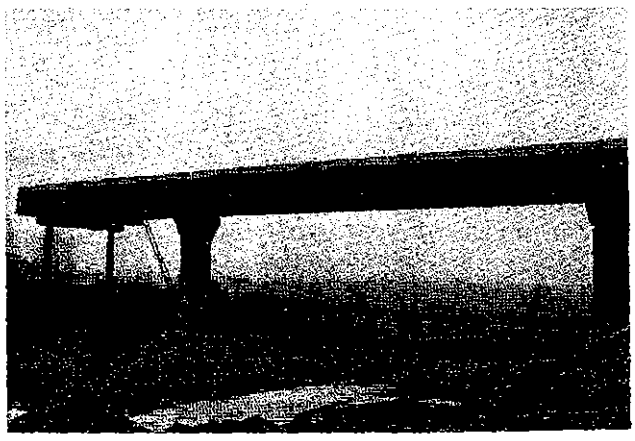
Transportation of segments



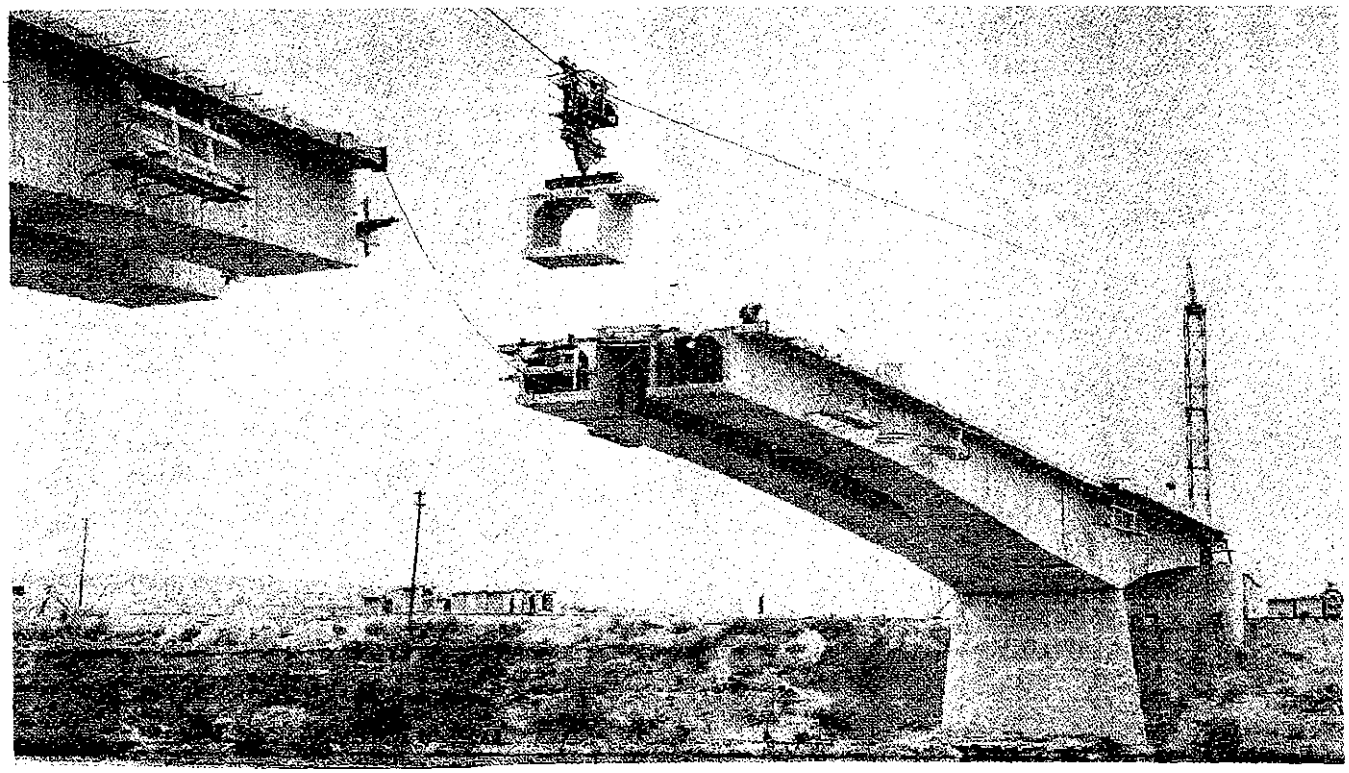
Cantilever support



Temporary cantilever prestress using bars

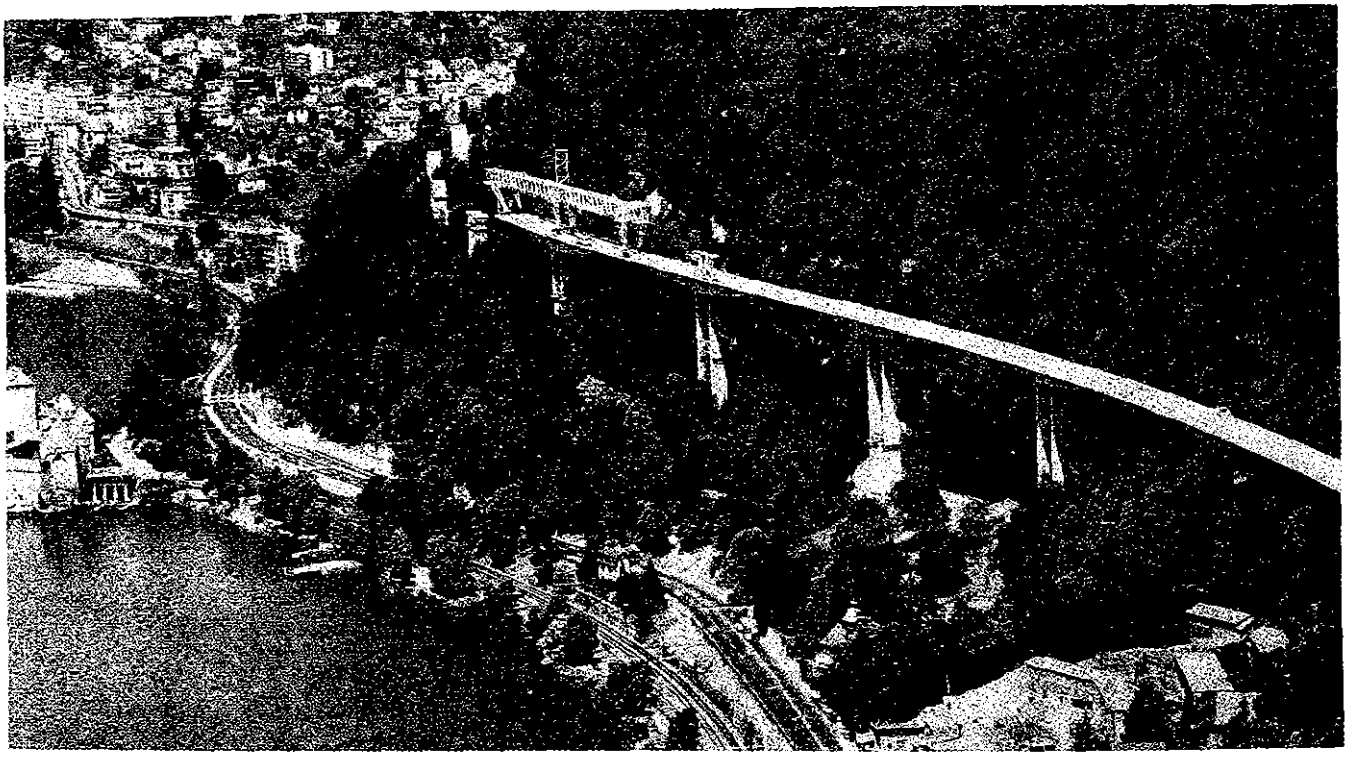


Completed overbridge

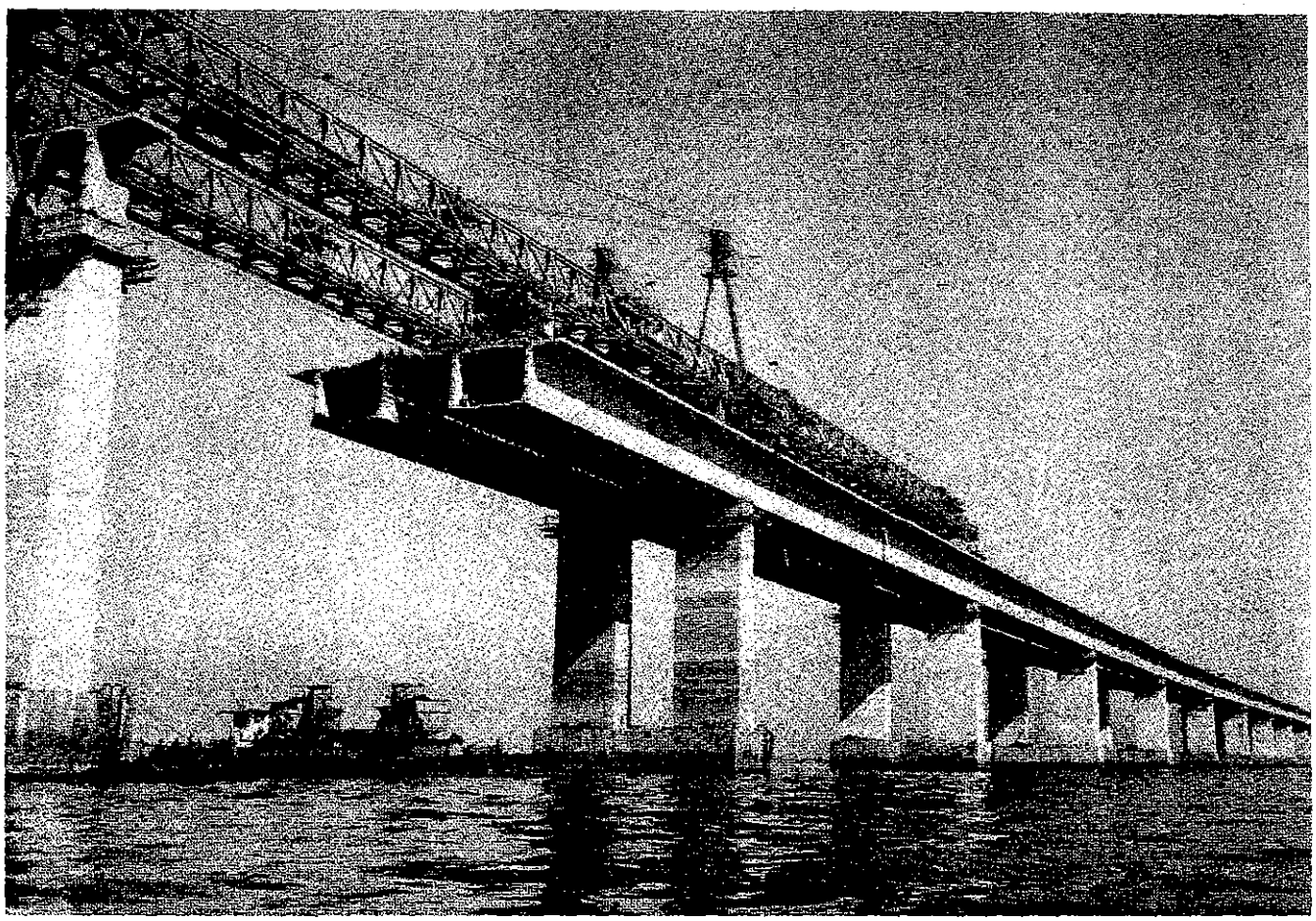


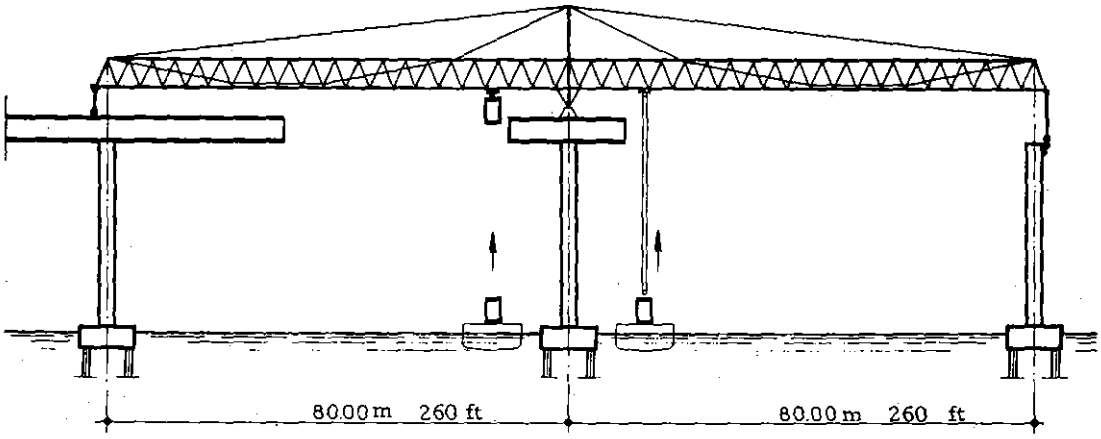
CASTEJON BRIDGE OVER THE RIO EBRO *SPAIN 1968*

Spans 25 m-101 m-50 m-3 × 28 m. 8 to 10 ton segments placed by cableway.
Design : Pr. C.F. Casado
Contractor : Agroman S.A.



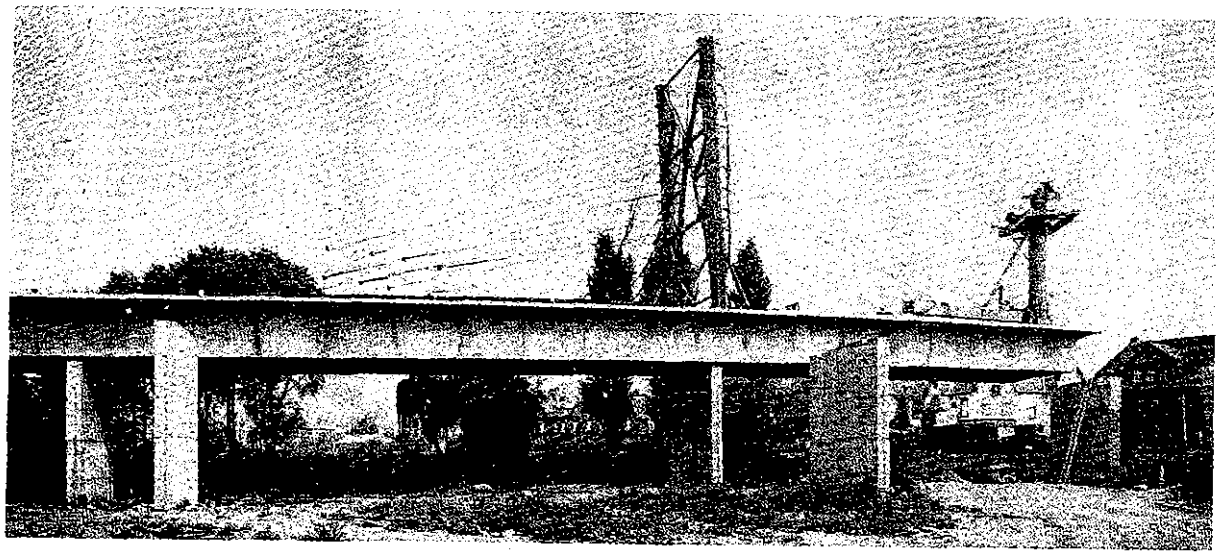
CHILLON VIADUCT ON THE GENEVA LAKE MOTORWAY *SWITZERLAND 1969*
2.210 m long structure with two separate decks. 92 m, 98 m and 104 m spans with superelevated curves.
Design : J.C. Piguet
Contractor : P. Chapuisat and Dantan Frères S.A.

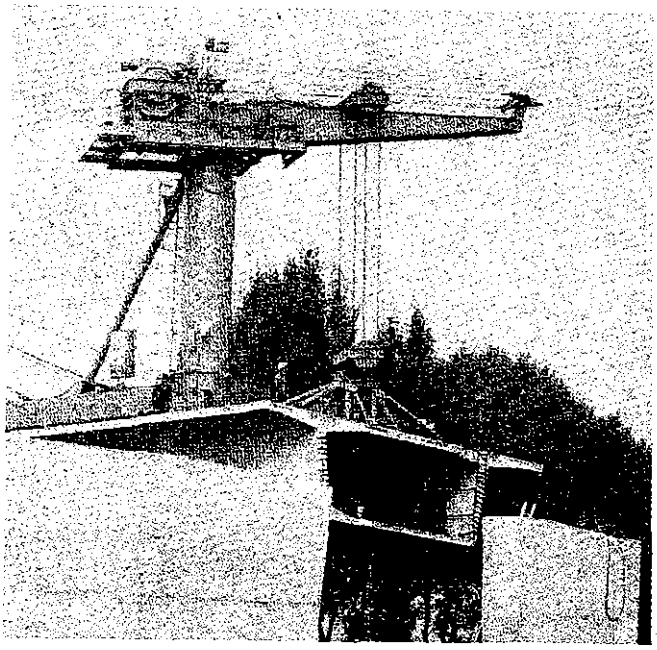




BRIDGE BETWEEN RIO AND NITEROI ACROSS THE GUANABARA BAY *BRAZIL 1973*

Longest cantilever built bridge in the world (7,884 m). 5 continuous sections in 80 m long spans. Segments placed by several launching girders.
Design : A.A. Naronha. Servicos de Engenharia S.A.
Contractor : Consorcio Constructor Guanabara Ltda
Equipment design and technical assistance : Campenon Bernard



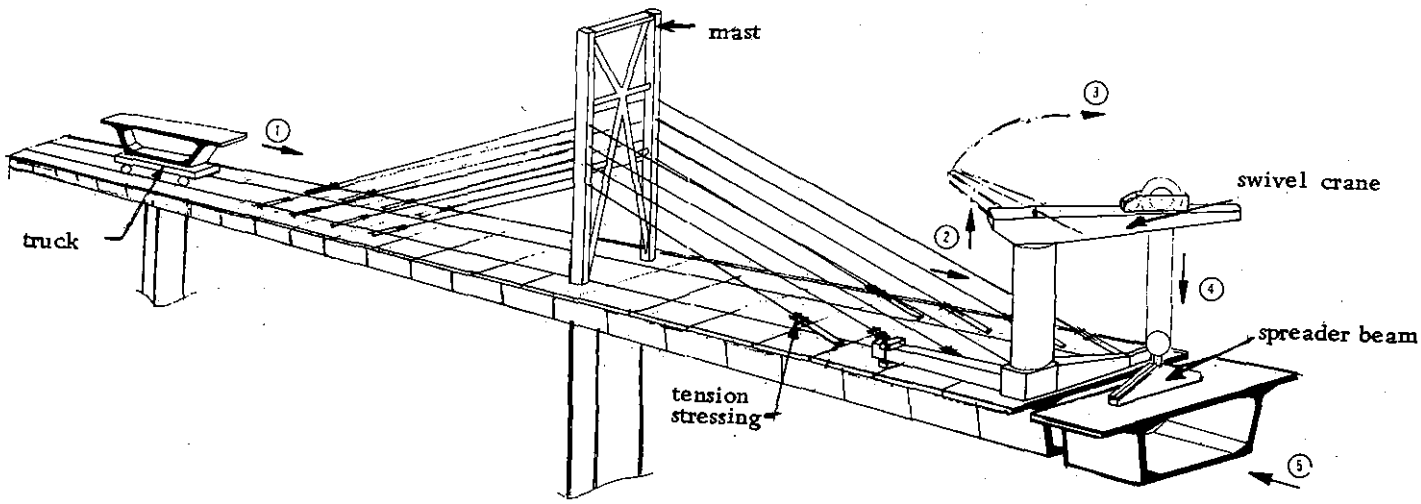


ROMBAS VIADUCT *FRANCE 1974*

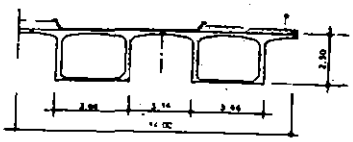
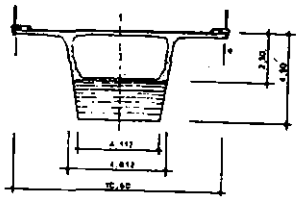
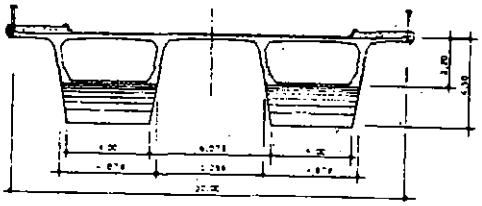
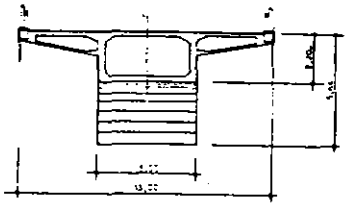
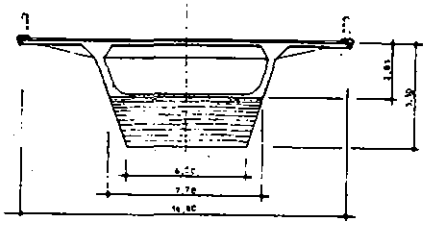
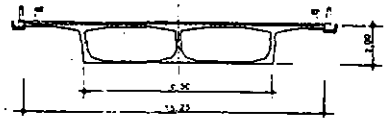
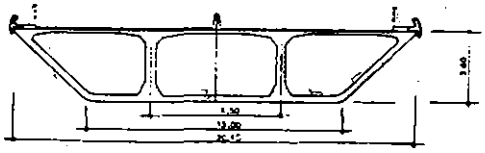
9 spans from 25 m to 45 m.

Design and Construction : Campenon Bernard

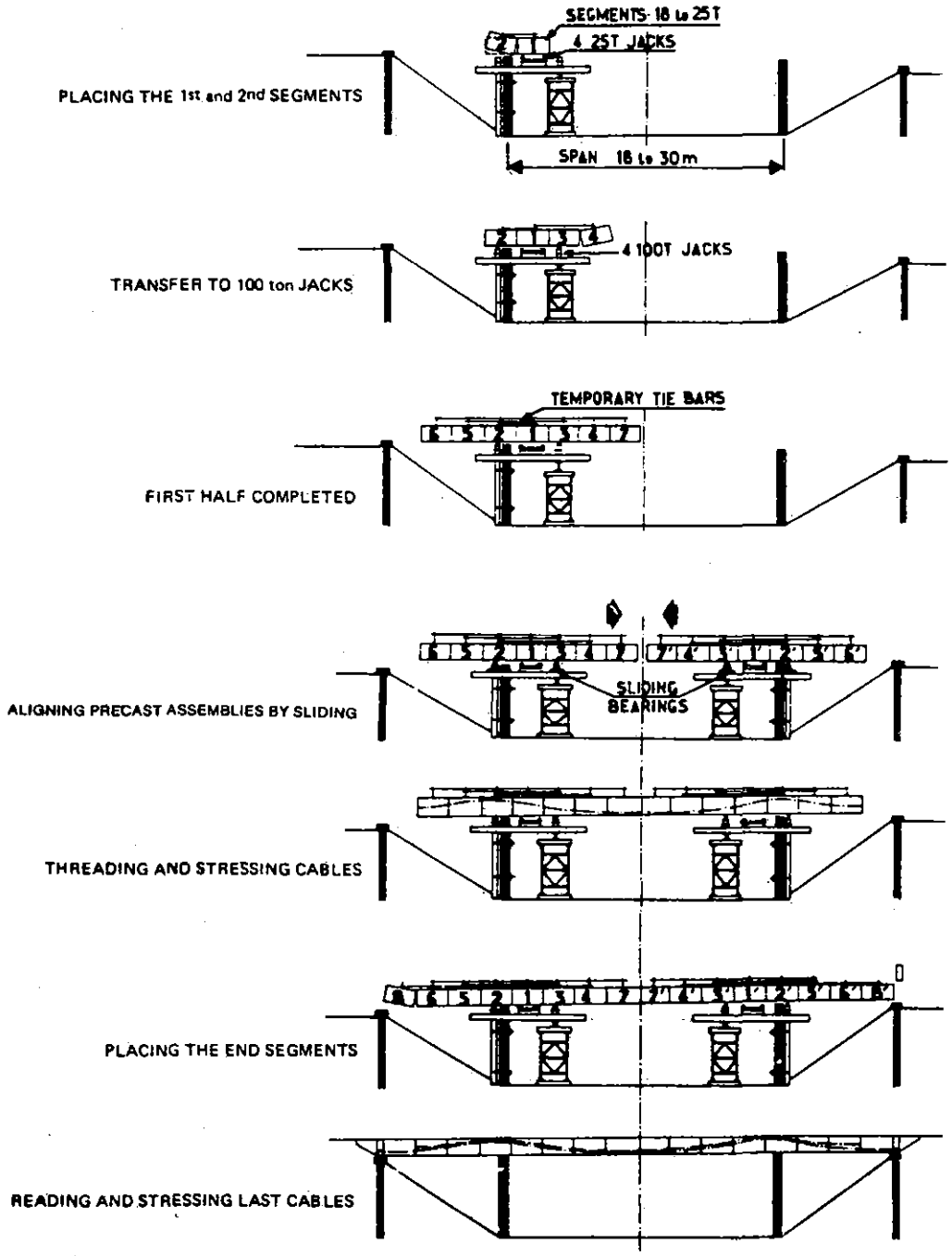
According to this new technique for bridges spanning up to 40-45 m, the cantilever construction proceeds directly from one-pier to the next. The first part of each span is cantilevered and the second part is supported by a mast and stays system. A pair of stays is placed for each new segment. Prestressing cables are threaded during cantilevering and stressed from the inside of the box section. Segments are placed by a swivel crane which is moved progressively along the deck.



EVOLUTION OF SEGMENT CROSS-SECTION AND WEIGHTS

BRIDGE	CROSS-SECTION (Dimensions in meters)	SEGMENT LENGTH	MAXIMUM SEGMENT WEIGHT (Tonnes)
CHOISY-LE-ROI		2.50 m 8.20 ft	25
SEUDRE		3.30 m 10.80 ft	75
BLOIS		3.50 m 11.50 ft	75
CHILLON		3.20 m 10.50 ft	80
SAINT-ANDRE-DE-CUBZAC		3.40 m 11.20 ft	80
B3 SOUTH		2.50 m - 3.40 m 8.20 ft - 11.20 ft	50
SAINT-CLOUD		2.25 m 7.40 ft	130

ALPINE MOTORWAY OVERBRIDGES: ERECTION SCHEME ~



CHAPTER 37

Considerations in the Design of Prestressed Concrete Box-Girder Decks with Special Reference to Cantilever-Construction

The design of concrete box girders is a complex task requiring extensive technical skill and construction experience. This subject is described, below in eight specific subheads 37.1 to 37.8.

37.1 INITIAL DESIGN

Why a Box Girder?

The popularity of the concrete box girder has risen steadily over the years to the point where it is now almost commonplace.

The reasons for this advance have been well documented by several authors in the past, and are listed briefly below:

(a) The box section is a highly efficient structural form which allows optimum use of materials on long-span bridges where reduction of dead weight is all-important.

The shape of the cross-section may be varied along the span to suit the loading envelope for the bridge, and the high lateral stability and torsional rigidity of the concrete box make it particularly suitable for curved bridges.

(b) In many cases it allows a finer pier design and thus a more attractive structure.

(c) Although its construction is more complex than other sections, recent advances in construction technology have overcome this disadvantage to the point where it is now competitive over a wide range of spans.

Types of Box Girder

Among the more important factors which must be considered are:

(a) *Forms of Cross-section*
- (i) Haunched or constant section longitudinally
- (ii) Multicell or multispine transversely (Fig. 37.1)
- (iii) Position of diaphragms

(b) *Method of Erection*
- (i) Cantilever, either symmetric [Figs. 37.2 (i)] or eccentric [Fig. 37.2 (ii)].
- (ii) Span-by-span on temporary staging (Fig. 37.3).

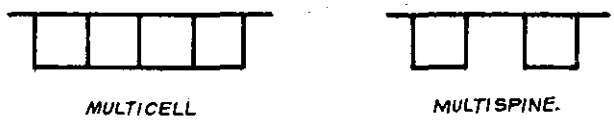


Fig. 37.1

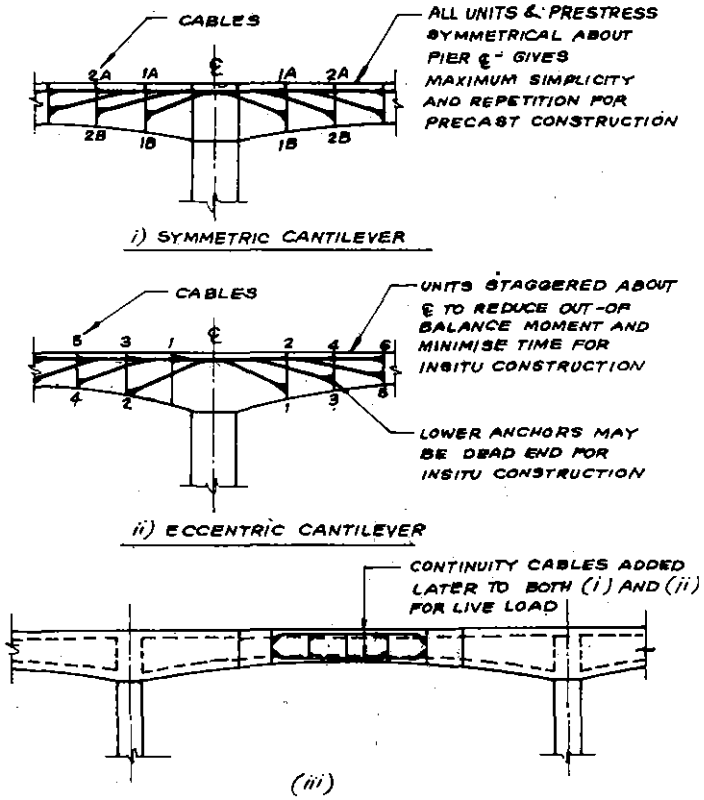


Fig. 37.2

(c) *Type of Construction*
- (i) In-situ.

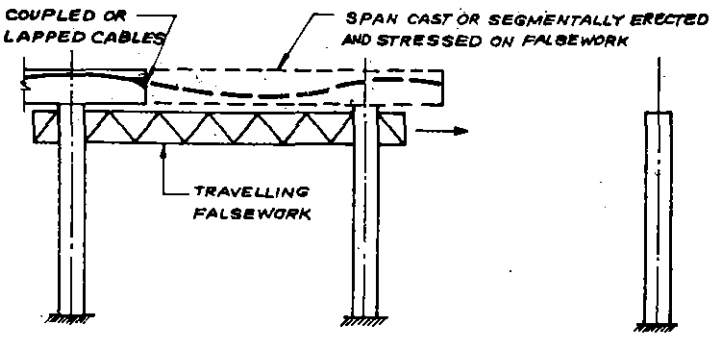


Fig. 37.3

(ii) Precast, including the method of jointing for this

(d) *Type of Prestress*

(i) Longitudinal [dependent on (a) above]

(ii) Transverse (for deck slab bending essentially)

(iii) Vertical (for web shear).

To assist the designer in this task the Cement and Concrete Association (London) has published a comprehensive survey of some major concrete box girders designed (1), with comparisons between them designed to indicate the most efficient ranges for the parameters involved.

Because of problems in detailing the thin sections used in these structures it is imperative that all initial sizes be reviewed in terms of anchorage location, duct trajectories, concrete placing, etc., before work on the final design is begun.

37.2 ANALYSIS

The accurate analysis of box girder bridges is a complex problem which has received considerable attention in recent years.

Dead Load Stresses

(a) *Straight Bridges*

For straight bridges, simple beam theory is generally sufficient to estimate dead load stresses. The dead load analysis must, of course, take full account of the various stages of construction.

Two secondary effects which should be evaluated for each structure are:

(i) *Shear lag*

In regions of high shear (i.e., at and near the supports) the 'stiffness' of the cross-section is effectively 'reduced' by the flexibility of the flanges in shear. For sections with large flanges the effect can be considerable. In prestressed structures, however, the effect may not be very important as a large percentage of the dead load is balanced by the prestress.

(ii) *Creep redistribution of moments* Wherever span-by-span construction is employed or wherever continuity between a number of spans is made some time after construction, some creep redistribution of the dead load moments will occur (Fig. 37.4)

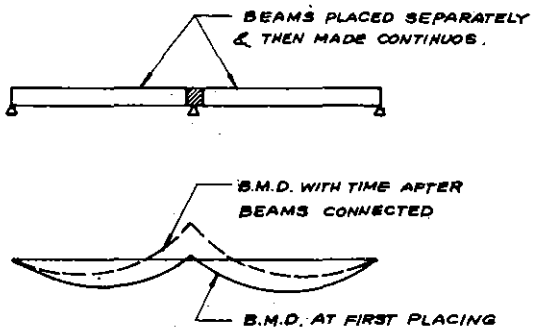


Fig. 37.4

(b) *Curved Bridges*

In the case of curved bridges the dead load moments are accompanied by torques for equilibrium.

For box structures of moderate curvature the method given in reference (2) and shown here in Fig. 37.5 is generally sufficient. The longitudinal bending moments are computed as for a straight beam and the expression for the incremental torque given in Fig. 37.5 is integrated along the beam from points of known zero torque to give the final torque at any section. (Also see separate chapter in this book, on this subject.)

Live Load Stresses

Methods of Analysis

The five methods generally pointed to (for the present purpose) are outlined below in principle.

(a) *Simple Beam Theory*

In its basic form, this theory assumes only simple bending theory and torsion of thin-walled members. Warping and distortion (Fig. 37.6) of the cross-section are ignored. Local bending moments in the deck slab are found from influence surfaces etc., and transverse moments, around the box by moment distribution etc., as shown in Fig. 37.7.

The structures may be analysed by a simple computer grillage analysis.

Representative (i.e., distributed) forces are first determined at each node of the grillage.

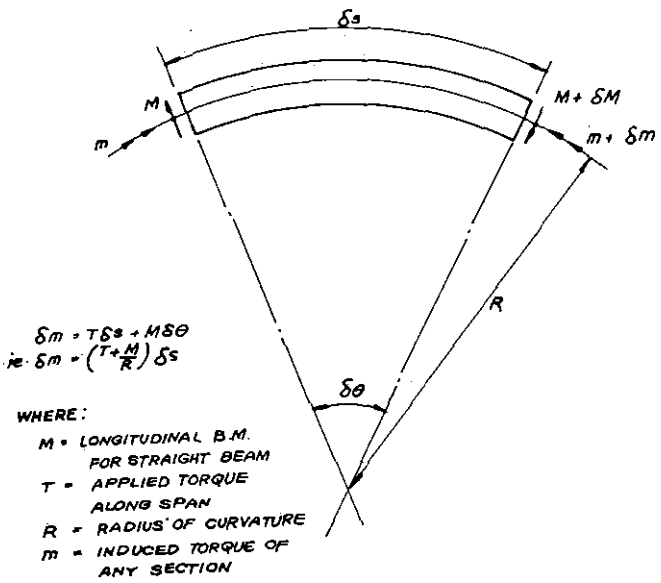


Fig. 37.5

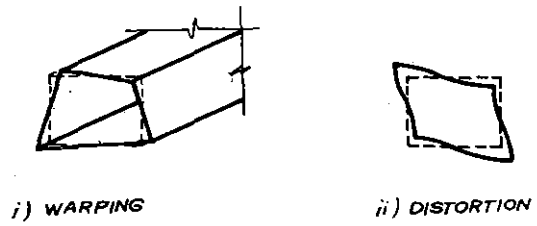


Fig. 37.6

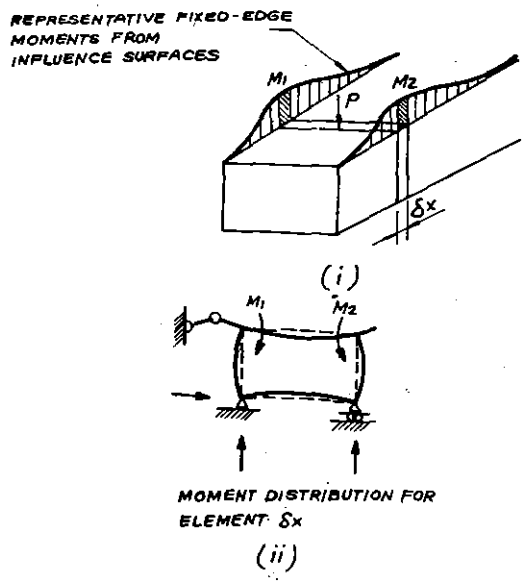


Fig. 37.7

These loads and the structure may then be separated into symmetric and antisymmetric components.

The structure may thus be analysed in each condition of symmetry and the final results obtained by superposition. Variation of section properties along the span (including arbitrary reductions in flange width for shearlag near the supports) can be handled in this way, and the method can be extended to curved structures also.

The accuracy of the method is acceptable only when the distortional 'stiffness' of the box is high and when the span length is such that warping stresses are small. If this is not the case, then one of the methods in (b) below must be used.

(b) *Approximate Allowances for Warping and Distortion*

A number of methods exist.

Some of these methods, e.g., that of Richmond (3) can be extended to grillage analyses of the form discussed in (a) above, and they can, therefore, be expected to give improved accuracy where the methods of (a) become inadequate.

(c) *Folded Plate Analysis (References 4 and 5)*

The basis of this method is shown in Fig. 37.8. The structure is represented as an assembly of plates joined rigidly along their longitudinal edges. All points on these edges have the four degrees of freedom shown in Fig. 37.8(i). Individual plate stiffness matrices for these displacements are derived from classical plate theory and the overall stiffness matrix for edge loading of the structure assembled from these. Wherever loads are applied away from the edges of a plate the fixed edge moments and forces must first be determined as in Fig. 37.8(ii). These are then applied to the structures as in Fig. 37.8(iii). In all cases, solution is by Fourier series and the final results are obtained by superposition of the results for each harmonic of loading.

The method is suitable for straight or curved, simply supported, structures of constant cross-section. Continuous structures can be handled by first treating the entire structure as simply supported and then eliminating displacements at the interior supports as a second stage in the analysis. Variation of overall depth or plate thickness in the longitudinal direction cannot be handled. However, the method is academically good, but is seldom used in practice.

(d) *Finite Strip (Reference 6)*

This method is essentially an approximate and more versatile version of the folded plate method. The loaded surfaces are subdivided into more plates, as shown in Fig. 37.9, so that variations in transverse thickness can be accommodated and the first stage in the folded plate analysis of (c) above can be eliminated by applying loads directly to the longitudinal edges between plates. The degrees of freedom chosen are the

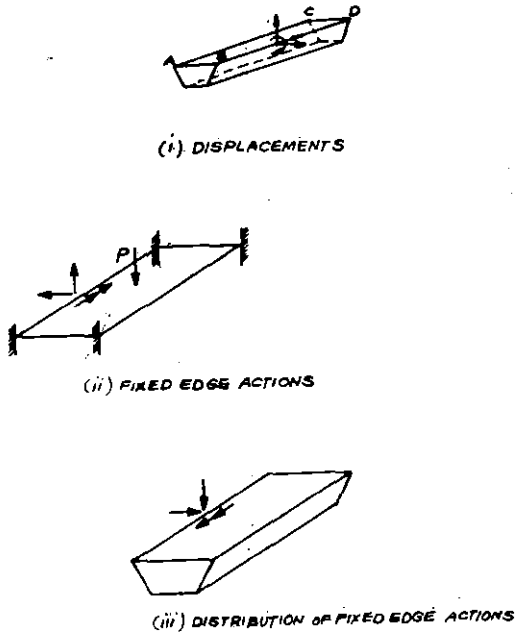


Fig. 37.8

same but in this case the variation of these displacements across a strip is approximated by polynomials. Solution is again by superposition of loading harmonics. The remarks made in (c) above concerning curved structures, continuous spans and variable depth structures apply equally to this method. Accuracy of the method is almost as good as for folded plate case (c). The method is seldom used in practice.

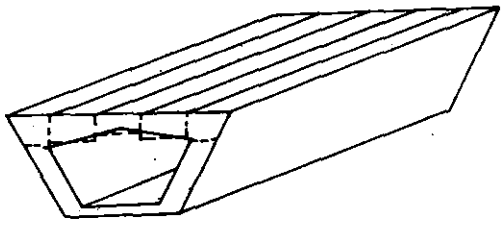


Fig. 37.9

(e) *Finite Element*

The basis of the method is shown in Fig. 37.10. The entire structure is divided into small elements and the stiffness of the structure is assembled from the membrane and plate-bending stiffness of each element.

The method is the only one that is truly general and powerful and can handle all types of structure. It has even been extended to the analysis of structures subject to cracking (7). Its accuracy depends on the nature and number of elements used. True compatibility of deformation

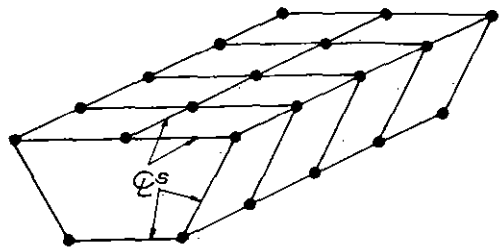


Fig. 37.10

is achieved only at the nodes, and the accuracy of the representation between nodes depends on the suitability of the assumed displacement functions for the strain field in question and upon the size of the grid element chosen. Considerable research has been directed recently towards finding the most suitable types of element for box girder work (8 and 9) and good agreement between experiment and theory can be obtained. The disadvantage of the method is the large computer storage required and the time consuming input-synthesis and output-analysis.

Comparison of Methods

In general it can be said that for single or multiple cell boxes, straight-in-plan, method (a) is good enough, while for curved-in-plan decks that are 'wide' and or 'shallow', method (e) is preferable.

As for "method versatility":

(i) Of all the methods discussed only the finite element method is truly versatile and accurate.

(ii) The approximate methods (a) and (b) are also versatile but their accuracy is inadequate for more complex structures.

(iii) The folded-plate and finite strip methods have limitations which make them unattractive to the design engineer.

Thus the approximate methods (a) and (b) are recommended for straightforward structures and the finite element method for more complex structures. (Also see Ch. 19 in this book on the associated subject of the distribution of the applied load transversely among the longitudinals of the deck.)

The approximate methods have been used successfully for the design of many major structures in the past and there is no reason why they should not continue to be used where appropriate in the future.

The finite element method is likely to be adopted ultimately as the general method for all this work. Simplified commercial programs for box girder structures are already available and it will not be long before even more sophisticated programs are in use.

Effect of Diaphragms

The influence of the number and position of diaphragms is discussed in reference (10). It is shown that the spacing of diaphragms must be selected carefully if the warping stresses in the box are not to be exaggerated. For the 'average' straight concrete box girder, it has been shown by several authors (e.g., References 11 and 12) that there is no need for transverse diaphragms except at the supports, and from the construction viewpoint also this is certainly the best solution.

Differential Temperature

This is an important effect which has been recognised only comparatively recently. It is now well documented, and a clear method of analysis is given in Ch. 30 of this book.

Prestress

(a) *Straight Bridges*

In general, simple beam theory is adequate for the prestress analysis of straight bridges. The more complex 'Finite Element' method may be used if the structural complexity warrants it.

(b) *Curved Bridges*

The comments in (a) above apply here with the additional point that care must be taken in the evaluation of torques induced by post-tensioning. For the usual case where the prestress is distributed symmetrically about the transverse centreline (Fig. 37.11) with cables moving essentially in the vertical plane, it can be shown (13) that it is only the secondary or parasitic prestress moments which induce torques in the structure. These torques may be evaluated from the secondary moments by the method referred to in Fig. 37.5 (as also the separate chapter on this subject in this book).

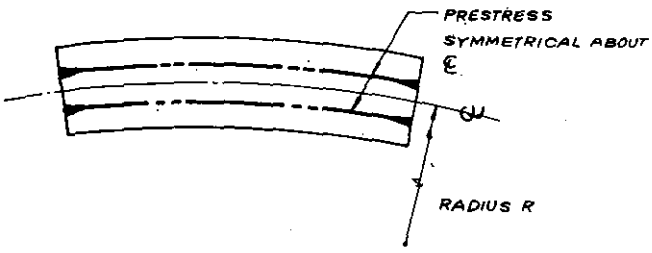


Fig. 37.11

Distortional Resonance and Instability

The minimum dimensions of concrete boxes are usually dictated by detailing requirements and the rigidity of the cross-section is thus usually more than adequate for the above effects.

Reference is made in (10) however to the case of a large thin-walled box where the natural frequency of distortional vibration was found to be in the normal range of traffic vibration.

Creep Redistribution within the Cross-section

In thin-walled and closed-cell structures like box girders there can be considerable differences in the rate of curing of the various components of the cross-section and consequent differences in the rate of shrinkage and creep.

This in turn, can give rise to variations in the distribution of the stresses within the cross-section, to the extent where the final distribution may be different from that prevailing initially.

37.3 FINAL DESIGN

Determination of the final concrete dimensions and reinforcement for concrete box girder bridges is a lengthy process requiring constant review of the initial member sizes and stresses.

The essential steps in this process may be summarised broadly as follows:

(A) Working Stress Design

i.e. (i) Determination of bending and shear reinforcement to control stresses and limiting the deflections at working loads.

(ii) A check on such miscellaneous effects as the influence of construction variables on design assumptions, and the fatigue limit of deck reinforcement.

(B) Overload Design (Check)

i.e., overload capacity

(C) Ultimate Load (Moment) Check

for both bending and shear.

(A) Working Stress Design

(a) Longitudinal Bending

Post-tensioning must be provided to control longitudinal stress. The determination of suitable prestress layouts for this purpose is a relatively straightforward problem and is not discussed here. The following points, more peculiar to box girder design are, however, important:

(i) The thin sections involved in box girder design can make detailing of the prestressing difficult. This should be given careful attention early in the design.

(ii) High stresses in design can give rise to large creep strains, particularly in in-situ construction, and consequently to large creep deflections if the prestress design is significantly different from a load balance.

The main cables may also be used for relief of vertical shear near the supports.

(b) Transverse Bending

Design of the (transverse) reinforcement for the bending within deck cross-section is also straightforward and is not discussed further here. Where, as is generally the case, the only resistance to deformation of the cross-section between support diaphragms is the bending strength of the cross-section itself, care should be taken to give the corners of the box adequate strength in transverse bending, e.g., as shown in Fig. 37.12.

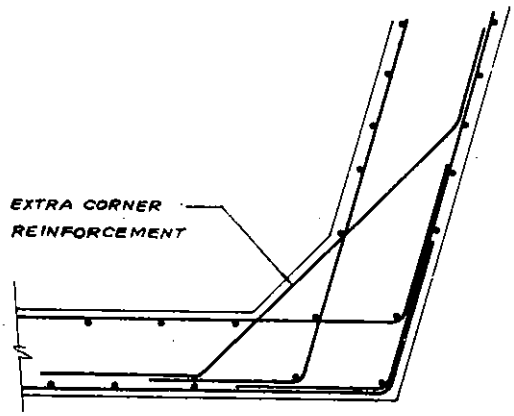


Fig. 37.12

Extra transverse reinforcement should also be provided in the flanges for the following effects (where appropriate):

(i) Splitting forces adjacent to any flange-anchored longitudinal prestressing cables,

(ii) Transverse forces due to inclination of the webs (from frame action of transverse section).

(c) Shear (Transverse Shear at Various Sections Longitudinally)

The behaviour of 'large' prestressed concrete beams in shear is still not clearly understood. Recommendations for the shear design of these members are correspondingly lacking. While the ultimate strength in shear may be estimated with reasonable confidence, it is not clear yet just how the various resisting mechanisms contribute to this strength throughout the loading range.

Current opinion (14, 15) on working stress design for shear is that it can be omitted if a number of requirements for ultimate load design are met. (See separate chapter in this book on the philosophy of design against shear.)

Principal tension is the webs may be controlled in any of the following ways:

(i) Increased web thickness (though not advisable beyond about 45 cm, from thermal stresses stand point).

(ii) Vertical prestressing in the webs [Fig. 37.13(i)]. This method is effective but it can give unacceptable increases in the principal compressions if used excessively. (Construction-wise cumbersome.)

(iii) Short cap cables for shear relief [Fig. 37.13(ii)]. (Note that this is in addition to the normal shear relief afforded by draped cables of the type shown dotted.)

(iv) Inclination of the longitudinal cables in cantilever construction [Fig. 37.13(iii)].

(v) Haunching of the member so that vertical component of the longitudinal flange force counteracts the shear [see Fig. 37.13(iv) and Reference 16]. For box girders with substantial bottom flanges, the above reduction may be approximated by the vertical component of the force in the bottom flange.

Extra reinforcement should also be included in the webs locally wherever longitudinal cables are anchored in them.

(d) Torsion

Torsion is usually of concern only in curved bridges.

Care should be taken with thin-flanged sections that the principal tensions in the flanges due to combined shear and torsion are limited [see (c) above]. The cross-sectional hoop reinforcement should be detailed to provide 'continuity' for torsional shear flow. The transverse reinforcement should be closely spaced to confine the longitudinal corner bars which should themselves be substantial. (See separate chapter on this subject in this book.)

(e) Miscellaneous Checks

(i) Construction Variables

In some cases assumptions may be made in design which are not fully realised in practice, e.g., the formwork for a continuous span may be assumed rigid when in fact it can never be fully so. An effort should always be made to establish the upper and lower bounds for these effects, in the same way as it is normal to allow for a small (5%) variation in the final prestressing force achieved.

(ii) Fatigue of Deck Reinforcement

In the thin flexible deck slabs of box girders there is a risk that continual overstressing of the transverse deck reinforcement can lead to premature fatigue failures. This is especially true in partially prestressed designs where the deck steel may be cycling anywhere between 300 kg/cm^2 compression and 1700 kg/cm^2 tension. It should be remembered that the design of the deck slab is

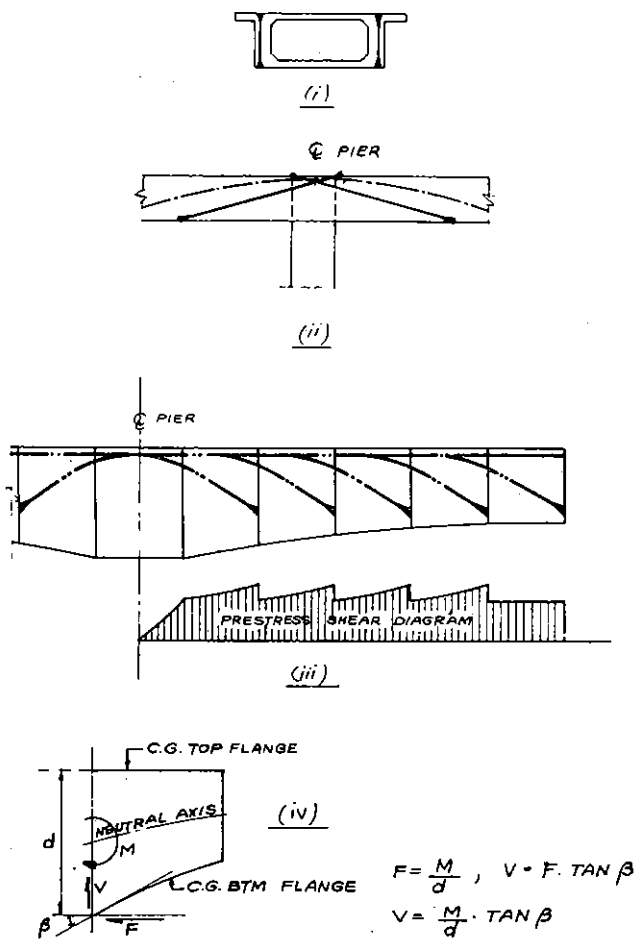


Fig. 37.13

frequently governed more by individual wheel loading than by overall truck loading, and that the fatigue life of the deck slab should, therefore, be appreciably higher than that of the girder.

The fatigue life of the deck slab reinforcement should thus always be checked by limiting the stress variation range in the steel and a reduced upper limit of its stress.

(B) Overload

The overload capacity check of prestressed concrete bridges involves a method that limits crack widths under overload according to the nature of the structure and the environment. The crack widths may be determined either by analysis from first principles of the cracked and partially prestressed section, or by the use of hypothetical allowable tensions which have been found to give comparable results for most common girder sections.

Extra non-prestressed reinforcement may thus be added to bring a structure up to a desired overload grading.

The longitudinal reinforcement should be closely spaced in the cross-section so as to minimise flexural crack widths. If the reinforcement is concentrated mainly in the webs, the crack spacing in the flanges will be governed mainly by the rate of shear transfer from the webs and will, therefore, be comparatively large and the cracks will be wide. If, however, the reinforcement (including where possible the bonded prestressed reinforcement) is distributed evenly over the cross-section, then the crack spacing will be governed mainly by the ultimate bond stress along the reinforcement in the normal fashion and the cracks will be correspondingly smaller in width and closer together. All post-tensioning ducts should be rigid, corrugated and well grouted to ensure maximum efficiency in this respect.

(C) Ultimate Load

(a) Bending

The ultimate strength of the box girder in flexure can be adjusted in the normal fashion by the use of non-prestressed reinforcement which is discussed in a separate chapter in this book.

Care should be taken in assessing the amount of plastic redistribution available from the girder, as discussed in the said separate chapter.

(b) Shear

See details in a separate chapter in this book.

37.4 DETAILING

The importance of detailing has been stressed constantly throughout this book. The following are some of the more critical factors which must be resolved as early as possible in the design stage.

Cable Layout

The prestressing layout adopted should be as simple as possible and should be directed towards the following objective:

(i) Optimum concrete flow access to webs
(ii) Minimum interference of cables with reinforcing steel
(iii) Maximum repetition and standardisation
(iv) Optimum speed of construction

Thus the arrangement of Fig. 37.14(i) for cantilever construction, for example, is preferable to that of Fig. 37.14(ii).

Duct Curvature Problems

Care must be taken wherever ducts change direction that there is sufficient concrete against the inside curve of

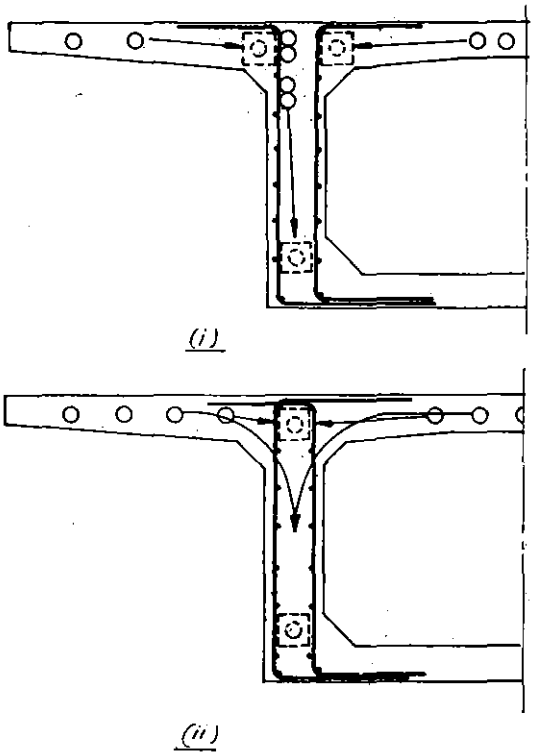


Fig. 37.14

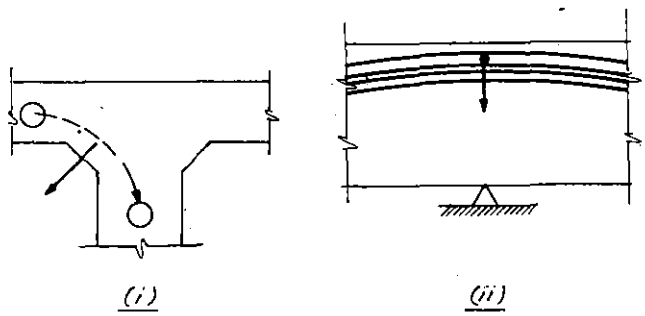


Fig. 37.15

the duct to resist the radial pressures from the tendon. Figure 37.15 shows two examples of this.

Where there is curvature over supports [Fig. 37.15(ii)], the ducts should be separated vertically by at least one and preferably two duct diameters.

Where the ducts run straight for some distance the opinion of the codes is divided but it seems acceptable to bundle the ducts at least in pairs either horizontally or vertically.

Care must also be taken during segmental construction that each new segment of duct is correctly aligned with the previous segment. [See reference (17) for example, where a special coupler had to be designed to keep friction losses down to the design limits.]

Construction Joints

Deliberate construction joints are generally of concern only in precast designs. In this case the designer may choose from the following:

(i) Normal (30–45 cm) in-situ concrete joints with reinforcement continuous through the joint.
(ii) Fine concrete (5–10 cm) in-situ joints with no reinforcement
(iii) Zero-tolerance epoxy joints with shear keys (Fig. 37.16)

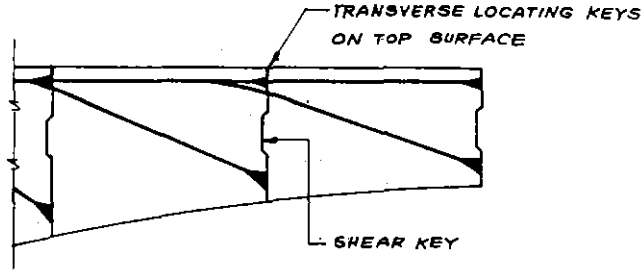


Fig. 37.16

The delays in construction inherent in (i) and (ii) must be weighed against the stringent precasting requirements for (iii).

Wherever possible it is generally recommended that the cross-section should be poured in one operation, namely:

(i) Pour bottom slab, [re-trowel the wet concrete in-slab... see (iv) ahead]
(ii) Pour webs, ensuring good compaction of the concrete in the critical fillet regions at the base of the webs. (Access holes for vibrators should be provided in the formwork above the fillets to facilitate this.)
(iii) Pour top slab.
(iv) Lightly revibrate the top slab as required in the vicinity of the webs to minimise the effect of concrete settlement (plastic settlement cracks). Also re-trowel the wet slab-concrete against plastic shrinkage cracks (as needed, depending on if surface evaporation is considerable).

Particular care must be taken to ensure good compaction around anchorage locations.

If construction joints in the cross-section cannot be avoided they should be placed away from anchorage locations at points of low shear and should be well 'prepared' before further concreting to ensure a degree of roughness at least equivalent to a fractured concrete surface.

Anchor Blocks (End Blocks, Anchorage Zones)

In any form of in-situ construction where travelling formwork is used, there is usually a conflict between design and construction requirements for the internal anchor blocks construction, whether it be for the continuity cables in cantilever construction [Fig. 37.17(i)] or the main cables in span-by-span construction [Fig. 37.17(ii)], poses a severe problem for the contractor.

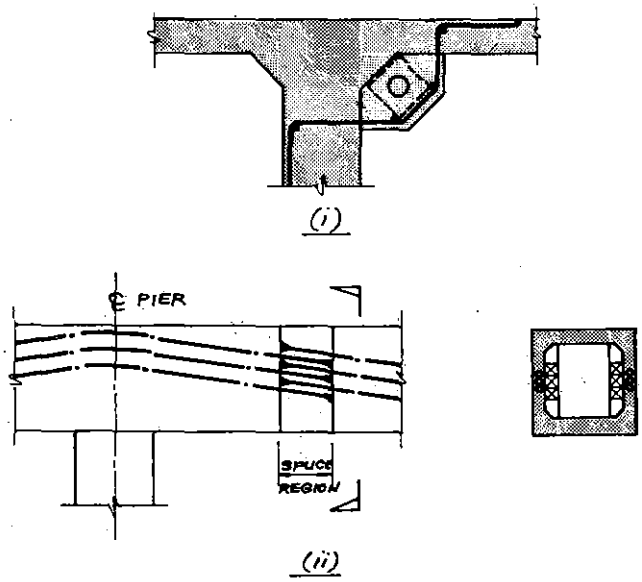


Fig. 37.17

37.5 CONSTRUCTION

The paramount requirements for box girder construction are the strictest supervision and quality control. Supervision should be directed by experienced engineers acquainted with aspects of the design, and all developments during construction should be examined closely.

The following items should be given special attention:

Formwork and Falsework

Formwork and falsework for concrete box girder bridges must be of the highest standard.

The entire falsework must be adequately rigid to ensure sound construction throughout and the formwork for the cross-section must be rigid and almost watertight. Access for inspection and concrete compaction is particularly important.

Specialist services for falsework design are now available and the use of these should be encouraged wherever possible.

Concrete

(a) Mix Design

The concrete for the superstructure must be durable and of the desired high strength, and satisfy a number of other conflicting criteria such as good workability and low creep and shrinkage characteristics.

Concrete mix designs and placing procedures must, therefore, be reviewed carefully beforehand, and wherever possible some form of trial run should be arranged to test these procedures.

(b) Curing

Careful curing is essential if shrinkage and thermal cracking of the cross-section is to be minimised.

The internal formwork should be removed as soon as possible to prevent undue 'propping' of the cross-section, and differential thermal expansion between formwork and concrete should also be limited. Steam curing procedures may need special review in this respect.

Erection

(a) Epoxy Jointing (Precast Only)

Where epoxy jointing is used in precast work the preparation of the mating concrete surfaces must be closely supervised.

Some preparation of the mating surfaces (e.g., by light sand-blasting) is generally required.

(b) General

All movements of the superstructure during erection should be constantly reviewed to ensure that they fall within anticipated limits and that the final state of stress throughout the structure will be acceptable.

The overall alignment of the superstructure must also be checked in cantilever construction to ensure successful span closure. The design should incorporate adequate facilities for the control of this alignment.

Grouting

Careful supervision of all grouting operations is essential if the desired durability and ultimate load capacity are to be achieved.

All joints between ducts should be carefully sealed and all cables in any one zone of the cross-section should preferably be grouted simultaneously to minimise the possibility of grout leakage between ducts.

Grouting should be performed only by skilled operators and facilities for emergency flushing with fresh water should be provided wherever possible.

37.6 DEFLECTION OF CANTILEVER BRIDGES AND CAMBER DESIGN

Each cantilever arm consists of several segments, fabricated,

installed, and loaded at different points in time. It is important, therefore, to predict accurately the deflection curves of the various cantilevers so as to provide adequate camber either in the fabrication plant for precast segmental construction or for adequate adjustment of the form-travellers for cast-in-situ construction.

While the structure is statically determinate, the cantilever arm deflections are due to:

(a) The concrete girder weight
(b) The weight of the travellers or the segment-placing equipment
(c) The cantilever prestress

If the structure is finally a continuous type, then after continuity between individual cantilevers is achieved, the structure becomes statically indeterminate and continues to undergo additional deflections for the following reasons:

(d) Continuity prestress
(e) Removal of travellers or segment-placing equipment (reverse loading, but on continuous structure)
(f) Removal of 'provisional supports' and release of 'deck to pier connections' if any (again a reverse loading)
(g) Placing of superimposed loads

Even if the structure is non-continuous, items (e), (f) and (g) still cause deflection.

Subsequent long-term deflections due to concrete-creep and prestress-losses will also take place. Compensation for the deflections resulting in the following three states must be provided-for by adequate camber or adjustment:

1. Cantilever arms (during construction)
2. Short term in-service deck } Statically determinate or
3. Long term in service deck } indeterminate

The concrete modulus of elasticity varies both with the 'age at the time of first loading' and with the 'duration of the load'. Deflections of states 2 and 3* above are easily accommodated by changing the theoretical longitudinal profile by the corresponding reverse amount in each section to offset the anticipated future deflections. A more delicate problem is to further accurately predict, allow for, and adequately follow the deflections of the individual cantilever arms during construction (at each stage there is a new cantilever). For this it is necessary to analyse each construction stage and to determine the deflection curve of each successive cantilever arm (as construction proceeds segment by segment). A simple case with a five-segment cantilever is shown in Fig. 37.18. The broken line represents the envelope of the various deflection curves or the 'space-trajectory', followed by the cantilever-tip at each construction stage (i.e., at the end of each segment addition). For more practical details see 37.7 ahead.

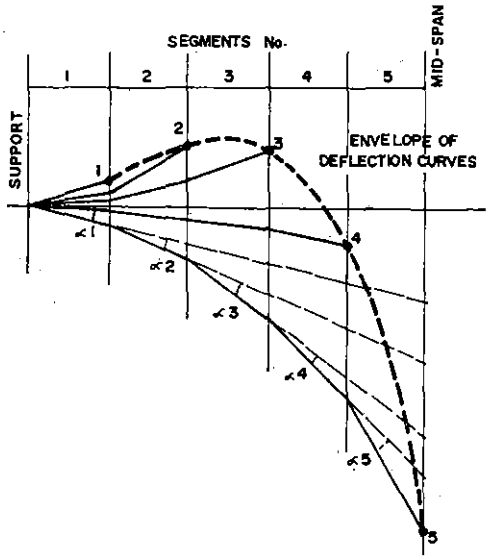


Fig. 37.18 *Deflections of a typical cantilever*

By changing the relative angular positions of the various segments by small angles, such as α_1, α_2, and so on, the cantilever should be assembled to its final length with a satisfactory longitudinal profile as shown in Fig. 37.19, for the simple case considered.

It is interesting to compare the relative importance of deflections and camber for cast-in-situ and precast construction. Figure 37.20 shows values for an actual structure, where computations have been made for the two different methods. The calculational assumptions given in Fig. 37.20 indicate that in most cases the difference would be even more significant if a cast-in-situ cycle of less than one week were employed and if precast segments were stored for more than two weeks.

37.7 PRACTICAL PROBLEMS IN CAST-IN-SITU CONSTRUCTION CAMBER CONTROL

Before proceeding with the cantilever construction proper,

* Deflection, at various sections, due to permanent loads (dead load and final prestress) as enhanced by creep of concrete over a long period of time (stabilising period is generally about 3 years), should be predetermined at each section and allowed—for during construction in the form of precamber in reverse direction on the actually required longitudinal profile of the deck. A good enough estimate of this deflection may be based on assuming the effective modulus of elasticity of concrete as 0.45 of its instantaneous value generally given in the codes. This will nearly take care of the changed concrete modulus on account of long term creep effect and the variations in humidity, temperature, water-cement ratio, etc., from section to section, during the period of construction. However, this simplification will apply more for atmospheric conditions not far from moderate, with atmospheric shade-temperatures of about (25 ± 20)°C. In a different environment, the designer may use more appropriate assumptions and the actual feed back from the construction site.

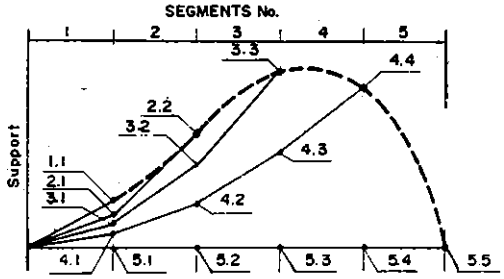


Fig. 37.19 *Choice and control of camber*

LEGEND

1.1 Means deflected position of the tip of segment 1 due to itself and its prestress.

2.1 Deflected position of the tip of segment 1 under the Wt. and prestress of segment 1 and 2.

2.2 Deflected position of the tip of segment 2 under Wt. and prestress of segment 1 and 2.

3.1 Deflected position of the tip of segment 1 under the Wt. and prestress of segments 1, 2 and 3.

3.2 Deflected position of the tip of segment 2 under the Wt. and prestress of segments 1, 2 and 3 and so on.

a starting base must first be completed on the various piers. This first special segment, called a 'pier-segment' or a 'pier-table', is generally constructed on a temporary platform anchored by prestressing the pier top, Fig. 37.21.

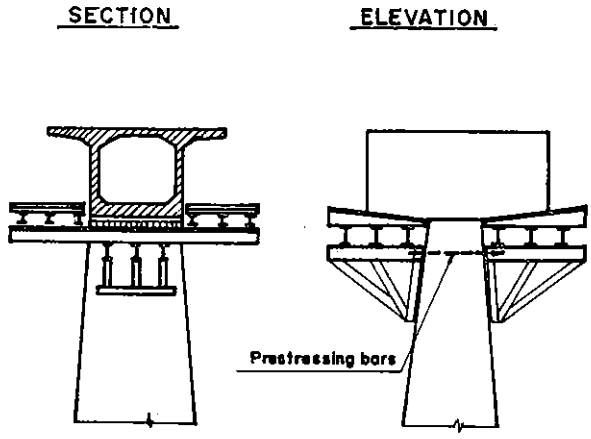


Fig. 37.21 *Construction of the pier-segment for a cast-in-situ cantilever deck*

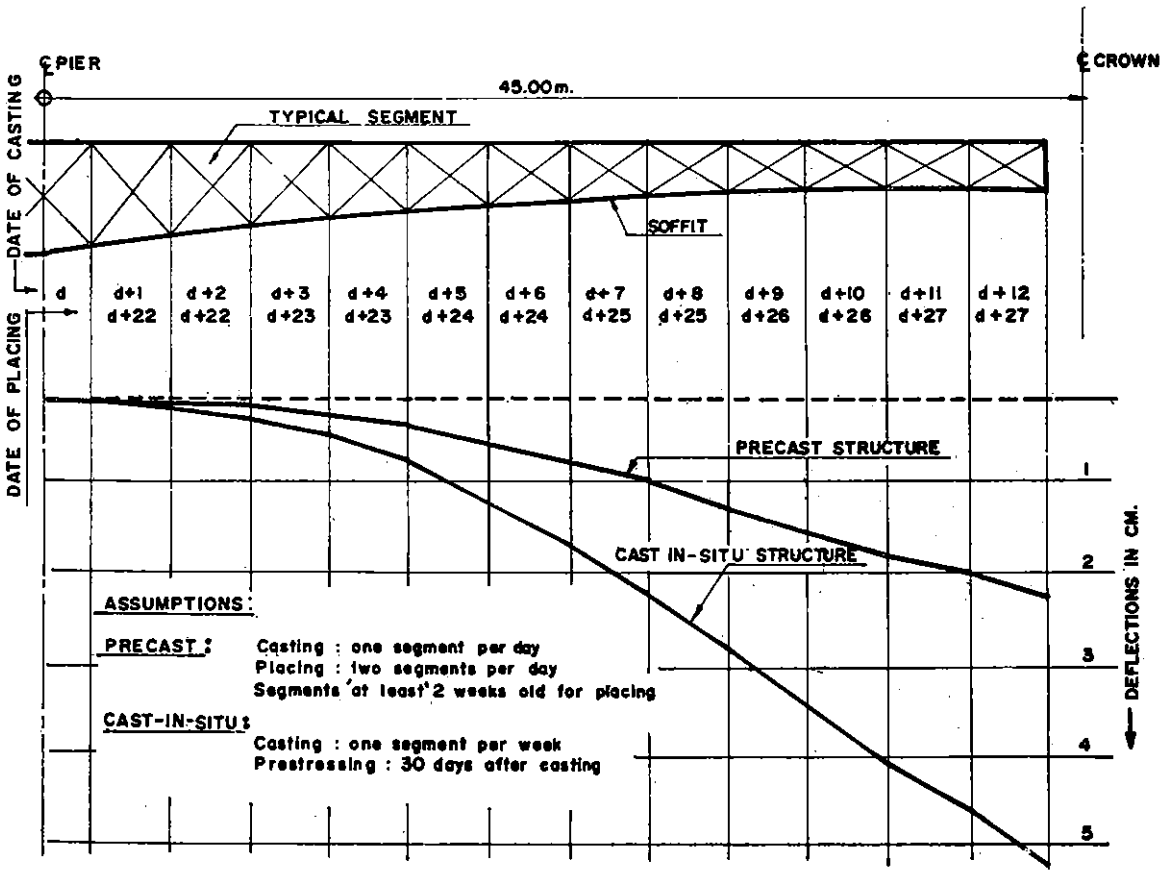


Fig. 37.20 *Comparison of soffit deflections between precast and cast-in-situ structures.*

This special segment may either be given the minimum length to insure adequate connection to the pier for the stability of the future cantilever or else be of such length as to allow both travellers to be installed simultaneously (Fig. 37.22).

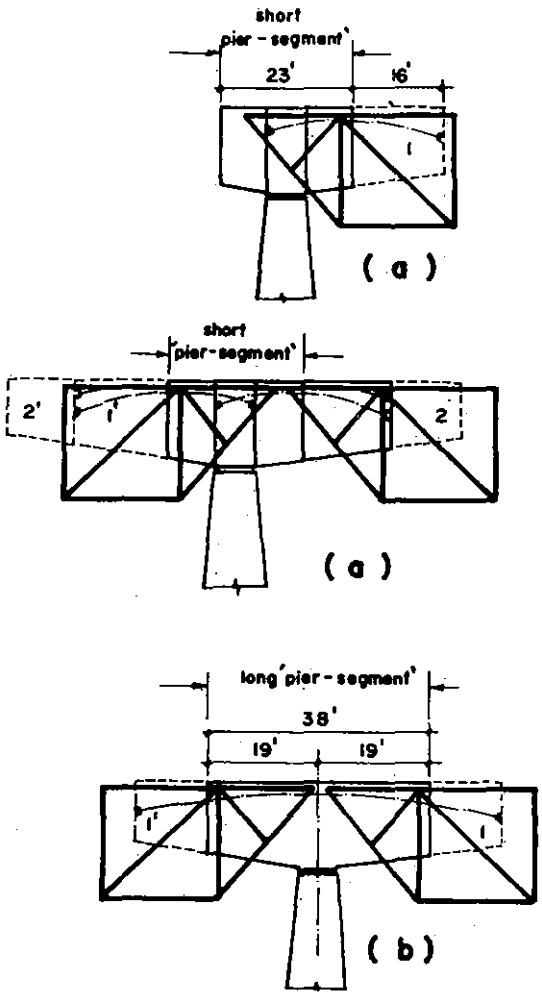


Fig. 37.22 *Start of cantilever construction from the pier-segment. (a) Short pier-segment—successive installation of travellers. (b) Long 'pier-segment'—simultaneous installation of travellers*

Another important problem relates to the safety of the travellers during construction. Apart from the difficulties of ensuring pier safety in the event a form-traveller fell during transfer from one position to the next, the difficulties would be even greater in the event of an accident during the casting operation. Consequently, all precautions must be taken both at the design stage and during construction to eliminate this potential hazard. The load-carrying members of the traveller must be carefully inspected and load-tested before use so as to practically eliminate the danger of structural failure.

The most critical areas are in the safety of the suspension rods and the transfer of the traveller-reactions to the concrete. Preferably all suspension rods and anchor bars should be doubled. Also, the prestressing tendons must have an adequate margin of safety. Use of only a single strand or a single bar in each web of the box should be avoided. Rather one or more multistrand-tendons with individual anchors for each strand or at least two prestress-bars should be used.

Worldwide use of cast-in-situ cantilever construction has established an extremely good safety record; no less than that for cast-in-situ construction on false work. Accidents are very few and far between; however, designers and constructions must always be safety conscious.

The most critical 'practical' problem of cast-in-situ construction is 'deflection control', particularly for long-span structures. There are five categories of deflections (or space geometrical movements of the structure) during construction and after completion:

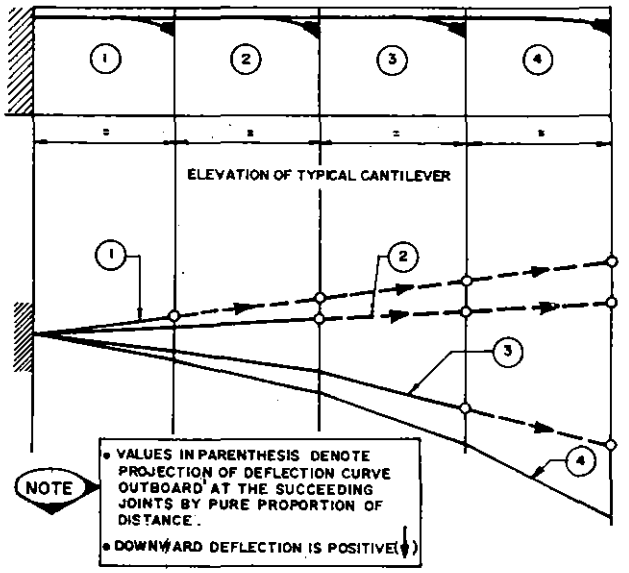
1. Deflection of the travellers themselves under the weight of the concrete segment. This value is given by the manufacturer or may be computed and checked at the site during the first operations.
2. Deflection of the concrete cantilever arms 'during' construction. For 'each' casting of a pair of segments, the weight of the concrete segments and the corresponding cantilever prestress forces impose upon the cantilever a new deflection curve.
3. Deflections of the various cantilever arms 'after' construction and after removal of the travellers (but before continuity is achieved with the other parts of the deck, if continuous).
4. Short- and long-term deflections of the completed structure, including the effect of superimposed dead loads (kerbs, railings, pavements, utilities) and live loads.
5. Short- and long-term pier shortenings and foundation settlements, if any.

Using the data available on concrete properties and foundation conditions, the designer should compute the various deflections mentioned under items 3, 4, and 5 above, assuming the bridge unloaded for foundation settlements. and for long-term concrete deck deflections, and half the design live load for computation of the short-term concrete deck deflections.

The sum of the various deflection values obtained in the successive sections of the deck allows the construction of a 'camber diagram', which should be added algebraically to the 'theoretical' longitudinal profile of the bridge deck to determine, for each cantilever arm, an adequate 'casting-curve'. (As pointed out earlier, this 'theoretical longitudinal

deck profile' is the 'actually required' deck profile with 'reverse' deflections due to all dead loads and final prestress added to it, taking into account, reduction in concrete E value due to long term creep.) This casting curve is the goal towards which construction proceeds during cantilever casting. The essential difficulty is that no absolute coordinates are available in a system where everything changes at each construction stage (transfer of traveller, concrete casting, or cantilever prestressing).

A very simple example may illustrate the solution of the problem of accommodating the deflections described under item 2 above. For simplicity, assume only a four-segment cantilever arm, for which a 'horizontal' longitudinal profile is the required 'theoretical longitudinal deck profile'. See Fig. 37.23.



DUE TO CASTING AND PRESTRESSING SEGMENT No.	VERTICAL DEFLECTIONS (in mm) Joint 1-2	Joint 2-3	Joint 3-4	end 4
1	-5	(-11)*	(-17)*	(-23)*
2	1	5	(9)†	(13)†
3	5	10	20	(30)°
4	8	18	29	49
∴ TOTAL DEFLECTION DUE TO SEGMENT WTS. AND THEIR PRESTRESS.	+9	+22 x	+41 y	+69 z

Fig. 37.23 *'Partial' deflections due to 'girder weight' and 'prestressing' at each construction stage ('partial', because the 'Theoretical longitudinal deck profile, referred to in the text, already includes the creep causing long-term deflections from permanent loads)*

* by proportion of -5 mm at 1–2
† by proportion of 1 mm at 1–2 and 5 mm at 2–3
° by proportion of 10 mm at 2–3 and 20 mm at 3–4

The designer analyses the various deflection curves for each construction step (casting segment or precasting). The typical results are shown in Fig. 37.23. The cumulative deflection curve is immediately obtained together with the camber diagram, Fig. 37.24. The use of the camber diagram for determining the adequate deflection at each construction stage is simple; however, it is much less simple to use in a proper manner in the field, and experienced surveyors have often made mistakes.

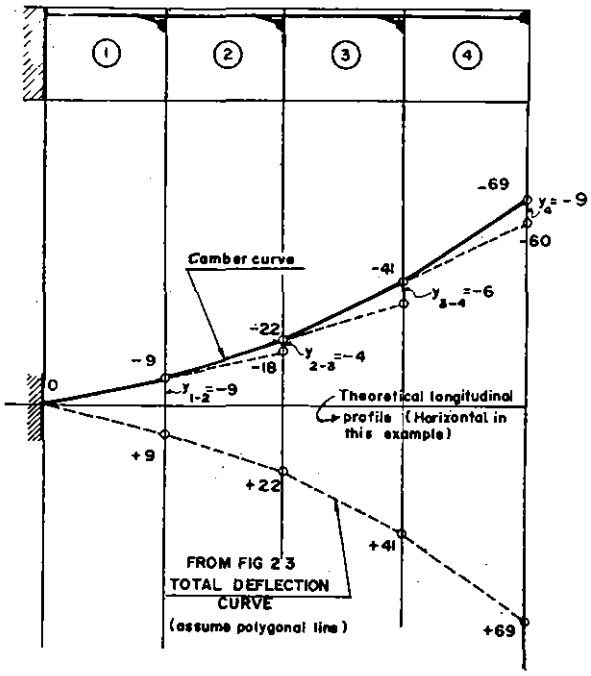
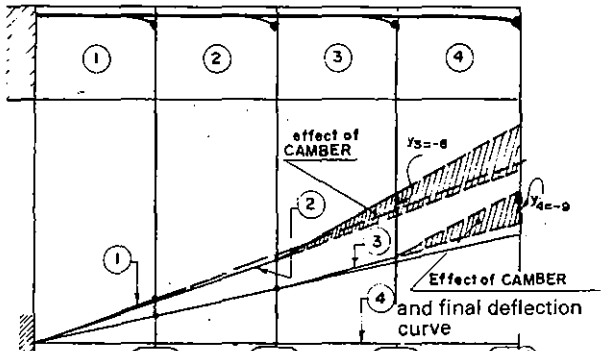


Fig. 37.24 *Cumulative deflection curve and choice of camber (i.e. pre-camber)*

When properly used, the camber diagram allows the determination, at each joint, of the offset values such as y_{1-2}, y_{2-3} and y_{3-4} at each point, which will bring the traveller in the proper position to realize the desired final geometry. The sketch and table in Fig. 37.25 show how to use the camber diagram properly. It is very important to realize that at no construction stage does the profile of the cantilever coincide with either the 'final deflection curve' or the 'camber diagram'. (Profile refers to the soffit-profile.)

The natural tendency would be to build-up the traveller to the required offset to make its nose fall exactly on the camber diagram. The results of this improper procedure are shown in detail in Fig. 37.26. The bridge is built with an undesired double curvature, particularly undesirable toward the end of the cantilever. When the mistake is discovered, it is usually too late to put into effect any remedial measures,



		1–2	2–3	3–4	end 4
SEGMENT 1	CAMBER	-9	(-18)*	(-27)*	(-36)*
	LOADS **	-5	(-11)	(-17)	(-23)
	TOTAL	-14	(-29)	(-44)	(-59)
SEGMENT 2	CAMBER	0	-4	(-8)∓	(-12)∓
	LOADS ***	+1	+5	(+9)	(+13)
	TOTAL	-13	-28	(-43)	(-58)
SEGMENT 3	CAMBER	0	0	-6	(-12)°
	LOADS ***	5	10	20	(+30)
	TOTAL	-8	-18	-29	(-40)
SEGMENT 4	CAMBER	0	0	0	-9
	LOADS ***	+8	+18	+29	+49
	TOTAL	0	0	0	0

Fig. 37.25 *Follow-up of Deflections with proper use of camber diagram*

* by proportion of −9mm at 1–2
∓ by proportion of −4mm at 2–3
o by proportion of −6mm at 3–4
* ... from Fig. 37.23 for respective segment.

because the final shape of a cantilever depends essentially upon the accuracy of the geometry near the piers, where the deck is subjected to the highest moments and where its deflections have the greatest effect at midspan.

37.8 CHARACTERISTICS OF PRECAST SEGMENTS AND MATCH-CAST EPOXY JOINTS

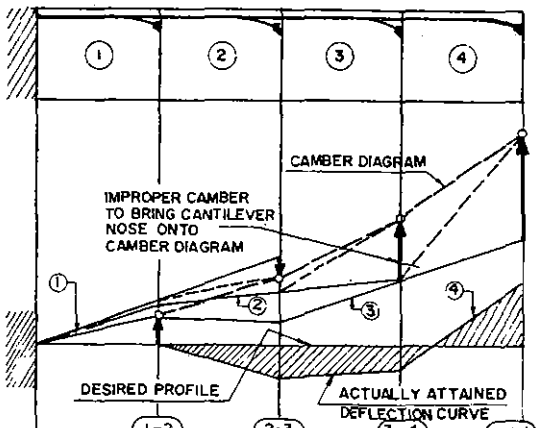
Developed originally to allow a rapid and safe assembly of precast segments at the construction site, the technique of 'match-casting' was progressively refined as experience was gained.

The epoxy resin plays several important roles:

1. During assembly but before hardening

(a) To lubricate the mating surfaces while final positioning takes place.

(b) To compensate for minor imperfections in the match-cast surfaces.



		1–2	2–3	3–4	end 4
SEGMENT 1	CAMBER	-9	(-18)	(-27)	(-36)
	LOADS	-5	(-11)	(-17)	(-23)
	TOTAL	-14	(-29) x_1	(-44)	(-59)
SEGMENT 2	CAMBER	0	$+29-22$ x_1-x = +7	(14)	(21)
	LOADS	1	5	(9)	(13)
	TOTAL	-13	-17	(-21) y_1	(-25)
SEGMENT 3	CAMBER	0	0 $+21-41$ y_1-y =	-20	(-40)
	LOADS	5	10	20	30
	TOTAL	-8	-7	-21	$-35z_1$
SEGMENT 4	CAMBER	0	0	0 $+35-69$ z_1-z =	-34
	LOADS	8	18	29	49
	TOTAL	0	+11	+8	-20

Fig. 37.26 *Follow-up of deflections with improper use of camber diagram (x, y, z from Fig. 37.23)*

2. In the finished structure after hardening

(a) To ensure the watertightness of the joints, especially in the top slab.

(b) To participate in the structural resistance by transmitting compression and shear forces. However, before hardening of the epoxy resin, the joints present no shear resistance whatsoever, because in this state, the epoxy behaves like a perfect lubricant. It is, therefore, necessary to provide shear keys in each web in order to ensure the shear-force transfer between segments. These keys, as well as those situated in the top slab, also allow a very accurate assembly of one segment with the another.

During assembly of the deck, some sort of temporary fixation, either mechanical or by means of prestress bars, allows the placing-equipment (launching girder or crane) to be quickly unloaded without waiting for the cantilever tendons to be stressed.

Figure 37.27 shows how a typical segment can be assembled into the existing structure using a temporary apparatus located on the top and bottom slabs, which is used to create forces F_1 and F_2 which ensure the equilibrium of

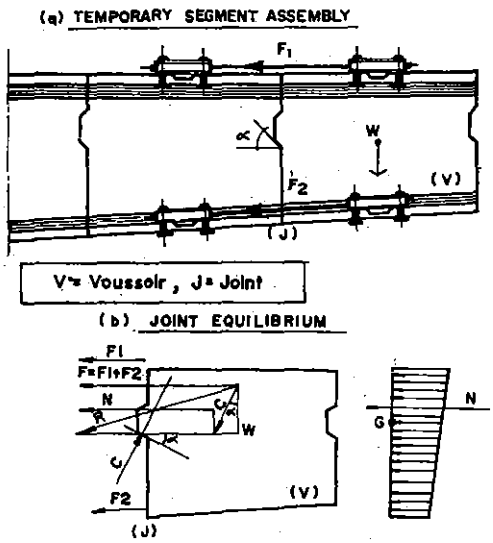


Fig. 37.27 *Temporary assembly of precast segments*

the new segment at the joint.

These two forces F_1 and F_2, combined with the weight W of the segment, give the resultant force R, which is inclined with respect to the joint. Because of the very small coefficient of friction of the epoxy, the vertical shearing component of R can be balanced only by the vertical component of the reaction C which exists normal to the inclined bottom face of the web shear keys. The resultant R is composed, therefore, of the oblique reaction C supported by the shear keys, and a horizontal reaction N, which is responsible for securing the joint.

The axial stress distribution at the joint cross-section differs in this case from what would be obtained by ordinary calculations. It is obvious that V is smaller than F (the sum of forces F_1 and F_2). Let α be the angle of the key support faces with respect to the horizontal: then $F - N = W \cdot \tan\alpha$ and for a typical case of $\tan\alpha = 0.50$, $F - N = W/2$. Consider a segment weighing $W = 50$ tons, temporarily assembled by a prestress force of 100 tons located in the slabs, the axial force reduction is $(F - N) = \frac{W}{2} = \frac{50}{2} =$ 25 tons—that is, 25% of the total applied prestress force.

If the rate of erection of the precast segments is sufficient to ensure the positioning of say four segments before the resin in the first joint has set, then the reduction in the effective axial force in this joint will be $4 \times 25 = 100$ T, which more or less corresponds to loosing one tendon of twelve 1/2 inch diameter strands. The same conclusion would be valid when the permanent prestressing was used to ensure the temporary stability of the cantilever.

This reduction of the effective prestress force should be taken into account while verifying the cantilever resistance and stability. Failure to do so may result in temporary joint opening, which is undesirable although not dangerous for stability.

It is also preferable to choose the intensity and the point of application of the forces F_1 and F_2 in such a manner as to allow the axial force N to be as close as possible to the section centroidal axis, thus ensuring a nearly uniform axial stress distribution over the total depth (and hence a resin-film of constant thickness) during construction.

Structural Importance of Epoxy Resins

The resistance or strength of different cantilevers is ensured by groups of tendons, which may be referred to as 'cantilever-tendons'. These may be straight or curved in profile, and anchored on the various segment faces. The stressing operations remain in the critical path of construction because no new pair of segments can be placed before the previous pair has been stressed to the existing cantilever.

The final group of tendons in continuous structures joins together the different cantilevers and makes the structure continuous. They are anchored either in block-outs in the bottom slab or in the 'fillets' at the 'junction between the top-slab and the webs', after upward deviation towards top slab level.

A bad choice, or improper use, of the epoxy resin can be a critical factor concerning the shear resistance of the joints, and for this reason joints of this type require strict quality control.

In general, the different types of epoxy resins available have final strengths substantially exceeding that of concrete, so they do not constitute a weak point in themselves. Several conditions must be satisfied, however, in order that the resin cures properly:

1. Mixing the constituents in their correct proportions.
2. Eliminating any solvents that have a fatal effect on the properties of the resin.
3. Avoiding any flexibility additives, such as thiokol, that greatly increase the deformability of the epoxy.
4. Mixing and applying the epoxy carefully.

With respect to the last point, the surfaces to be joined must be specially treated if the best results are to be obtained. Comparative tests have shown that sand blasting the concrete surfaces gives the most satisfactory results, the surfaces being kept clean, dry, and free from grease, etc. during placing. In damp or rainy weather alcohol is burnt on the joint surfaces to eliminate surface moisture. The water present in the concrete itself has no detrimental effects on the performance of the resin.

It has also been established, that rapid placing of successive segments has a favorable effect on the properties

of the resin. The additional compressive stress applied to an epoxy joint while still not fully set (i.e., while still under polymerization) when the next segment is prestressed, improves the resin's ultimate mechanical properties.

In addition to the precautions taken to ensure correct curing, one may provide against the risk of bad results by including shear keys in the bottom slab also.

It is preferable that the precast segments are equipped with interlocking keys in the top and bottom slabs and in most of the height of the webs. This configuration of shear keys at regular intervals, improves the behaviour of joints by relieving the epoxy of its structural role.

37.9 FATIGUE IN PRESTRESSED CONCRETE BRIDGES

Basically, prestressed concrete resists dynamic and cyclic loadings very well. Eugene Freyssinet demonstrated this fact sixty years ago. He tested two identical telegraph poles under dynamic loading. One was of reinforced concrete and the other of prestressed concrete; both were designed for the same loading conditions. The reinforced concrete member failed earlier than the prestressed concrete member which sustained the dynamic load for several million cycles.

Fatigue in concrete itself has never been a problem in any known practical structure, because a variation of compressive stress in concrete may be supported indefinitely—so long as it never turns tensile and either the applied load never exceeds about 0.6 of ultimate load or the maximum compressive stress does not exceed about 0.55 of cylinder crushing strength. When reference is made to fatigue in prestressed concrete, it is always inferred that fatigue problems arise in the prestressing steel or conventional reinforcing steel as a result of ingress after the cracking of concrete due either to bending or to shear. If concrete cracking could be avoided in prestressed concrete structures, the fatigue problem would be almost completely eliminated.

Figure 37.28 shows fatigue behaviour of prestressing strands currently used in prestressed concrete structures. The diagram shows the limit of 'stress variation' causing fatigue-failure versus the 'mean stress' in the prestressing steel. For convenience, both values are expressed as a ratio with respect to the ultimate tensile strength. For a steel stress of 60% of the ultimate, the acceptable range of variation is ± 8% of the ultimate for a number of cycles between 10^6 and 10^7. Using for example 270 ksi quality strand, this variation is therefore about ± 22,000 psi, or a 'total' range of about 44,000 psi.

Because dynamic loading on a bridge is of a short-term nature, the concrete modulus is high and the ratio between steel and concrete moduli is of the order of 6. Consequently, the maximum concrete stress in an uncracked

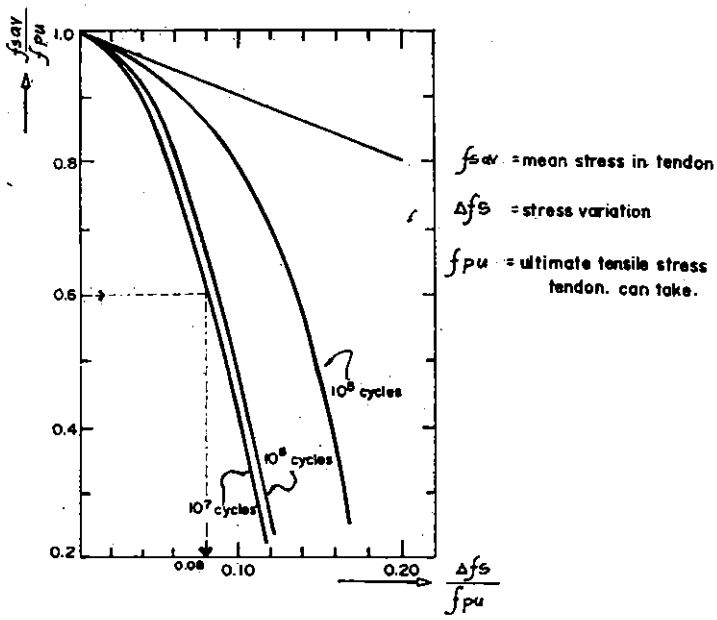


Fig. 37.28 *Fatigue behaviour of prestressing strands*

section that would cause a fatigue failure would have to be about 44,000/6 = 7330 psi (516 kg/cm^2)—a value which is probably nine times the 'stress variation' under design live loads in highway box girder bridges. An uncracked practical prestressed concrete structure is, therefore, completely safe with respect to fatigue, regardless of the magnitude of live loads. A limited amount of cracking, although considered inadvisable from the corrosion point of view, is not critical if kept under control.

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CHAPTER 38

Design and Construction of Cable-stayed Bridge Decks — Some Information*

Present day construction techniques use cable-stays for many types of structures, such as stadia, aircraft hangars, telecommunication towers, large chimneys and cooling towers, electricity pylons and, perhaps the most important, long-span stayed bridges, which are undergoing an impressive and fast development.

38.1 BRIEF HISTORY

The construction of cable-stayed bridges did not really start until the very early years of the 18th century. Though initially expected to develop very rapidly, its development was halted by two serious accidents:

Pryburg bridge, Great Britain, collapsed in 1818 due to oscillations caused by the wind; and the Saale river bridge, Germany, collapsed in 1824 due to overloading by a crowd watching a regatta.

These accidents were no doubt caused by the insufficient technology of the 'stays' and their 'connections'. Composed of bars and chains, their mechanical properties were poor and their cross-section not large enough. During construction, there were no proven methods of applying the correct tension to the stays which were, therefore, ineffective, causing the deck to attain dangerous deformations.

The French scientist Navier, who studied these accidents, recommended suspension bridges rather than stayed, and for about 100 years, only a few stayed bridges were built, of which, Whealing bridge over the Ohio river and the rail bridge over the Niagara, both built by Roebling in the US in 1846 and 1855, may be cited. The aerial ferries built in Marseille and Nantes (France) by Arnodin, in 1903 and 1905, are early examples of pure cable stayed bridges.

In the 1950s, the necessity to rebuild the bridges destroyed during World War II, specially in Germany, together with the progress made in steel construction technology (steel and cables of high mechanical properties, orthotropic slab, etc.) were factors which influenced the revival and development of cable-stayed bridges. Engineers like Dischinger, and others proved the superiority of stayed bridges over suspended bridges for spans of some hundreds of metres.

It is generally admitted that the first modern cable-stayed bridge is the Stromsund bridge, built in Sweden in 1955.

More than 70 other major bridges of this type, with steel or concrete decks, have since then been built or are under design or construction.

38.2 THE EVOLUTION OF THE STAYS

The stays of the first modern bridges consisted of single or multiple arrangements of the following main basic elements: locked-coil wire ropes, twisted strand ropes, bars, strand of twisted wires, and compacted parallel wires. The ultimate tensile strength of these elements was in the range of 1100 to 1500 N/mm^2.

The 'stress variation' in cable-stays may reach high values and the fatigue properties are, therefore, of the utmost importance. Fatigue here is so important because of large stress variation caused in the stays even by the passing of live load on the bridge. Approximately, it can be assumed that the deck is very flexible so that the stress in the stays is nearly directly proportional to the applied load. Moreover, the stays, essentially the direct tension carriers, also suffer bending stress from the bending moment caused by the combined effect of axial tension and self weight 'sag' in them. If this is not 'subdued' (by using damping guides near anchorages), the resulting stresses and, therefore, the stress variation can be even higher. This can enhance the fatigue effect further.

Although some of the above-mentioned elements, specially parallel wire ropes, possessed relatively good fatigue properties, their anchorages were unable to withstand high 'stress amplitudes', and it became necessary to find new types of stays.

Through development and research undertaken basically in Germany, mainly by Prof. F. Leonhardt, a type of stay has been developed which uses parallel wires with ultimate strength of about 1700 N/mm^2, of the same type as those

* The author acknowledges with grateful thanks Engr. P. Xercavins for the information reproduced here.

used for prestressed concrete.

During the 1970s 7-wire strand to a very large extent superseded the wire in post-tensioning technology, and following this, research was directed towards the use of prestressing strand for cable-stays.

Practice has proven the satisfactory dynamic properties of good quality strand with ultimate static strength up to 1900 N/mm^2.

Parallel strand stays have been used for long span bridges, among which two landmarks in this new technology are worth noting, Brotonne Bridge over the Seine river, France, and Rande Bridge near Vigo, Spain.

Numerous other applications of this technology may be envisaged, such as the anchorage of semi-submersible off-shore platforms, and cable stayed roofs similar to the type used for the Olympic Stadium in Munich, designed by German Engineers and for which strand was adopted.

Economy

It is worth pointing out that cable stayed 'concrete-deck' bridges have the potential to outbid the concrete box girders in such 'low' span ranges as 150 to 250 m. The large concrete box girder alternatives have some inherent problems of durability and serviceability which are likely to be magnified with growth in span-length. The large free cantilevers suffer a big time dependent deformation which may take as long as five or seven years to stabilise. Errors in their prediction at design stage lead to a 'break' in profile at the central hinge (commonly preferred by some) and result in serviceability defects which are difficult to rectify. Precast segmental construction also needs additional attention. The first generation voussoirs in use rely on the epoxy adhesive for transfer of shear and impose severely low tolerance on joint geometry, field mixing, curing and application of epoxy compound. Errors can pose grave durability risks. Discontinuity of reinforcement across the joint face leaves little room for control of cracking caused by non-linear temperature distribution which has led to distress of many box girder bridges abroad in the recent past.

Many of these problems could perhaps be cut down with the adoption of cable stayed concrete decks. The biggest advantages are the simplicity of the form and accessibility of the deck and stays to inspection. The low depth simple beam and slab deck of cable stayed bridge does not require the large volume of prestressing steel consumed in concrete box girders, interiors of which are not easily accessible to inspection. In case of damages, repair and restoration can be a costly experience. The cable-stays on the other hand are replaceable by design.

Aerodynamic Stability

It is common knowledge that a cable stayed deck has inherent superiority over suspension bridges in dynamic behaviour. The multiple stay system provides additional system-damping of larger number of cables, 'all with different natural frequencies' and a shallower depth of deck! The resulting improved aerodynamic behaviour of multiple stay cable deck has influenced new thinking on efficient deck geometry. It may be recalled, that streamlined aerofoil profiling of deck geometry was found necessary for aerodynamic stability of suspension bridges. The flat sharp nosed closed box section of Severn, Bosphorous and Humber suspension bridges are typical examples of application of this principle enunciated by Leonhardt in 1952. Later, it was found that an open section like the one adopted by Leonhardt for the cable stayed Knie bridge (1969) at Dusseldorf with two triangular boxes at edges was equally aerodynamically stable. However, for the design of twin rail-cum-road cable stayed bridges over Parana-Palmas and Guazu (1972-77), a dynamic model of the whole bridge was tested. It proved that the multiple stays provided such heavy damping as to wipe out any probability of large amplitude resonance oscillations. The flutter theory and the sectional wind tunnel tests have little applicability in checking the aerodynamic stability of the full multiple stay system deck configuration. This new knowledge has led to a striking simplification of the deck geometry of modern bridges.

Concrete Deck

The multiple stay system lends itself to the use of pre-fabricated concrete decks with ease. The all-reinforced concrete Dnepr bridge in USSR built in 1963, is a pioneering example. The modest 144 m span and 6.3 m wide deck was erected by suspending precast reinforced concrete deck elements from cable stays. A nominal amount of steel was used in deck except at cable connections. Leonhardt's more modern Pasco bridge made full use of the advantages of the multistay system in deck erection. The pre-fabricated deck elements were 24.33 m wide and 8.33 m long. These were erected by a pair of stay-cables located in the segment. An overview of the Pasco bridge design shows that dead load bending moments can be largely neutralised. Normal forces can be significant. Live loads and temperature differential between the steel stays and the deck are the major sources of bending moment in deck. 'Partially prestressed concrete' is the obvious solution for the normal cable stayed concrete deck. The Pasco deck was essentially proportioned by ultimate limit state requirements. Tensile stresses were permitted for full live load and temperature combination. But these were kept low.

Unlike steel bridges, shrinkage and creep effects comprise an important design consideration in concrete decks. Improved knowledge and advanced mathematical

modelling tools have aided modern design greatly. In practice, initial forces introduced in the stays during erection, and allowing for subsequent redistribution of differential moments by the end of erection and in service, makes liberal room for economical design. To avoid creep problems, 'composite deck' with concrete slab on steel girders is rarely used in long span cable-stayed bridges. The Hooghly bridge is an isolated exception. Very long span bridges of all types (beams, cable-stayed and suspension), in modern times, invariably use an all steel orthotropic plate deck. Considerable saving in dead load and steel consumption makes its use almost mandatory. For Hooghly, lack of infrastructure facilities and fabrication experience posed problems and were the main reasons for the isolated exception.

"The construction of spans of up to 400 m with concrete decks (even longer spans with steel decks) using cable-stay techniques, derived from prestressing that enable simple installation and the eventual replacement of cable-stays, have nearly pushed beyond the 500 m limit the field previously reserved exclusively for suspended bridges."

The specifications relative to cable-stayed bridges are numerous since this technique is still relatively recent.

38.3 ANALYSIS (*Brief Note*)

A cable-stayed bridge is a statically indeterminate structure with high degree of redundancy. Analysis of cable stayed bridges considering three dimensional space action is a relatively complex problem but if suitable assumptions are made, the problem may be solved by two dimensional analysis and results can be obtained with reliable accuracy for practical design. Several investigators have used two dimensional approach and many bridges have been designed on that basis and constructed. Undoubtedly the cable-stays display a non-linear behaviour and the non-linearity is due to large displacements, 'catenary-action' of cables (which, therefore, are non-linear) and bending moment-axial force interaction in stays, girders and towers. Hence in strict terms, the principle of superposition is not applicable and consequently stresses and displacements cannot accurately be determined by use of simple influence lines. However, various investigators have studied the non-linearity effect on cable stayed bridges and found that non-linearity is not significant and linear analysis gives satisfactory results particularly if damping guides are used near anchorages. Nevertheless, for very large spans and important structures non-linear analysis must be carried out and its effect studied.

The modern approach to structural analysis can be broadly classified as:

(i) Stiffness (displacement) approach,

(ii) Flexibility (force) approach.

Any computerised analysis procedure generally uses one of the above approaches, depending upon the problem in question and convenience of the analyst. A mixed method of analysis has also been by Podolony and Scalzi. The finite element technique is another method used for the analysis of cable-stayed bridges. The well known 'stiffness' approach seems to be the most commonly employed technique of analysing a cable-stayed bridge. In the stiffness method, following assumptions may be made:

(i) The structural system is idealised as interconnected system of linear elements joined to one another at nodal points only.

(ii) The towers are idealized by beam-elements capable of taking axial deformation as well as bending.

(iii) The stays are assumed as uniaxial truss elements capable of resisting axial force only, not bending. (This, infact is wrong, unless damping moments in the stays caused due to axial tension and 'sag' in them).

(iv) The nodes may be selected at the points of interaction of cables and girders.

(v) Foundations may not be considered in this analysis.

38.4 DEFLECTION CRITERIA

A task committee of the American Society of Civil Engineers has prepared tentative recommendations for cable-stayed bridge structures according to which the static deflection in highway bridges under live load should not exceed 1/500 of the span length between piers i.e. central span, unless static and dynamic analyses indicate that serviceability requirements are met. This static live load deflection of 1/500th of the span length is excess of that allowed by American Association of State Highway and Transportation Officials (AASHTO) specifications. The guideline is generally for short length bridges (for spans of 150 m or less) and if the same specification is adhered to for long span bridges of a flexible nature, it would result in an unduly conservative design. In Germany, bridges are being designed with deflection limit ranging from 1/500 to 1/225 of the span length.

AASHTO recommends that until further experience is gained, the 1/500 limit should be adopted for design.

The deck superstructure can be of steel or concrete. Many of the bridges built around the world have used steel decks but many others have been built using concrete decking and generally a prestressed box girder system. A concrete girder will have more weight per unit length (and hence will need relatively more cable material to support the girder) and heavier section of towers and substructure. Steel deck will undoubtedly be a lighter construction.

Proportions

For an economical design of a cable-stayed bridge the ideal side to central span ratio may be about 0.40 and the tower height in such bridges should be about 1/6 of main span for minimum tension in the cables (and hence for less area of cables), minimum girder moments and minimum base moment in the tower. However, economy can be achieved within a range of 0.35 to 0.45 of side to central span ratio under different span lengths and with different types of cable arrangements within a tower height of 1/4 to 1/8 of central span.

With the increase in allowable deflection limit the total cable area required decreases. Effect of tower height on cost of superstructure is almost linear for a central span range of 200–300 m.

The economical range of main span for a cable-stayed bridge of prestressed concrete box girder type appears to be 200–300 m, and if deflection criterion is relaxed then a cable-stayed bridge with central span of up to 500 m could become competitive, particularly if lighter steel deck is adopted. Steel deck, involving high grade steel and special welding electrodes and technology, may be costlier in certain countries, but this has to be offset against reduced cable steel and propagation of technology.

38.5 OPERATION OF A CABLE STAY

By its structure and its installation, the cable stay is sometimes confused with a prestressing cable.

The comparison between a prestressed concrete bridge built in cantilever and a cable-stayed bridge will enable the fundamental differences to be distinguished (Fig. 38.1).

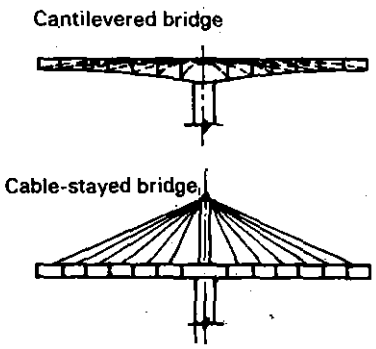


Fig. 38.1 *Segmental bridges*

Both types of bridge use the same construction procedure that consists of building the deck with successive segments from a pier, but the basic difference may be realised from the following considerations:

(a) In the case of a cantilever bridge, the prestressing cable acts as an 'active' force on the deck, latter being very rigid compared to the actual deformation of the cable. Because of this rigidity (of the deck) the cable can be stressed to a high proportion, even independently of the applied dead and live loads. The subsequent stress variations in the cables will result from those undergone by the concrete, in proportion to the moduluses of the two materials, steel and concrete. Under the action of the variable loads, a stress variation of the order of 50 Mpa in the steel and 10 Mpa in the concrete may occur.

(b) In a cable stayed bridge, the cables are the 'stressed elements' of the cross-linked framework formed by the deck, the pylon and the cable stays. They are *passive* suspenders (but adjustable) and should not be stressed to 'values' other than the tensions resulting from the loads applied at the nodes (the deck-stay combined-effect points) as otherwise this will cause deformations that are incompatible with the actual response of the deck (Fig. 38.2).

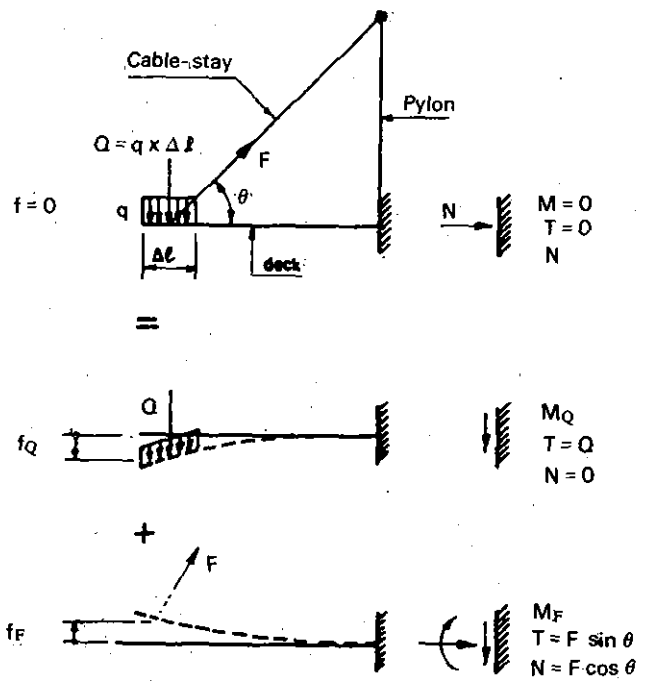


Fig. 38.2 *Cross-linked framework formed by the deck, the pylon and the cable-stay*

In this model, the deck is a 'continuous beam on multiple elastic bearings' latter being the suspenders, i.e., the cable-stays. The aim of the adjustments is to result in the lowest possible moments in the deck, in keeping with the spacing of the stays. Any 'variation of applied load' is immediately expressed in the stays by a 'stress variation' which, in a

preliminary approximation of an infinitely flexible deck, is proportional to variation in the applied load. The 'order of magnitude' may be easily estimated.

The dead load of a concrete-deck cable stay is in the region of about 1500 kg per m^2 of bridge (overall width), or 1,800 kg per m^2 when reduced to the width of the carriageway, whereas the live load may be taken as about 400 kg per m^2 for spans of some 200 metres or so.

Hence, the ratio between the live load and the overall loads is about:

$$\frac{400}{400 + 1800} = 0.182$$

If the maximum stay tension is fixed at 0.45 RG, then, with the characteristic ultimate stress RG = 1800 Mpa (N/mm^2), the stress variation in the stays is about: $0.182 \times 0.45 \times 1800 = 147$ Mpa(N/mm^2) (i.e. 1470 kg/cm^2), which is large.

It can be seen from this calculation that the steel used for stays is subjected to large stress variations. Hence a detailed analysis of the strand (and cable stay) 'fatigue' performance is thus necessary.

As a secondary consequence, the assumption of a stay behaving as a linear 'tension rod' introduces a theoretical non-linearity since the self weight of the tension rod causes a slight deflection (sag) that varies with the tension (Fig. 38.3). The result is a non-linearity of elongation with tension.

In practice, for the analysis of the structures, it is enough to consider a constant apparent modulus owing to very slight variation in this modulus in service.

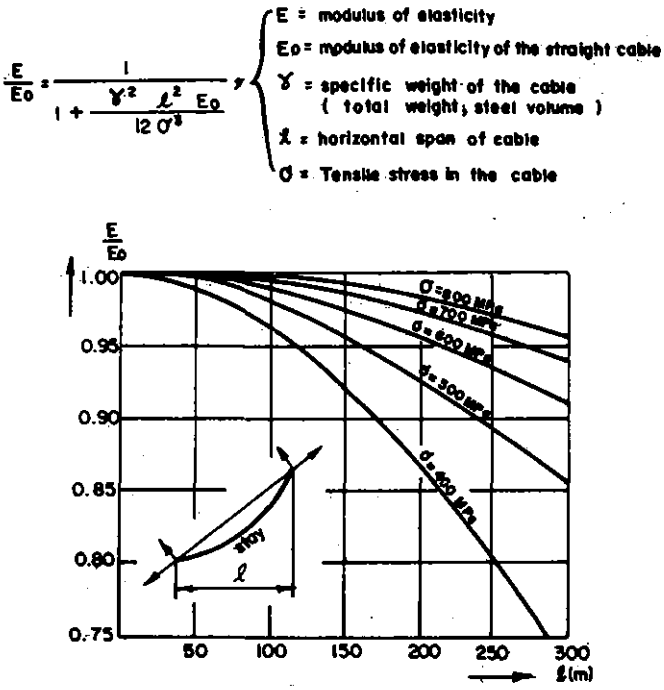


Fig. 38.3 *Effective modulus of elasticity E in terms of horizontal span and tensile stress in the stay cable. (Ernst's formula)*

38.6 ALTERNATE STRESSES IN CABLE-STAYS

Two complementary aspects must be taken into consideration in the analysis of alternate stresses:

(a) The first concerns the working loads to be taken into account for the large number of loading cycles.

(b) The second concerns the bending stresses that superimpose on the simple axial stresses when the cable-stays are subjected to load variations, bending or vibrations (due to wind effects, etc.).

Aspect (a)

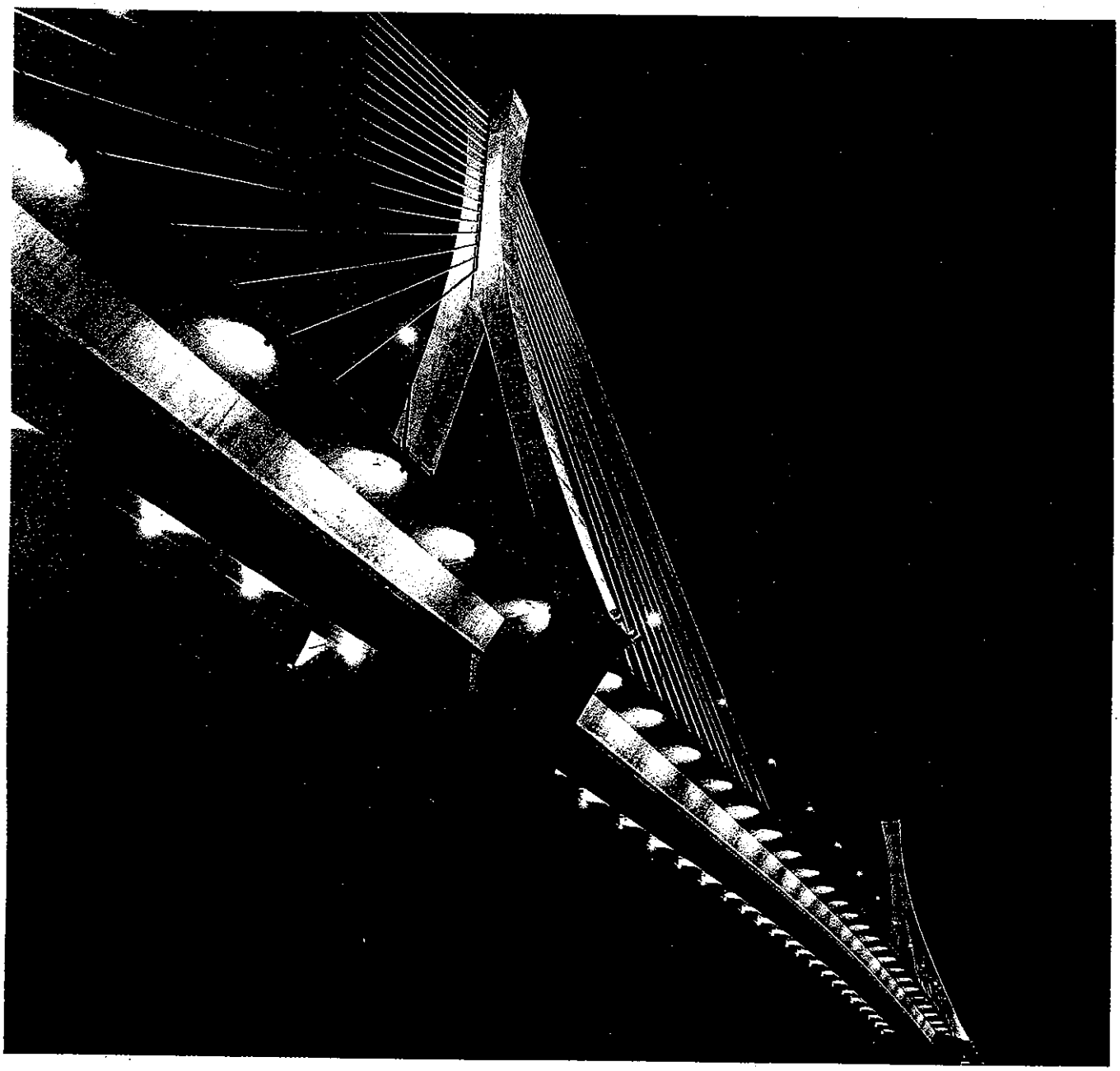
The daily live loads that are applied frequently enough and cause fatigue are much lower than the conventional design live loads. Fatigue is caused only by tens of thousands of loading cycles. The usual value of 2 million cycles in terms of fatigue would require a daily frequency of 54 passages of live load for 100 years. The live loads corresponding to this frequency are far much lower than the design loads, and very often too low to be taken into consideration with regard to fatigue. However, the design loads must be taken into account: for example, if the deck is very light (steel), or for particularly high operating loads, or in the case of short cable-stayed spans, or for railway bridges where the probability of heavy loads is much higher.

Therefore, the intensity of loads to be taken into account depends on the one hand on the number of envisaged cycles and, on the other, on the type of load.

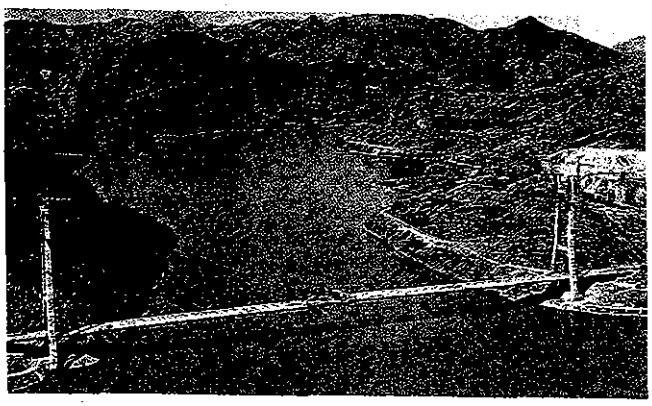
This intensity, related to the log of number of cycles, is expressed by a curve known as 'loading distribution spectrum'. The aspect of this curve may vary considerably, depending on whether it concerns an action related to a natural phenomenon (Fig. 38.4a) or a "determinist" action, for example the loading spectrum of a railway bridge where the loads and frequencies are more clearly defined (Fig. 38.4b). Generally speaking, for road bridges, the spectrum is unknown as it depends upon numerous parameters: its aspect may, however, be taken as that of the spectrum of a natural phenomenon.

In the absence of precise information, assumptions are made that often result in considering a nominal rated load (that tries to take into consideration the different parameters mentioned) and a probability of loading which may have repetition of 2.10^6 cycles.

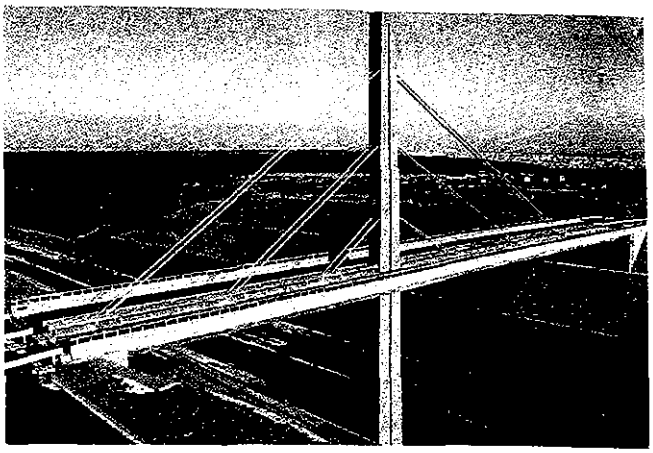
The French working loads are usually rated with a coefficient of 0.5 , whereas the less intense AASHTO live loads can be rated with a coefficient of 0.7. It should be



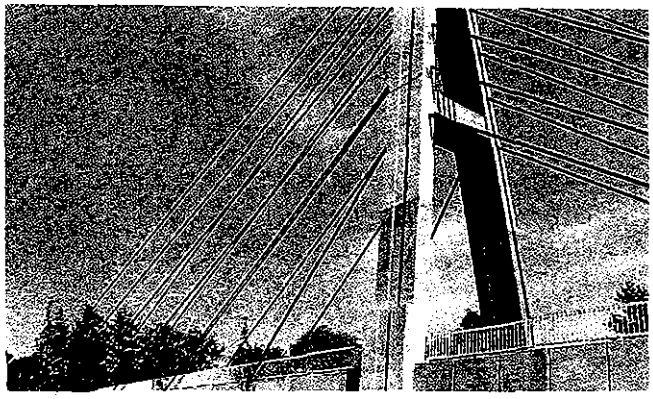
A cable-stayed bridge deck



Barrios de Luna Bridge (Spain)

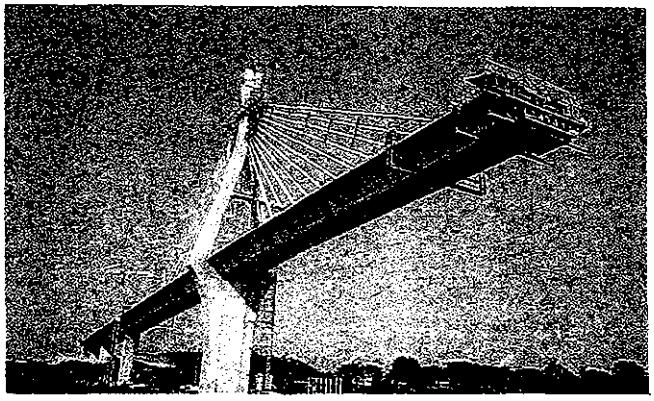


Nishikigaoka Bridge (Japan)

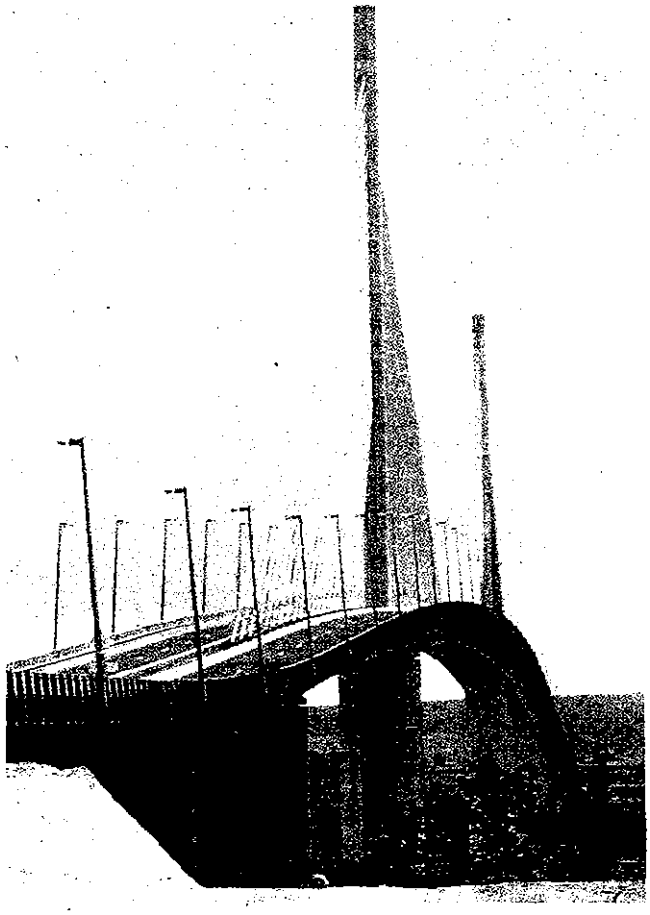


Illhof Footbridge (France)

Meylan Footbridge (France)



Coatzacoalcos Bridge (Mexico)



Brotonne Bridge (France)

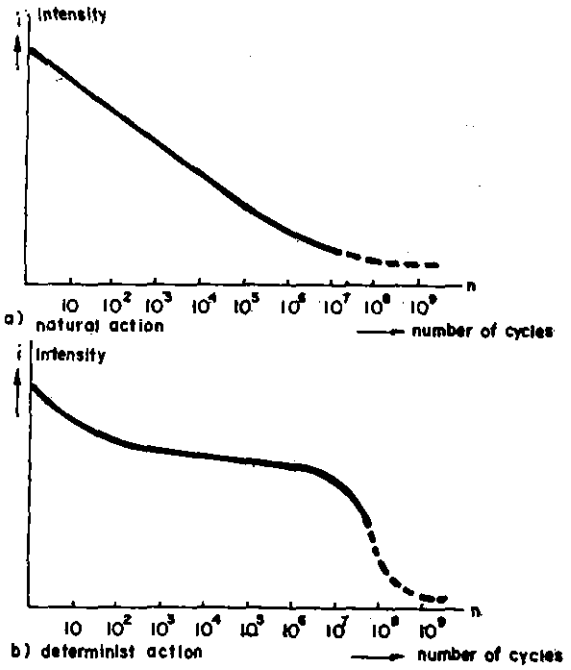


Fig. 38.4 *Spectra of intensity of loading i in terms of the number of cycles n*

noted that the West German DIN (Deutsches Institut fur Normung) standards define a variable coefficient, according to the type of their live load:

- 1.0 for railway loading
- 0.8 for category 30 traffic loading
- 0.5 for category 60 traffic loading

Aspect (b)

This aspect has often been under-estimated and even neglected if reference is made to the specifications of numerous tenders that insist only on the axial stress variations without a mention of bending stresses. The latter are produced:

- Either by the variations of stay tension and distortion, impeded at the fixed anchorages (Fig. 38.5),
- Or by stay vibrations caused by wind effects.

An 'articulation' at the anchorage could theoretically eliminate these stresses, but the relatively intricate and costly arrangement for this, renders this solution hardly applicable in practice.

Even the oscillation due to wind, and the deflection of the deck under applied loads, generate bending stresses which, if added to those previously mentioned and with the axial stress, involve making special arrangements in order to resist fatigue effects.

By analysing the induced bending stresses, the value of the bending moment in the stay may be obtained at any point

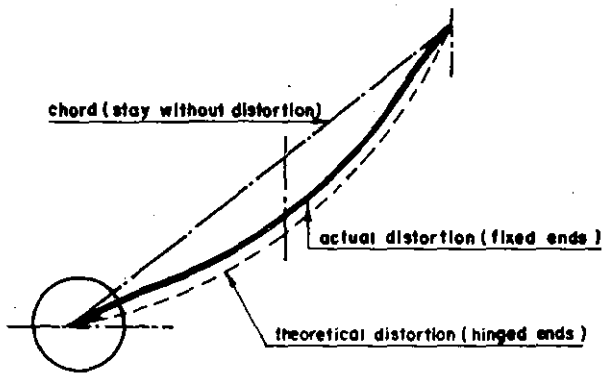


Fig. 38.5 *Distortion of a cable-stay*

x distant from the anchorage, as a function of the angle α between the tangent at the fixed end and the tangent at the distortion (Fig. 38.6).

$$M = \alpha\sqrt{EI \cdot F} \cdot e^{-kx} = \sqrt{EI \cdot \sigma_t A} \cdot e^{-kx}$$

and

$$k = \sqrt{\frac{F}{EI}}$$

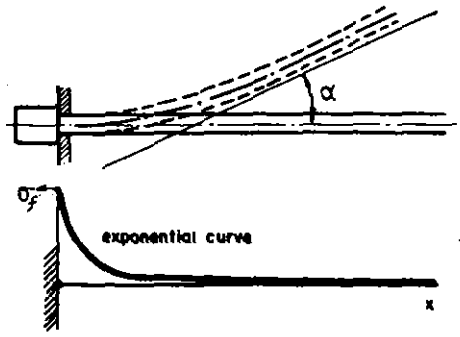


Fig. 38.6 *Distorted stay cable at its extremity and corresponding bending stress in terms of α*

where F: Force in the stay cable
E: modulus of elasticity of steel
I: inertia of the stay cable.*

* The inertia I to be considered is:

— Either n times the inertia of one of the n constituent wires of the strands, when they are independent of each other, which is the case with strands grouted with a flexible product inside a sheath, or individually protected strands.

— Or the inertia of the entire group of constituent wires when they are bounded together by a stiff, adhesive product, e.g., cement grout.

The corresponding 'bending stress' is equal to: $\sigma_f = \dfrac{M}{I} \cdot \dfrac{d}{2}$

giving:

$$\sigma_f = 2\alpha \frac{d}{4i}\sqrt{E\sigma_t}e^{-kx}$$

where

i = radius of gyration corresponding to 'd' ($i = \sqrt{\frac{I}{A}}$)

σ_t = 'axial stress' in the stay

$$k = \sqrt{\frac{F}{EI}} \quad \text{or} \quad k = \frac{1}{i}\sqrt{\frac{\sigma_t}{E}}$$

$$\left(\because \sqrt{\frac{F}{I}} = \sqrt{\frac{\sigma_t \cdot A}{I}}\right)$$

d = diameter of reference**

** The diameter d to be considered is:

— either, the diameter of one constituent wire when the wires of the strands and independent of each other e.g., strands with flexible injected protection.

— or, the diameter of the 'group' of constituent wires when they are bonded together by a stiff, adhesive product, e.g. cement grout.

If one assumes a uniform circular cross-section for the stay, then: $i = d/4$

so that:

$\sigma_f = 2\alpha\sqrt{E\alpha_t} \cdot e^{-kx}$

From its maximum value $(\sigma_f)_{max}$ at the fixed end ($x = 0$) σ_f decreases exponentially:

$\sigma_f = (\sigma_f)_{max}e^{-kx}$

where $(\sigma_f)_{max} = 2\alpha\sqrt{E\alpha_t}$ = 'reference bending stress'.

For angles of deflection of the order of 10^{-2} to 10^{-3} radians and

$$E = 200\ 000 \text{ MPa and}$$
$$\sigma_t = 500 \text{ MPa, we get}$$
$$(\sigma_f)_{max} = 20 \text{ MPa for } \alpha = 10^{-3} \text{ radian, and}$$
$$(\sigma_f)_{max} = 200 \text{ MPa for } \alpha = 10^{-2} \text{ radian.}$$

Thus the bending stress 'variation' for restrained rotations at the fixed end may be of the same order of magnitude as the axial stress 'variations'. Taking this observation into account, it is recommended that adequate steps be taken to reduce to a minimum the bending stresses at the anchorage position.

Local reinforcement of the stay by means of a tube of appropriate thickness, bonded to the wires, is one of several solutions. Another method, both economical and effective, which is used on Freyssinet stay cables, consists in passing the cable through a guide situated at a distance l from the anchorage. The reduction in bending stress is quite considerable, once the distance is sufficiently great, as compared with the design diameter d. Figure 38.7 values of reduction coefficients β_A (A = anchorage) and β_G (G = guide).

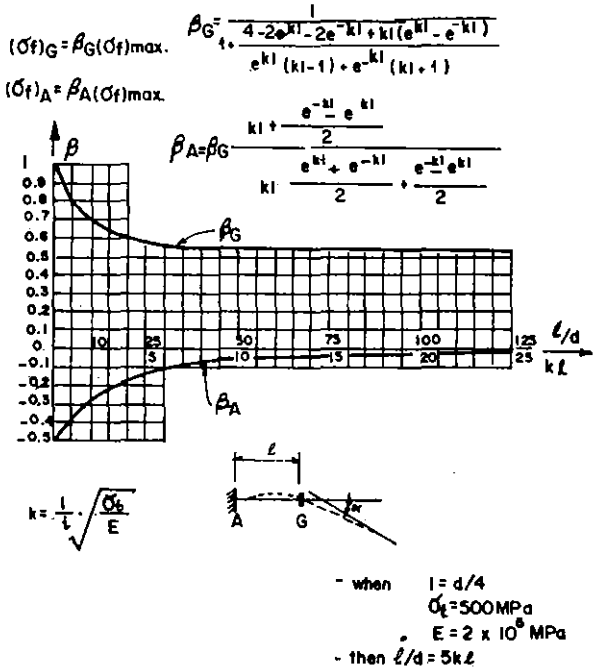


Fig. 38.7 *Reduction coefficients β_A and β_G for bending stress σ_f*

For a ratio $l/d = 30$, the bending stress at the anchorage position becomes only 10% of the reference bending stress $(\sigma_f)_{max}$ and the reduction of stress at the guide position is approx. 50%.

Figure 38.8 shows two cases of bending stress distribution:

(i) The first is that of a stay composed of wires bonded together (e.g. by rigid grout) where d may be taken as being equal to the external dia. 0.20 m and where a guide is placed at 2.00 m from the anchorage. In this case $l/d = 10$ and the residual value of the bending stress at the guide is about 69% of the reference bending stress. This value could be further reduced by local strengthening of the protector tube.

(ii) The second case is that of a stay composed of independent strands (i.e. with flexible protection), where d may be taken as being equal to the diameter of a constituent wire (0.005 m for a 15 mm dia. strand), and where the guide is placed at 2.00 m from the anchorage.

In this case $l/d = 400$, and the bending stress at the guide position is 51% of the reference bending stress, which may be considerably reduced by using a guide of adequate length and nature: the peak of the curve is flattened out and the residual values is very low.

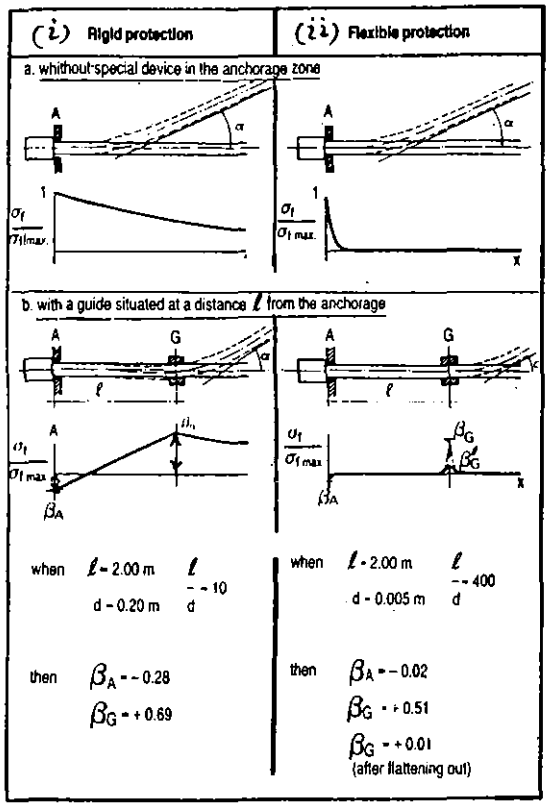


Fig. 38.8 *Bending stress distribution according to the protection type*

This brief analysis shows that, without special precautions, the bending stresses are far from negligible and that it is necessary to provide devices capable of reducing them considerably

38.7 CABLE STAY RELIABILITY

Cable-stays, the principal elements of cable stayed bridges, must present a total operational guarantee through their design and type. It is for this reason that the protection against corrosion must be particularly well designed and carefully executed.

Furthermore, to be in a position to easily and rapidly repair a cable stay damaged by a traffic accident that is never absolutely unavoidable, it is strongly advised to have cable-stays that can be replaced without impairing the structure of either the pylon or the deck.

38.8 GENERAL CABLE STAY ARRANGEMENT

The technique of modern cable-stays, derived from that of prestressing cables, also used construction procedures inherited from prestressed concrete and particularly the cantilever construction method, as indicated earlier.

In this method of construction, the distribution of the cable-stays is consistent with the progression, connected with the length of segments, and this determines the strength required of the cable stay tendons. The length of deck supported by a cable stay system is in the order of 5 to 10 metres for concrete decks and 10 to 20 metres for steel decks. The load capacity of each cable stay can be between 200 tonnes and 1000 tonnes, or even more, during the operational phase. A cable stay generally supports between 50 m^2 and 250 m^2 of concrete deck and 100 m^2 to 500 m^2 of steel deck.

The 'arrangement' of the cable-stays determines the design of both the pylon and the deck.

In the transverse section of the bridge the staying may be formed by a single axial layer or two lateral layers, depending upon the carriageway layout and the width of the bridge (Figs. 38.9 and 38.10, respectively):

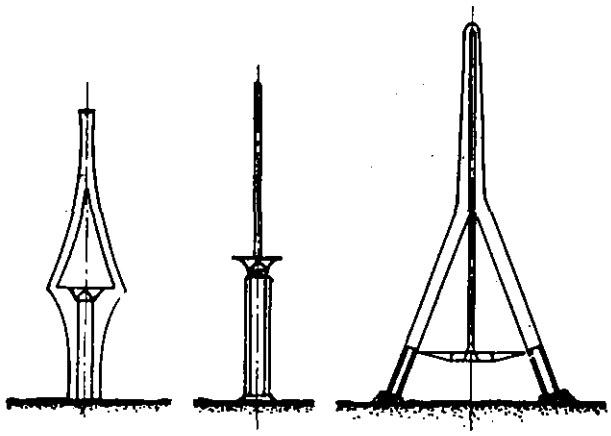


Fig. 38.9 *Transverse arrangement of pylons... One axial layer of stays*

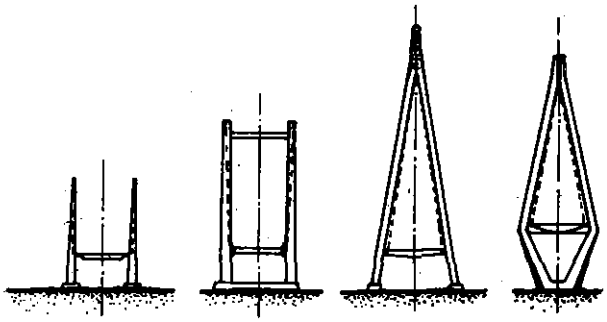


Fig. 38.10 *Transverse arrangement of pylons... Two lateral layers of stays*

- Two layers (or two planes) of stays are recommended:

— when the deck has only two lanes, so as not to separate them by the otherwise single axial set of stays, hence avoiding traffic jams in the event of an accident
— when the deck carries and odd number of lanes
— when the bridge is very wide (so that two-plane pick-up is better).

- In the other cases, and particularly for carriageways with two lanes in each direction, the solution consisting of an axial layer is possible:
  — A central, i.e. single (axial) layer implies a box-girder deck (Fig. 38.11) in order to withstand twisting effects due to asymmetrical load variations. In this case, the pylon may be either a single mast (BROTONNE Bridge) or an inverted-Y (COATZACOALCOS Bridge) with the cable-stays anchored at the top, the splayed legs absorbing transverse forces when imposed by local conditions (wind, earthquake, etc.).

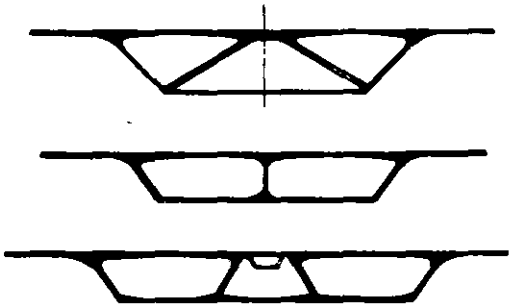


Fig. 38.11 *Cross-section of the deck for one axial layer of stays*

- Two lateral layers result in a layout with two longitudinal bearing systems at right angles to the layers in the form of lattice or plate girders for steel bridges or ribs in the case of a concrete deck, the two bearing systems being connected together (Fig. 38.12).

The pylon in this case has:
— either two independent masts, that may or may not be braced at or near the top, depending upon transverse stresses
— or the form of an inverted-Y or A, the cable-stays in this case being anchored in the vertical portion of the Y or along the inclined portions of the A.

Longitudinally, three typical layouts are encountered (Fig. 38.13):
- fan type
- harp type
- mixed type

The choice of the system is derived from a compromise between a purely economical pursuit and an aesthetic pursuit.

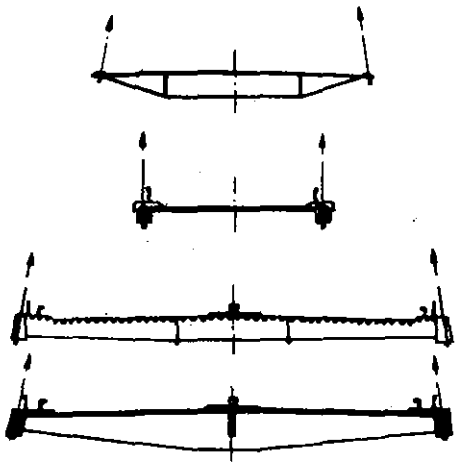


Fig. 38.12 *Cross-section of the deck. Two lateral layers*

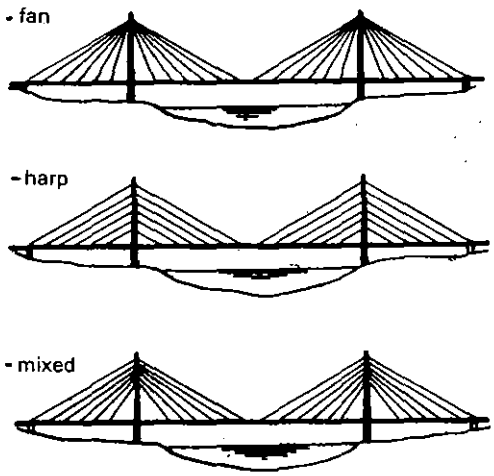


Fig. 38.13 *Longitudinal arrangements*

The fan type solution is more aesthetic and is, as a rule, the most economical for a pylon slenderness ratio $h/l \leq 0.30$ (Fig. 38.14), because for an equal pylon height, the average inclination of the cable-stays is obviously greater in fan system, and the average stress in the cable-stays is, therefore, lower. They are, however, longer but with a more favourable anchorage cost repercussion. But, as the cable-stays converge towards a single point at the pylon top, the arrangement becomes very complex and any subsequent stay replacement is difficult.

It is for this reason that a 'mixed' solution is often adopted, by spreading the anchorages along the mast according to their dimensions, closer to the top.

The structural response of the pylon varies, depending on the chosen solution: the fan-type solution increasing buckling problems obviously (greater effective strut length),

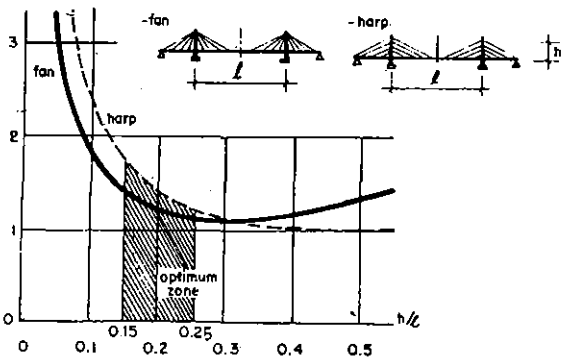


Fig. 38.14 *Compared weight of stay cables in terms of slenderness*

the harp-solution increases the bending moments. This has to be understood very clearly.

Arrangement at the Pylon

The transfer of forces from the cable-stays to the pylon may be achieved by three different arrangements:

(a) A 'saddle', permitting the continuity of the cable stay (Fig. 38.15)
(b) The crossing of rectilinear cable-stays 'inside' the pylon (Fig. 38.16)
(c) 'Relay' devices incorporated into the pylon, a relay connecting the upper anchorages of associated cable-stays (Figs. 38.17 and 38.18).

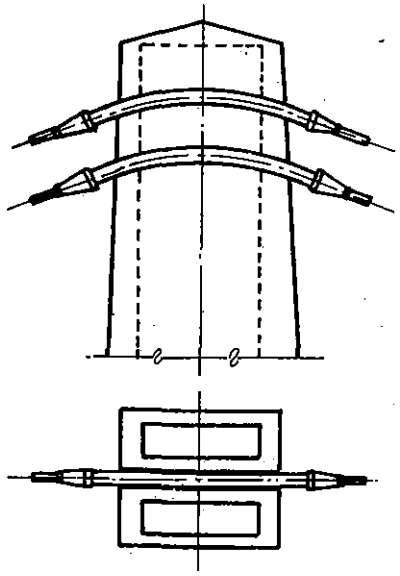


Fig. 38.15 *Saddle*

(a) The merits of the saddle are continuity and the elimination of upper anchorage but the delicate sequencing of the strands at installation results in preference being given in certain cases to systems with upper and lower anchorage that ensure improved control to fatigue endurance uniformity. The eventual 'replacement' of cable-stays is more difficult with saddle arrangement than in the other solutions. A longitudinal retaining system must be incorporated with the saddle because of the asymmetry of forces (Fig. 38.15).
(b) The crossing of rectilinear cable-stays inside the pylon is a simple solution for the installation and eventual replacement of a cable stay. It does not

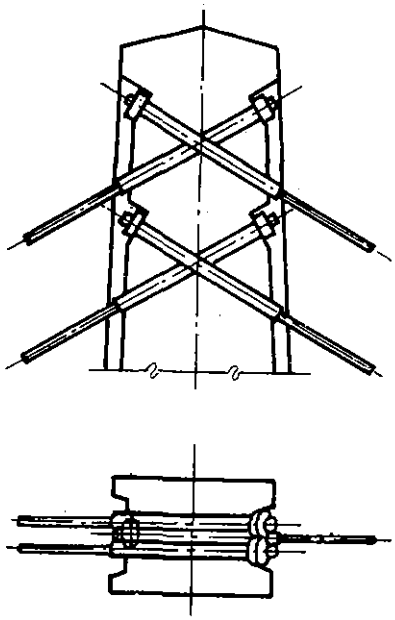


Fig. 38.16 *Crossing of stay cables*

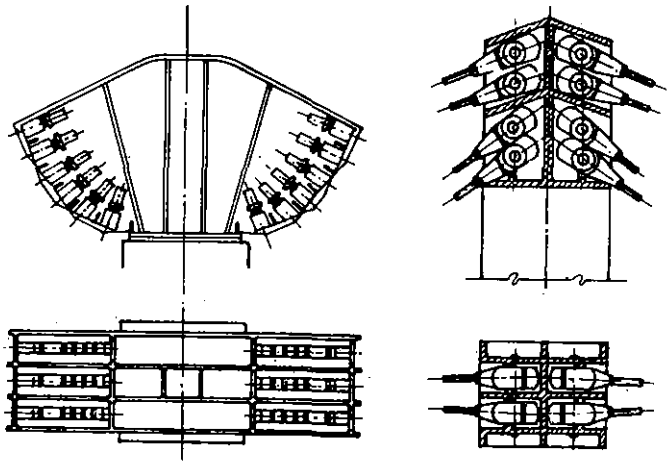


Fig. 38.17 *Incorporated relay device. Grouped anchorages (FAN system)*

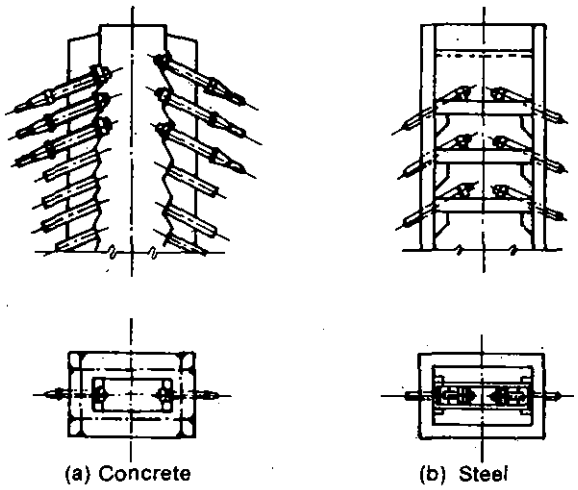


Fig. 38.18 *Incorporated relay device. Distributed anchorages*

permit a symmetrical arrangement of the cable-stays on either side of the pylon (a simple layer on one side and a divided layer on the other), and it also causes twisting moments in the pylon by couple alternation (two opposite cable-stays cannot cross over in the same plane), if an asymmetric solution is adopted (Fig. 38.16).

(c) There are many types of 'relays' that can be incorporated into the pylon, as for example:
- the concrete of the pylon itself compressed by cables or short prestressed bars [Fig. 38.18(a)].
- a coupling (connecting the two anchorages) consisting of steel sections or prestressed concrete [Fig. 38.18(b)], a multiple connecting box.
- a special saddle connected to the upper ends of the cable-stays by 'couplers'.

Arrangement of Stays at Deck Level

The first cable stayed bridges comprised relatively of 'few and rather widely spaced apart stays'. The cantilever construction method, the desire to limit cable stay tendon capacity, and also the possibility of 'rapidly' being able to replace a cable-stay without causing excessive stresses in the deck, resulted in bridge designers adopting a 'closely spaced' cable stay arrangement.

Depending on the type of cable stay chosen (a central plane layer of stays or two lateral planes or layers of stays) access to the anchorage will be either from inside the box-girder or externally by means of mobile platforms.

Furthermore, certain special arrangements must be made depending on the case:

- special protection of the cable stay over a certain height from the carriageway
- incorporation of special vibration-damping system

38.9 DIFFERENT TYPES OF CABLE-STAYS

The first cable-stays used the same cables as for suspension bridges, particularly locked-coil cables. The latter consists of stranded cables with an outer layer of Z-shaped wires that are interlaced to form a locked unit. These cables were factory made including the anchorages which are formed by pouring metal into a socket between the flared ends of wires. The yield point fluctuating between 1100 MPa and 1500 MPa, depending on the elements employed. The modulus of elasticity is in the neighbourhood of 150,000 MPa (1.5×10^6 kg/cm^2) Fig. 38.19.

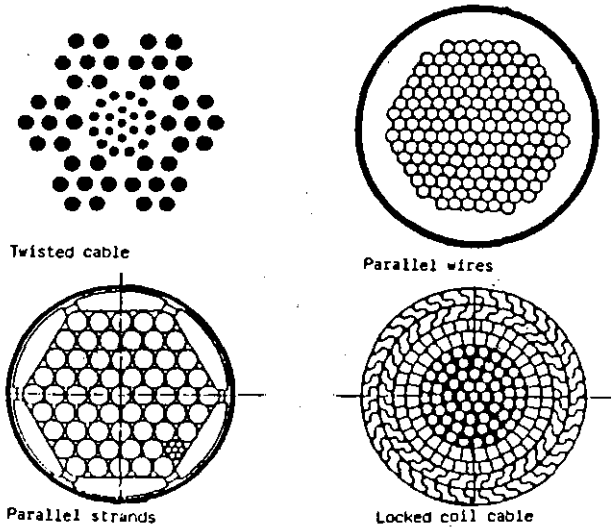


Fig. 38.19 *Different types of cable-stays*

The first cable-stayed bridges were mainly of steel, the stress 'variations' — as is the case with these decks — were high, and the locked-coil cables were not entirely satisfactory.

Parallel wire cables, evolved from prestressing techniques, were then proposed and used after having successfully undergone fatigue tests.

During the end of the sixties and throughout the seventies 7-wire strands were widely developed for prestressing and for this reason the research work undertaken on cable-stays resulted in the use of strands with a very high yield point (1860 MPa) and excellent endurance qualities and a modulus of elasticity similar to that of the wires.

BROTONNE Bridge was one of the first large cables stay bridges to use strands. It was followed by RANDE Bridge in Spain, and COATZACOALCOS Bridge in Mexico.

38.10 THE FREYSSINET CABLE STAY

Freyssinet stays consist of a bundle of parallel strands of 15 mm (0.6″) dia., of the same type as currently used for prestressed concrete. For this application, however, the strand must be properly selected for its fatigue strength. Generally, good quality steel will have a fatigue performance which makes it suitable for this purpose.

Over the free length, the strands are enclosed in a tube, generally of polyethylene, which allows the threading and grouting of the stay. A spiral of steel wire inside the polyethylene sheath ensures that proper grout cover is provided around the bundle of strands (Fig. 38.20).

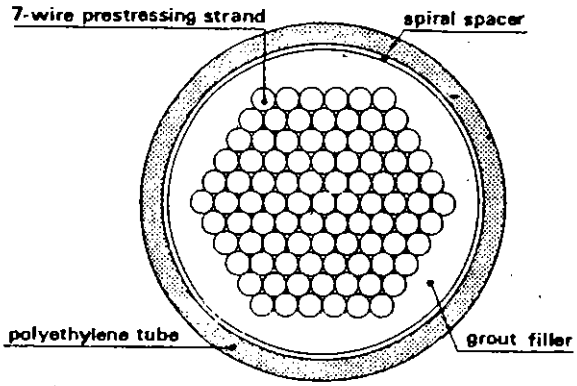


Fig. 38.20 *Section of a Freyssinet cable stay*

The cable-ends are locked into anchorage devices which have been specially designed for this purpose, incorporating technological features which have proven their security and economy within the limits of prestressed concrete techniques.

The strands are individually anchored in the blocks by means of jaws or swaged sleeves, depending on stressing and installation procedures.

Other components of the anchorage devices are:

- a trumpet, which guides the strands to the anchor block with the appropriate angle and spacing.
- a trumpet extension (heavy steel pipe), the length and gauge of which have been selected in order to reduce the range of stresses transmitted to the anchorage.
- a light pipe used to extend the stay anchorage device for structural reasons.
- a neoprene damper is generally inserted between the light steel pipe and the structure in order to restrict wind-induced oscillation in the stays.

The stay anchor exists in two versions: fixed and adjustable. The adjustable type is provided with an outer thread which allows fine adjustment of the stay tension by turning a collar. Both versions can be used at the upper or lower end of the cable stay.

The Cable Stay Comprises of 3 Types of Zones (Fig. 38.21):

(a) The free length zone.
(b) The transition zones (between the anchorages and the free length zone, particularly concerned by the problems of flexural fatigue).
(c) The anchorages themselves.

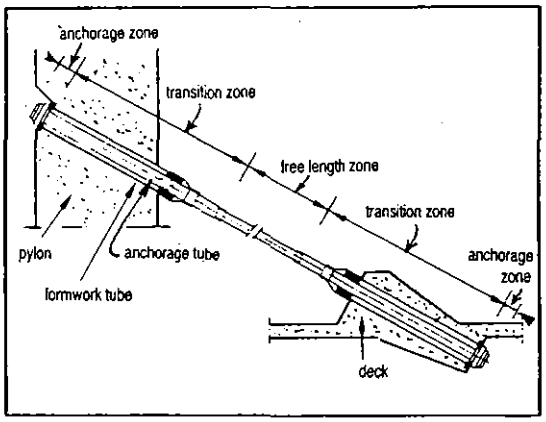


Fig. 38.21 *General layout of the Freyssinet cable-stay*

(a) *The free length zone* consists of a bundle of 15 mm (nominal) diameter parallel strands, number of strands in the bundle depending on capacity required. (See ahead for 'Protection of Stay' for purpose of protecting the stay.)

(b) *The transition zone* comprises essentially of:
- an 'anchorage-tube' connected to the anchorage, of equal or greater length than the formwork-tube, permitting sliding (and hence the complete stay-structure independence) to allow for re-stressing, adjusting or destressing if necessary, both during construction and during the service life of the bridge.
- a 'guide' placed at the end of the anchorage tube. Its functions are:
  - to reduce bending stresses in the cable at the anchorage outlet and at the guide outlet.
  - to reduce the dimension of the cable as it issues from the structure.
  - finally, to act as a barrier between the transition zone and the free length zone in the case where the types of protection are different in the two zones (for example: flexible in the transition zone, rigid in the free length zone).

The type and the dimensions of the guide are such that bending stress variations are rendered negligible compared to the axial tensile stresses in a cable-stay with flexible protection, and to within controlled limits (by means of appropriate dimensioning of the anchorage-tube) in the case

of rigid protection (as explained earlier in some detail).

(c) *The anchorage itself* is the essential element, being naturally the most sensitive part with regard to fatigue endurance in the absence of special precautions.

The design of anchorages for cable-stays is based on the following ideas:

- use of prestressing techniques by anchoring the strands with jaws specially designed for their fatigue endurance,
- arranging the anchorage and the transition zone so as to obtain the perfect distribution of strands, eliminating 'contact' between the strands. (These 'contacts' are responsible for the group effect that reduces the fatigue strength of a bundle compared to that of its components.) This is made possible:
  - by the use of strands, each having high yield point and fatigue endurance qualities, hence a limited number of strands to obtain a given force and the possibility to sequence these strands,
  - by the miniaturization of jaws (made possible by using high-performance materials to manufacture the anchorage), allowing a reduction of the centre to centre distance of strands, and, therefore, of the deviations.

Protection of Stay

This is an essential operation to guarantee a dependable service life. In most of the existing modern cable stay bridges, the protection consists of injecting cement grout, a technique derived directly from prestressing.

Through experiences on numerous nuclear containment vessels and cable-stay bridge sites, special equipment complying to the requirements of international specifications related to grouting have been developed. This has not prevented the continuation of research work to reduce grouting operations on site to a strict minimum. This is in fact the case for cable stays with 'flexible' protection, where the free length zone consists of self-protected sheathed strands and where grouting is reduced to the transition and anchorage zones.

A 'flexible' product of the epoxy pitch, grease, wax, etc., ... type is then used.

For the protective elements a choice between the different protection systems (Fig. 38.22) may be classified as follows:

- rigid type protection, for example:
  - * bare strands in a steel sheath injected with cement grout.
  - * same arrangement, but in a high density polyethylene sheath (PEHD)
- flexible type protection, for example:
  - * bare strands in a PEHD sheath grouted with epoxy pitch, grease or wax
  - * individually sheathed strands (in PEHD) grouped together either by hose clamps and placed in an eventual outer sheath, or by a spirally wound band.

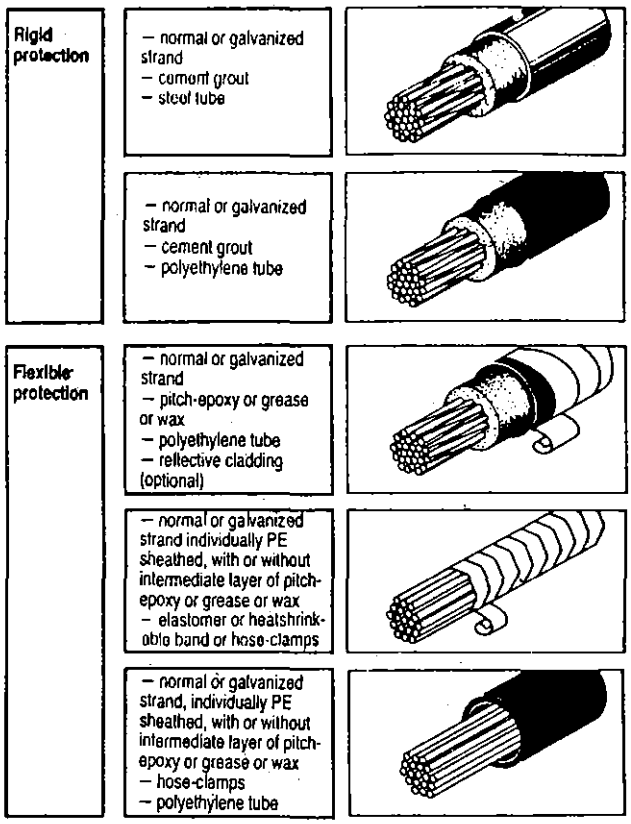


Fig. 38.22 *Various types of protection*

Depending on the environment, a 'reflective coating' may also be applied to the protecting surface to avoid radiation problems.

Durability of the cables is a major concern for the stay cables exposed to ambient atmosphere. Extensive use of highly stressed steel tendons all over the world in bridges even at marine locations discounts inhibition, but leaves little room for complacence.. Bare ropes, protected by mere painting or by mere galvanising may be vulnerable to corrosion. In the Kohlbrandt cable stayed bridge in Germany, all stay cables of locked coil ropes had to be replaced in 1979 after about five years of service due to corrosion damages.

A more effective protective system for parallel wire cables comprises two successive barriers—the first one is the PE tube sheathing and the second is cement grout injected into the annular space round the wire bundle.

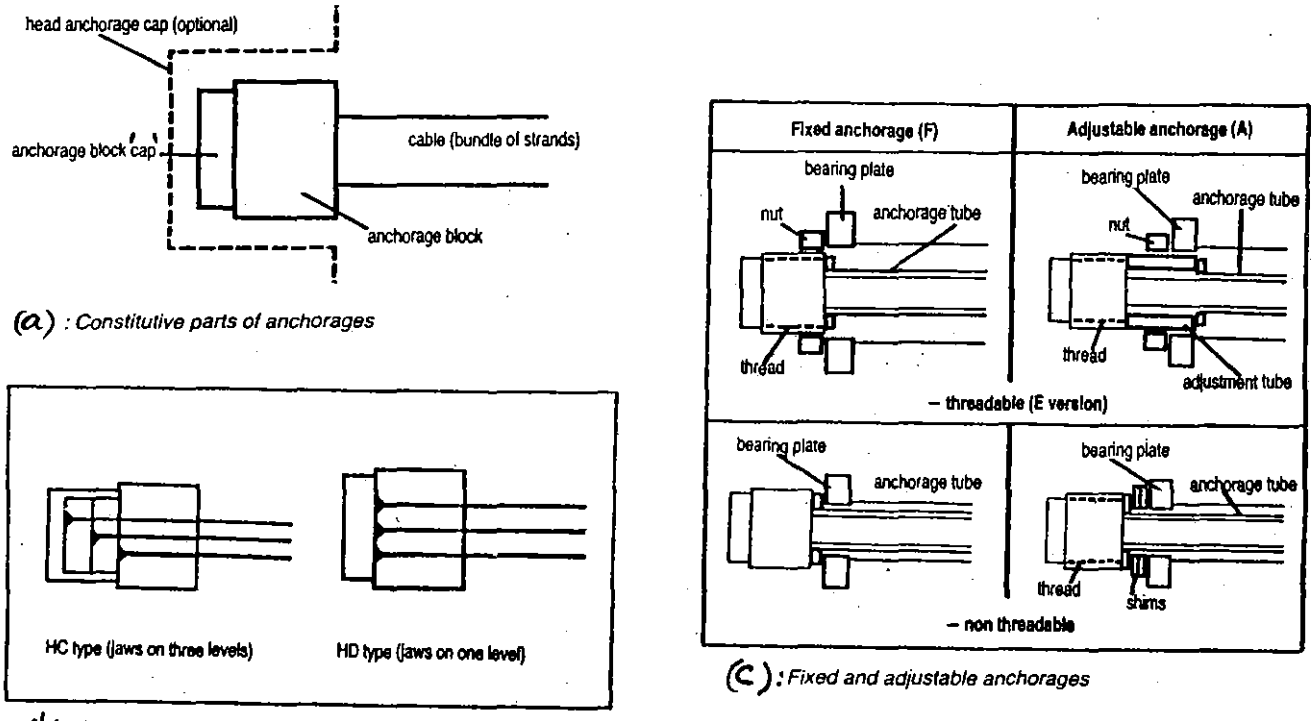
Grout continuity and no damage to the duct is ensured by rigid controls on grouting pressure and grouting sequence. An epoxy enriched cement is used. In some cases like Brotonne, France, or Mainbruck, Hochst, West Germany, steel tubes have been used for sheathing. Rigid steel tubes have some advantages in contributing to stiffness. PE tubes, however, offer comparative ease in handling and are generally preferred. The typical thickness of a 160 mm PE tube is 8.6 mm and the duct is not sensitive to handling, flexing and lateral pressures. The corrosion protection effectiveness of the cement grout is reported to have been tested by BBR for the Schiller-Strasse bridge, after 12 years of service. A high pH value of pore water in dense grout, rust free cable-surface, and no degradation in mechanical properties of the PE tubes were confirmed.

Going by the above evidence, the durability life of the stay cables can be predicted as 'high'. However, a modern system of deck and tower anchoring does provide for easy replacement of stays to cater for accidental damages.

38.11 BBR STAY AND ANCHORAGE

BBR-HIAM (high amplitude) parallel wire cables, *with their own anchorage system*, have been developed to perfection by the combined efforts of the famous Swiss firm BBR (well known for its prestressing system) and the German consulting firm Leonhardt and Andra. The individual wires forming a parallel wire stay bundle are no different from the familiar 7 mm wires used for prestressing. The wires are bundled into a hexagonal section shape and enclosed by a PE (polyethylene) duct. A strand, 10 mm in diameter, is wound round the wires as a spacer. The wires are provided with a variant of BBR buttonhead anchorage. The buttonheads can provide adequate static strength. But, in order to statisfy high fatigue strength demands, the anchorage system has some special features. The wires in the bundle are flared out inside a steel casing before being held at a steel plate by usual BBR buttonheads. The conical space inside the casing is filled with a mixture of small steel pellets of 1.5–2.0 mm diameter and epoxy resin.

The cold cured mass in epoxy matrix, which carries the tendon load by arching action developed in the pellets, is claimed to ensure anchorage efficiency with superior fatigue. In the molten-zinc-filled-sockets of ropes, used earlier, the steel was damaged due to high temperature. A degradation of elastic modulus and a stress range as low as 100 MPa resulted. The Freyssinet and the HiAm BBR anchorages, in sharp contrast, do not reduce either the static or fatigue strength of the wires. Conclusive evidence have been derived from static and dynamic tests in which failure has invariably occurred in free length of wires. The static and dynamic properties of the stay cables are, therefore, determined by the mechanical properties of the constituent wires.



(a) : *Constitutive parts of anchorages*

(b) : *Types of anchorages for Freyssinet stay cables*

(c) : *Fixed and adjustable anchorages*

Fig. 38.23 *Constitutive parts of the anchorage*

38.12 FREYSSINET STAY ANCHORAGE

Two ranges of anchorages are proposed by the *Freyssinet System:* the HC anchorage and the HD anchorage (Fig. 38.23).

- The HC anchorage features a staggered arrangement of the jaws in 3 levels, resulting in the virtually complete suppression of deviations. The resistance of the anchorage to fatigue is practically that obtained with a mono-strand anchored in the same conditions. (The HC anchorage was used for the COATZACOALCOS Bridge in Mexico.)
- The HD anchorage, which is based on the same principles, comprises of a single level of jaws. It has a slightly large diameter than the HC anchorage, but is more rapidly installed.

For both types of anchorage, a cap covers the rear-end to ensure impermeability when grouting. A second cap filled with grease may be added to protect the emergent extremity of the anchorage.

Finally, to allow for re-stressing operations, two adjustable load transfer systems are proposed:

- by means of screwing a threaded collar on a threaded adjustment tube, placed in front of the anchorage
- by means of inserting shims, in a simplified version.

In both systems, the anchorage block at the extremity of the cable stay where re-stressing is carried out is itself threaded, enabling the whole anchorage assembly to be pulled i.e. stressed without direct re-stressing of the previously stressed and eventually grouted strands.

The E version (slip-on) allows the front of the anchorage without its threaded collar to pass through the bearing plate for the installation of prefabricated cable-stays.

38.13 STRENGTH OF THE STAYS

In typical cable stayed structures the stays are subjected to large load variations so that their static strength as well as their fatigue strength become critical, as indicated earlier.

Stays may be assembled from strand conforming to current standards for prestressed concrete strand, but additional fatigue requirements have to be considered.

Static Strength

The stays and their anchorage devices have been so conceived that their ultimate static strength can be considered equal to the sum of the guaranteed ultimate loads of the strands of which they are composed. The ultimate strength for instance of FREYSSINET cable-stays is obtained through bond and wedge action.

Table 38.1 gives some physical characteristics and ultimate loads for the FREYSSINET range of stays. It has been prepared for use with strand conforming to various standards and corresponds to cables containing the full number of strands which the anchor can hold. It is obvious that a stay may contain a lesser number of strands than its anchors allow for.

Operating Stress in the Cable Stay

The 'operating stresses' of cable-stays are traditionally limited to a fraction of their characteristic ultimate tensile stress, the normally imposed values vary between 40–45% of the characteristic ultimate tensile stress.

This percentage is lower than that considered in standard structural steel work because of the enhanced fatigue problems here, mentioned previously. Also, higher stresses would give rise to ductile cable-stays (increased deflection), ***and it is necessary to have a sufficient margin for the transfer of cable stay reactions to the neigbouring cable-stays in the event of changing a stay.***

A closer analysis of deformability and fatigue would enable in many cases, particularly for road bridges with concrete decks, to increase the maximum stress. It is not correct to go beyond 50% of the characteristic ultimate tensile stress, as long as this value has not been ratified by

Table 38.1 *Some characteristics and ultimate load for the Freyssinet stay size**

Type of Strand		Single Strand	Freyssinet Stay Size*						
			19H15	27H15	**37H15**	48H15	**61H15**	75H5	**91H15**
15.24 mm (0.6″).	A	140	2660	3780	5180	6720	8540	10500	12740
grade 270. to	Afpu	260.7	4953	7039	9646	12514	15902	19552	23724
ASTM A-416	g	1.102	20.94	29.75	40.77	52.90	67.22	82.65	100.28
15.7 mm super	A	150	2050	4050	5550	7200	9150	11250	13650
strand to	Afpu	265	5035	7155	9805	12720	16165	19875	24115
Euronorm EU 138	g	1.19	22.42	31.86	43.66	56.64	71.98	88.50	107.38
15.2 mm. class III	A	139	2641	3753	5143	6672	8479	10425	12649
to French Circular	Afpu	252.1	4790	6807	9328	12100	15378	18907	22941
N° 73–175	g	1.091	20.73	29.46	40.37	52.37	66.55	81.82	99.28

A—nominal cross section in mm^2; Afpu—ultimate force of the element in kN; g—nominal weight in kg/m.

* Recommended standard sizes are indicated in bold characters. The other sizes can be envisaged when quantities justify.

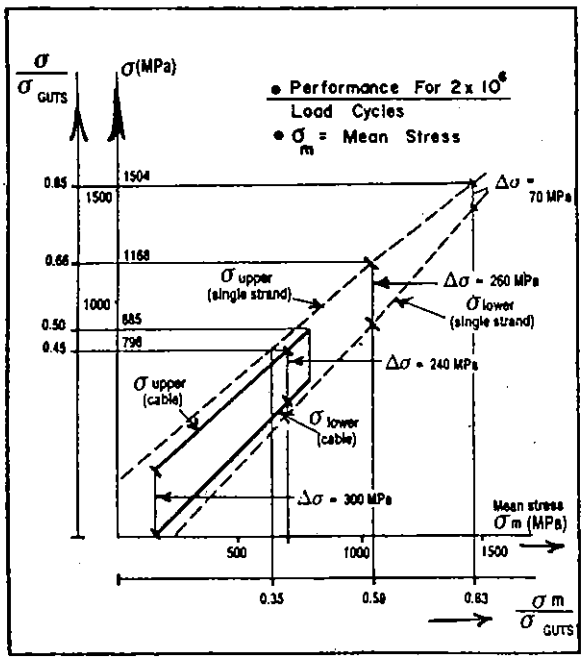
fatigue experience. (For strands, the characteristic ultimate tensile stress is sometimes converted to their 'breaking load'—this stress times the actual steel section area of the strand, and referred to as such.)

'Upper Stress' Limited by Fatigue Endurance Performance

The endurance of the constituent strands should be close to that of the total cable stay itself and the 'group effect' should be relatively low.

The 'anchorage performance' also depends on the 'strand performance', and the latter itself is the subject of specifications with relation to fatigue.

Generally, the steel suppliers draw-up 'WÖHLER curves' and 'SMITH diagrams', latter deduced from a group of WÖHLER curves (Fig. 38.24).



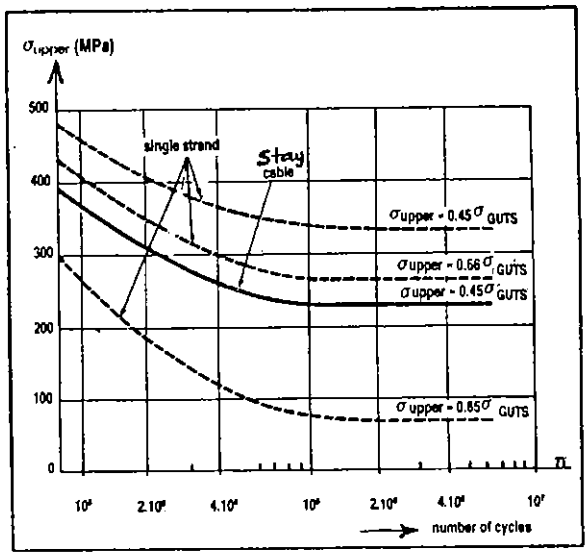
(a) Compared Smith's Diagrams for a Single Strand Dia. 15mm & a Stay Cable. Number of Cycles 2x 10⁶

Fig. 38.24 *(a) Smith's diagrams*

The performance of stay-anchorages is also presented in similar forms from unit tests and tests on complete anchorages.

An interesting point concerning anchorage performances is their ability to support a stress variation $\Delta\sigma$ of 240 Mpa for 2 million cycles with a maximum stress of 0.45 σ_{GUTS}, i.e. 0.45 RG. (σ_{GUTS} or RG, as defined earlier, is the characteristic ultimate tensile stress.)

From this experimental data, the upper stress (expressed as a fraction of σ_{GUTS}) may be plotted against the ratio 'q' of extreme stresses (Fig. 38.25) where



(b) Wöhler's Curves for a Single Strand Dia. 15mm and a Stay Cable with Freyssinet anchorages

- σ_{GUTS} = RG = *Characteristic or Guranteed ultimate tensile strength*
- σ = *stress in tendon*
- $\Delta\sigma$ = *stress variation*
- $\sigma_{upper} = \sigma_{maximum}$ (or $\sigma_{max.}$)
- $\sigma_{lower} = \sigma_{minimum}$ (or $\sigma_{min.}$)

Fig. 38.24 *(b) Wöhler's curves*

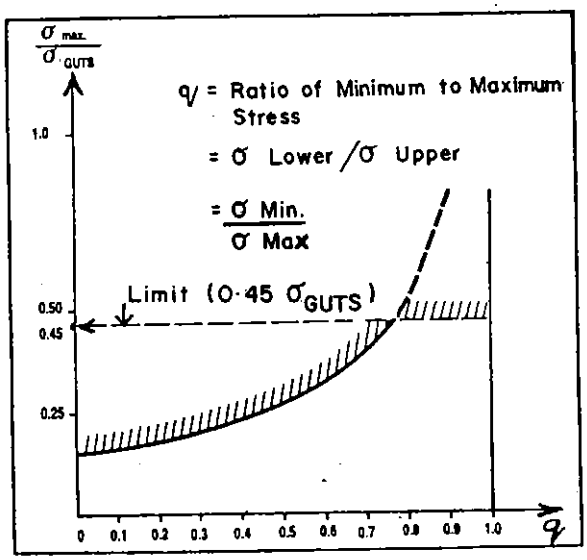


Fig. 38.25 *(Maximum stress in terms of σ_{GUTS}) vs. 'q'*

$$q = \frac{\sigma_{min}(\text{or } \sigma_{lower})}{\sigma_{max}(\text{or } \sigma_{upper})}$$

and also against the stress variation $\Delta\sigma$ (Fig. 38.26) where $\Delta\sigma = (\sigma_{max} - \sigma_{min})$ or $(\sigma_{upper} - \sigma_{lower})$.

These diagrams have been limited to the maximum stress (σ_{upper} or σ_{max}.) of 0.45 RG (i.e. $0.45\sigma_{GUTS}$). They do not include a safety factor, this being left to the designer and state of art applicable to the concerned site and country.

The margin for this safety factor may be obtained either by reducing the 'amplitude' of allowable stress (for a realistic amplitude of variable loads causing fatigue), or by increasing the variable loads, or partially on both.

38.14 SPECIFICATION OF STRANDS FOR CABLE-STAYS

The strand used for stay cables must comply with the requirements of one of the following standards:

- ASTM A 416-80
- EURONORM 138-79
- B.S. 5896: 1980
- any other equivalent standard

They must in all events comply with clause relative to fatigue in B.S. 5896: 1980, or in EURONORM 138-79, that is to say ability to withstand 2.10^6 cycles with:
$\sigma_{upper} = 0.80$ of actual breaking stress and $\Delta\sigma = 195$ MPa.

Standard	ASTM A 416-80	EURONORM 138-79	BS 5896: 1980
Strand designation	• 600 strand Grade 270	Strand Fe-7S. 1770-15.7-Relax 2 EU 138/6	BS 5896/3 Super strand-1770 15.7-relax 2
Nominal diameter (mm)	15.24	15.70	15.70
Nominal tensile strength (MPa)	1862	1770	1770
Nominal steel area (mm^2)	140	150	150
Nominal weight (kg/m)	1.102	1.180	1.180
Specified characteristic breaking load (kN)	260.7	265.0	265.0
Specified characteristic 0.1% proof load (kN)		225.0	225.0
Load at 1% elongation (kN)	221.5	233.0	233.0
Minimum elongation at max. load (%)	3.5	3.5	3.5
for L = (mm)	610	500	500

Dimensions and properties of strands used for Freyssinet stay cables (L = length of test piece).

38.15 SPECIFICATION OF ANCHORAGES FOR CABLE-STAYS

All the component elements of for instance Freyssinet anchorages are designed to ultimate strength requirements.

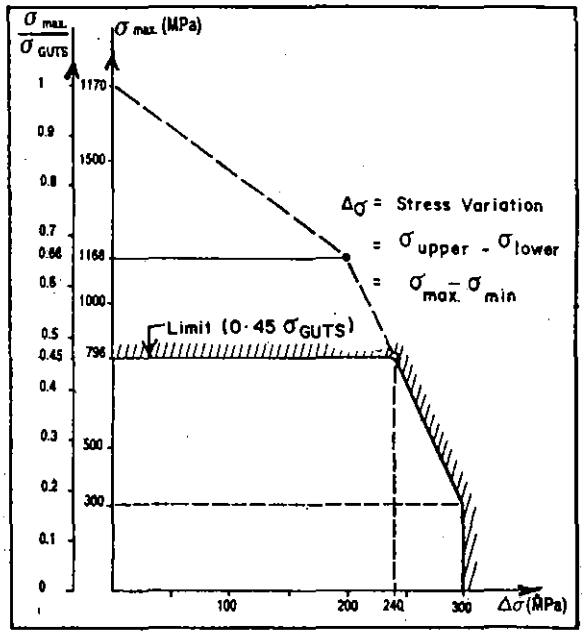


Fig. 38.26 *(Maximum stress in terms of σ_{GUTS}) vs. ($\Delta\sigma$)*

High strength alloy steels are used and are treated in order to give the following mechanical characteristics:

—*Nut and adjusting tube*—steel complying with French standard NF A 35-552

- Breaking strength : 600 MPa min.
- 0.2% proof stress : 400 MPa
- Elongation at rupture : 17% min.
- Charpy U Notch (KCU) : 50 J/sq. cm. min.

—*Anchorage blocks*—steel complying with French standard NF A 35-552

- Breaking strength : 800 MPa min.
- 0.2% proof stress : 600 MPa min.
- Charpy U Notch (KCU) : 60 J/sq. cm. min.

—*Bearing plate*—steel complying with French standard NF A 35-501

- Breaking strength : 490 MPa min.
- 0.2% proof stress : 305 MPa
- Charpy V Notch (KCV) at 0°C : 35 J/sq. cm. min.

38.16 SPECIFICATION OF PROTECTIVE MATERIALS FOR CABLE-STAYS

Stay cables are protected by means of either rigid or flexible materials depending on the design.

The rigid materials are:

—Steel tube complying for example with French standard NF A 49-112 or equivalent. The steel employed has the following characteristics:

- Breaking strength : 540 MPa min.

- 0.2% Proof stress : 300 MPa
- Elongation at rupture : 22% min.

—cement grout for injection of the stays, of the same type as that used for prestressing tendons.

The grout must therefore comply with the specifications and grouting procedures of the latter.

The flexible materials are:

—high density polyethylene tube or coating of the same nature on individually protected strands.

The polyethylene (HDPE) complies for instance with French standard NF T 54–072 class 5.3, or with the equivalent ISO specifications 161/1 and 3607, which require:

- Yield stress : 19 MPa min.
- Elongation at rupture : 350% min.
- Carbon black content : 2.3% ± 0.3%

The internal parts of for instance the Freyssinet anchorage are protected by injecting flexible epoxy resin having the following characteristics:

- Viscosity at 20°C : 1800 mPas (cPs)
- Pot life (for 500 g) : 5 1/2 hr, at 20°C
  : 1 1/2 hr, at 50°C

38.17 SUPPLY OF 'STAY' STEEL

The stay cables may be supplied to site fully fabricated from the factory. ***The sequence of operations for instance for a BBR stay comprises the following:*** 'Individual wires are cut to exact length required, on a special cutting bench. The wires are coated with oil, pulled through a die to form the bundle, wrapped by the spacer strands, and then the anchor head is fitted at one end by button heading. The PE duct is drawn over the bundle and the duct joints are welded. The second anchor head is fitted. Epoxy resin and steel pellets are filled in the end cone held vertically and cured with programmed tempering. The cables are then reeled on bobbins, for shipping to site. Even long hauls from factory to site are not uncommon. For the Novi Sad bridge, the cables were delivered from Switzerland (Stahlton AG) to the Yugoslavian site, by rail, on 48 steel drums, the largest one weighing 18 tonnes.

The wires used for stay cables are of the familiar prestressing steel grade of S 1500–1600 for instance. An equivalent elastic modulus of 1.9×10^6 kg/cm^2 normally ensures adequate stiffness and good dynamic behaviour. ***In view of the critical importance of the mechanical properties, elaborate testing programme, especially for 'stress range' $\Delta\sigma$ has been a significant feature of the cable stayed bridges. A few salient results for two bridges, Novi Sad bridge, Yugoslavia, and Pasco, USA, are given below:***

"... From the 916 coils of wires used for the bridge over the Danube, Novi Sad, 210 specimens were tested. They have been randomly chosen over the whole lot and tested at 5 ranges of stress amplitudes, each having 42 samples. The number of cycles of failure was checked up to a maximum of 2×10^6 cycles. From the results, arranged according to statistical methods, the necessary inference (Wöhler field) of the lot could be drawn".

"... Design criteria are in general taken from the values related to 2×10^6 load cycles assuming that this level represents the material's infinite fatigue resistance. In the case of Novi Sad bridge the intrinsic fatigue strength (variation) of the lot was found to be 378 MPa taking into account a rupture probability of 5% on an upper stress level of 735 MPa. The fatigue strength of the cables can be calculated as 240 MPa with a factor of safety of 1.6. The fatigue damage of these cables subjected to 2×10^6 load cycles in this range will not be more than 5%, even in the case of accidental-overloading of the cables".

"... For the Pasco-Kennewick Bridge two cables with 83 wires each have been tested (with excellent results) at the University of Texas at Austin. The cables were subjected to an elevated temperature creep test, a room temperature creep test, a fatigue test (after 2 million cycles between 0.25 and 0.45 of breaking load, fatigue (stress variation) 160 MPa, no wire broke) and an ultimate strength test upon completion of the fatigue test: a maximum load of 4636 KN was developed corresponding to 1.04 of σ_{GUTS}. Only the first wire broken indicated a fatigue crack, no other wires showed any evidence of fatigue damage. The distribution of wire failures along the cable represents the statistical distribution very closely".

38.18 INSTALLATION OF A CABLE STAY

These may be installed in their final position, by individual threading of strands or by threading the whole bundle of strands, raising the 'cable' which may be prepared on the deck or fabricated/assembled in the factory.

However, in the case of factor prefabrication, the use of a flexible sheath is absolutely necessary because of transportation, and special precautions must be taken to limit the consequences of winding-stresses on a large diameter sheath.

The ease with which certain types of stays (e.g., FREYSSINET cable-stays) are installed renders their factory prefabrication unprofitable, as transportation is more expensive due to the special transport and handling equipment required.

The stay cables may be *installed in many different ways* while in each case providing the same performance guarantees.

They may be installed in their final position equally well:

- either by threading the strands one by one or in a complete bundle;
- or by raising the cable complete with anchorages, prefabricated on site or in the factory.

In fact, the ease with which the Freyssinet stay cables are installed renders it unnecessary, in most cases, to have recourse to prefabrication in the factory — a costly operation due to the special handling and transport equipment which it requires, particularly for the larger range of stays.

The choice between complete in-situ assembly or on-site prefabrication depends on the method and the speed at which the deck is constructed.

The different construction sites in which Freyssinet International has participated provide evidence of the flexibility of the techniques proposed.

'On-site' Fabrication and Installation (Fig. 38.27)

This method has proved to be the most economical due to the limited amount of special equipment required. For example, in the case of a stay with polyethylene sheath, the general scheme of operations is as follows:

1. Installing of the anchorages and anchorage tubes in their final position.
2. Welding together 12 m lengths of polyethylene tube on the deck or in a prefabrication area.
3. Moving into position and lifting the upper end of the pre-assembled sheath by means of a lifting system placed on the pylon.
4. Bringing the lower end as near as possible to the lower anchorage.
5. Threading the first strands through the upper anchorage block, the sheath and the lower anchorage using a single-strand pushing machine.
6. Stressing the first strand to a predetermined value using a monostrand jack. This operation can be carried out either at the adjustable or the fixed anchorage position.
7. Successive threading and stressing of all the strands, as for the first, while checking forces and displacements.
8. Control and adjustment stressing of the complete stay.
9. Protecting the anchorage zones (grouting).
10. Corrosion protecting the free length (grouting, for example).

This method applies to straight stay cables, without deviations, having upper and lower anchorages. In the case of a stay with a deviation saddle the procedure is a little different although based on the same principles of threading with a pushing machine as for Coatzacoalcos bridge.

Prefabrication and Installation of a Cable Stay

Prefabrication theoretically offers the advantages of a more easily controlled production and a reduction of work-time on site.

On the other hand, factory prefabrication is virtually limited to PE sheathed low capacity cables with a small-diameter flexible-sheath so as to limit the transport equipment to reasonable dimensions and weight to comply with drum diameters (latter preferably not greater than 20-25 times the diameter of the cable stay sheath of PE).

For pre-assembled cable stays all the components of the stay, excluding the permanent corrosion protection of the free length, are assembled and controlled in the plant. The main operations are:

Stay assembly

The prestressing steel is cut to length, with temperature corrections as required, within very close tolerances. The polyethylene sheath is pulled through the bundle and welded, and the anchors and other accessories assembled at each end. The anchorage resin compound is poured into the anchorages. The complete stays are finally coiled in reels, reusable or disposable.

Shipment

Loaded reels containing one or more cables may be shipped by truck, rail or ocean freight. Dependent upon the cable sizes and lengths, individual reels may weigh up to about 20 tons.

Erection

At the jobsite the reels are installed in 'dispensers' with automatic breaks. An erection module installed on the top of the pylon, which includes a winch and a 'messenger cable' provided with hangers, are used to bring the cable stay in place. Specially designed tensioning equipment is used to place the anchorages in position and further to adjust to the desired tension.

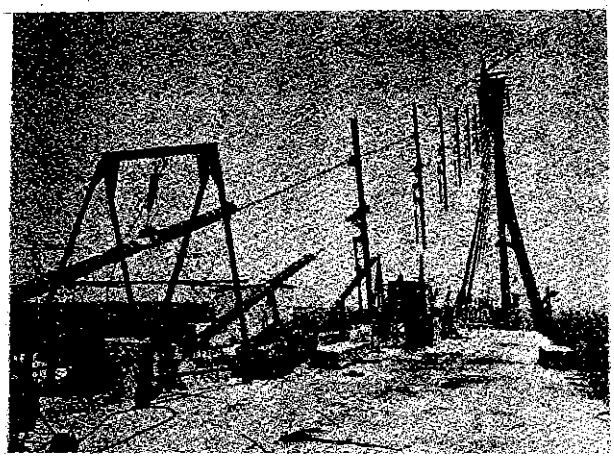
Corrosion protection of the free length

This operation is carried out, as in the case of assembled in-situ stays, by injecting cement grout in the sheath.

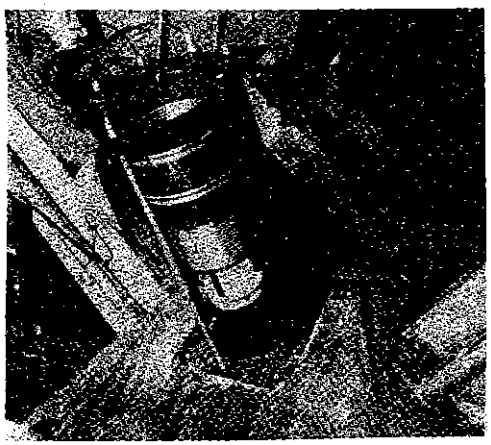
Stressing; Adjusting; Detensioning of Stays

The design of Freyssinet stay cable anchorages offers the same advantages for stressing as those used for prestressed concrete. The cable passes through a centre hole jack and the slack is taken up without the necessity of cutting to a precise length (Fig. 38.28).

Moreover, in parallel-strand cable stays, due to the sequencing of the parallel strands, stressing may be



Installation of stay cables



Installation of a lower anchorage (Freyssinet)

Threading and classifying the strands through the tube (Freyssinet)

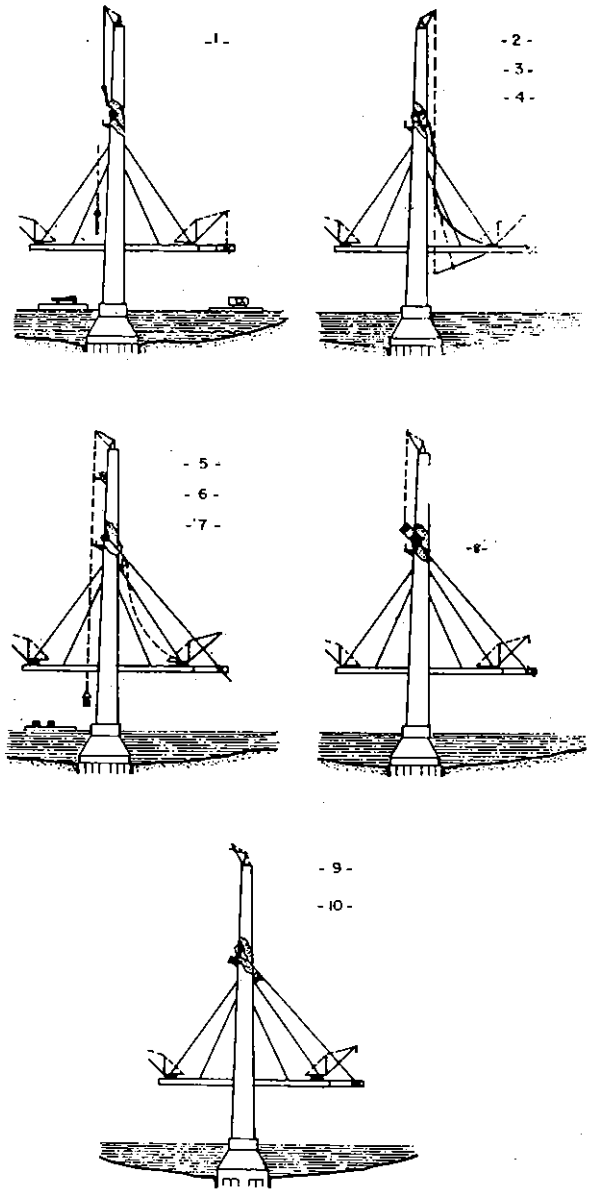


Fig. 38.27 *On site installation (typical scheme of operations)*

performed advantageously on site, strand by strand, with a lightweight easily handled jack. For a semi-prefabricated cable stay (i.e., the one with a simple pre-blocked anchorage) again the stressing may be carried out strand by strand, whereas for a completely prefabricated cable stay (in which both anchorages are pre-blocked) stressing is performed with a jack drawing all the strands simultaneously. Table 38.2 indicates the jacks to be used according to the tendon to be stressed.

Furthermore, an adjusting or a control jack may be used. This jack may remain available to the client for the

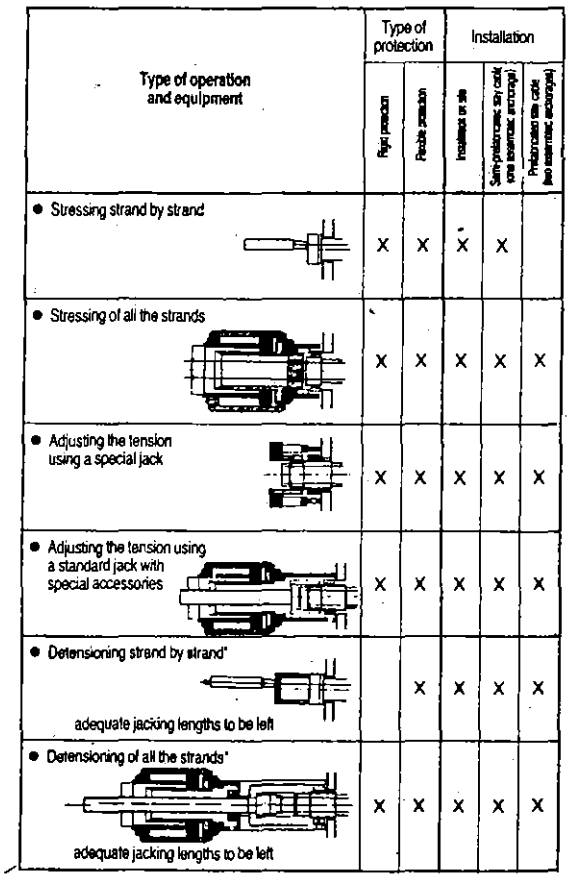


Fig. 38.28 *Stressing, adjusting and detensioning of a Freyssinet stay cable*

Table 38.2 *Principal Characteristics of Freyssinet Tensioning Jacks*

Number of strands	Type of jack	Length* (mm)	External diameter (mm)	Weight (kg)	Internal diameter (centre hole jack) (mm)	Tensioning area (cm^2) (ram)	Max. tensioning pressure (bar) kg/cm^2
19	K 350	502	440	430	156	490	625
31 to 37	K 500	718	515	780	203	766	600
55 to 61	K 700	727	640	1160	250	980	625
73 to 91	K 1000	783	770	1450	310	1431	625

* Dimensions without accessories.

supervision of cable stays. It is screwed to the anchorage block and enables restressing. Considering its function (adjustment), the stroke may be limited to reduce the weight.

The stressing procedure is confined to the compliance with the regulations in force in the country concerned.

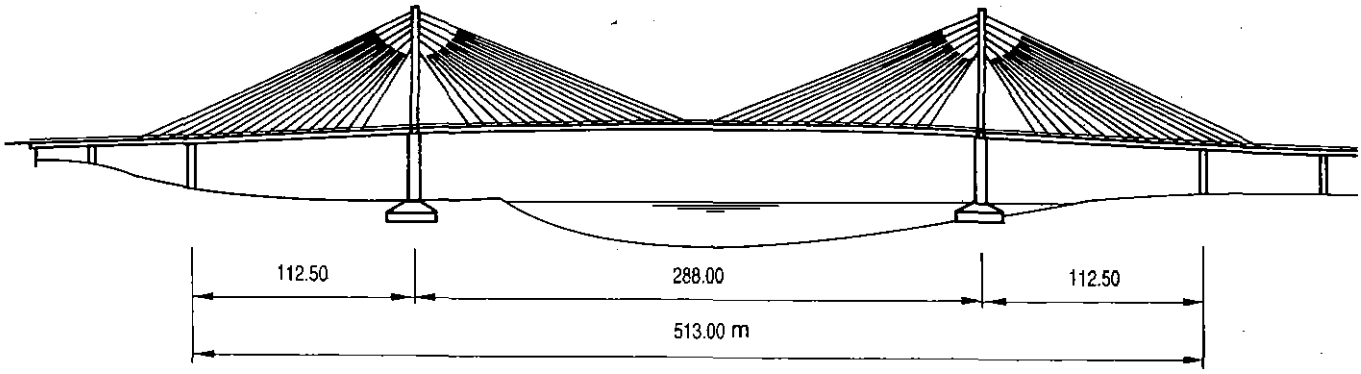
The cable stay may be completely destressed, either strand by strand (in the case of a flexible protection), providing that the necessary extra length has been provided for, or, the 'whole stay' (by the clamping of a special 'friction-piece' taken up by the stressing jack, adapted for destressing, whatever the type of protection).

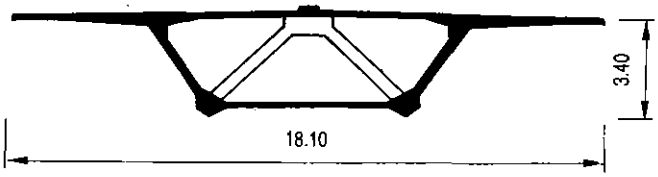
Salient Details of the Cable-stayed Hooghly Bridge (Calcutta, India):

~ **Main Bridge:**

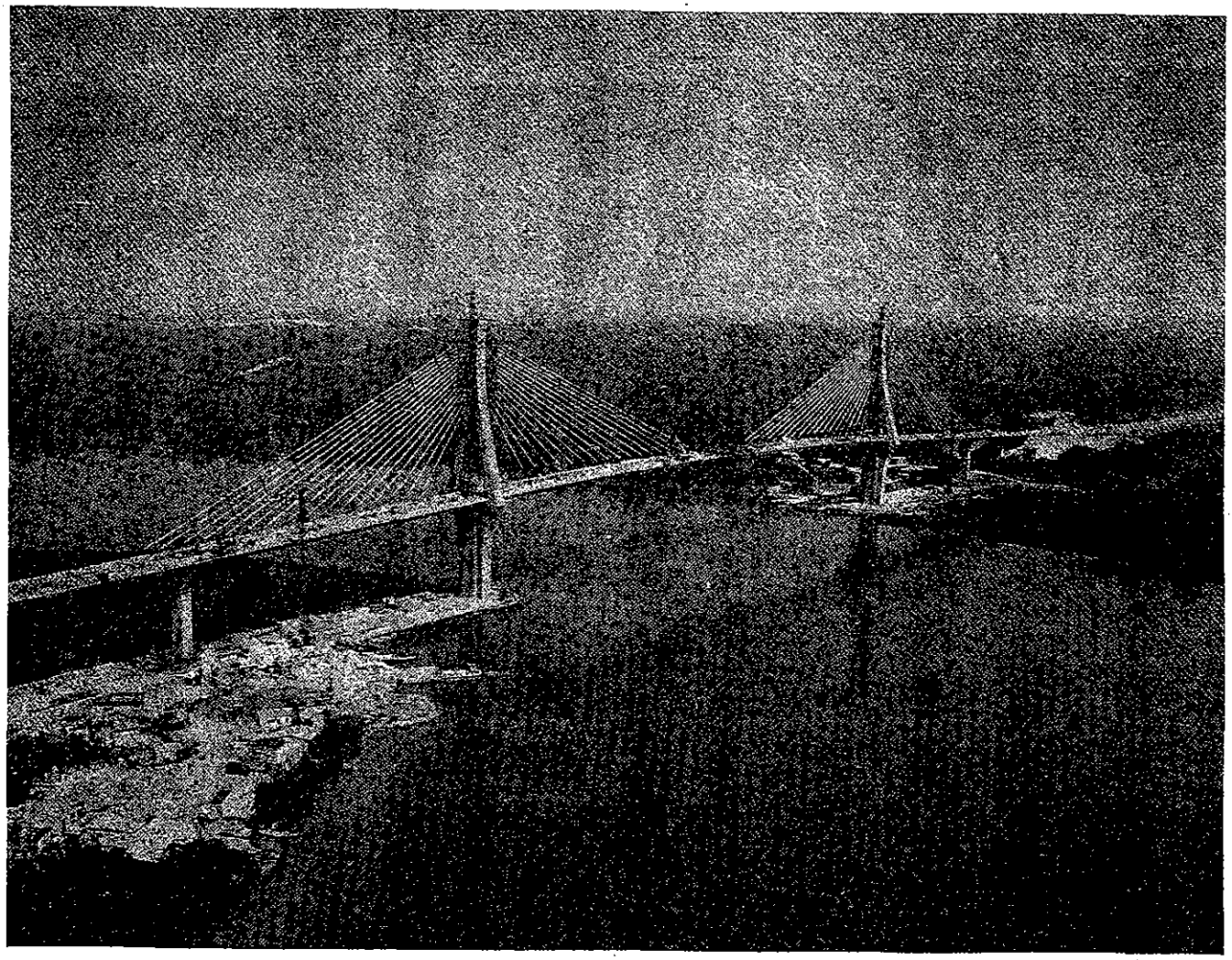
—Type of structure	Cable stayed 'composite deck'—reinforced concrete carriageway slab on steel I-beams.
—Length:	822.96 m.
—Main spans	457.20 m.
—Side spans (2)	182.80 m., each
—Navigational clearance	33.87 m/34.38 m.
—Roadway	Two 3-lane carriageways, 12.3 m each, 1.7 m. median, 2.5 m. footways
—Deck:	Concrete slab 230 mm thick, two outer steel I-girders 28.10 m apart and a central I-girder.
—Cable:	Parallel wire cable, σ_{GUTS} = 1600 MPa, BBR (HiA) (HiAm anchorage) (σ_{GUTS} = characteristic ult. strength).
—Towers:	Two steel towers. Height 122.30 m from pier top.
—Piers	Concrete piers. Height 12.39 m/16.05 m from foundation top, for main towers.
—Foundations:	Well foundation—twin wells, Main foundations—well diameter: 20.60 m/21.35 m Concrete Steining thickness—1.50 m/2.25 m (9 cells) Depth—21.96 m/29.95 m. Cellular space unfilled, watertight and accessible to maintenance inspection.
—Contractor	M/S. Bhagirathi Bridge Construction Co., India (A consortium of Gammon India Ltd. and BBJ).
—Consultant (Prime)	Leonhardt and Andrä, West Germany.
—Advisory Consultant	FFP, UK
—Approx. Cost (Bridge only)	Rs. 600 million*
—Executing Agency:	Hooghly River Bridge Commission

* The cost figure is only indicative of the project volume.



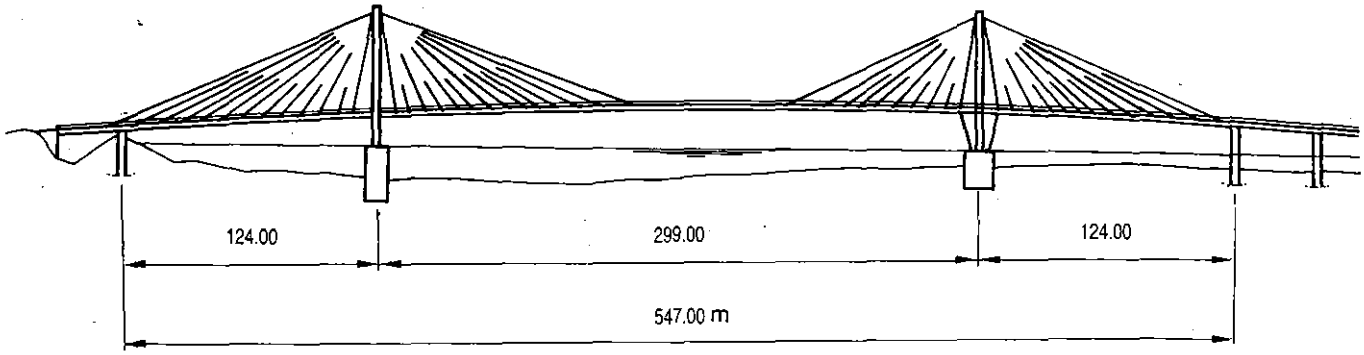


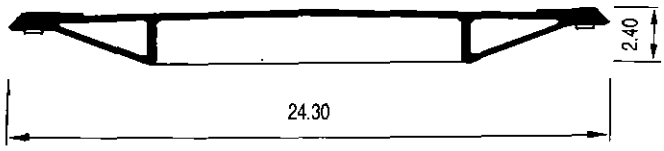
Owner :Secretaria de Communicaciones y Transportes
Designer : SOGELERG (France)
Contractor : ICA (Mexico)
Sub-contractor for stays : Freyssinet de Mexico SA (Mexico)
Period of construction :1980-1984
Type of stay-cables: 37 and 61 HC 15 Freyssinet stay-cables.



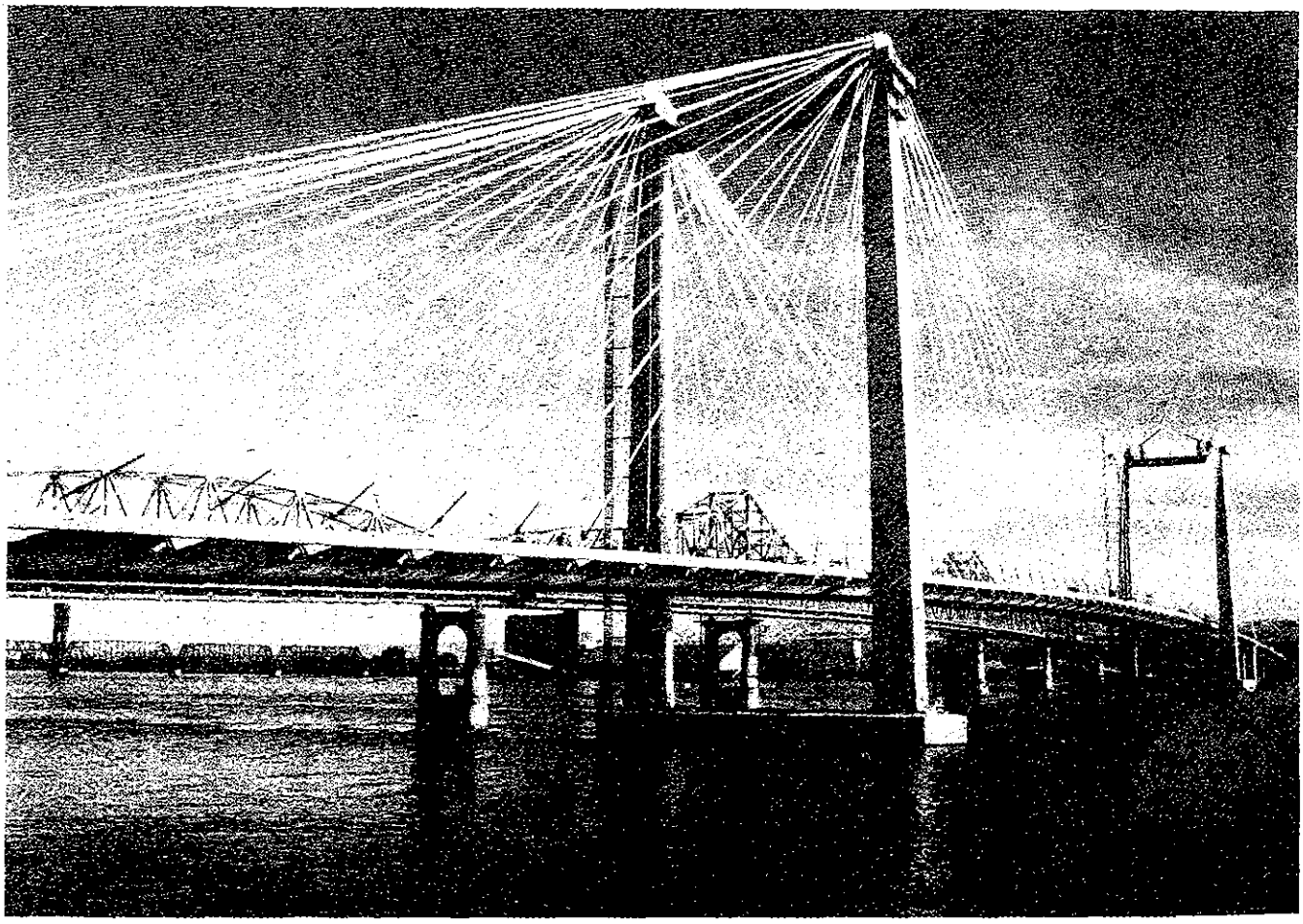
COATZACOALCOS BRIDGE

MEXICO

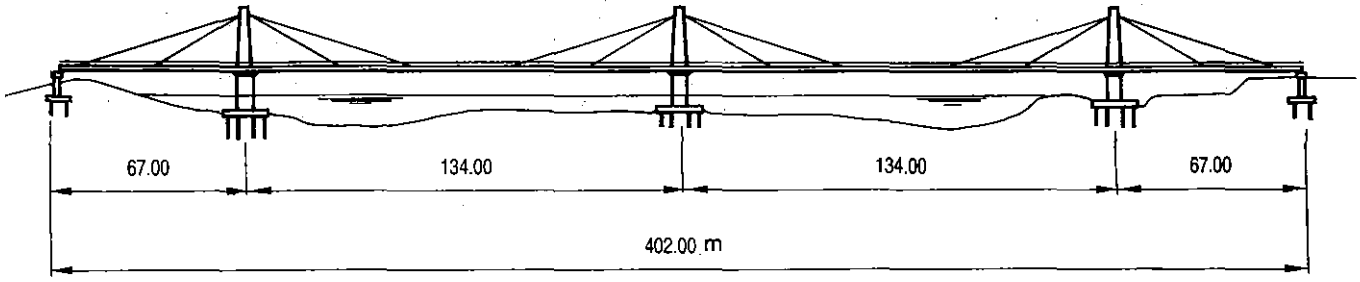


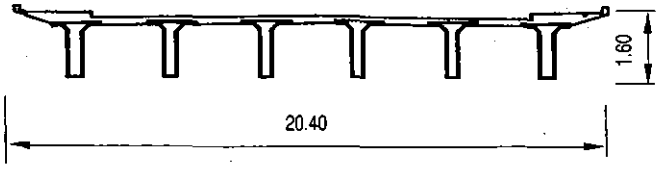


Owner : Federal Highway Administration and Cities of Pasco and Kennewick (Washington)
Designer : A. Grant and Associates (USA) in collaboration with Profs. Leonhardt and Andrä (Germany)
Contractor : Peter Kiewit Sons Co (USA)
Sub-contractor for stays : The Prescon Corporation (USA)
Period of construction : 1976-1978
Type of stay cables : HIAM cables with wires dia. 1/4"



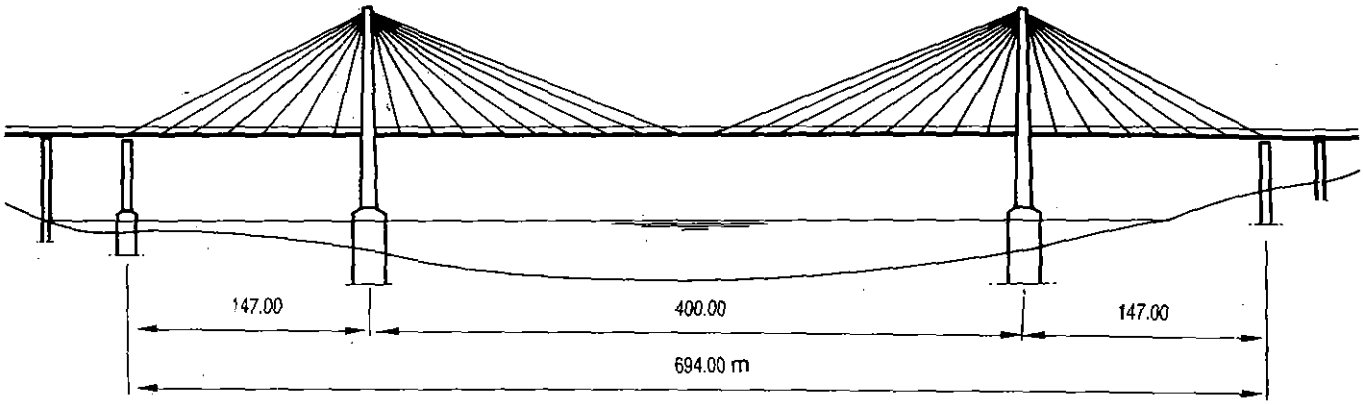
PASCO AND KENNEWICK BRIDGE *UNITED STATES OF AMERICA*

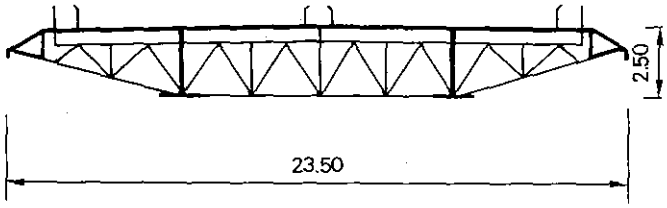




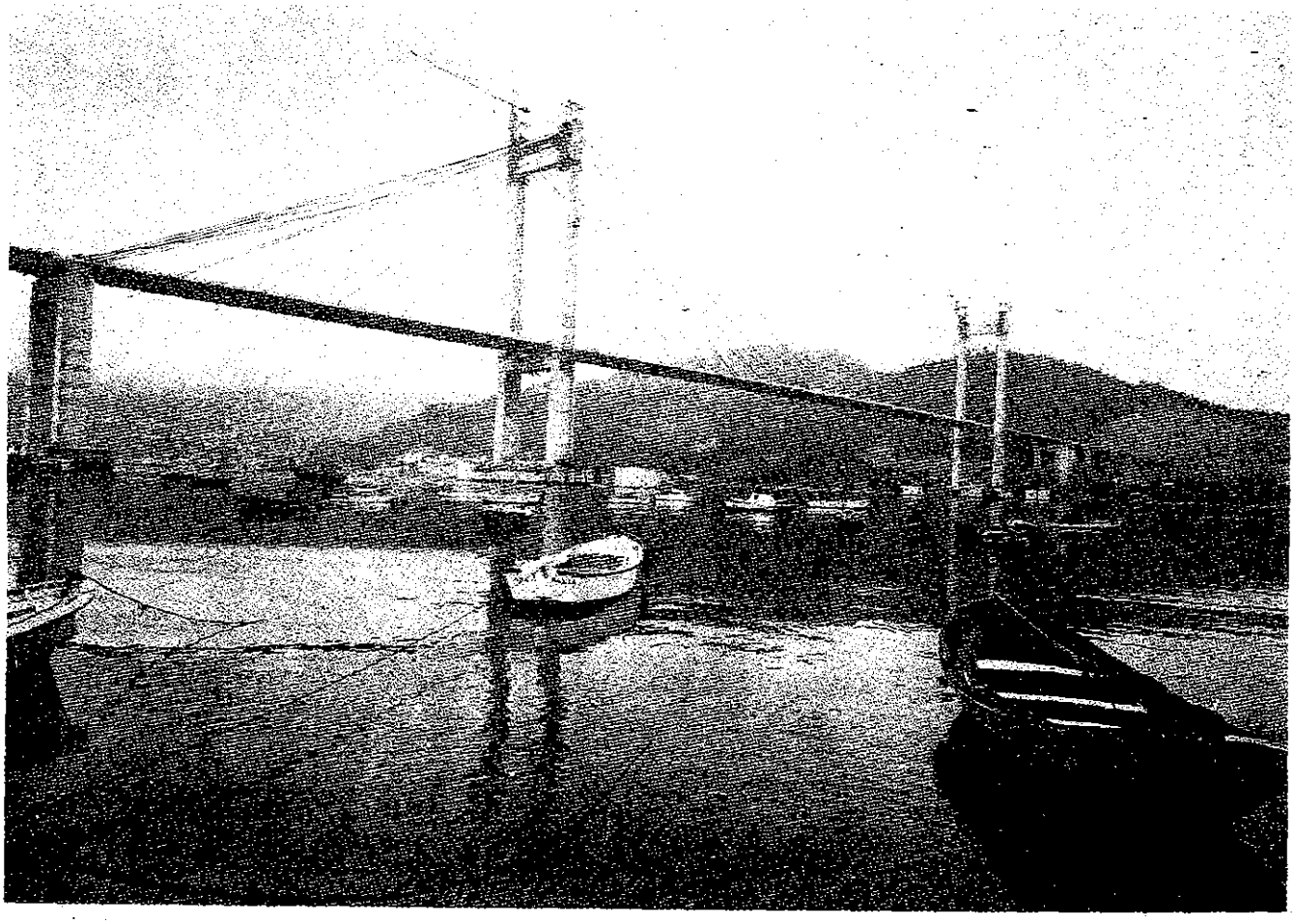
Owner : Taiwan Highway Bureau
Designer : TY Lin International (USA)
Contractor : Kung Sin Engineering Co (Taiwan)
Sub-contractor for stays : FKK (Japan)
Period of construction : 1975-1977
Type of stay cables : bundles of 14 (12 T 13) cables

KWANG FU BRIDGE *TAIWAN*

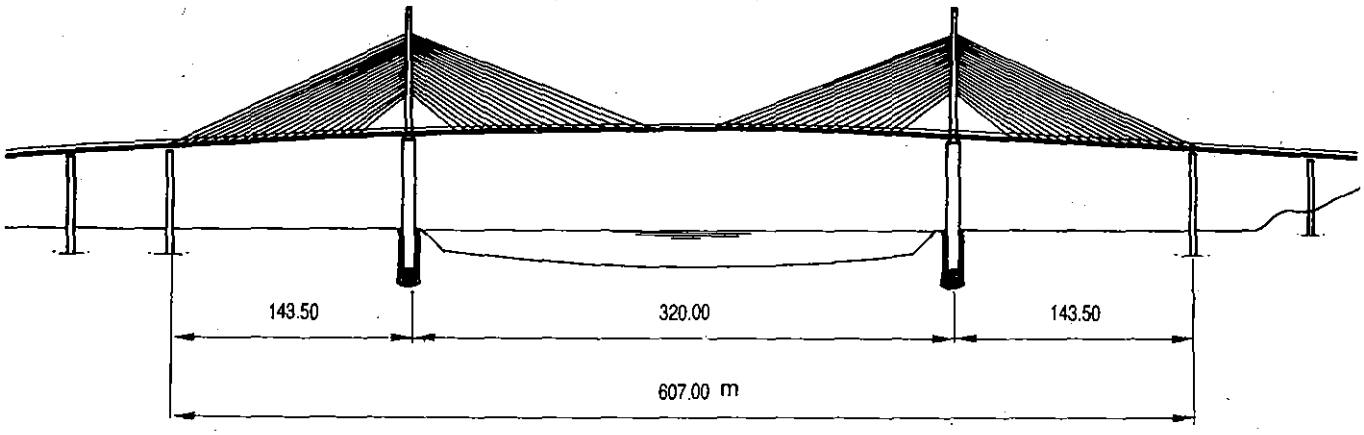


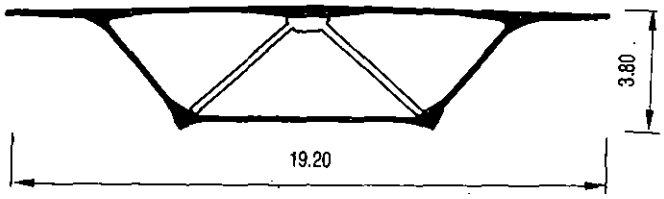


Owner : Audasa
Designer : Profs del Pozo, de Miranda and Passaro (Spain)
Contractor : C.Y.T.-M.Z.O.V. (Spain)
Sub-contractor for stays : Freyssinet SA (Spain)
Period of construction : 1975-1978
Type of stay cables : 37 H 15, 61 H 15 and 91 H 15 Freyssinet stay cables



RANDE BRIDGE ***SPAIN***





Owner : Departement de la Seine-Maritine
Designer : Campenon Bernard (France)
Contractor : Campenon Bernard (France)
Sub-contractor for stay anchorages : Freyssinet International (France)
Period of construction : 1974-1977
Type of stay-cables: 39 to 60 T15 continuous stays with lower adjustable anchorages.

BROTONNE BRIDGE *FRANCE*

Some Noteworthy Cable Stayed Bridges

Name	Country	Type of Structure	Main Span (m)	Deck	Inauguration	Freyssinet Group Member
Danube Canal bridge	Austria	Road bridge	119.00	Concrete	1975	Vorspann Technik (Austria)
Brotonne bridge	France	Road bridge	320.00	Concrete	1977	Freyssinet International (France)
Pasco-Kennewick bridge	USA	Road bridge	299.00	Concrete	1977	Prescon Corporation (USA)
Kwang Fu bridge	Taiwan	Road bridge	134.00	Concrete	1978	FKK (Japan)
Rande bridge	Spain	Road bridge	400.00	Steel	1978	Freyssinet SA (Spain)
Meylan footbridge	France	Footbridge	79.00	Concrete	1979	Freyssinet International (France)
Ilhof footbridge	France	Footbridge	63.50	Concrete	1980	Freyssinet International (France)
Cotton Tree Drive footbridge	Hong Kong	Footbridge	60.00	Concrete	1980	Freyssinet Hong Kong Ltd (Hong Kong)
Nishikigaoka bridge	Japan	Road bridge	52.00	Concrete	1982	FKK (Japan)
Luling bridge	USA	Road bridge	362.00	Steel	1983	The Prescon Corporation (USA)
Barrios de Luna bridge	Spain	Road bridge	440.00	Concrete	1983	Freyssinet SA (Spain)
Coatzacoalcos bridge	Mexico	Road bridge	288.00	Concrete	1984	Freyssinet de Maxico (Mexico)
Abidjan Cathedral	Ivory Coast	Cable stayed roof	90.00	Concrete	1985	Freyssinet International (France)
Sambre bridge	Belgium	Railway bridge	108.00	Steel	under construction	Laboratoire de Cinematique (Belgium)
Wandre bridge	Belgium	Road bridge	183.00	Concrete	under construction	Laboratoire de Cinematique (Belgium)

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CHAPTER 39

Vibration of Bridge Decks

39.1 INTRODUCTION

The passage of any load over a bridge causes the span to deflect from its equilibrium position relatively quickly. Thereafter, the inherent elasticity of the structure tends to restore the bridge deck to its equilibrium position, causing a series of oscillations. This phenomenon continues till either the structure comes back to its equilibrium position or is again activated by the passage of another load. The effects of bridge vibrations are two-fold, causing (i) structural damage in extreme cases, and (ii) unpleasant physiological and psychological reactions on humans.

The effect of vibration is to cause additional stresses in the structure over and above the static effects. The earlier studies conducted in the field aimed at the evaluation of those additional effects leading to the provision of suitable impact factors in the Codes of Practice. This step contemplated to account and provide for the extra stresses over and above the static effects but did not qualitatively analyse the exact role played by vibrations in rendering a structure unsafe. Certain thumb rule provisions incorporated in the Codes of Practices, such as limiting the ratio of deflection to span and restricting the span-depth ratios, tend indirectly to make the structure more rigid and thus less prone to vibrations. But even these provisions are not based on the evaluations of the frequency and amplitude of vibration likely to occur, and hence cannot be taken as a guarantee against the occurrence of undue vibrations even under normal loads.

As regards wind, it can affect either directly or indirectly. The direct action is envisaged by the wind force acting on vehicles, pedestrians and bridges, which could be a serious problem on exposed bridges like suspension bridges or bridges who approaches are shielded by surrounding objects acting like wind breakers. To meet this contingency, the design could provide for protective parapets, etc. The indirect effect relates to the effect on the user of the bridge due to the wind force 'exciting' the structure, tending to create a feeling of instability. The solution of this problem lies in making the structure more stiff with as high a value of structural damping as possible. Studies conducted do not tend to result in specific recommendations/guidelines for the designer.

Vehicles provide the major force causing excitation on bridges. Studies conducted indicate that the vibration set-up is influenced by the following parameters:

(i) rigidity and first-mode natural frequency of vibration of the bridge deck,
(ii) natural frequencies of the vehicle system and the suspension systems,
(iii) vehicle speed,
(iv) ratio of vehicle weight(s) to bridge weight,
(v) irregularities in bridge deck and approaches,
(vi) ill functioning of expansion joints,
(vii) frequency of live load application due to the passage of multiple axles,
(viii) motion induced in the bridge before application of the live load (particularly important for continuous spans), and
(ix) the damping characteristics of the bridge and the vehicle.

Actual observations on bridges have shown that the foregoing factors have wide variations in their relative influence in bridges, making it practically impossible to evolve a generalised solution applicable to all bridge cases. Whatever studies have been conducted, the results are applicable to only specific cases and probably only some of the broad conclusions reached may apply to other cases as well. Further, due to the large number of interacting parameters between the bridge and the vehicle, often attempts are based on quite a few simplifying assumptions which are far from reality, tending to make the conclusions all the more limited to a particular case. This only emphasizes the fact that each case will need to be treated on its own merit rather than have a solution based on the results of other cases.

From the point of view of pedestrian force, the effect of marching troops seems to be the only force worth considering, causing excitation to bridges. But the troops should break step while crossing a bridge and thus not affect the bridge response materially. The effect of pedestrian force may be of relevance only in case of footbridges.

The vehicular force constitutes the main exciting force causing vibrations in bridges. This is also reflected by the

large number of studies conducted to evaluate the effect of moving loads.

39.2 STUDIES

The studies conducted can be divided basically into two parts i.e. (1) theoretical analyses and model studies, and (2) field observations. In some cases where observation of dynamic response was made on actual bridges, the results were also substantiated by theoretical studies. Sometimes the two differed, making a conclusion difficult, the primary aim of all the tests conducted being to evaluate the free response and forced response of a bridge so as to determine the increase in static deflection, moment and shear. Based on the studies conducted, some of the conclusions that have been reached are as follows:

(i) For predicting the motion of a loaded simple 'beam and slab' span at points directly under the path of the load, the initial conditions need to be completely defined.

(ii) Simple span bridges, when loaded, vibrate with their first mode of natural frequency, or a forced frequency depending upon the loading vehicle.

(iii) The observed rigidity of simple span bridges is more than that arrived at by usual design computations (due partly to wearing course and parapets—and even the kerbs—contributing to the deck section properties).

(iv) Pavement irregularity, causing fluctuations in the magnitude of the applied axle load, is the key factor influencing the amplitude of motion of simple span bridges.

(v) The amplitude of vibration for single span bridges is not generally well related to vehicle speed even for a particular vehicle.

(vi) Vibration intensity measured in terms of human reaction to sustained vibration, indicated that "unpleasant" levels of vibration are not uncommon.

The results of the studies indicated that designers should consider the inter-relationship between vehicle loads, pavement irregularity and bridge vibration. The irregularity in the bridge approaches and at the expansion joints tend to introduce significant motion in the vehicle system increasing vibratory motion.

The above findings are generally true for continuous spans also. Although continuity increases the strength and stiffness of each span in a continuous bridge, natural vibration occurs at the same frequency as if the span were simply supported.

Since continuous spans are usually lighter than simply supported spans, it is evident that the continuous spans in general will vibrate at lower frequencies than simply supported spans of the same strength.

Compared to 'frequency' of vibrations the results for 'amplitude' of vibration were far less consistent. Amplitude is significantly affected by the condition of bridge deck and approaches and the characteristics of the bridge and vehicle. Correlation of data of vibratory amplitude, obtained with various parameters and design factors, showed a significant relationship between amplitude factor and stiffness, indicating thereby the bridges of less stiffness showed a clear tendency for much larger values of vibratory amplitude.

Oechler compared simply supported, continuous and cantilever type bridges and concluded that cantilever bridges have the following characteristics:

(i) greatest susceptibility to vibration,
(ii) greatest maximum duration of vibration,
(iii) lowest natural frequency,
(iv) highest amplitude of vibration,
(v) greatest psychological discomfort, and
(vi) those designed with composite action have larger amplitudes of vibration.

The findings reached are of a very general type, leading to only broad conclusions and dependent on the bridge and vehicle parameters, emphasizing that individual problems have to be tackled independently to arrive at desired solutions. For the purpose of a designer's need, more detailed tests seem to be necessary which will clearly prescribe the upper limit to vibrations likely to occur and suggest ways and means of restricting them within acceptable limits, so that the deflection, bending moments and shear forces, as accounted for in the design, are not exceeded in actual practice by the passage of moving loads.

39.3 CODAL PROVISIONS

The various Codal provisions which correlate indirectly with the dynamic response of bridges are those pertaining to impact factors, limitation in the ratio of deflection to span and restriction in the span-depth ratios. These provisions have been evolved with time, based on a quantitative analysis of the bridge behaviour when subjected to live loads, i.e., after accounting for the increase in the static values due to the dynamic effect. Even these provisions vary from one Code of Practice to the other, indicating a lack of uniformity in approach as well as actual values. Whatever studies have been conducted, they do not help in arriving at a generalised qualitative solution, so that the provisions made in the Codes of Practices could be linked up with vibrations, amplitude, etc., likely to occur. However, a larger number of these studies generally seem to conclude that the additional safety factors incorporated in the design by way of impact considerations or limiting the deflection and depth of member with respect to span, are sufficient to account for the overstresses caused due to the dynamic

effect of live loads, thus indicating that the overall structural safety is not impaired on account of the vibration effect.

Some schools of thought recommend separate "amplification factors" to be applied for calculation of deflections, moments and shears. The deflection amplification factor being higher than the others because of the severe restrictions placed on dynamic deflections considering human susceptibility to vibration. However, so far there is lack of research and lack of conclusive findings to substantiate the above concept for being included as Codal requirements, though the basic reasoning seems to be appropriate.

Studies that have been conducted with idealised axle systems to seek information with respect to the frequencies of vibration of the bridge when loaded and unloaded, are not conclusive enough for incorporation in the Codes. The mathematical equations that have been evolved are based on a number of assumptions, and may not be obtainable in and applicable to other cases. This thus indicates that the prediction of frequencies whether natural or forced, for any bridge, needs to be evaluated individually by obtaining actual test values rather than by any theoretical equation. Impact is a dynamic feature related mainly to the surface characteristics of the bridge and the spring constant of the vehicle. No doubt these will not be the same in all countries, and, because of this, there may be some differences between the impact values in different countries. But it is difficult to appreciate the wide variation even in the basic approach of different countries in allowing for impact, as brought out below:

(i) The British Standard specifies an impact allowance of 25 per cent to be added to the axle load, or the pair of adjacent wheels which produces the greatest bending moment or shear as the case may be. The stipulations in the Norwegian standard are similar: the only difference is that instead of 25 per cent, 38.5 per cent impact is added to the heaviest axle load.

(ii) In the majority of the remaining countries, impact is related to the loaded length (span length) although the exact relationship varies considerably from country to country.

(iii) Austria, India and Japan specify different impact for concrete and steel bridges, the factor for steel being more than that for concrete. (This apparently is based on the principle that a lighter structure will be subjected to a more dynamic effect.) Finland too has a similar approach, but it distinguishes only timber bridges from the others by specifying a lower impact factor for them, presumably on account of the damping effect of timber.

(iv) The British standard and the loading standards of West Germany and Italy ignore impact when the span length exceeds 30m, 50m, and 100m, respectively. But even for longer spans, countries like Australia and India specify certain minimum values of impact.

(v) All the standards give an upper limit for the impact allowance either with respect to the type of vehicle (tracked or wheeled) or in relation to the type of bridge (concrete, steel or timber) and the value of this varies from 25 per cent to 65 per cent in different standards.

(vi) Unlike other countries, Belgium and France relate the impact factor to the dead load of the bridge structure. The principle behind this is, however, implicit in the impact formulae being used by many other countries which relate impact to the type of bridge and length of span. The impact formula of Belgium is further complicated by including the speed of the vehicle in it.

(vii) Austria specifies different impact factors for the directly loaded and indirectly loaded main girders on concrete bridges, the factor for the former being more than that for the later. Again in the case of steel bridges, it distinguishes between the first and second lanes of steel bridges, specifying higher impact factors for the first than the second and allowing no impact for lanes in excess of two.

The formulae for impact allowance specified by some countries are unnecessarily complicated and not fully justified, particularly since the effect of live load on the bridge is comparatively less than that of dead load for span lengths of approximately 25 m and above. These clearly show that the present knowledge about the dynamic behaviour of live load on bridge structures is inadequate and there is need for more research in this field.

39.4 HUMAN ASPECT

Bridge response to a moving load is dependent on mass, stiffness, damping properties of the bridge and dynamic properties of the moving loads, resulting in vibrations either at the natural frequency or at the frequency of the applied exciting force. The normal range of fundamental frequency of bridges varies between 1 and 20 cps. This may coincide with the range of frequencies of vehicles, resulting in the possibility of resonance.

The human reactions to vibrations are peculiar and need to be examined from two points: (i) physiological effects on the body and (ii) psychological effect on the person. The human body has its own standards for feeling the effects of natural frequency, and is capable of detecting small levels of vibration. The human reaction to these vibrations results in the physiological effects. The psychological effects are

more difficult to define, varying from one human being to the other. This aspect is more linked with the human feelings and is a problem of educating and familiarising the user in such a situation.

39.5 FREQUENCIES CAUSING PHYSIOLOGICAL AND PSYCHOLOGICAL EFFECTS

The assessment of the nature of human reactions, as related to the large variations in the stimulus causing the vibration, is rather complex. However, studies conducted to evaluate physiological response of humans have been successful in laying down the following frequencies at which feeling of discomfort or resonance may occur in the human body:

1/4-1 cps — associated with motion sickness,
2 cps — head resonance (for horizontal movement); also associated with motion sickness,
4–6 cps — major resonance of the whole body,
7–9 cps — abdominal resonance,
10–12 cps — unspecified trunk resonance.

The predominance of the above frequencies will result in adverse physiological effects like discomfort, annoyance, etc., specially when subjected to such conditions for a long period.

However, laying down similar limiting frequencies from the point of view of adverse psychological effects is rather difficult, as no standardised scale can be evolved due to the wide variation in human reactions. For instance, one person may react unfavourably with heights and even a small degree of vibration on a bridge whereas another person may not feel this at all. As such this is a problem wherein the response varies largely from user to user, depending upon familiarity and usage.

39.6 FATIGUE AND TOLERANCE LEVELS

Apart from the physiological and psychological effects, is the fatigue effect and the difficulty the user feels in performing normal duties when subjected to long duration of vibrations. To analyse this problem, researchers have considered the reaction of the pedestrian as the deciding factor in comparison to the passengers in the vehicle, as the latter remain on the bridge relatively for a very short duration. Studies conducted have revealed that there is an appreciable reduction in 'tolerance level' if a person is standing still rather than walking on a bridge. This was observed due to the flexing of the knees in the process of walking creating a hinge effect reducing the amount of excitation transmitted to the body! To study the levels of human tolerance to vibration, work has been done by Reiher and Meister, Dieckman, Goldman, Janeway and Bryce. However, these works do not seem to cover vibration effects on people when walking, or the tolerance of people to transient vibrations. In 1965, the Road Research Laboratory, U.K. conducted some tests to determine the tolerance limits for pedestrians walking and standing on a bridge. The amplitude and frequencies were varied and human reactions ascertained. The acceptable and non-acceptable levels of vibration for all subjects based on the above tests for walking and standing were reproduced as an amplitude frequency plot and in the form of acceleration against frequency.

Tests indicated that cantilever bridges were of the livelier type, needing more care in design. However, the R.R.L., U.K., concludes that test data available are insufficient to enable comparisons with other types of bridges.

39.7 WORK ALREADY DONE AND ITS RELATION TO THE PRESENT PROBLEM

Extensive studies have been carried out by the University of Illinois, U.S.A., Queen's University, Ontario, Canada, and Technical University of Prague, Czechoslovakia, to study the dynamic response of highway bridges. On many occasions, the experimental data on models or prototypes, have been supplemented by theoretical analyses involving simplified assumptions and idealisations. These tend to limit the application of the results on similar problems existing in case of other highway bridges. Based on past studies, it is felt that the main input parameters which influence the dynamic response of a continuous bridge can be summarised as below:

(i) Assumptions and idealisations made for simplification of the study,
(ii) Aspect ratio of the bridge (span/width), and other relevant geometrical and structural details,
(iii) Boundary conditions at the ends,
(iv) Nature of moving vehicle system,
(v) Speed parameter, generally defined as $VT/2L$, where V is the velocity of the moving vehicle system, T is the period of the bridge and L is the characteristic length of the bridge.

In addition to the above parameters, the following four factors will also influence the validity of the results obtained:

(i) How the response data are obtained,
(ii) Method adopted for calculating the natural frequencies and the numerical results obtained,
(iii) Method adopted for analysing the forced response of the bridge and the results obtained, and
(iv) General conclusions drawn with respect to the dynamic increments for deflections and stresses.

The tests consist of two parts: (1) dynamic load-tests, and (2) static load-tests. In the dynamic load-tests, different types of loadings are employed for the purpose, with differing numbers of trucks and convoys moving in one or both directions, maintaining certain speeds and spacings.

The accelerations are measured by an instrument-probe

and acceleration vs. time plots are recorded automatically through an instrumentation set up on a photosensitive paper. From these measured accelerations for the various points of the bridge, obtained for different types of test loadings, different speed parameters, and for different directions of movement of trucks, the required amplitudes of dynamic displacements are calculated from which the corresponding dynamic increments are derived. From these data it has been possible to summarise what are commonly called as 'impact factors', both for deflections and for loads.

The static tests are planned in order to obtain the deflection factors for salient points.

39.8 PRACTICAL FACTS

Prestressed or reinforced concrete bridges usually do not have vibration problems because the amplitudes of these vibrations in concrete bridges are very small and there is really no adverse effect on traffic or on human beings. Only a certain range of frequencies may disturb the feelings of some human beings, but this disturbance usually is in a tolerable range unless it is a light pedestrian bridge where people are not quickly passing across.

Again, it is usually not so strong that it could not be accepted by pedestrians. If you sit in a car, you even do not feel all these vibrations. Therefore, we do not really need to go into any 'limitations' of such vibrations. Also, the stresses which come to these bridges by vibrations or by frequent loadings are such that there is no reason for being worried for the structural safety, if the codal impact factors on live load are included and the road surface is smooth.

The impact factors which have been measured in many concrete bridges under traffic in Germany show that the impact is so small that the stresses caused by the impact are small and never reach the maximum stresses by extreme loads which are specified for ultimate strength. It is different in steel bridges and specially different with steel bridges for railroads. There we have to consider, of course, the impact especially for the welded structures which are sensitive against frequently repeated loadings leading to the reduced fatigue strength. Impact factor actually reduces with higher speed, speeds higher than about 50–60 kilometres per hour. The deformations become smaller because the structure does not have sufficient time to react and get the large deflection which we calculate for statical loading. Therefore, high speeds of vehicles do not really affect the highway bridges.

The quantitative value and mode of vibration of the vehicle, which causes various physical discomforts and structural criticality, depends mainly on the type of vehicle, vehicle-suspension, surface characteristics and the structural system. Generally, for concrete structures, vibration, apart from causing a certain amount of unpleasant physical feeling to pedestrians, is not a significant factor. The vibrations are normally more pronounced in prestressed concrete structures as compared to reinforced concrete structures in view of their smaller mass and hence that of the damping system. Again, a continuous bridge generally manifests a higher level of vibration as compared to what is experienced with simply supported spans, on account of possible superimposition of the vibration modes of multiple vehicles traversing the deck, and lighter mass, other things being equal.

39.9 PRACTICAL APPROACH FOR VIBRATION ANALYSIS

Descriptions given above are all very well but not very helpful to the regular designer who finally still needs a guideline. For this purpose reference may be made to B.S. C.P. 117-Part 2—1967 and Lenzen's Criteria (Fig. 39.1). The following procedure may be adopted for a quick and workman-like solution of the problem.

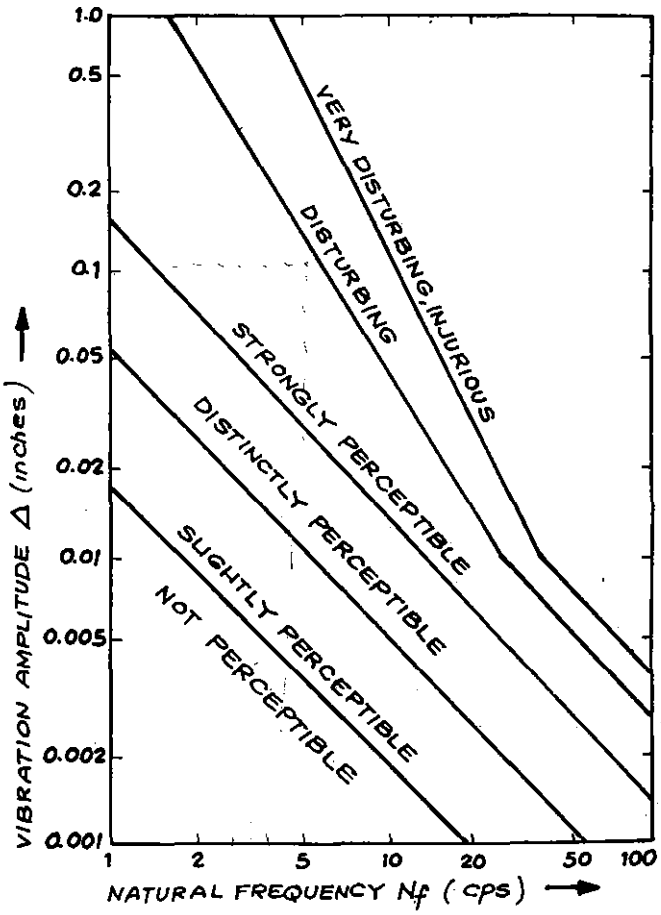


Fig. 39.1 *Lenzen's criteria*

Step 1 Estimate the maximum span deflection δ (in inches) under a single 20-ton hypothetical point-load placed in the centre of span and cross-section, using fully composite (average) flexural rigidity of the deck section of full-width (EI).

Step 2 Estimate the fundamental natural frequency N_f of the span from:

$$N_f = \frac{2}{L^2}\sqrt{\frac{EIg}{w_d}} \text{ cycles per second}$$

where w_d = dead weight, including finishes, in tons per foot of bridge span,
g = 32.2 ft./sec^2
EI = Flexural rigidity, in foot and ton units, of the full-width section of the bridge deck,
and L = span (ft)

Step 3 Take $\Delta = 0.40\delta$ if $N_f > 4$ cps
and $\Delta = 0.75\delta$ if $N_f > 4$ cps
where Δ is the maximum amplitude of vibration, in inches (δ also in inches).

Step 4 Estimate the maximum acceleration A from:

$$A = 40\Delta N_f^2 \text{in/sec}^2$$

Step 5 Ensure that the product A $\Delta > 5$ in^2/sec^2 and that the vibration characteristic from Fig. 39.1 (from known values of N_f and Δ) is as desirable.

CHAPTER 40

Use of Freyssinet Flat Jacks

40.1 INTRODUCTION

The Freyssinet flat jack is a thin hydraulic jacking device capable of exerting large forces simply and economically.

Devised by M. Freyssinet and used by him as long ago as 1938 for strengthening dams, it soon became evident that its field of application was extremely varied and it has progressively extended to all spheres of construction, either to improve the quality of foundations, to simplify the building of structures and to increase their resistance due to the prestress thus applied, or for carrying out repairs, reinforcing and controls, and ensuring a maximum of operating safety.

Over the past thirty years, Freyssinet International has acquired considerable experience in a wide variety of possible applications, using flat jacks of varied shapes and forces.

The Freyssinet flat jack (Fig. 40.1) is a hydraulic capsule in the form of a flat double saucer, made of two steel plates welded together around the exterior of the pressformed peripheral beading. Two nozzles, one of which acts as a vent, enable the flat jack to be inflated. The upper and lower plates are forced part, exerting a uniform pressure on the surfaces of both parts of the structure between which the flat jack is placed. The pressure inside the jack may reach 15N/mm^2 and the opening (or stroke), that is to say the final clearance between the two plates, corresponds to the diameter of the beading (normally 25 mm).

40.2 ADAPTATION AND COMBINATIONS

- The table below indicates the sizes of standard circular jacks. Jacks of different sizes and shapes, rectangular, triangular or oblong, for special application, can be supplied by Freyssinet International.

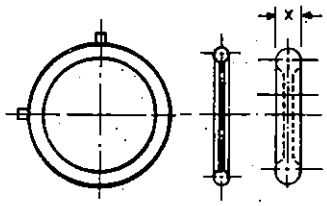


Fig. 40.1

- Jacks with openings in excess of those shown can also be supplied on condition that the bearing capacity is reduced, the permissible pressure in the beading decreasing when the diameter increases.
- To increase the movement, however, it is preferable to compound the jacks in layers (Fig. 40.2).

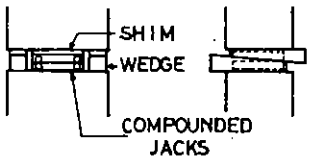


Fig. 40.2

- If a load has to be distributed over a large area, it is advisable to use a series of flat jacks interconnected and inflated simultaneously using a common manifold (Fig. 40.3).

40.3 FORCE EXERTED ACCORDING TO THE OPENING

The force exerted by a Freyssinet flat jack varies slightly

Standard Sizes

Max-force in Tonnes	2	7	15	40	50	60	80	110	170	240	370	600	830	900
Outside diameter in mm	70	120	150	220	250	270	300	350	420	480	600	750	870	920
Thickness in mm (x)	15	25	25	25	25	25	25	25	25	25	35	35	35	35
Minimum access* gap in mm	25	35	35	35	35	35	35	35	35	35	45	45	45	45
Opening in mm (x)	15	25	25	25	25	25	25	25	25	25	35	35	35	35

* Minimum gap required to place flat jack and steel thrust plates.

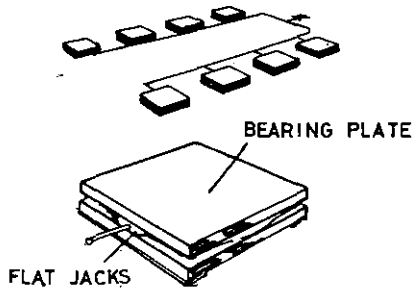


Fig. 40.3

according to the opening (or stroke) and the pressure. The active surface (quotient of the force divided by the hydraulic pressure) decreases when the opening increases.

It should be noted, however, that the force exerted is, in all events, mainly dependent on the reaction acting against the jack.

40.4 METHODS OF 'INSTALLATION'

The double-saucer shape of the flat jack requires the use of intermediate thrust membranes between the jack and the structure. A number of methods are used:

Steel Plate

The method is used when the jack is to be recovered on completion of duty (Fig. 40.4).

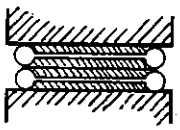


Fig. 40.4

A steel saucer plate is placed in each of the saucer recesses. The steel plates may be designed with handling lugs for ease of manipulations and for extraction after inflation.

Epoxy Resin

This method is used when the jack is to remain permanently in position. The dished surfaces are filled and levelled with a hard setting epoxy resin which has the advantage of being light, easy to handle and non-corrodible (Fig. 40.5).

Pre-Cast Concrete Blocks

Used as a permanent installation where the concrete block is to remain part of the visible structure, or where a number of jacks are compounded in series (Fig. 40.6).

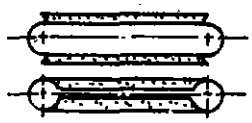


Fig. 40.5

The concrete blocks are reinforced with one or several layers of reinforcing mesh depending on their thickness.

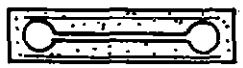


Fig. 40.6

40.5 'INFLATION' TECHNIQUE

(a) Temporary Installation

When the jack is to be used for a limited period and subsequently removed, inflation is normally done with hydraulic fluid using normal pumping techniques and a circuit designed according to the structural balance or 'fail-safe' requirements. Experience in circuit design is necessary, and discussion is advised on all but the simple applications.

(b) Permanent Installation

Where Freyssi Flat Jacks are to remain permanently installed, inflation by chemical hard setting resin is normal. Often in these cases, temporary oil inflation is required for periods of up to six months for structural stability to be achieved or to allow for the effects of a continuing construction programme.

This can be done quite simply where each jack in turn can temporarily be depressurised for replacement of the oil by the chemical resin: where this cannot be permitted, a more advanced technique, involving a double-banked jack circuit, must be used—developed by Freyssinet International.

Freyssi Flat Jacks may equally be inflated with cement grout, but, in these cases, the working force should be reduced to 50% of normal to ensure correct inflation free of blockages at a pressure not exceeding 7.5 N/mm^2.

40.6 RECOVERY AND RE-USE OF JACKS

Freyssi Flat Jacks remain in their inflated shape on release of hydraulic pressure and, on recovery, may be returned to their closed position in a press. The number of re-uses that can be obtained is dependent on the extent of the opening used and the working pressure, and, as a guide, from 5–10

uses may be expected under average conditions.

However, if the conditions of use demand reliability or calibrated accuracy from the jacks, *it is recommended that they be used only once* as the cost of replacement is comparatively low.

40.7 APPLICATIONS

Freyssi Flat Jacks find wide application in the civil and structural engineering industry wherever application or control of large forces are required or where structural or foundation strains have to be induced. Essentially simple and compact, they are so often used in situations for which their use was not foreseen, such as remedial measures or structural additions, as they are used in new constructions in which they form part of the structural concept.

Amongst the many applications can be listed:

* Control of thrust forces
* Pre-stressing between abutments
* Adjustment of support reactions
* Structural pre-loading
* Structural lifting
* Underpinning
* Measurement of forces
* Thrust maintenance

Five Specific examples of actual use are described ahead to illustrate the adaptability of Freyssinet Flat Jacks.

• Maintaining Constant Thrust

The present 'Minster' at York (England) dates back to the eleventh century with later additions to the tower up to the fifteenth century and some rebuilding following a fire in the nineteenth century. Due to gradual foundation settlement causing severe cracking, extensive structural repair has had to be undertaken.

The Consulting Engineers wished to apply a steady thrust to the East Wall, which was, at the start of operations, more than 2 ft. out of vertical. A series of steel-framed flying shores were erected and double-banked Freyssi Flat Jacks inserted at their bases to provide in each an axial thrust of 160 tons (Fig. 40.7)

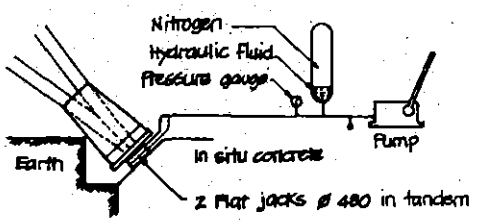


Fig. 40.7

However, due to the considerable rigidity of the 14 ft. thick masonry walls and of the foundations of the shores, temperature effects would have caused the thrust to vary from zero to twice the force required; this was not acceptable structurally, but, in any case, would have doubled the cost of the steel frames.

A hydraulic circuit was, therefore, fitted with nitrogen-filled compensating accumulators designed to maintain the actual thrust to within 5% of the given mean under the most adverse temperature conditions. Control records show that this was achieved.

• Prestressing between Abutments

A length of $1\frac{3}{4}$ miles of the road tunnel section, constructed through Mont Blanc to link France and Switzerland, was pre-stressed longitudinally with Freyssi Flat Jacks (Fig. 40.8).

The end abutment sections were each designed to withstand a maximum thrust of 7700 tons and, between them, the concrete section was stressed such that a minimum of 225 lb/in^2 was maintained under the most adverse ambient conditions.

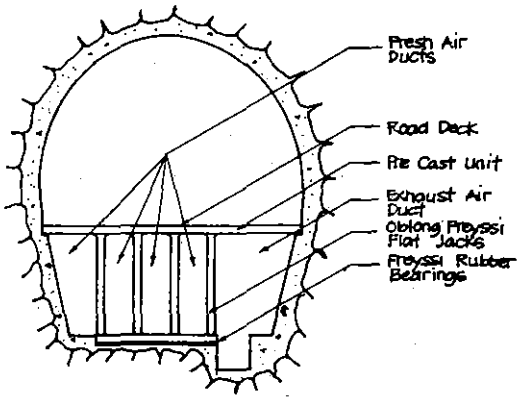


Fig. 40.8

The concrete sections, containing, in the lower chambers, the air ducts and, on the top, the road surface, were pre-case in 32 ft. lengths and jointed in-situ. Each sixteenth joint was 'active' and Freyssi jacks were inserted across almost the whole section. These were case in pairs into lightly reinforced concrete block.

In order to provide adequate strain capacity at each joint, which had to compensate for concrete shrinkage, elastic shortening and creep under pre-stress and for temperature movement, itself estimated at $\pm 1.5''$ per joint, the Freyssi Flat Jacks were arranged in seven layers giving a maximum capacity of $7''$ movement. 'Active' joints had to be provided at frequent intervals in order to localise the restraints to longitudinal force caused by the rubber bearings on which the road section was supported.

The Freyssi Flat Jacks were inflated by oil in layers, each layer provided 1 in. of movement; they were later cement-grouted. The prestress was readjusted through a period of nine months as shrinkage and creep took place and observations continued so that further correction may be carried out as necessary. More than 2,000 oblong shaped Freyssi Flat Jacks were used in this project.

• Control of Arch Thrust

The Gladesville Bridge, constructed over the Parramotta River just east of Sydney, is a concrete arch bridge with a clear span of just over 1000 ft. It is the largest of its type in the world.

Constructed as four independent arches, later made monolithic, it is entirely made of pre-cast voussoirs erected on steel centring. A battery of Freyssi Flat Jacks was inserted at both third points of the arch, by means of which the whole span has lifted clear of the support frame for de-centring; this created the arch thrust which was then adjusted to the correct point of application (Fig. 40.9).

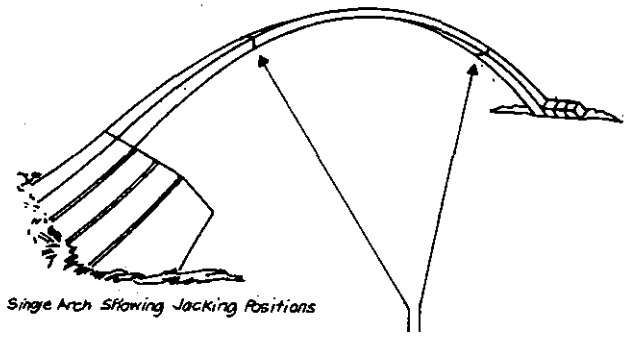


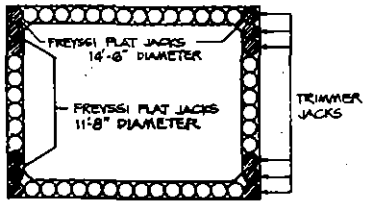
Fig. 40.9

As each of the arches, in its independent condition, was extremely sensitive to horizontal curvature, due to differential drying and the thermal effects of sun, a sophisticated circuit, including trimmer controls, was provided.

At each section of each arch, 56 Freyssi Flat Jacks, capable of exerting a maximum total thrust of 6500 tons, were installed; these were banked in layers of four jacks to provide a maximum of 4″ movement (Fig. 40.10).

• Transfer of Forces and Lifting

(a) Freyssi Flat Jacks were used to help preserve a matchless example of cultural history. The temple at Abu Simbel was due to be inundated by the rising waters of the Nile behind the newly constructed Aswan High Dam. A scheme to raise the temple just over 200 ft. was financed through UNESCO



Voussoir section showing Freyssi Flat Jacks at Active Joint

Fig. 40.10

just in time to carry out the work which, in principle, involved cutting away the surrounding rock and sawing the temple into blocks of 20–30 tons weight for re-erection at the higher level.

To take the load of the roof as the vault-effect was removed, a series of vertical columns were erected, into which strut forced were gradually induced by Freyssi Flat Jacks as the dismantling proceeded (Fig. 40.11).

The magnitude of the operation can be judged from the fact that 540 Flat Jacks of 220 mm dia. were used.

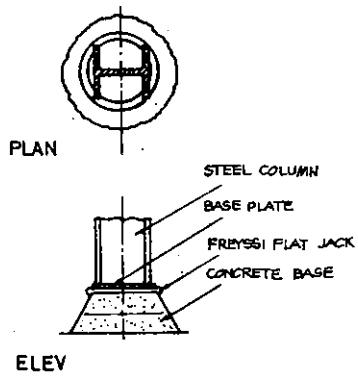


Fig. 40.11

(b) These jacks can also be used conveniently to temporarily lift bridge-girders (momentarily) in order to 'relieve' the neoprene bearings under them so that these bearings need to be designed only for the subsequent deformation permanently. This leads to economy in their design. These jacks can also be used to lift the super structures momentarily in order to reset the steel bearing plates and rollers to save on their design travels.

• Structural Levelling

The spectacular project, called Habitat'67, is a new concept of modern urban living. It was erected as a part of the Montreal Expo'67.

Habitat has 158 apartments each made of individual precast units so arranged that each forms the roof garden for the one above. The structure rises to 120 ft., contains six elevator shafts and has a system of longitudinal 'streets' for horizontal communication. All the elements were of prestressed concrete.

The 'street' girders were each made of six pre-cast elements individually erected with a special crane; these elements were supported temporarily on scaffolding and adjusted to correct line and level using Freyssi Flat Jacks of 18.9″ diameter with a maximum thrust of 222 short tons. 12/0.5″ Freyssinet pre-stressing cables were then inserted and stressed, after which the flat jacks were recovered and subsequently re-used.

CHAPTER 41

Fire Resistance of Structural Concrete

41.1 GENERAL

Fire resistance of structural concrete is assumed to have reached its limit when:

(1) The service load cannot be supported any longer.
(2) The structural member ceases to prevent the passage of flames and combustion gases.
(3) An excessive transfer of heat to the unexposed side occurs.

The latter two criteria are applicable to floors only, which perform a separating function. Although they are the criteria for failure in fire, in a structural sense they could be compared with the serviceability limit since the structure can be repaired after the fire provided the deformations are not excessive.

The conditions for simply supported beams are much more unfavourable than those for continuous beams or for beams which are built-in at the ends or capable of having thermal restraint. In various countries specific regulations have been made which do not quite agree with each other and are often of an arbitrary nature, except in the United States where more practical considerations apply from the point of view of fire insurance and the requirement of possible repair. There is a great difference between theoretical tests and practical experience in the case of actual fire.

In a newly built bungalow in North Carolina at which a fire occurred following a collision involving an oil drum and lasted for 1 1/2 hr. (during which temperature of 1400°F (750°C) must have been reached since glass melted), the walls were badly damaged and had to be rebuilt, but the prestressed lightweight concrete floor and roof consisting of double tee-beams without special insulation retained their full carrying capacity. This was ascertained from a subsequent loading test.

• *The different methods of dimensioning prestressed concrete members for a definite fire resistance rating:*

There are three possibilities of dimensioning prestressed concrete members to resist fire for a definite time.

(1) Test of full-sized members giving specific minimum dimensions of standard constructions.
(2) Tables showing minimum dimensions in which the cover to the steel (in fact the distance from the centroid of the steel) and the minimum dimensions are specified. These tables vary for different countries and are mostly based on conclusions made from tests. Obviously method (1) which is based on test results should have preference.
(3) Computation method. When the individual conditions are known, or can be reasonably assumed, fire resistance may be established from computation. In this case, the effect of continuity and/or actual restraint can be taken into account and thermal expansion and restraint, if any, ought to be considered.

The excellent performance of the prestressed concrete construction in Avianca Building, Bogota, was, no doubt, largely due to good detailing of the individual precast members as well as the entire structure.

41.2 SIMPLY SUPPORTED (UNRESTRAINED) SLABS AND BEAMS

If the underside of a slab is exposed to fire, the bottom of the slab will expand more than the top, resulting in a deflection of the slab. The tensile strength of the concrete and steel near the bottom of the slab will decrease as the temperature increases. When the strength of the steel at elevated temperature reduces to that of the stress *in the steel, flexural* collapse will occur.

41.3 CONTINUOUS SLABS AND BEAMS

Structures that are continuous or otherwise statically indeterminate undergo changes in stresses when subjected to fire. Such changes in stress result from temperature gradients within structural members, or changes in strength of structural materials at high temperatures, or both.

If the underside is exposed to fire, the bottom of the beam becomes hotter than the top and tends to expand more than the top. This differential heating causes the ends of a continuous beam to tend to lift from their supports thus increasing the reaction at the interior support. This action results in a redistribution of moments, i.e., the negative moment at the interior support increases while the positive moments decrease.

During the course of a fire, the negative moment

reinforcement (at top) remains cooler than the positive moment reinforcement because it is farther and hence better protected from the fire. Thus, the increase in negative moment can be accommodated. Generally, the redistribution that occurs is sufficient to cause yielding of the negative moment reinforcement. The resulting decrease in positive moment means that the positive moment reinforcement can afford to be heated to a higher temperature before failure will occur. Thus, it is apparent that the fire endurance of a continuous reinforced concrete beam is generally significantly longer than that of a similar simply supported beam loaded to the same moment intensity.

ASTM E119-79 specifies the test methods and procedures for determining the fire resistive properties of building components, and is the common standard.

41.4 FIRE ENDURANCE OF FLOORS AND ROOFS WHICH HAVE RESTRAINT TO THERMAL EXPANSION

If a fire occurs beneath a small interior portion of a large reinforced concrete slab, the heated portion will tend to expand and push against the surrounding part of the slab. In turn, the unheated part of the slab exerts restraining compressive forces on the heated portion. The compressive force, or thrust, acts near the bottom of the slab when the fire first occurs, but as the fire progresses the line of action of the thrust rises. If the surrounding slab is thick and heavily reinforced, the thrust forces that occur can be quite large, but considerably less than those calculated by use of elastic properties of concrete and steel together with appropriate coefficients of expansion. At high temperatures, creep and stress relaxation play an important role. Nevertheless, the thrust is generally great enough to increase the fire endurance significantly. In most fire tests of restrained assemblies, the fire endurance is determined by temperature rise of the unexposed surface rather than by structural considerations, even though the steel temperatures often exceed 815°C (1500°F).

41.5 HEAT TRANSMISSION

In addition to structural integrity, ASTM E119 limits the average temperature rise of the unexposed (top) surface of floors or roofs to 139°C (250°F) during standard fire tests. For concrete slabs, the temperature rise of the top surface is dependent mainly upon the thickness, unit weight, moisture content, and aggregate type. Other factors that affect temperature rise, but to a lesser extent, include air content, aggregate moisture content at the time of mixing, maximum size of aggregate, water-cement ratio, cement content, and slump.

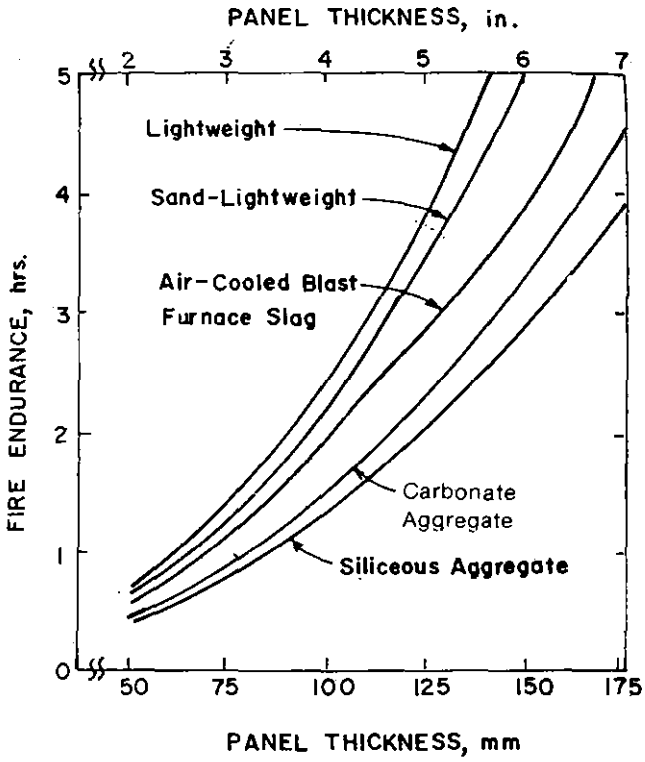


Fig. 41.1 *Effect of slab thickness and aggregate type on fire endurance of concrete slabs. [Based on 139°C (250°F) rise in temperature of unexposed surface.]*

• *Effect of Slab Thickness and Aggregate Type*

Figure 41.1 shows the relationship between slab thickness and fire endurance for structural concretes made with a wide range of aggregates. The curves are for air-entrained concretes fire tested when the concrete was at the standard moisture condition (75 per cent RH at middepth), made with air-dry aggregates having a nominal maximum size of 19 mm (3/4 in.). On the graph, lightweight aggregates include expanded clay, shale, slate, and fly ash that make concrete having a unit weight of about 1520 to 1680 kg/m^3 (95 to 105 pcf) without sand replacement. The unit weight of air cooled blast-furnace slag aggregate has little effect on the resulting fire endurance of the normal weight concretes in which it is used.

• *Effect of Unit Weight*

Fire endurance generally increases with a decrease in unit weight. For structural concretes, the influence of aggregate type may overshadow the effect of unit weight. For low density concretes, a relationship exists between unit weight (oven-dry) and fire endurance, as shown in Fig. 41.2. The

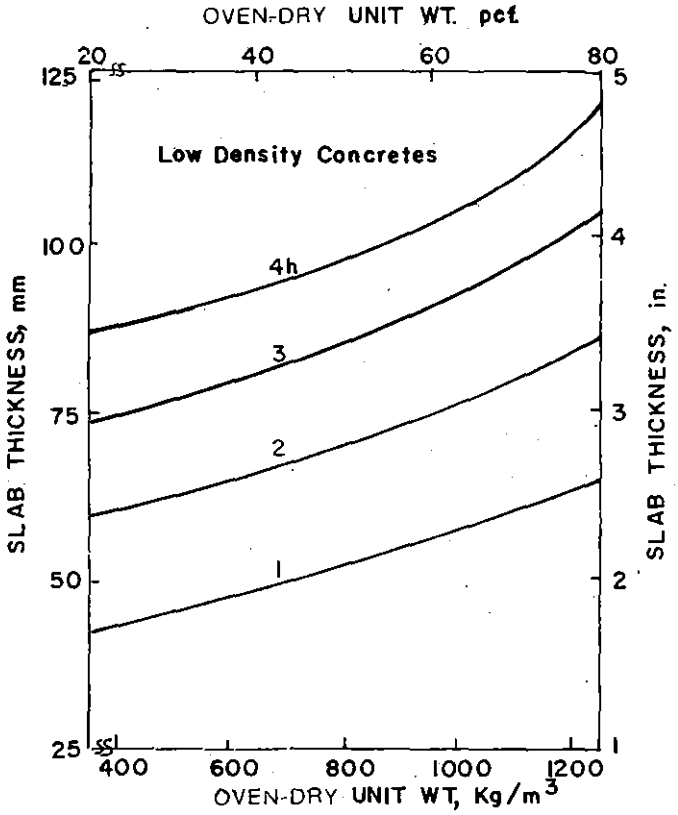


Fig. 41.2 *Effect of dry unit weight and slab thickness on fire endurance of low density concretes. [Based on 139°C (250°F) rise in temperature of unexposed surface]*

curves in Fig. 41.2 represent average values for concretes made with dry vermiculite or perlite, or with foam (cellular concrete), with or without masonry sand.

- ***Effect of Moisture Condition***

The moisture content of the concrete at the time of test and the manner in which the concrete is dried affect fire endurance. Generally, a lower moisture content or drying at elevated temperature of 50 to 95°C (120 to 200°F) reduces the fire endurance. A method is available for adjusting fire endurance of concrete slabs for moisture level and drying environment (Appendix 5, ASTM E119-79).

- ***Effect of Air Content***

The fire endurance of a concrete slab increases with an increase in air content, particularly for air contents above 10 per cent. Also, the improvement is more pronounced for lightweight concrete.

- ***Effect of Sand Replacement in Lightweight Concrete***

As indicated in Fig. 41.1, replacement of lightweight aggregate fines with sand results in somewhat shorter fire endurance periods.

- ***Effect of Aggregate Moisture***

The influence on fire endurance of absorbed moisture in aggregates at the time of mixing is insignificant for normal weight aggregates but may be significant for lightweight aggregates. An increase in aggregate moisture increases the fire endurance. Thus, the fire endurances obtained from Fig. 41.1 represent minimum values.

- ***Effect of Water-cement Ratio, Cement Content and Slump***

Results of a few fire tests indicate that these factors, per se, within the normal range for structural concretes, have almost no influence on fire endurance.

- ***Effect of Maximum Aggregate Size***

For normal weight concretes, fire endurance is improved by decreasing the maximum aggregate size.

41.6 FIRE ENDURANCE OF WALLS

In fire test of walls consisting of plain concrete, reinforced concrete and concrete masonry units, the fire endurance is generally governed by heat transmission rather than structural consideration assuming that the structural requirement of the design code had been satisfied.

41.7 REINFORCED CONCRETE COLUMNS

Reinforced concrete columns have performed well during exposure to fire throughout the history of concrete construction.

Table 41.1 *Fire Endurances Proposed by Hull and Ingberg*

Aggregate type	Minimum area of round or square cross section, cm^2 (sq in.)	Concrete cover, mm(in.)	Fire endurance classification, hr.
Siliceous	710 (110)	38 (1-1/2)	1-1/2
Siliceous	1,290 (200)	38 (1-1/2)	2-1/2
Siliceous	1,290 (200)[1]	38 (1-1/2)	3-1/2
Siliceous	1,613 (250)	64 (2-1/2)	3
Siliceous	1,613 (250)[1]	64 (2-1/2)	6
Traprock & slag[2]	1,290 (200)	38 (1-1/2)	4
Carbonate	1,290 (200)	38 (1-1/2)	6

1. Mesh in cover
2. Air-cooled slag

Columns larger than 305 mm (12 in.) in diameter or

305 mm (12 in.) square are assigned 3 hr and 4 hr fire resistance classifications in most building codes in America.

It is suggested that the information in Table 41.1 be used for designing reinforced concrete columns for exposure to fire. This information is based on the results of a comprehensive series of fire tests on concrete columns reported in 1925. Only those types of columns which are commonly used today are included in the table.

Columns designed in accordance with the requirements of Table 41.1 have been used in concrete buildings for about 50 years. These ratings combined with requirements for structural adequacy have given economical column sizes that have performed well.

Recently analytical procedures have been developed for estimating temperature distributions in concrete columns during exposure to fire and for designing concrete columns for specific fire endurances and loads.

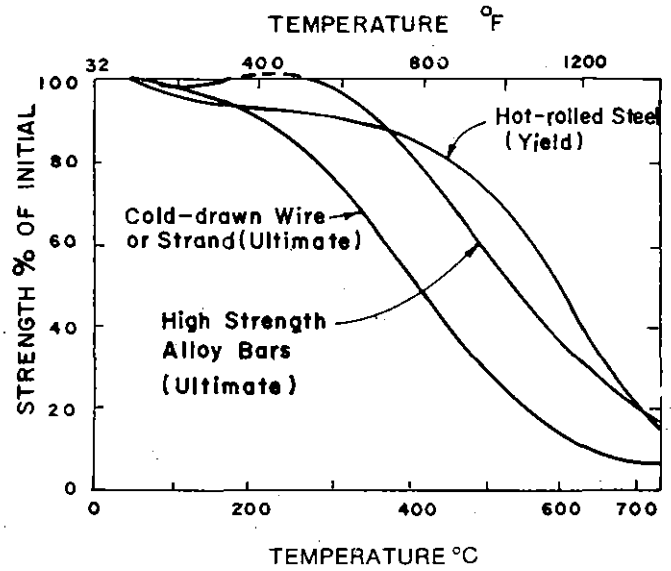


Fig. 41.3 *Strength of certain steels at high temperatures*

41.8 PROPERTIES OF STEEL AT HIGH TEMPERATURES

• *Strength*

Figure 41.3 shows the influence of temperature on the strength of certain steels. Included are data on the yield stress of structural steels and ultimate strengths of cold-drawn steel and high strength alloy steel bars used in prestressed concrete. Generally, the strengths of steels decrease with increasing temperature but ultimate strengths of hot-rolled steels are often slightly higher at temperatures up to about 260°C (500°F) than they are at room temperature.

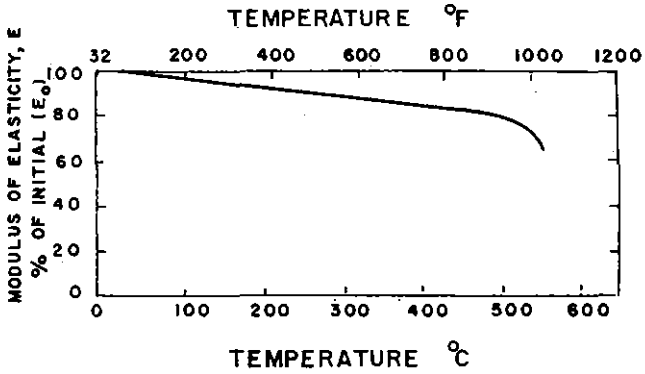


Fig. 41.4 *Modulus of elasticity of steel at high temperatures*

• *Modulus of Elasticity*

The modulus of elasticity of steel decreases with increasing temperature as shown in Fig. 41.4. Young's modulus for ferritic steels decreases linearly up to about 480°C (900°F). Above 480°C (900°F) the modulus decreases at a higher rate. Most investigators indicate that modulus of elasticity is not dependent upon microstructure of the steel. Consequently, the curve in Fig. 41.4 is representative of the types of steel used in concrete construction. However, there is a greater decrease in Young's modulus for prestressing steel than that shown in Fig. 41.4.

• *Thermal Expansion*

The average linear thermal expansion of ferritic steels over a temperature range of 200 to 650° (400 to 1200°F) is shown in Fig. 41.5. The coefficient of thermal expansion is not constant over this temperature region but increases as temperature increases. The temperature dependence of the coefficient of thermal expansion, α, is approximately by the formula:

$$\alpha = (11 + 0.0036\theta_1) \times 10^{-6}/°\text{C or}$$
$$\alpha = (6.1 + 0.002\theta_2) \times 10^{-6}/°\text{F}$$

in which $\theta_1(\theta_2)$ is temperature in degrees Celsius (degrees Fahrenheit)

• Stress-Strain relationships

Stress-strain relationships for several types of steel have been reported by Harmathy and Stanzak.

Such curves for an ASTM A-36 steel are shown in Fig. 41.6. Figure 41.7 shows a family of stress-strain curves for cold-drawn prestressing steel, ASTM A421.

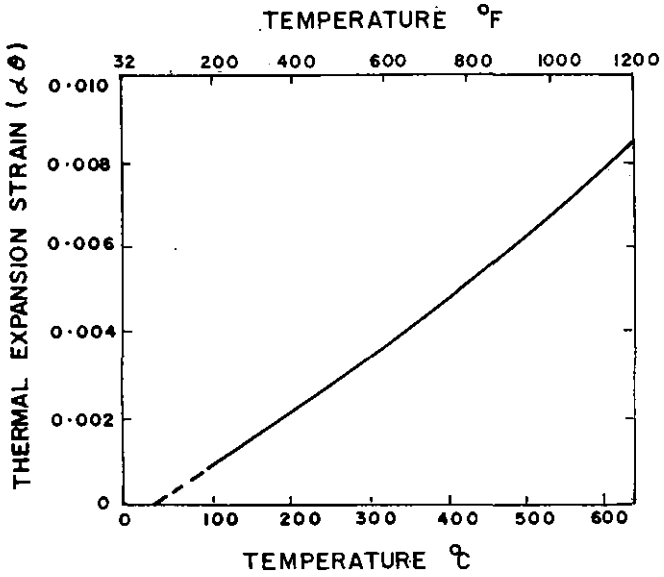


Fig. 41.5 *Thermal expansion of ferritic steels at high temperatures*

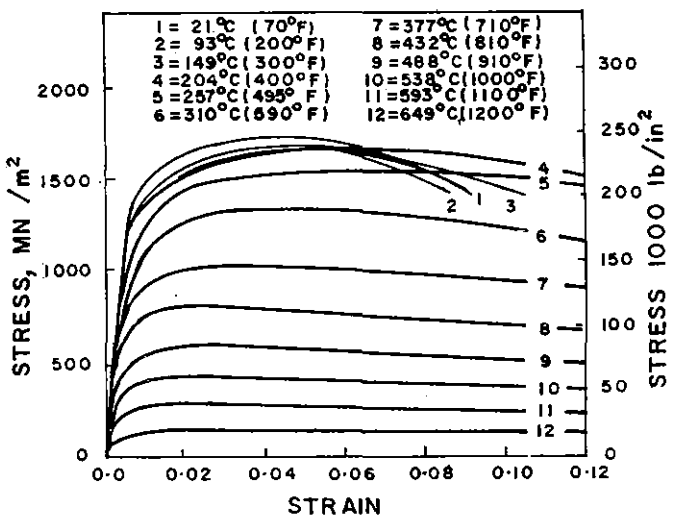


Fig. 41.7 *Stress-strain curves for prestressing steel (ASTM A-421) at various high temperatures*

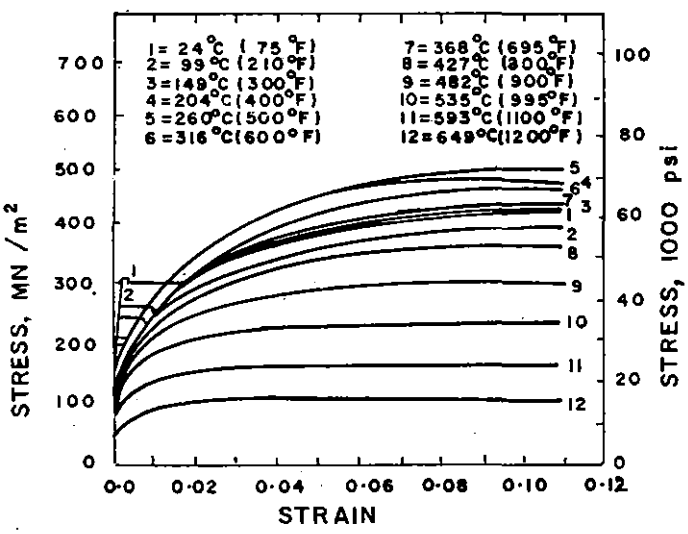


Fig. 41.6 *Stress-strain curves for structural steels (ASTM A-36) at various high temperatures*

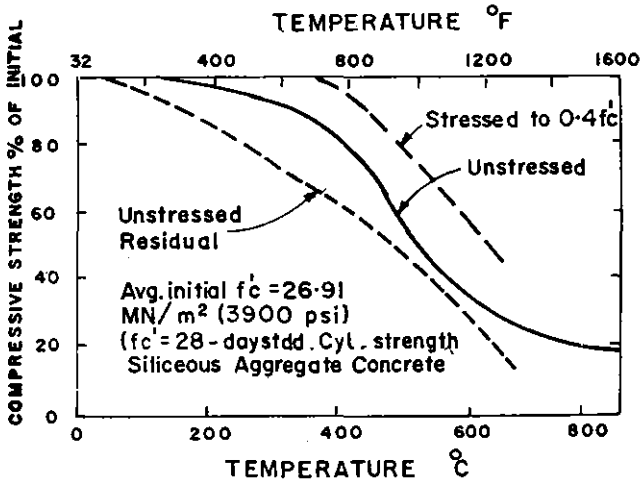


Fig. 41.8 *Compressive strength of siliceous aggregate concrete at high temperature and after cooling*

41.9 PROPERTIES OF CONCRETE AT HIGH TEMPERATURES

• *Compressive Strength*

Compressive strengths of concretes made with different types of aggregates are shown in Figs. 41.8, 41.9, 41.10. Curves designated "unstressed" are for specimens heated to test temperature with no superimposed load and tested hot. Strengths of specimens heated while stressed to 0.4 f'_c (f'_c = 28-day cylinder compressive strength of concrete) and then tested hot are designated "stressed to $0.4f'_c$". The "unstressed residual" strengths were determined from specimens heated to test temperature, cooled to room temperature, stored in air at 75 per cent RH for 6 days and then tested in compression. Note that the "stressed" strengths are higher than the "unstressed" strengths. Abrams found that stress levels of 0.25 to 0.55 f'_c had little effect on the strength obtained. The "unstressed residual" strengths were in all cases lower than the strengths determined by the other two procedures. Abrams also noted that original concrete strengths between 27.6 and 44.8 MPa

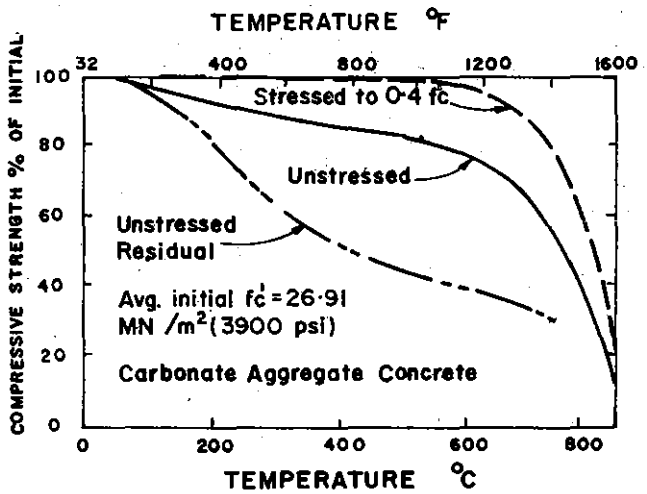


Fig. 41.9 *Compressive strength of carbonate aggregate concrete at high temperature and after cooling*

(4000 and 6500 psi) had little effect on the percentage of strength retained at test temperature. In Fig. 41.10 the "sanded" specimens were made with sand replacing 60 per cent of the lightweight fines, by volume.

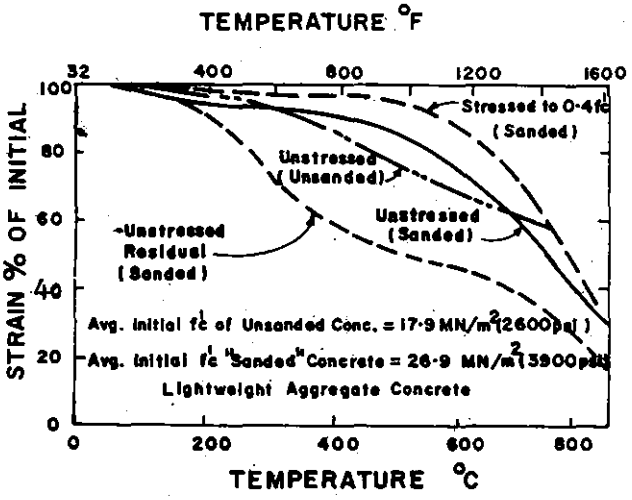


Fig. 41.10 *Compressive strength of lightweight concrete at high temperature and after cooling*

The "unsanded" concrete was the kind used in masonry block manufacture. Harmathy and Berndt reported data on the compressive strength of cement paste and a lightweight concrete from tests performed on specimens held at the target temperature in no-load condition for a period of 1 to 24 hr.

Further data on the strength of concrete at high temperatures have been reported by Zoldners, Malhotra, Saeman and Washa, Binner, et al., and Weigler and Fischer.

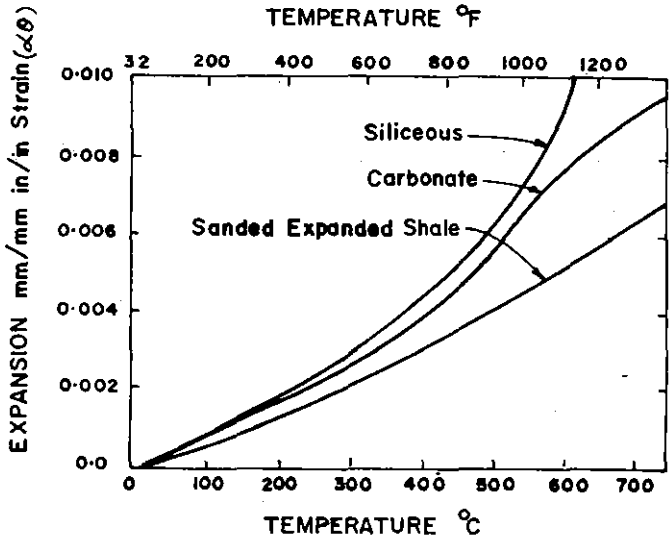


Fig. 41.11 *Thermal expansion of concrete at high temperatures*

- ***Thermal Expansion***

Figure 41.11 shows data on linear thermal expansion of concretes made with different aggregates. The data were obtained by Cruz.

Harmathy and Allen studied the thermal expansion of 16 different concretes used in masonry units. Among these, pumice concretes were found to exhibit considerable shrinkage at temperatures above 315°C (600°F). Dettling pointed out that thermal expansion of concrete is influenced by aggregate type, cement content, water content, and age. Philleo performed tests on a carbonate aggregate concrete using a different technique. He obtained somewhat higher values than those obtained by Cruz at temperatures above 370°C (700°F)

- ***Modulus of Elasticity and Shear Modulus***

Figures 41.12 and 41.13 show the effect of high temperatures on the moduli of elasticity and shear of concretes made with three types of aggregate. The data were obtained by Cruz.

From Cruz's data, it appears that aggregate type and concrete strength do not significantly affect moduli at high temperature.

Philleo obtained values for modulus of elasticity of a carbonate aggregate concrete using a dynamic method. His results agree closely with those obtained by Cruz up to about 370°C (700°F). From 370 to 650°C (700 to 1200°F),

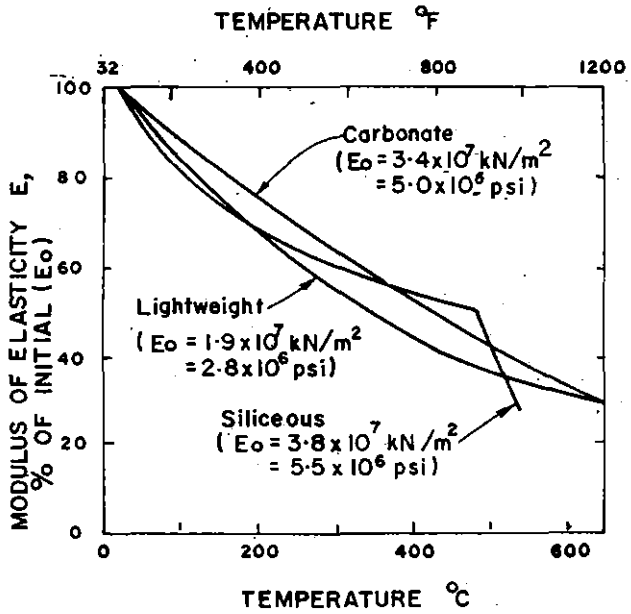


Fig. 41.12 *Modulus of elasticity of concrete at high temperatures*

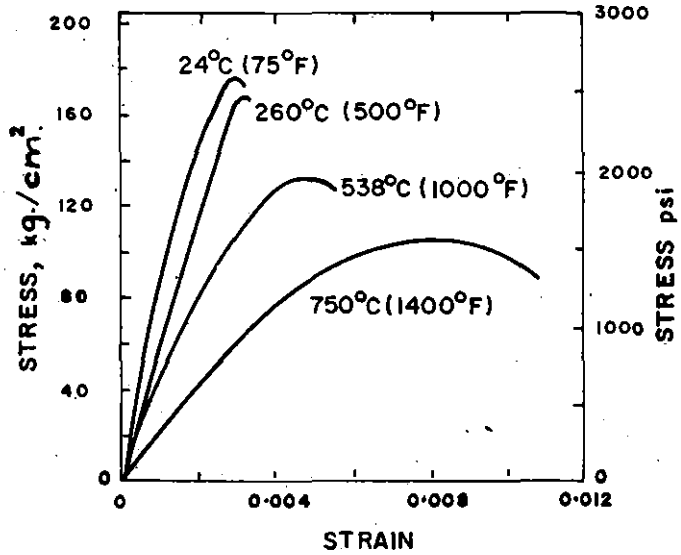


Fig. 41.14 *Stress-strain curves for a lightweight masonry concrete at various high temperatures*

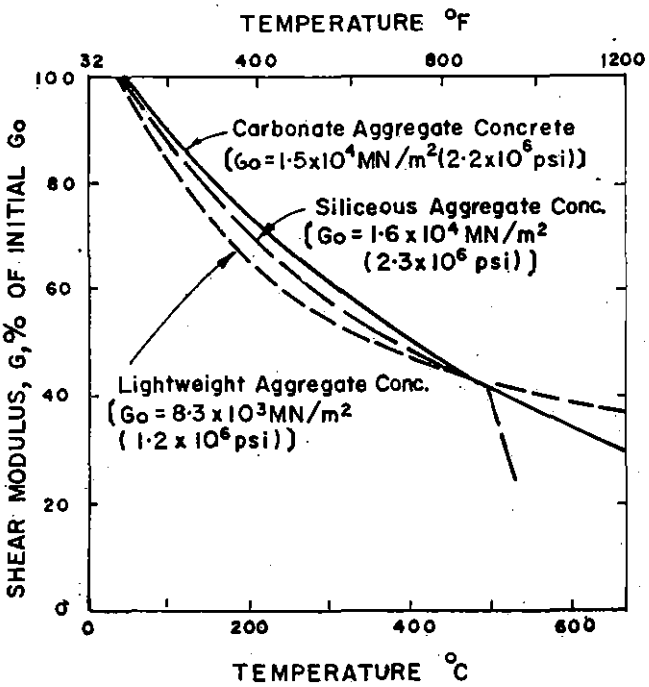


Fig. 41.13 *Shear modulus of concrete at high temperatures*

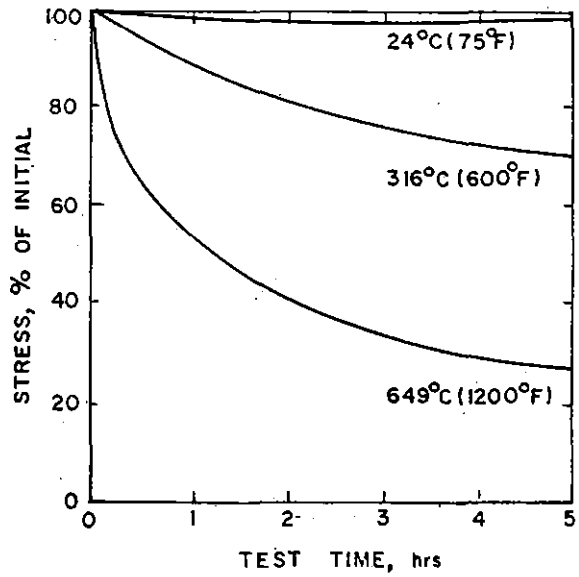


Fig. 41.15 *Stress relaxation of a carbonate aggregate concrete*

Philleo obtained higher values. Harmathy, and Saeman and Washa determined the modulus of elasticity in compression and found little change up to about 200°C (400°F).

- ***Poisson's Ratio***

Philleo and Cruz reported data on Poisson's ratio of concrete at high temperatures. Even though Philleo indicated a decrease in Poisson's ratio, both he and Cruz pointed out that results were erratic and no general trend of the effect of temperature was clearly evident.

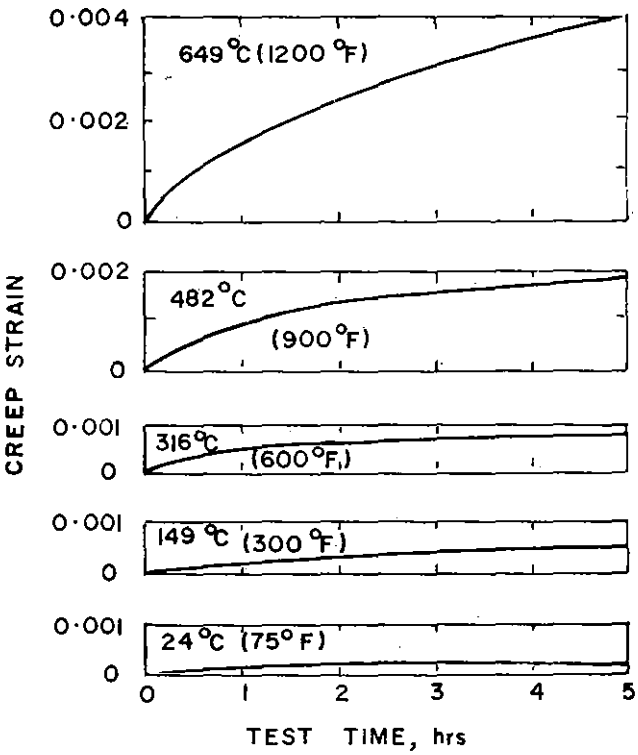


Fig. 41.16 *Creep of a carbonate aggregate concrete at various temperatures (applied stress = 12.42 MN/m² (1800 psi), f'_c = 27.6 MN/m² (4000 psi))*

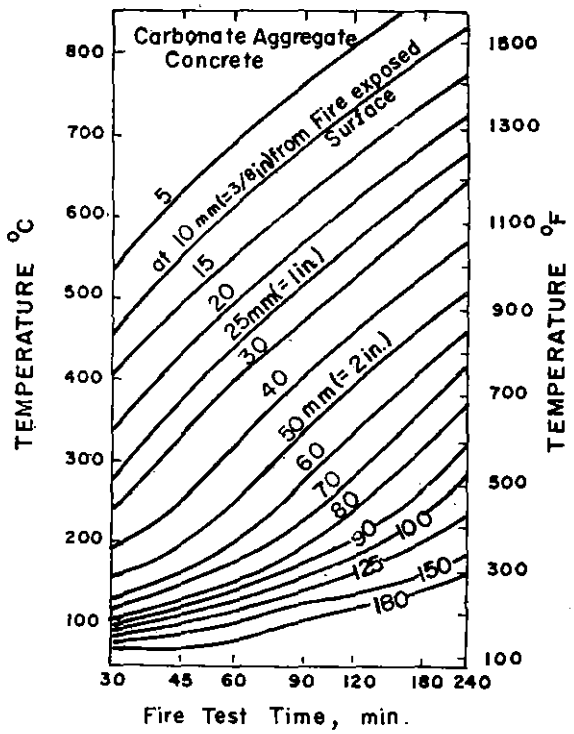


Fig. 41.17 *Temperatures within slabs during fire tests — carbonate aggregate concrete*

• ***Stress-strain Relationships***

Stress-strain relationships (in compression) of a lightweight masonry concrete (expanded shale aggregate) were reported by Harmathy. Figure 41.14 shows some of the data. Kordina and Schneider studied the stress-strain response of normal-weight concretes at variable temperatures under a number of loading conditions.

• ***Stress Relaxation and Creep***

Some data on stress relaxation and creep at high temperatures of a carbonate aggregate concrete were reported by Cruz. Figures 41.15 and 41.16 show the data graphically for a 5-hr test period. Nasser and Neville reported that age, moisture-condition, type and strength of concrete, and stress-strength ratio affect creep of concrete at high temperatures. Mukaddam and Bresler conducted studies on the creep of concrete at variable temperatures.

41.10 TEMPERATURE DISTRIBUTION WITHIN CONCRETE MEMBERS EXPOSED TO FIRE

• ***Slabs***

Figures 41.17, 41.18 and 41.19 show temperatures within concrete slabs during fire tests. Total slab thickness did not significantly affect the temperatures except for very thin slabs or when the temperatures were less than about 204°C (400°F). Temperatures in slabs were obtained from specimens 3 × 3 ft in plan with protected edges.

• ***Rectangular and Tapered Joists***

Computed and measured temperatures within rectangular beams made with quartzitic gravel have been reported by Ehm. The beam sizes he tested ranged in size from 64 × 305 mm (2.5 × 12 in.) to 279 × 559 mm (11 × 22 in.).

Figures 41.20, 41.21 and 41.22 show temperature distributions along the "centre-line" at various distances from the bottom of the beam and for widths up to 254 mm (10 in.) for normal-weight carbonate aggregate concrete for fire endurance periods of 1, 2, and 3 hr. The width b is the beam width for rectangular members and the width at a distance u from the bottom for the tapered member. These charts were generated from test data obtained from tests of rectangular and tapered members. Tests were carried out in Underwriters' Laboratories Floor Furnace, Northbrook, Illinois, and Portland Cement Association's Beam Furnace, Skokie, Illinois. Temperature distributions obtained in other furnaces may differ from those shown due to differences

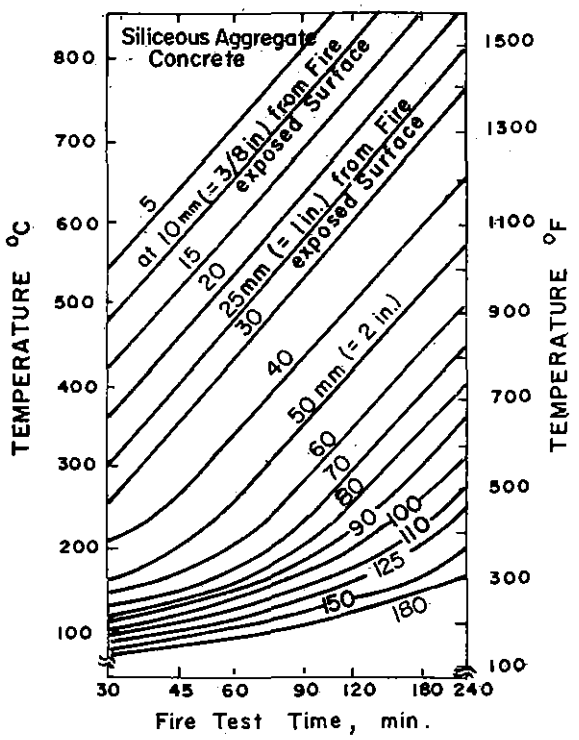


Fig. 41.18 *Temperatures within slabs during fire tests—siliceous aggregate concrete*

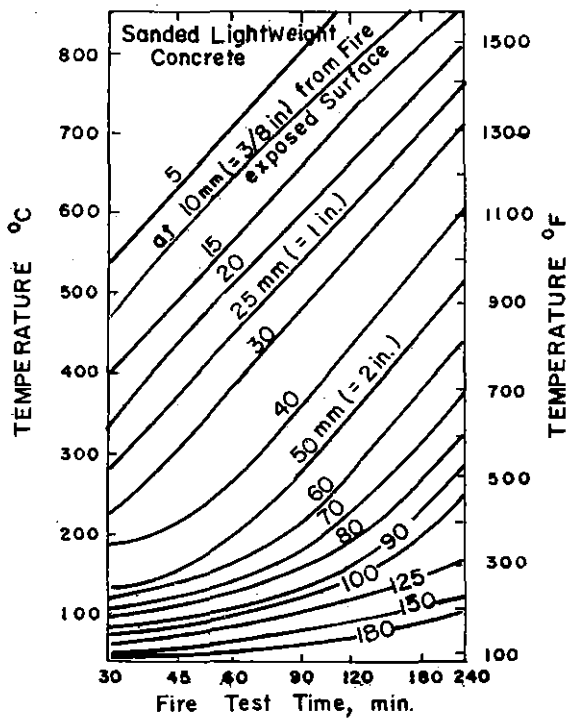


Fig. 41.19 *Temperatures within slabs during fire tests—sanded lightweight concrete*

in furnace size and design, furnace wall construction, and flame type.

Some National Codes of Practice specify certain "cover" and/or "minimum dimensions" to achieve a specific period of fire rating. These vary to a great extent in different countries. In the USA the P.C.I. Committee on Fire Resistance Rating adopted a Code provision in December 1971 which, however, has not been published on the consideration that some values are rather conservative.

In Table 41.2 the British Code CP 110 requirements, and the P.C.I. proposals (1971) have been incorporated.

In the British Code the end-restraint or continuity has not been directly taken into consideration. Advice has been given in such a case to use the values for the next lower rating-time.

In the American proposal a continuous beam or slab is defined as that which has been designed for continuity at least for superimposed dead load and full live load, and has at least 20 per cent of the maximum negative moment reinforcement made continuous "throughout the span".

The American proposal also includes restrained beams and slabs which are framed into a building in such a manner that restraints to thermal expansion occur although structurally the support condition could be simply supported.

By comparison it appears that the main difference between the American proposal and the British Code lies in the fact that the former recognizes the different behaviour of continuous beam "spaced up to" 4 ft (1.2 m) on centres and beams "speed over" 4 ft (1.2 m) on centres. Closely spaced beams act like an anisotropic slab showing better transverse distribution of load. The capacity of load distribution to adjacent ribs significantly improves fire performance since it appreciably reduces the risk, generally present in primary beams, of sudden collapse due to serious spalling. Obviously there is no clear-cut demarcation (beams having slightly more spacing than 4 ft (1.2 m) may not act very differently) but the demarcation line has been drawn on the basis of practical experience, especially with standard double-T units having rib centres on 4 ft (1.2 m).

For continuous beams spaced up to 4 ft (1.2 m) centres the dimensions (width and cover) are considerably higher in the British Code (except the cover for 1 hr rating). It should be noted that for a fire rating period of over 2 hr. the dimensions in the American proposal are almost halved for this category of beam. Even for beams spaced over 4 ft (1.2 m) centres the dimensions in the American proposal are appreciably less for fire rating periods of 2 to 4 hr. whereas the dimensions for 1 to 1-1/2 hr. period are slightly higher.

Regarding simply-supported *beams*, the British and American results are comparable although the American figures are slightly on the high side.

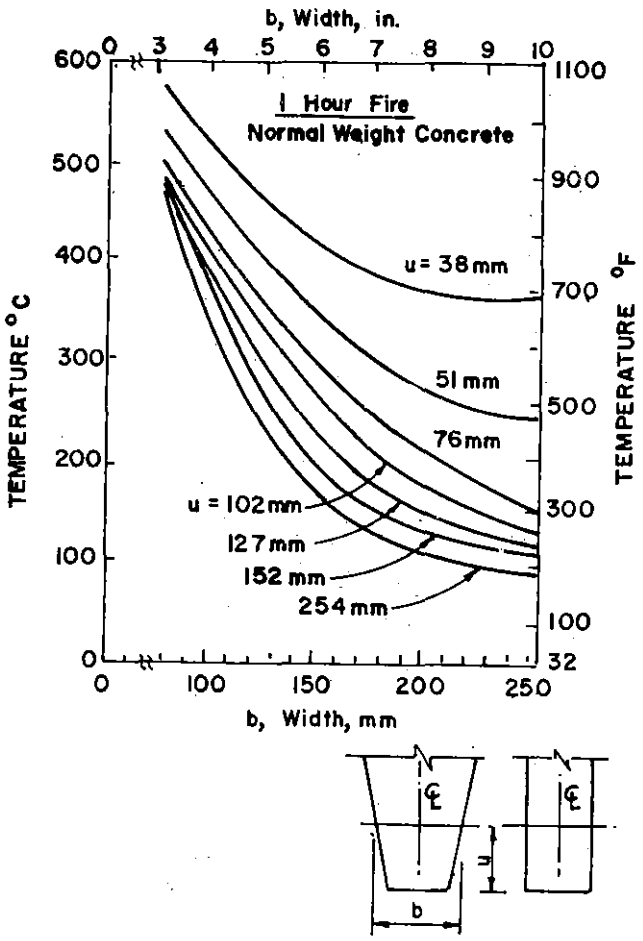


Fig. 41.20 *Temperatures in normal weight concrete rectangular and tapered units at 1 hr of fire exposure*

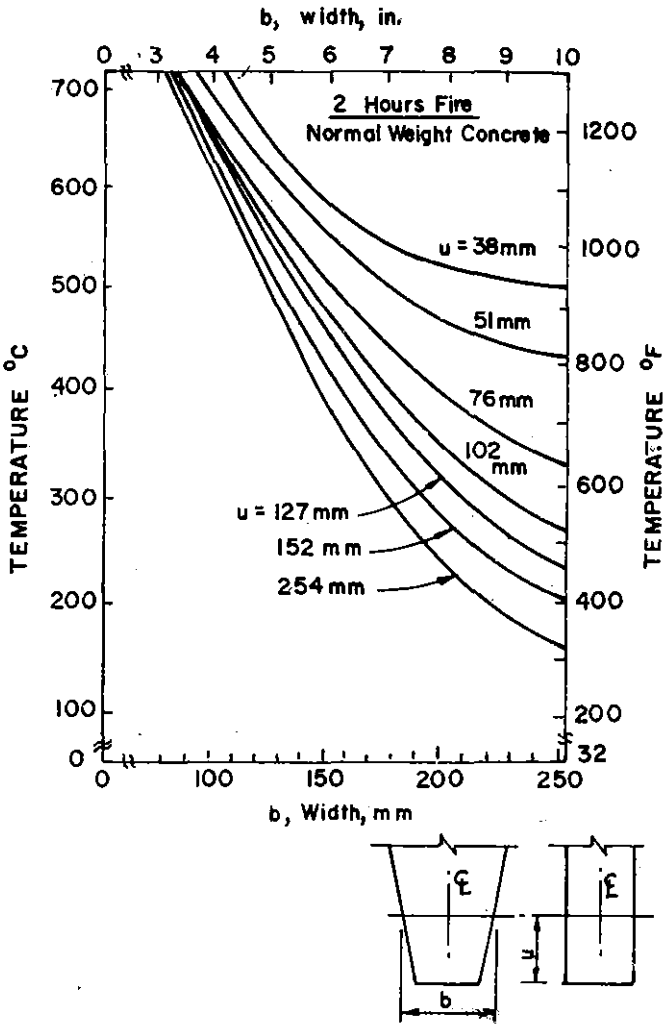


Fig. 41.21 *Temperatures in normal weight concrete rectangular and tapered units at 2 hr of fire exposure*

With regard to simply supported *slabs* the American proposals for 2 to 4 hr. range are less favourable, but below 2 hr. the difference is very small. As far as continuous slabs are concerned, the cover in the American proposal is considerably less than in the British Code expect for the 1 hr. rating. The thickness, on the contrary, is slightly higher in the American proposal.

Fire rating dimensions below 1 hr. have not been included in the Table, as the British Code gives dimensions relating to a 1/2 hr. period whereas the American proposal gives a 3/4 hr. period.

With regard to slabs the British Code also includes specifications for cored slab, hollow ribbed floor, upright T-slabs and inverted channel or U-slab. It is apparent that the P.C.I. results are quite progressive in many respects.

For prestressed lightweight concrete members Table 41.3 shows the dimensioning as proposed in the C.E.B. Manual on Lightweight Concrete. The definition of continuous and restrained members in this case is the same as in the American proposal.

In the case of a tapering beam the width refers to that at the centroid of steel. Where the cover exceeds 40 mm, supplementary reinforcement in the form of welded mesh should be provided to avoid spalling.

In the thickness of a floor, non-combustible floor finishes may be included.

If the end-span of a continuous beam or slab is so designed that the end-support can take the same moment as the intermediate support, the end-bay can be considered as continuous.

P.C.I. published a report on the Fire Resistance of Post-Tensioned Structures. The specification for cover corresponding to different periods of fire rating is virtually identical to that in the American proposal shown in Table 41.2.

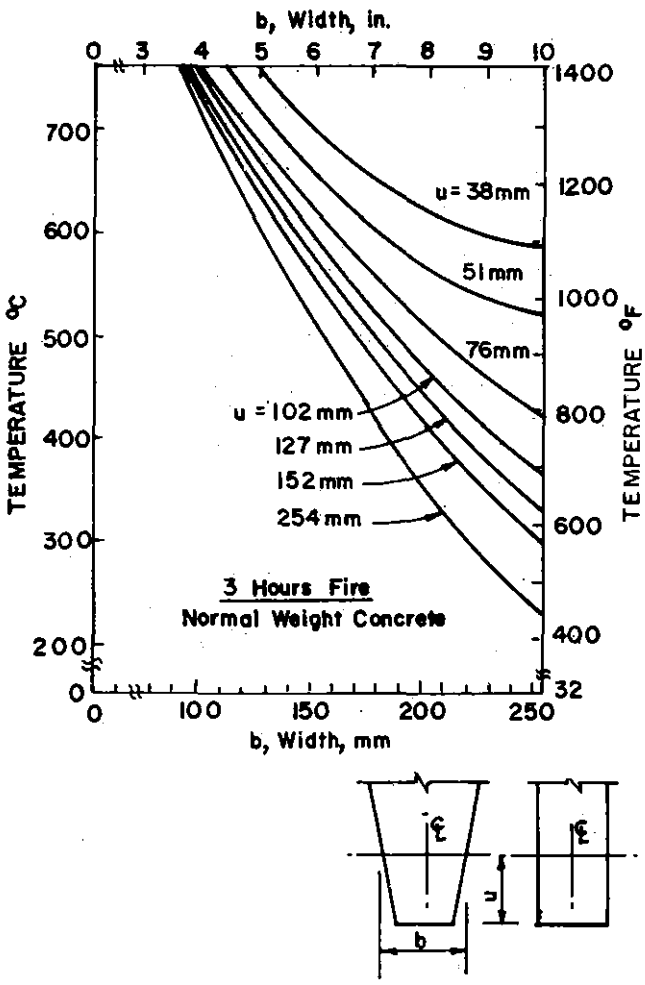


Fig. 41.22 *Temperatures in normal weight concrete rectangular and tapered units at 3 hr of fire exposure*

Table 41.2 *Minimum Dimensions in Millimetres for General Concrete*

	End condition				4 hours	3 hours	2 hours	1 1/2 hours	1 hour
BEAM	Simply supported	British code		Width	280	240	180	140	110
				Cover	100	85	65	50	40
		American proposal		Width	305	255	195	155	115
				Cover	95	80	65	50	45
	Continuous beams	British code		Width	240	180	140	110	80
				cover	85	65	50	40	25
		American proposal	1.2 m or less on centres	Width	110	98	79	65	65
				Cover	45	38	33	33	33
			Over 1.2 m on centres	Width	195	155	115	115	115
				Cover	65	50	45	45	45
SLAB	Simply supported	British code		Thickness	150	150	125	125	100
				Cover	65	50	40	30	25
		American proposal		Thickness	178	158	127	110	90
				Cover	71	59	46	38	28
	Continuous beams	British code		Thickness	150	125	125	100	90
				Cover	50	40	30	25	15
		American proposal		Thickness	178	158	127	110	90
				Cover	25	25	20	20	20

Table 41.3 *Minimum Dimensions in Millimetres for Lightweight Concrete*

	Member type and support condition	Minimum dimension	Minimum dimensions for fire resistance rating indicated in millimetres				
			4 hours	3 hours	2 hours	1 1/2 hours	1 hour
1	Beams, simply supported	Width	250	200	160	130	100
		Cover	100 (80)	90 (65)	75 (50)	65 (40)	50 (30)
2	Beams, composite with slab and continuous, spaced 1200 mm or less on centres	Width	100	90	70	65	65
		Cover	35	35	35	30	30
3	Beams, composite with slab and continuous, spaced over 1200 mm on centres	Width	200	180	130	100	90 (80)
		Cover	70 (65)	60 (50)	50 (40)	45 (30)	40 (20)
4	Simply supported slab	Thickness	135	115	95	80	65
		Cover	60	50	35	30	25
5	Continuous solid slab	Thickness	135	115	95	80	65
		Cover	30	25	20	20	20

NOTE In this table the figures in brackets indicate the appropriate values as specified in the British Code of Practice CP 110.
In CP 110 no provision has been made for continuous beams spaced up to 1200 mm on centres. Therefore, the approximate values for continuous beams in the British Code have been shown.
For prestressed lightweight concrete slab the British Code (CP 110) suggests the same dimensions as the normal-weight aggregate concrete.

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CHAPTER 42

Economics and Quantity-Trends in Alternative Bridge Structure Schemes

42.1 SYNOPSIS

Talking alone never pulled out any stumps. Only first-hand, first-rate, commercially biased, result-oriented practising professional experience, gained in cold-blooded competition, gives a workman-like understanding of the true mechanics of what actually constitutes 'economics' in the study of alternative bridge structure schemes in given situations. This, tempered with scientific unbiased analysis of the results, and sifted with clinically inquisitive curiosity of a practising professional, can unravel the 'quantity figures' on which the economics is based.

Such 'quantity figures' represent a fund of practically useful information which is not found in straight textbooks, class-room lectures or in laboratory exercises and research reports. (Real professionals are generally too busy to dwell in mundane publications.)

Such distilled 'quantity figures', when presented in a tool-kit manner, reflect the usefulness of hundreds of competitive engineering man-months of top quality computer aided analysis and design of myriads of alternative actual bridge structures, with many of which the author was personally involved first hand, more as a doer than as just a checker.

With this as the background, the author presents here first the facts of bridge economics and then the results of certain alternative case-studies that show the relative economics of different structure schemes. This is followed by various practical inferences and finally are presented the actual all-important 'quantity trends' in various alternative bridge structure schemes. These represent an invaluable amalgam of high strung perspiration and inspired slog.

42.2 INTRODUCTION

There is no one form of design which would be always most economical. It is only by comparing a few designs that the economic design can be found in a particular set of conditions.

However, sometimes the quantities of concrete and steel expressed per square metre of deck area are quoted as indicative of economy although these figures are not the only ones which govern the cost of bridge. Other considerations which influence the cost are: the length of the individual spans, the type of deck cross-section, the number of longitudinals in cross-section, the width of the bridge, the depth and type of foundation, excavation and fill involved, the cost of formwork, the cost of materials and labour, whether reinforced concrete or prestressed concrete, whether precast or cast-in-situ, the method of erection, the size of contract, etc.

Difficult foundations alone may justify increase of span lengths to reduce the number of foundations, in spite of increase in the cost of the deck owing to longer spans.

For smaller spans, i.e. below 30 m, the comparison is even more difficult and depends entirely on local circumstances.

Figure 42.1 shows the usual relationship between the length of span and the cost of the bridge per unit of plan area of the deck. It is comparatively high for short spans, reduces for medium spans with repetitive elements, and rises for longer spans.

Figure 42.2 indicates average relative quantities of concrete and steel per square unit of deck plan area for various types of decks for a given and design criteria. It appears that for spans smaller than about 35 m, simply-supported beams are probably the cheapest type of construction from purely materials point of view.

For spans larger than about 45 m, the cost of simply supported beams increases rapidly and becomes practically prohibitive at more than about 60 m.

Where the deck is less than about 8 m above the ground and foundations are simple and shallow, shorter spans of the order of about 18 m to 25 m can be more economical. Still shorter spans, using about 10 m span, 3 to 4 span continuous modules, reinforced concrete solid slab deck, may be more convenient if not more economical but this might adversely affect the aesthetics and also require more subsequent maintenance, particularly if protection against scour at its numerous piers is called for at a later date.

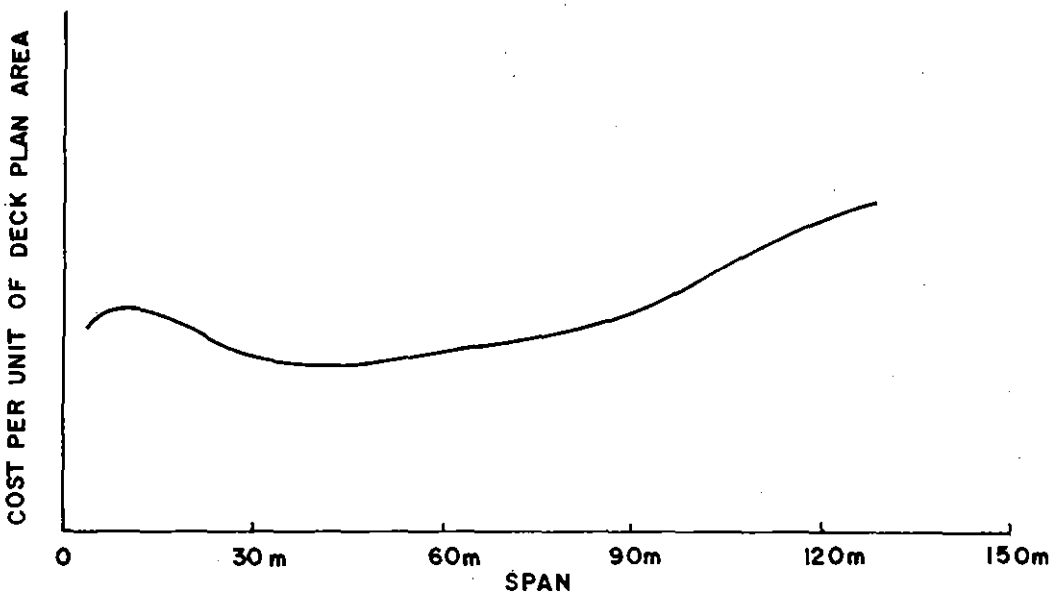

Fig. 42.1

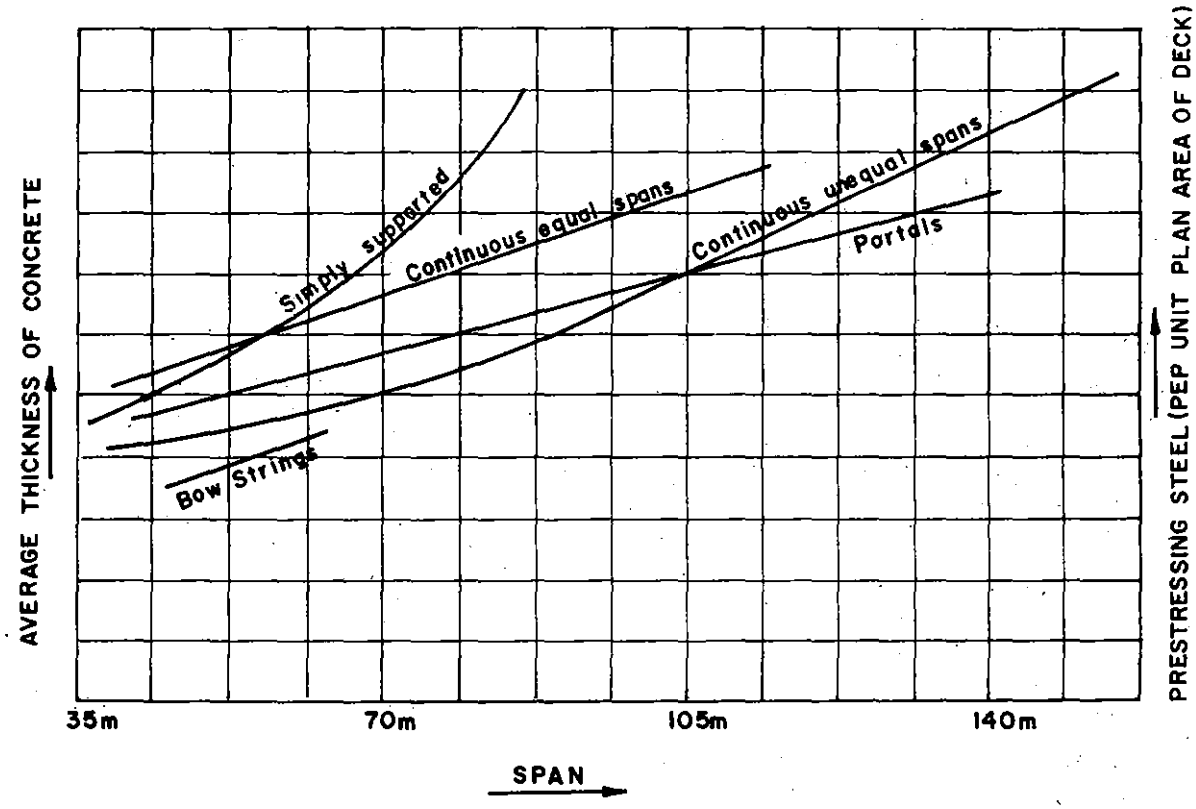

Fig. 42.2 *Approximate quantities of concrete and steel in various types of bridge decks*

NOTE Actual values will depend on the loading and the design specifications. The above curves are only indicative and relative.

In the 18 m to 25 m span range, under the circumstances, the most economical deck for a multi-span bridge could be:

- precast prestressed concrete simply supported girders which may be made continuous for finishings and live load by the in-situ reinforced concrete top slab (assuming that the effect of differential settlement of foundations is small).
- Precast reinforced concrete girders simply supported for self weight but temporarily propped at one-third points till the cast in place top slab becomes composite with the beams. (May be made continuous for finishings and live load.)
- voided prestressed concrete slab, continuous if differential settlement of foundations is not a problem.

Continuous beams of equal spans do not seem to bring significant savings up to about 45 m spans, but become more economical above that span length, particularly if the cost of foundations is not, in general, different from what it would be in case of simply supported beams.

Continuous beams with unequal spans offer noticeable economics. The range of applicability of such solutions depends on the ratio of short span to long span. With this ratio approaching 0.3, the bridge deck may be considered 95% fixed and the longer span may be increased even to about 90 m. For these very high degrees of fixity obtained by alternating short spans, foundations may become expensive because of the extreme piers needing to resist the uplift. Counter-weights or anchorages may become necessary which can add considerably to cost.

Portal frames lead to quantities somewhat similar to those in continuous beams with unequal spans. Generally, portals would be slightly more expensive than continuous beams with unequal spans for spans less than about 90 m. The cost of foundations, however, may tilt the balance.

It appears that for portals the quantities of steel increase slowly with the span. This is because of the thrust, although this thrust must be paid for in the foundations, depending on the nature of the ground.

Concrete arches are seldom prestressed because prestressing adds very little to the natural advantage of thrust in arch design.

Bowstring girders are cheaper solutions from the point of view of quantities of materials, but the cost of formwork is high and may effect the overall economy. Their better aesthetic appearance, however, cannot be debated out in terms of economy alone.

A tenderer's policy may involve putting large sums of money against certain items of the work because those particular items would be completed earlier in the game, and the money will thus be recouped at an early stage in the contract. All such factors can distort the data collected from tender bids as a basis for building up cost and quantity information data bank. However, scathing exposure to and a workman-like analysis of a very large number of fiercely competitively designed and actually constructed bridges, of various types, first hand, in cold commercial competition (which is far different from class-room lecturing, pure academic noise, or restrictive staid office experience, where one does not actually evolve, conceive and design the alternatives — and build first hand, but essentially only proof reads and oversees someone else's work) can unveil the treasure of 'quantity-trends' and unfold the 'economics of alternative bridge structure schemes'.

42.3 ECONOMICS — VARIOUS CONSIDERATIONS

This chapter is intended to provide a guide for the preliminary planning basically of reinforced and prestressed concrete bridges. Discussion and typical geometries are presented for several commonly used types of bridge structures. The list is not intended to be complete. Suggestions made should be regarded as general guidelines only.

Preliminary design and dimensional proportioning are necessary for preliminary cost estimates and comparative economics of all bridge structures, and also for the analysis for all indeterminate bridge structures. Through advance layout and approximate dimensional proportioning, based on theory, practice, experience, and judgement, one may establish probable geometries and dimensions which will permit the designer to analyze and design the structure. With further revisions, the engineer can then produce a complete set of contract drawings for the structure.

Before moving on to the more specific considerations, it would be appropriate to review several factors which will have a direct bearing on the decision-making process. First of all, it should be recognized that the high cost of construction-labour has forced changes toward industrialized construction methods. Technological progress in mass production of high performance precast and prestressed concrete with compressive strengths of 450 to 600 kg/cm^2 are already available. Methods and equipment for transporting and erecting concrete elements weighing over 100 tonnes are now available.

In longer span bridges, more attention will be paid to reduction in superstructure dead loads. As a result, the utilization of thinner sections and lower density concretes will receive more consideration in future.

Factors to be Considered

In selecting the type of structure for a particular site, the following factors should be debated, decided and established:

Geometry

1. Horizontal and vertical clearances
2. Permissible structure-depth and support-widths
3. Detour or traffic accommodation during construction
4. Utilities: those to be crossed and those to be carried.

First-cost and Ease of Maintenance

1. Sub-surface conditions
2. Total time of construction
3. Construction details—repetitive elements
4. Total cost including approach fills
5. Requirements for any future extensions.

Safety

1. During construction—to adjacent property, utilities, construction-crew, and highway/channel traffic
2. Minimizing traffic hazard in completed structure.

Appearance

1. Similarity to and harmony with adjacent structures
2. Congruence with the environment
3. Use of attractive shapes and surface treatments.

Waterway Crossing—Special Requirements

1. Bridge length, waterway, and pier spacing, to accommodate flood discharge
2. Backwater effects
3. Maintenance of waterway-traffic during construction
4. Navigational requirements and approvals.

Railway Crossings—Special Requirements

1. Overhead clearances
2. Loadings, impact, and vibration
3. Operational restrictions and approvals.

Additional Factors Particularly Associated with Precast Prestressed Construction

1. Cost of tooling (formwork, cranes, etc.)
2. Length of production run
3. Plant production versus on-site production
4. Weights of elements
5. Transportation and erection costs
6. Material production, transportation and erection strategy.

In general, the items to be finalized as early as practical are as follows:

(a) Typical sections and alignment (vertical and horizontal)
(b) Span lengths and arrangement
(c) Structure type and span to depth ratios—including type of materials
(d) Other considerations.

Typical Sections and Alignment

Many bridges have had to be redesigned because of modifications of roadway alignment, indecisions on the number of lanes, change of waterway clearances, width of shoulders, type of curb and railing, and size of pipelines to be carried. It is necessary that these items be established as early as possible to avoid expensive and time-consuming redesign.

Span Lengths and Arrangement

For grade separation bridges the total length is established by the facility crossing under the structure. Location and type of supports are dictated by foundation conditions, required clearances, safety, and aesthetics. Long spans with few supports make the most graceful composition, constitute the least obstruction and, aesthetically, create sensations that are very satisfying.

For river crossings the span lengths may be dictated by any combination of the following:

(a) Foundation requirements
(b) Height of piers
(c) Waterway area and other hydraulic considerations
(d) Navigation requirements
(e) Type of debris expected during flood stages
(f) Facility of construction
(g) Economy

Structure Type, Span to Depth Ratio and Various Other Considerations

In the following subsections the general characteristics of the common types of concrete bridge structures are indicated for reference in preliminary studies. In addition to structural aspects, various other considerations are listed (Fig. 42.3).

Reinforced Concrete Slab Bridge

Structural: Span to depth ratios should be about 15 for simple spans and 19 to 24 for continuous spans; solid slabs are used for spans from approx. 5 to 14 m; cored or voided slabs are used for spans from 14 to 25 m.

Appearance: Neat and simple; desirable for low short spans.

Construction: Details and formwork simplest.

Traffic: May be impeded by falsework if cast-in-place, due to reduced clearances. Guide-rail should protect falsework openings for traffic lanes.

Construction time: Shortest of any cast-in-place construction.

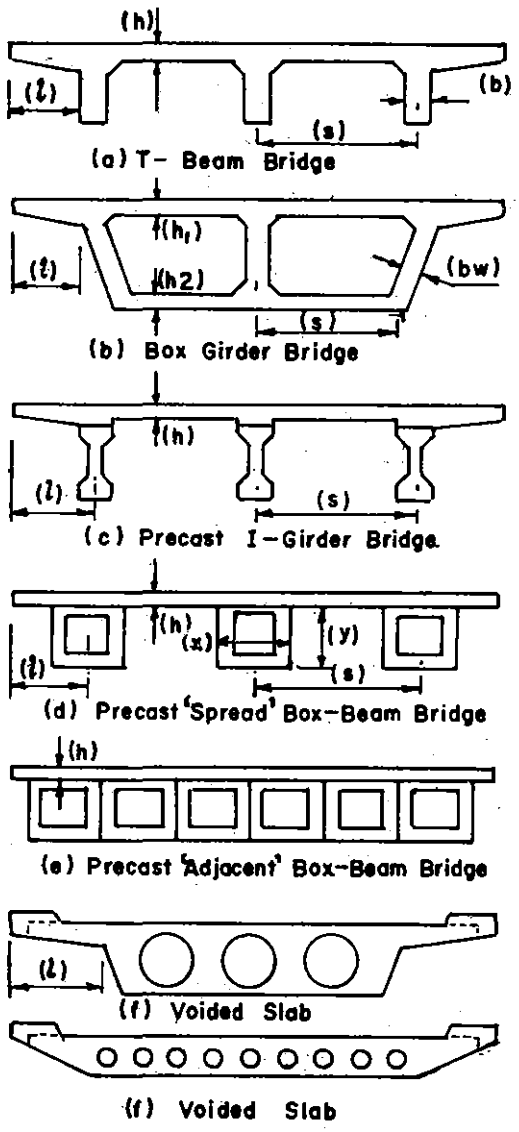

Fig. 42.3 *Concrete bridge deck configurations*

Maintenance: Very little except at hinges. Future widening may be difficult.

Reinforced Concrete T-Beam Bridge

Structural: Span-depth ratio is generally 14 ± for simple spans; 16 ± for continuous spans; higher ratios are possible, but riding qualities are affected by creep characteristics of concrete; span range: 10 to 25 m.

Appearance: Cluttered in view from bottom; elevation is neat and simple.

Construction: Requires good finish on all surfaces; formwork is complicated.

Traffic: May be impeded by falsework if cast-in-place, due to reduced clearances. Guide-rail should protect falsework openings for traffic lanes.

Construction time: More than for slabs due to forming, but not excessively longer.

Maintenance: Low, except that bearing and hinge details may require attention.

Reinforced Concrete Box Girder Bridge

Structural: Span-depth ratio is generally 16 ± for simple spans; 18 ± for continuous spans; higher ratios are possible, but riding qualities are affected by creep characteristics of concrete; high torsional resistance makes it suitable on curved alignment; span range of 25 to 70 m.

Appearance: Neat and clean lines from all views; utilities, pipes and conduits can be concealed.

Construction: Rough form finish is satisfactory on inside surfaces; formwork is complicated.

Traffic: May be impeded by falsework due to reduced clearances. Guide-rail should protect falsework openings for traffic lanes.

Construction time: More than for slabs or T-beams due to staging of concrete placement, but still not excessively long.

Maintenance: Low, except that bearing and hinge details may give some trouble. Future widening may be difficult.

Prestressed Concrete Slab Bridges

Cast-in-place, Post-Tensioned

Structural: Voided slabs up to about 40 m. where a high span-depth ratio is required; either simple or continuous spans; span-depth ratios as high as 40 ± have been used (generally 30); high torsional resistance, making it very suitable for curved alignment, especially on single columns.

Appearance: Neat and simple; desirable for low short spans.

Construction: More complicated than conventional reinforced concrete. Sequence of stressing and grouting should be supervised by specialists.

Traffic: May be impeded by falsework due to reduced clearances. Guide-rail should protect falsework openings for traffic lanes.

Construction time: About same as conventional reinforced slabs.

Maintenance: Very little except at hinges.

Precast, Pretensioned or Post-Tensioned

Structural: Voided slabs for spans 10 to 25 m.; span-depth ratio: 25 to 30.

Appearance: Neat and simple; desirable for low short spans.

Construction: Details and formwork very simple; plant fabrication methods are favourable and applicable; field erection may be fast.

Traffic: No falsework required; units placed by cranes; no prolonged impediment to traffic.

Construction time: Time for erection of precast elements at site a minimum.

Maintenance: Very little except at hinges.

Prestressed Concrete Girder Bridges

Cast-in-place, Post-Tensioned

Structural: Generally applicable to spans 25 to 200 m., either T or box girder form; either simple, cantilever, or continuous; span-depth ratio is usually 20 ± for simple spans and 25 ± for continuous spans; as high as 33 ± have been used; haunched box girders, to 200 m, with span-depth ratio at mid-span 50 ± and at supports 33 ±, both cast-in-place and precast combinations.

Appearance: Beams may be cluttered in view from bottom; elevation neat and simple.

Construction: More complicated than conventional reinforced girders.

Traffic: May be impeded by falsework unless using cantilever construction which is practical for major structures only.

Construction time: Slightly longer than for conventionally reinforced T-beam or box girders.

Maintenance: Low except that bearing and hinge details may require attention.

Precast T, I, U and Box Shaped Beams, with in-situ Slab

Structural: Applicable to spans 10 to 50 m.; the span-depth ratio for I-beam is 18 ± for simple spans and 20 ± for continuous spans; for spread box beams is 18 to 22; and for adjacent box beams is 25 to 30.

Appearance: I-beam is similar to T-beam, except for bulbous bottom of prestressed girder sections; spread box-beam is similar to T-beam; adjacent box beam is similar to cast-in-place box girder.

Construction: Fabrication is more complicated than in conventional reinforced concrete girders; longer span girders require very careful handling after fabrication in plant and at job site; standard girders are preferred; special sections are more expensive; standard forms are stocked by fabricators; can be used as simple spans, but preferably used as continuous spans under live load; precast girders with deck slab cast-in-situ.

Traffic: No falsework is required except for long spans with precast segments to be post-tensioned unless stage stressed by a cantilever method.

Construction time: Time for erection of precast element at site a minimum, but may require slab falsework.

Maintenance: Low, except that bearing and hinge details may require attention.

Rigid-Frame Bridges

Structural: Integral rigid negative-moment knees greatly reduce the positive span moment and (overturning) moment at foundation level; single rigid portal frames will adapt to narrow water channels, railways, subways, and divided or undivided highways underneath; double-span rigid frames suitable for divided multi-lane highways underneath. Rigid frames (with or without side spans) can cross multi-lane divided highways with a wide centre median. The horizontal member may be of any of the construction forms described earlier. The advantage of a variable moment of inertia can be easily incorporated; preliminary proportioning can start with a thickness at the knee equal to approximately twice that at the crown.

Appearance: Graceful and clean—well adjusted to stone facing.

Construction: Usually requires curved form-work for variable depth.

Traffic: May be impeded by falsework due to reduced clearances. Guide-rail should protect falsework openings for traffic lanes.

Construction time: Similar to that of other types.

Maintenance: Low, except for potential back-fill settlement, limited flexibility for future widening.

Arch Bridges

- *Fixed arch (Filled or open spandrel)*

Structural: Horizontal reactions created an arch greatly reduce the otherwise large positive moment in the centre; constant depth for small spans and variable moment of inertia for medium and long spans; spans as long as 300 m have been built; rise-to-span ratio varies with topography; thickness at springing lines usually is slightly more than twice that at the crown; filled spandrels are used only with short spans; for medium and long deck spans, open spandrels with roadways carried by columns are the rule; in a through arch, the centre deck usually is carried by hangers and side decks by columns; use long single spans over deep waterways and shorter multiple spans over wide shallow waters with rocky bed.

Appearance: Graceful and attractive, especially over deep gorges, ravines, or a large waterway.

Construction: Either using falsework or cantilever method.

Traffic: When traffic cannot be diverted, the cantilever method may be used in lieu of falsework.

Construction time: Usually longer than other structures.

Maintenance: Low.

- *Two-Hinged Rib Arch*

All statements made for the fixed arch apply, except the following.

Structural: The two hinges usually are placed at springing lines, either at the same level or not, depending on topography; sections at the hinges, being moment-free, take thrusts, constant sections are used more with two-hinged rib arches than with fixed arches; design of hinges becomes complicated for heavy long-span arches where fixed arch should prevail; roadways usually are supported on columns carried by the deck arch; in a through arrangement, only the side decks are carried by columns while the centre deck is supported by hangers; multiple two-hinged arch spans are uncommon.

- *Tied Arch Bridge*

All statements made for the two hinged arch rib apply, except for the following structural aspects

(a) Tied-arch bridges are used where the supporting rock foundation cannot resist the arch thrust.
(b) The horizontal thrust of the arch is entirely taken by the tie.
(c) The tied arch is always used as a through span.
(d) The deck floor of a tied-arch bridge is always carried by hangers.
(e) Multiple tied arch spans are a succession of individual tied arches.

Truss Bridge

Applicable to spans 35 to 80 m.

Structural: A truss bridge may be supported as a simple, cantilever, or continuous structure. It may have parallel chords, inclined upper chord in a through type, or inclined lower chord in a deck type. It may be prestressed, precast, and field jointed; or precast, and post-tensioned. For spans longer than usual for girders it may prove competitive. A span-depth ratio of 10 or less should be sued for a deck truss. For about equal quantity of materials, a truss has less deflection than its girder counterpart. However, concrete trusses are uncommon.

The Vierendeel truss has not at all been widely used. A rational analysis is available to solve the three unknowns in each member with computer programmes.

Appearance: Probably the least attractive of any bridge type.

Construction: Complicated forming makes the construction slow.

Traffic: Requires extensive falsework, makes it difficult to continue traffic under the bridge during entire construction.

Construction time: Usually quite long.

Maintenance: Moderate because of complexity of trusses. Future widening is a real problem.

Suspension and Cable Stayed Bridges

Structural: Cable stayed girder bridges have been used for spans of 400 to 500 m; with cellular piers, to carry towers and cable stays. Suspension type bridge serves well for spans exceeding about 300 m.

Appearance: Graceful and delicate; well adapted to both urban and rustic environment.

Construction: At difficult crossings the suspension bridge is simpler to construct than the other common bridge types; stayed-type bridges are especially adapted to long spans.

Traffic: Falsework usually is not necessary; deck may be constructed by the cantilever method or suspended from cables.

Construction time: Considerable.

Maintenance: Moderate, because of complexity of suspension system.

Initial Geometric Proportioning of Deck

The deck slab overhang, designated as in Fig. 42.3 should be provided as required to produce desirable aesthetic effect and reduce the span moment.

Girder spacing S may vary as justified by comparing the cost of more girders against that of additional slab thickness. It is generally more economical to use greater depth of girder (or stem) and wider spacing. Usual ranges are 1.6 to 2.5 m for T-beam bridges, 2 to 3.5 m. for box-girder bridges, 2 to 4.5 m for precast prestressed I-beam bridges, and 2 to 4 m for precast, prestressed box-beam bridges.

Stem width, b, shall be a minimum of 27 cm for up to 8 longitudinal bars. It is often necessary to increase this thickness over continuous supports to accommodate compressive forces.

For prestressed girders the stem width should preferably be 30 cm minimum to facilitate provision of prestressing tendons. The bottom slab thickness should be approximately 1/20 of clear span between webs, but should not be less than 15 cm.

The thickness of the bottom slab, h_2, may be increased near continuous supports to accommodate compressive forces.

The webs of box girder superstructures should have a minimum thickness b of 20 cm. It is often useful to increase the thickness of webs near supports to provide adequate concrete shear resistance.

Precast box beams ordinarily have a width of 100 to 120 cm and heights in the range of 60 to 120 cm.

Abutments

There are two basic types of abutments: open type or spill-through type and closed type or non-spill through type.

Open type abutments are more economical than closed type since they allow the backfill to spill out in between their columns thereby reducing the active earth pressure. But, due to this very reason, the backfill may settle down, should it be washed through under heavy rains. In such a case a deep curtain wall should be provided extending downward from the abutment cap, for at least 1.5 m below the spill-line.

Piers or Bents

The term "pier" pertains to an intermediate substructure unit between the abutments. A "bent" is a pier consisting of one or more columns in the substructure unit, with or without cap.

Pile Bents Type are economical but are suitable for stream crossings where floating debris is minimal. They may be competitive in placid waters and in dry beds where soil is poor and footings laid at shallower depths might settle. The preferable minimum width of cap is 75 cm.

Pile bents are unsightly for bridges 6 m or more in height and unsuitable if scour is significant.

Solid Piers They are located in streams where floating debris and rolling boulders accompany the fast current. They are preferable for long spans, and may be supported on spread footings, pile foundations or caissons, as appropriate. A pier wall should have a low thickness/height ratio, but should not be less than 30 cm thick. This is the minimum thickness that should be used with two planes of reinforcing steel.

Multi-Column Bents and Multi-Columns They generally support dry land structures. They normally replace pile bents when spread footings are recommended. They may be supported on either spread footings or pile foundations. Columns may be circular, rectangular or variable in section to provide architectural effect.

The thickness/height ratio of normal height uniform section columns should be in the range of 1/12 to 1/15 for appearance as well as structural reasons. Unnecessarily thick columns can invite high temperature and shrinkage stresses and very slender ones will require buckling analysis.

Single-Column Bents and Single Columns They can sometimes be used to avoid skewed piers. They are adapted to viaducts over city streets where the location of the columr may be restricted.

Foundations

Footings are appropriate where scour is low and soil bearing capacity is enough not to create settlement problems, or if rock is available at a shallow depth. In any case, if subsoil water is encountered footings are feasible only if dewatering is possible. Otherwise piles are preferable, which enables transferring the loads directly to lower and firmer substrata, without requiring dewatering. These are costlier than footings, in general. However, in case of deep scour, rolling boulders, floating debris and strong current, the (generally costlier) caissons become necessary. Caissons are relatively massive and generally suit spans not less than 25 m., preferably longer.

Economic Evaluation

If the form of construction chosen for each component of a bridge is functional and well proportioned, for the conditions imposed by the layout and location of the structure, it is likely that the overall solution is an economic one. ***The appropriateness of various forms of construction to differing situations and ranges of span is a judgement usually attributed to professional experience. Involvement in the first hand design and construction of varying types of structures is the best means of developing a keen sense of their suitability in differing situations.***

It is certainly a waste of effort to carry our lengthy calculations on optimization for component parts within a solution which is itself inappropriate. In most engineering design situations there are a myriad practical constraints on the proportions and options open.

Some of the most commonly quoted factors relating to economy can yield disappointing results. It is often stated that contractors prefer methods of construction which are speedy to erect. Once committed to a project the contractor's preferences are likely to be very much orientated in that direction, but this is not to say that he would give the best tender price for the solution which offers the most rapid construction. ***Within a highway contract there may well be individual structures which are critical in terms of allowing access through a site, but differences in the overall cost on the project which might result from the delayed or early completion of such a structure are unlikely to be reflected in the costs attributed to it in the overall bills of quantities.***

There are no permanent rules about the comparative costs of differing solutions for construction, because changes take place in the relative cost and availability of labour and materials from one time to another, and developments in constructional techniques.

There are inevitable difficulties in forecasting the cost of civil engineering works, and these are aggravated during times of economic uncertainty or rapid inflation. Quite apart from the general financial climate of the country, the rates adopted by contractors are obviously influenced by their current workload, so that low pricing

cannot be expected during a period when the construction industry is heavily committed.

Costs in tenders submitted by several contractors frequently show marked disparities in the sums of money assigned to individual portions of the work, even where the total tender prices may be similar. Bridge designers have always been sensitive to this problem because the cost of the bridges is commonly only part of the total cost of a highway project. There may therefore be wide discrepancies between the prices placed against bridgeworks by different competitors, within overall tender prices which may be close. There can also be widely differing rates against indentical items in comparable works within separate contracts.

Difficulty arises in making comparisons between works that are not similar. A comparison of the difference between deep spread footings in difficult ground and the use of piling would be sensitive to certain rates which therefore require detailed examination to confirm their reliability. Similar items may demand significantly different rates within alternative solutions. An obvious example is the formwork required to abutment-wall-construction if the options are between a plain surface for a cantilevered wall and the shaping required to form counterforts. The rates for the formwork in such cases will be markedly different.

It is also important to identify an appropriate unit for making cost comparisons. It can be misleading to assume idealized conditions when making comparisons between alternatives. Most choices of this kind bring, in their wake, a number of peripheral issues, some of which may be significant in terms of cost. There can be other valuable guidelines to the economy of a form of construction, apart from direct calculations of cost. For example, the contents of materials—concrete, reinforcing or prestressing steel—expressed per unit area of bridge deck, etc. may be useful guidelines to economy; indicating the quantity trends.

42.4 SOME USEFUL INFERENCES

Given below, in a workman-like summary form, are certain inferences relevant to quantities in the more usual types of bridges. These are essentially in the vein of a guideline, indicating the relative trend in the quantities in well designed alternatives, all designed for same loads using the same set of design specifications.

For same deck-depth, a box section deck has less concrete than that in a voided deck, and ultimately is cheaper. However, practical considerations regarding access for formwork eliminate box decks of less than 1.2 m depth. In such cases, voided decks are preferable (Fig. 42.4).

There is an 'optimum depth' for a 'given' span for 'minimum prestress'. Very shallow decks require high prestress. Box decks require lesser prestress than

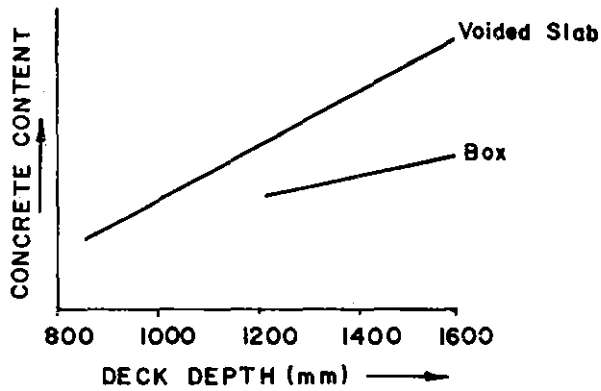

Fig. 42.4

corresponding voided decks (Fig. 42.5).

A box with thin walls may require heavy reinforcement to resist local bending, but there is little change in it with increase in deck depth, and overall quantity may still be less in the case of box deck when compared with a well detailed voided deck ***(where void formers require to be held down against floatation)*** (Fig. 42.6)

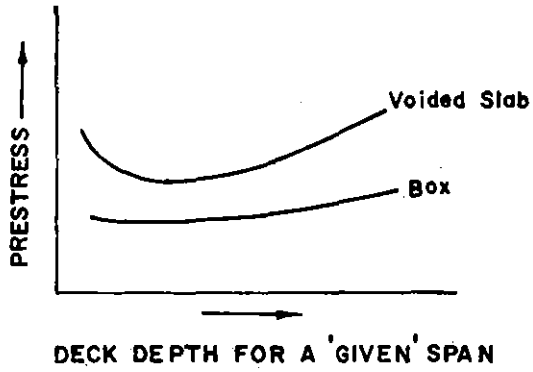

Fig. 42.5

Where deck depth enables a box construction, its overall cost can be cheaper than that of the voided slab. But the cost comparison is sensitive to the slenderness of the deck in relation to the span. An actual comparison is therefore necessary for the span under consideration (Fig. 42.7).

Whatever form of construction is adopted the unit cost per square metre of a bridge deck tends to increase with the span (Fig. 42.8).

If the cost of an individual pier and its foundation is fairly constant, it is apparent that the total cost of the pier substructures will reduce with increase in the span. However, for a given bridge length the cost of bridge finishings is constant, regardless of the span lengths adopted,

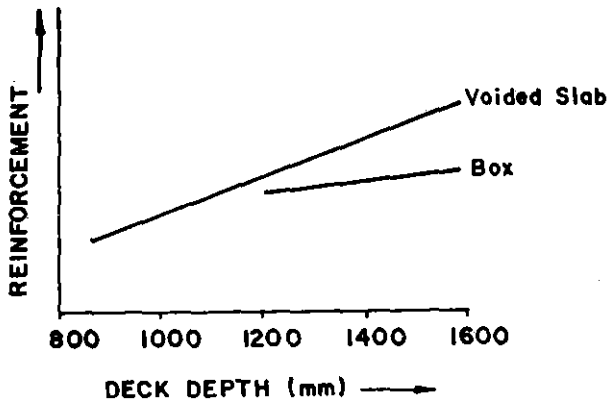

Fig. 42.6

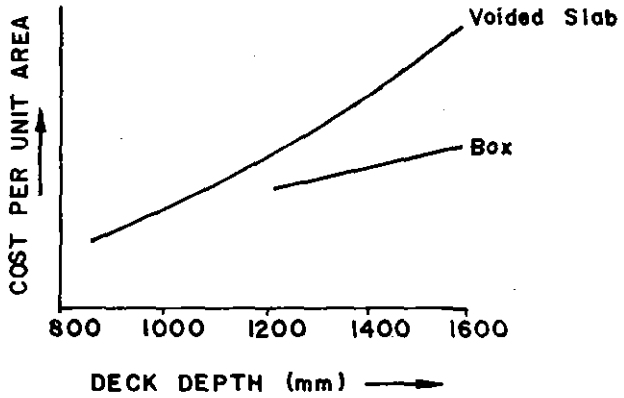

Fig. 42.7

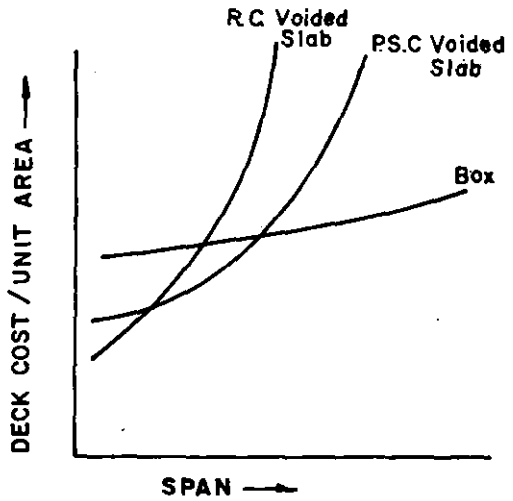

Fig. 42.8

and nearly so is the cost of its abutments (Fig. 42.9).

As the span increases, cost of deck becomes greater. Since pier costs are primarily related to height, longer spans

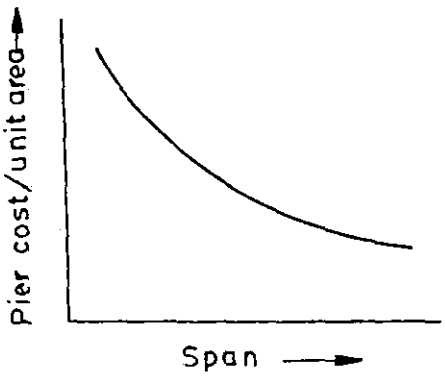

Fig. 42.9

will be economical where piers are tall and foundations deep.

For a given span length, a particular deck depth is optimum; prestress will increase for lesser or more depth. If a very slender deck is adopted, high stresses develop which demand a high prestress. The shallowest deck is not, therefore, the most economical (Fig. 42.10).

For a given span, the quantity of reinforcement increases with increase in deck-depth in prestressed concrete but reduces in reinforced concrete. However, some details, such as diaphragms, require less steel with greater depths (Fig. 42.11).

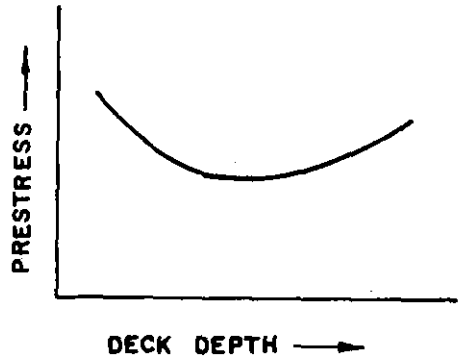

Fig. 42.10

Formwork for the soffit is only marginally affected by increases in depth. However, side shutters and void forms obviously increase with increasing depth (Fig. 42.12).

Even with the lower grade of concrete called for in mass abutments and consequent lower unit rates for such concrete, mass abutments may be cheaper only when either other rates are high and/or seismic activity is very low (Fig. 42.13).

For high walls the reinforcement needed in cantilever abutments can become very heavy, particularly if there is

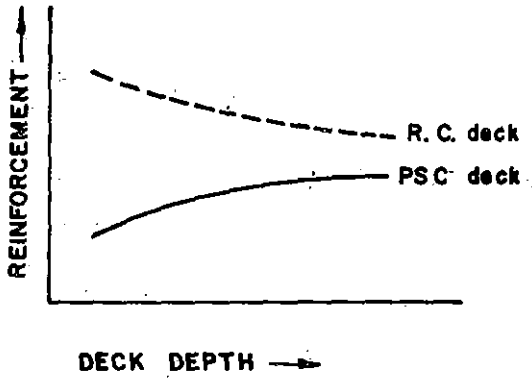

Fig. 42.11

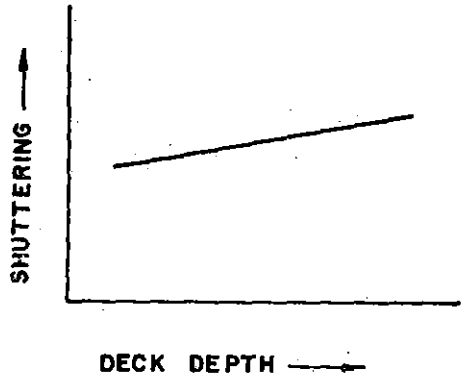

Fig. 42.12

difference in the rates between large and small diameter bars (Fig. 42.14).

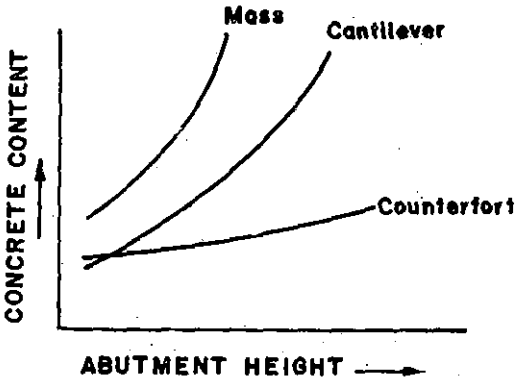

Fig. 42.13

Complicated shuttering and details needed in counterfort abutment construction attract higher rates (Fig. 42.15)

Apart from theoretical economic considerations in abutments, the choice between wall types may be influenced by the speed of construction and practical considerations

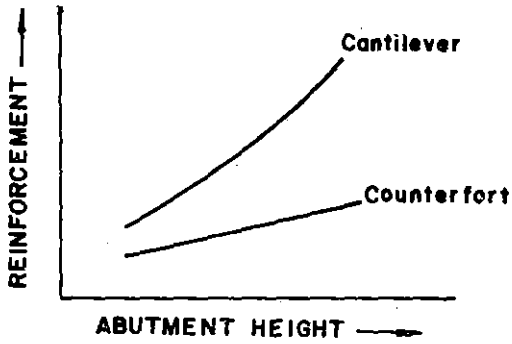

Fig. 42.14

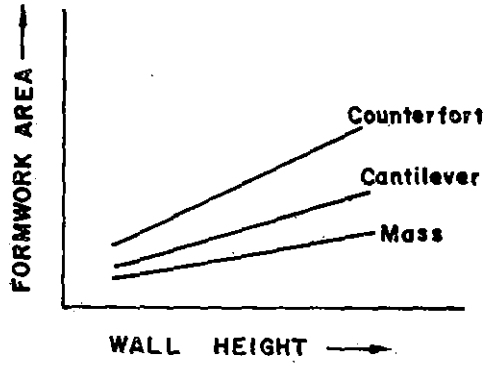

Fig. 42.15

relating to the concreting of wall sections at a given height.

42.5 ADDITIONAL FACTORS

Some of the additional factors that have a marked effect on the overall cost and hence on the relative economy of the bridge structure are:

(a) current material costs in the market or as supplied,
(b) current cost of transportation of materials to site including all handling and storage,
(c) economic climate in the country at the time of costing the bid,
(d) whether there is abundance or scarcity of civil engineering works in the current market,
(e) current labour wages and labour availability,
(f) possibility of increase in material costs and labour wages during the currency of contract, i.e. price escalation incident to inflation,
(g) location of the site of work with respect to the nearest rail and road head (effect on hauling and movement costs),
(h) length of working season per year,
(i) quantity of work involved,
(j) difficulties in construction (e.g. extents of detours required, including their maintenance; whether construction is on a new alignment which may be undisturbed by

traffic or whether the construction is in an urban area and therefore handicapped by heavy traffic volumes, etc.),

(k) whether any major equipments and apparatus are required to be bought afresh,

(l) contractor's overhead expenses, profit margin and promotional expenses (whether any special provisions to be built in!) and

(m) whether a low 'first-cost' is the over-riding constraint, which may result in higher ultimate cost (partly because of possible underestimation in design and costing at bid time, and partly because of higher subsequent maintenance cost), or a more sedate costing (including provision for better looks) would be encountered.

The present chapter deals basically with 'economy and quantity-trends' as a result of alternative structure schemes. Details about 'costing', i.e. 'pricing the costs' of 'constructing' a particular item of work, the 'consultancy fee' and the 'overheads', have been discussed separately in the author's other books.

42.6 TYPICAL COMPARISON—CASE STUDY 1

Problem

To design in detail suitable structural alternatives for a right-bridge, crossing a six lane divided highway, and to compare their strategic dimensions, quantities and relative costs.
Deck width: about 10.4 m (number of design lanes: 3)
Soffit of Deck: about 5.5 m above the highway.
Abutments: as dwarf walls resting on approach embankments. *Piers:* 2-column frame with a cap, on 45T or more working load capacity cast-in-situ piles (each 12 m or more in length). Minimum length of bridge (end to end) 46.4 m (to clear the dual-carriageway highway including its median and hard shoulders). Length may be suitably increased for aesthetics and to offer better proportioned end spans. *AASHTO Live Load:* HS 20–44 (3-axle 32.66 T Truck per lane). AASHTO Design Specifications. Seismic activity at site: G/20. Wind intensity: normal, Neoprene bearings.

Span-layout Selection

Four arrangements for 'span layout' were chosen:

(i) 8 m-18.3 m-18.3 m-8 m ('4-span' arrangement)
(ii) 14.5 m-36.5-14.5 m... ('3-span' arrangement)
(iii) 22.9 m-22.9 m..... ('2-span' arrangement)
(iv) 45.7 m..... ('single-span' arrangement)

These four span-arrangements were considered to be the upper and lower limiting arrangements by which the same highway could be crossed, some span arrangements affording a 'lighter' look owing to additional end spans, and others creating a blunter look. (Some clients prefer the lighter look for aesthetics, even if the first-cost is higher.) Arrangements (iii) and (iv) can also permit a future widening of the highway at minimal cost.

Structure-Schemes

Scheme A: cast-in-situ R.C. beam-and-slab fully continuous deck.
Scheme B: cast-in-situ R.C. box-girder fully continuous deck.
Scheme C: precast p.s.c. I-girders with cast-in-situ diaphragms and deck slab, simply supported for these dead-loads, continuous for finishings and live load.
Scheme D: welded structural steel girders and cross beams with cast-in-situ R.C. slab, made composite for finishings and live load; each span simply supported (continuity would have meant either no shear-connectors in hogging moment zones, or, if shear connectors were provided there, then additional longitudinal reinforcement in such zones to take bending tension in order to ensure integrity of concrete in tension zone of continued composite action. In balance, only simple spans were therefore adopted).
Scheme E: cast-in-situ p.s.c. box girder fully continuous deck.
Scheme F: cast-in-situ p.s.c. box girder, simple span.

(i) in the '4-span' arrangement: Schemes A, B, C and D.
(ii) in the '3-span' arrangement: Schemes B, E, and D.
(iii) in the '2-span' arrangement: Schemes B, E, C and D.
(iv) in the 'single-span' arrangement: Scheme F only.

Main Quantities

The main quantities for various structure schemes for each of the four span-layouts (sub- and super-structure) are indicated in Table 42.1. In its last row are also shown the 'comparative' overall costs in proportion to that of the most economical alternative assumed equal to 1.00. (For unit prices, etc., see the next sub-section.)

Unit Prices, Comparative-Cost Picture, and Deck-Depths

Suffice it to say that prevalent all inclusive unit prices (as were relevant to the site at the time the investigation was made) were considered. Their individual values are of little significance in the discussion here. What is of significance here are the relative overall costs of the various alternative structural schemes expressed 'as proportions of the lowest-cost alternative'. The latter is shown as 100 per cent in Table 42.1A.

Conclusions

As can be seen from above, the most economical scheme here appears to be the '2-span' arrangement in R.C. box girder, fully continuous deck (i.e. in Scheme B), designated

as 100 per cent. The '4-span' arrangement in cast-in-situ R.C. beam and slab fully continuous deck (i.e. in Scheme A) follows very closely with 100.9 per cent. This is followed by the '2-span' arrangement in the cast-in-situ p.s.c. box girder fully continuous deck (i.e. in Scheme E) with 102.7 per cent, a figure which is of some general interest since it shows that the depth of a structure can be reduced at not much additional cost. The variation in the relative position of the precast prestressed "I" girder (scheme C) in the 4-span and 2-span alternatives arises from the fact that the shorter prestressed units are relatively more costly due to the essentially certain fixed costs of prestressing operations. The composite welded steel girder type bridge (scheme D) shows a consistent differential greater than 20 per cent, a result which issues from the high cost of structural steel at the concerned site at the considered time.

Table 42.1

	4-span alternative				3-span alternative			2-span alternative				1-span alternative
Item	SCHEME A R.C. BEAM AND SLAB CONT-INUOUS	SCHEME B R.C. BOX GIRDER CONTI-NUOUS	SCHEME C PRE-CAST PRES-TRES-SED 'I' GIR-DERS MADE SEMI-CON-TINU-OUS	SCHEME D COM-POSITE WEL-DED STRUC-TURAL S.G. SIMPLE SPANS	SCHEME B R.C. BOX GIRDER CON-TINU-OUS	SCHEME E PRES-TRES-SED CAST IN SITU BOX GIRDER CON-TINU-OUS	SCHEME D COMP-OSITE WEL-DED STRUC-TURAL STEEL GIRDER SIMPLE SPANS	SCHEME B R.C. BOX GIRDER CON-TINU-OUS	SCHEME E PRES-TRES-SED CAST IN SITU BOX GIRDER CON-TINU-OUS	SCHEME C PRE-CAST PRES-TRES-SED 'I' GIR-DERS MADE SEMI-CON-TINU-OUS	SCHEME D COM-POSITE WEL-DED STRUC-TURAL STEEL GIRDER SIMPLE SPANS	SCHEME F PRE-STRESSED CAST IN SITU BOX GIRDER SIMPLE SPAN
Structure excavation (m^3)	142	147	147	142	178	178	171	82	81	70	70	33
Structure backfill (m^2)	77	71	71	77	92	90	86	77	73	62	62	47
Prestressed girder (No.)			10							10		
Prestressing Steel and operations (kg.)						2417			4456			10318
Class 'A' concrete (m^3)	317	314	268	222	468	152	236	298	105	201	197	83
Cast in place prestressed concrete (m^3)						314			191			274
Reinforcing steel (T)	45.5	53.5	39.7	22.8	79.4	36.8	25.0	43.6	20.9	24.9	20.5	27.0
Structural Steel including paint (T)				45.5			79.4				46.3	
Furnishing concrete piling (m)	408	408	408	384	658	658	610	518	518	449	449	375
Driving Piles (No.)	34	34	34	32	50	50	46	36	36	32	32	22
Barrier Railing (m)	122	122	122	122	150	150	150	114	114	114	114	144
Comparative overall 'costs'	1.009	1.041	1.175	1.255	1.491	1.524	1.741	1.000	1.027	1.076	1.232	1.220

Table 42.1A

	4-span alternative				3-span alternative			2-span alternative				single span alternative
Scheme	A	B	C	D	B	E	D	B	E	C	D	F
Girder depth (m)	1.22	1.07*	1.16	1.07	1.98	1.68	2.13	1.37	0.99	1.30	1.30	1.98
Comparative overall costs	100.9 %	104.1 %	117.5 %	125.5 %	149.1 %	152.4 %	174.1 %	100.0 %	102.7 %	107.6 %	123.2 %	122.0 %

*Practically it will be very difficult to construct a box section deck less than about 1.20 m deep, so, to that extent, Scheme B in this 4-span alternative is more of academic interest than of any practical value.

The results of the study also point out the cost effect of significantly increased span lengths. In the 3-span alternative, the main span was increased from 18.3 m to 36.5 m and costs increased by about 50%. With a problem of the kind under study, the effect of the removal of one support is lost in the much greater increased cost of the superstructure.

It is also apparent in the above case that, for example, an increase in pile lengths to 24 m would have increased the cost of all bridges, but the cost of the steel bridges would have increased at a slower rate since fewer piles are required in their foundations owing to lesser dead load. Conversely, shorter piles tend to decrease overall costs and favour the concrete bridges here. From this, it should be apparent that the precise cost relationships among the various types of bridges can be determined only by the kind of detailed study which gives due consideration to all such variations.

42.7 TYPICAL COMPARISON—CASE STUDY 2

Problem

To design in detail suitable structural alternatives for a right-bridge, crossing a wide and almost flat-bedded stream, and to compare their strategic dimensions, quantities and 'relative' costs per unit plan-area of deck. The stream is subject to minor seasonal floods, scour depth is small, open foundations on sandy strata possible. Deck width: 15.85 m. Clear roadway (Number of design lanes: 4). Soffit of deck: about 10 m above soffit of footing. Abutments: non-spill through type. Piers: either 5-column frame with a cap, or a slender shaft with a hammer head, or a wall type, resting on spread footing. Minimum length of bridge: about 51 m, but could even be about 55, 60, 75 or even about 90 m, end to end. Could even be much longer, in which case the above modules could be filled in various combinations as required and as feasible. Aesthetics: not any significant criterion as the bridge is located out in wilderness, away from any urban area. Low cost main consideration. Live Load : Saudi Ministry of Communications new 3-axle 600 KN Truck, per lane. AASHTO design specifications, Seismic activity nil. Wind intensity normal. Neoprene bearings.

Alternative 'Span-layouts' and 'Structure-schemes'

Seven arrangements were chosen.

Scheme A: R.C. fully continuous cast-in-situ constant depth slab spans, with column-frame piers, spans being:
7.75 m-10 m-10 m-10 m-10 m-7.75 m = 55.50 m

Scheme B: R.C. fully continuous cast-in-situ haunched slab spans, with hammer headed shaft piers, spans being:
15.25 m-20 m-15.25 m = 50.5 m.

Scheme C: p.s.c. fully continuous cast-in-situ voided slab-spans, with wall type piers, spans being:
15.25 m-20 m-15.25 m = 50.5 m

Scheme D: Similar to Scheme C, but spans being:
22.75 m-30 m-22.75 m = 75.5 m

Scheme E: p.s.c. fully continuous cast-in-situ box girder with wall type piers, spans being:
22.75 m - 30 m - 22.75 m = 75.5 m

Scheme F: 20 m precast p.s.c. girders with cast-in-situ R.C. diaphragms and deck slab, simply supported for these dead loads, and rendered continuous for finishings and live load (i.e. semi-continuous deck), with hammer headed shaft piers, spans being:
20.25 m - 20 m - 20.25 m = 90.50 m

Scheme G: Similar to Scheme *F* but with 30 m precast p.s.c. girders, spans being:
30.25 m-30 m-30.25 m = 90.50 m

Main Quantities

The main quantities for these seven structure-schemes are indicated in following Table 42.2. In it are also shown the 'comparative' overall costs per unit plan-area of the respective decks in proportion to that of Scheme G~ the most economical alternative (assumed as 100 per cent). Rather than local unit prices, the 'relative' costs per unit plan area of deck are of significance here: These are indicated in Table 42.2 and the inference is rather interesting. Also shown in the last row in Table 42.2 are the structural deck depths as a matter of interest to practitioners.

Conclusions

As can be seen from Table 42.2 the most economical scheme here appears to be Scheme G, consisting of 30 m precast p.s.c. girders simply supported for self-weight and that of cast-in-situ R.C. diaphragms and deck slab but rendered continuous over piers (by longitudinally appropriately reinforced deck slab) for finishings and live load, designated as 100%. This is followed by a similar deck arrangement, Scheme F (107%), which utilises shorter spans and a slightly smaller deck depth. Then follow, Scheme E (fully continuous cast-in-situ p.s.c. box girder, 117.6%) and Scheme A (fully continuous cast-in-situ R.C. slab spans of constant depth, 123%). However, the latter involves too many more foundations (apart from the crowding of columns) which is not helpful for subsequent maintenance activity. Schemes D and C (fully continuous cast-in-situ p.s.c. voided slabs of longer and shorter span modules, respectively) relative costs: 127.6% and 130.4%, and Scheme B (fully continuous cast-in-situ R.C. haunched slab-spans): 130%, appear to be the costliest in the present

Table 42.2

QUANTITY ESTIMATES								
DESCRIPTION	Unit	Scheme A	Scheme B	Scheme C	Scheme D	Scheme E	Scheme F	Scheme G
Unclassified structural excavation	m^3	2480	2090	2015	2050	2050	2015	2050
Concrete in R.C. Substructure	m^3	1008	960	1025	1171	1171	966	977
Concrete in R.C. Superstructure	m^3	557	600	—	—	—	—	—
Concrete in p.s.c. Superstructure	m^3	—	—	532	978	664	439	742
Concrete in retaining walls	m^3	—	—	—	—	—	—	—
Elastomeric bearings	1000 cm^3	90	130	150	200	200	160	227
Metal hand railing	m	120	110	110	160	160	130	190
Reinforcement bars Grade 60	T	166	146	86	119	163	114	152
Prestressing steel (including anchorages, cost of ducts, cable making, stressing and grouting)	T	—	—	9.5	20.5	20.5	9.5	20.5
R.C. Slope protection	m^3	38	38	38	38	38	38	38
Expansion joint	m	33.6	33.6	33.6	33.6	33.6	33.6	33.6
Structural steel (minor)	T	0.15	0.15	0.15	0.15	0.15	0.15	0.15
Cost in Saudi Riyals per m^2 of deck-plan area *(Riyadh—1984)*		1919	2028	2036	1992	1836	1670	1561
Comparative costs per unit deck-plan area compared to that in the lowest cost alternative (Scheme G)		123%	130%	130.4%	127.6%	117.6%	107%	100%
Structural deck depth (m)		0.56	0.60 increasing to 1.0 at piers	0.80	1.20	1.20	1.40 (beam : 1.15)	1.60 (beam : 1.38)

case. The structural scheme that was cheaper in case study 1 (i.e. fully continuous 2-span cast-in-situ R.C. box girder, each span 22.9 m) was in fact tried here also with span arrangements of (i) 15.25 m-20- m-15.25 m (= 50.50 m), and (ii) 25.25 m-25.25 m (= 50.50 m). The latter arrangement came very close (105%) to Scheme G, while the former was nor far (110%). However, precast p.s.c. girders had an edge in the present case owing to the relatively low price assumed for these girders, based on the centralised location of the precasting yard relative to the total length of the highway project and the location of various bridges in it where precast girders could be used.

42.8 QUANTITY TRENDS

42.8.1 Appreciation

Since prices of materials, wages of labour, operational costs of plant and equipment and even the margins for overheads and profits can vary, depending on incidence of inflation, availability of materials and personnel, logistics of site, availability of works, and scores of other factors (some of which have been outlined earlier), it is futile to talk in terms of any lasting 'cost' trends. Instead, 'quantity' trends have a better utility in practice. These too of course will depend on the design specifications (codes of practice) and road live load adopted and the incumbent magnitudes of seismicity, wind, flood velocity, scour depth at foundations, safe bearing capacity of substrata at founding levels, properties of soil below design scour level and of the backfill, etc., which again can vary from site to site.

Given this back-drop, certain quantity-trends mentioned in the succeeding subsections (although they are based on a vast number of actually professionally designed bridges in commercial practice), must still be regarded essentially to be 'indicative' only, and that too valid for the particular loading and design specifications mentioned against them.

Having said this, ***it must however be clearly understood that such 'quantity figures' nevertheless represent a fund of practical information that is not found in straight textbooks, class-room lectures or in any research reports. Talking alone never pulled out any stumps!*** These 'figures are a treasure, and represent the essence of hundreds of engineering man-months of competitive hard-hitting, practice-oriented, computer-aided, commercially-biased professional analysis and the design of myriads of alternative bridge structure schemes, with many of which the author was personally involved first-hand (more as a doer, than as just a checker).

Summarised in the Succeeding Paragraphs are Certain "Quality-Trends" Arranged as Follows

... in subsection 42.8.2—For HS-20-44 (32.5^T 3-axle) AASHTO Truck per lane ~

Item A. Continuous beam-and-slab R.C. decks:

... average concrete thickness and span to depth ratios for particular beam spacings, for various spans.

Item B. Continuous beam-and-slab R.C. decks:

...reinforcement in deck for different span to depth ratios for particular beam-spacings, for various spans.

Item C. Continuous box-girder R.C. decks:

...average concrete thickness and span-to-depth ratios for particular web spacings, for various spans.

Item D. Continuous box-girder R.C. decks:

...reinforcement in deck for different span-to-depth ratios for particular beam-spacings, for various spans.

Item E. Simply supported precast p.s.c. girders with cast-in-situ R.C. diaphragms and deck slab:

...girder depth, and quantities of concrete, reinforcement, and prestressing steel per girder, for various spans.

Item F. Simply supported cast-in-situ p.s.c. box section deck:

...prestressing steel per unit deck plan area for different span-to-depth ratios with particular web spacings, for various spans.

Item G. 2-equal spans continuous cast-in-situ p.s.c. box section deck:

...prestressing steel per unit deck plan area for different span-to-depth ratios with particular web spacings, for various spans.

Item H. 3-equal spans continuous cast-in-situ p.s.c. box section deck:

...prestressing steel per unit deck plan area for different span to depth ratios with particular web spacings for various spans.

Item I. 3-span continuous (0.75-1-0.75) cast-in-situ p.s.c. box section deck:

...prestressing steel per unit deck plan area for different span-to-depth ratios with particular web spacings, for various spans.

Item J. 4-equal spans continuous cast-in-situ p.s.c. box section deck:

...prestressing steel per unit deck plan area for different span-to-depth ratios with particular web spacings, for various spans.

Item K. 4-span continuous (0.75-1-1-0.75) cast-in-situ p.s.c. box section deck:

...prestressing steel per unit deck plan area for different span-to-depth ratios with particular web spacings, for various spans.

Item L. Simply supported composite steel girders with R.C. deck slab:

...structural steel per unit deck plan area for particular girder spacings, for various span to depth ratios, for various spans.

Item M. Concrete and reinforcement in non-spill through type abutments on spread footings or piles, for spans up to about 40 m (piles excluded):

...in cantilever type, counterfort type, hinged-to-deck type and rigid-frame type cases.

...in subsection 42.8.3 —for 3-axle 600KN Truck per lane (new Saudi loading), three design lanes roadways ~

Item N. Simply supported precast p.s.c. girders with cast-in-situ R.C. slab and only (half-depth) end cross beams (no intermediate cross beams, so that, where needed, the slab shutters move on trolleys that move on girder bottom bulbs, and hence a much speedier construction is possible), assuming static transverse load distribution among girders: 3 design-lanes carriageway:

...quantities of concrete, reinforcing steel and prestressing steel, in 15-, 20-, 25- and 35-m spans, respectively.

Item O. Concrete and reinforcement in cantilever type non-spill through abutment of different heights, (on footings or piles) for spans up to about 40 m (piles excluded):

...quantities of concrete and reinforcement per unit length of abutment for skew angles of 0° and 35°, and quantity of reinforcement per cubic content of concrete, for overall abutment heights of 5.5, 6.5, 7.5, 8.5, 9.5 and 10.5 m, respectively.

Item P. Concrete and reinforcement in cantilever type spill-through (2-columnar abutment of different heights, on footings or piles, for spans up to about 40 m (piles excluded):

...quantities of concrete and reinforcement in the abutment for skew angles of 0° and 35°, and quantity of reinforcement per cubic content of concrete, for overall abutment heights of 9.5, 11.5, 13.5, 15.5, and 16.5 m, respectively.

Item Q. Concrete and reinforcement in 2-columns pier of different heights, on footings or piles, for spans up to about 40 m (piles excluded):

...quantities of concrete and reinforcement in the pier for skew angles of 0° and 35°, and quantity of reinforcement per cubic content of concrete, for overall pier heights of 8, 10, 12, 14, 16, 18, 20 and 22 m, respectively.

Item R. Cantilever type retaining wall, retaining the roadway:

...quantities of concrete and reinforcement per unit length of wall and quantity of reinforcement per cubic content of concrete, for overall wall heights of 5, 7, 9, 11, and 13 m, respectively.

Item S. Cantilever type retaining wall, retaining sloped

embankment:

... quantities of concrete and reinforcement per unit length of wall and quantity of reinforcement per cubic content of concrete, for overall wall heights of 8, 10 and 12 m, respectively.

... in subsection 42.8.4 ~

Item T. p.s.c. butterfly deck, constructed by free cantilever method:

... some typical section sizes and quantities for a 2 design-lanes wide deck with footpaths, comprising 54.5 m free cantilever arms supporting 6.0 m riding (floating) spans, designed for British HA and 45 units of HB loading and Indian Class A and Class 70R loadings.

... in subsection 42.8.5 ~

Item U. Approximate quantities in certain important items of work in a regular riverbridge designed to I.R.C. Specifications (e.g. average thickness of concrete deck in R.C. and p.s.c.; reinforcement in R.C. deck slab, in p.s.c. and in R.C. decks, in pier caps, in pier shafts, in well-caps, in well-steinings, prestress-steel; steel structurals for well cutting-edges and strake panels.

... in subsection 42.8.6 ~

Item V. Approximate quantities in shutter form-work, in staging falsework and platform in trusses supporting cast-in-situ p.s.c. girders of up to about 35-m span, and quantities in travelling gantries used in cantilever construction of superstructure.

42.8.2 Items A to M, below, are based generally on following Design Parameters

1. design live load: AASHTO HS-20-44 3-axle truck of 32.5 T (or the specified lane load) per lane, No. of design lanes: 3.
2. design based on: AASHTO Specifications.
3. reinforcement bars: 2800 kg/cm^2 yield.
4. concrete: 210 and 400 kg/cm^2 in R.C. and p.s.c., respectively (28-day cylinder).
5. prestressing steel: 16600 kg/cm^2 minimum U.T.S.
6. structural steel: 2800 kg/cm^2 yield.
7. spacing between webs or girders: 2.1 to 3.2 m in the limit, but generally 2.1 to 2.6 m.
8. web width: 35 cm in R.C. beam-and-slab
   20 cm in R.C. box girder
   30 cm in p.s.c. box girder
   (web width flared near supports to twice the thickness).
9. wobble and curvature friction coefficients: 0.003/m and 0.25/rad. respectively.
10. bottom slab thickness in box girders: 14 cm generally, but flared to 30 cm between 0.9 span and 0.95 span and constant beyond.
11. Soil:
    - safe bearing capacity: 3 kg/cm^2 at founding level
    - angle of internal friction: 30° (non-cohesive)
    - angle of wall friction: 18°.
12. Seismicity: G/20 horizontal, ± G/40 vertical or wind of normal intensity.

NOTE : (i) Quantities of parapets, kerbs and wearing course excluded.
(ii) Span/depth ratio is associated with sum total of web width.

Item A. Continuous beam-and-slab R.C. decks (cast-in-situ)

Span lengths (m)	15	23	30
Span/depth	12.5	13.5	14
Av. thickness of concrete in deck (cm)	36.2	44.5	55.2

• Item B. Continuous beam-and-slab R.C. decks (Cast-in-situ)

Span lengths (m)	15				23				30			
Span/depth	15	14	13.5	12.5	15	14	13.5	12.5	15	14	13.5	12.5
Reinforcement bars in kg/m^3 of concrete in deck	178	172	165	156	159	154	150	145	150	145	—	—

• Item C. Continuous box girder R.C. decks (Cast-in-situ)

Span lengths (m)	15	23		30		38	
Span/depth	16.6	18.75	15	20	16.6	20.8	17.8
Av. thickness of concrete in deck (cm)	39.7	42.8	45.2	46.5	49	49.4	53

• Item D. Continuous box girder R.C. decks (Cast-in-situ)

Span lengths (m)	15		23			30				38		
Span/depth	15	16	15	17	20	15	17	18	20	17.5	18.8	20
Reinforcement bars in kg/m^3 of concrete in deck	174	184	182	195	221	184	198	212	228	203	217	224

• Item E. Simply-supported precast prestressed girders with Cast-in-situ R.C. diaphragms and deck-slab

Span (m)	12			18			24			30			36	
Girder* depth (m)	0.92	1.22	1.53	1.07	1.38	1.68	1.22	1.53	1.68	1.38	1.53	1.68	1.53	1.68
Concrete/girder (m^3)	4	4.6	5.9	6.3	7.6	9.1	8.9	10.7	11.8	12	13.1	14.3	15.6	16.8
Wt. per girder (T)	10	11.5	14.75	15.75	19	22.75	22.25	26.75	29.5	30	32.75	35.75	39	42
Reinforcement/girder (kg)	274	320	364	400	482	568	568	704	750	772	872	954	1067	1136
Reinforcement in kg/m^3 of girder concrete	68.5	69.6	61.7	63.5	63.4	62.4	63.8	65.8	63.6	64.3	66.6	66.7	68.4	67.6
Prestressing steel/girder (kg)	182	160	138	378	287	251	600	464	428	909	800	709	1250	1081
Prestressing steel in kg/m^3 of girder concrete	45.5	34.8	23.4	60	37.8	27.6	67.4	43.4	36.3	75.8	61.1	49.6	80.2	64.4

*Girder spacing = 2.10 m.

N.B: For reinforcement in R.C. deck slab, see ahead.

• Item F. Simply-supported P.S.C. box girder (Cast-in-situ)

Span (m)	30				38				45				53			
Span/depth	20	22	25	28.5	20	22	25	28.5	20	22	25	28.5	20	22	25	28.5
Prestressing steel in kg/m^2 of deck-plan area	12.5	13.5	15.7	18.1	14.7	15.9	18.1	20.3	17.6	19.1	21	23.5	21.5	23	25	27

• Item G. 2-equal spans continuous P.S.C. box deck (Cast-in-situ)

Span (m)	30					38					45					53					60				
Span/depth	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33
Prestressing steel in kg/m^2 of deck-plan area	10.9	12	13.8	15.9	22.2	13.4	14.4	16.3	18.8	24.2	15.9	17.3	19.2	21.7	26.6	18.8	20.3	22.7	25.7	29.5	21.7	23.1	26.1	29.1	33.4

• Item H. 3-equal spans continuous P.S.C. box deck (Cast-in-situ)

Span (m)	30					38					45					53					60				
Span/depth	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33
Prestressing steel in kg/m^2 of deck-plan area	11.4	12.8	13.8	15.8	22.6	13.3	14.4	16.3	18	23.6	15.3	16.7	18.2	20.7	25.4	18	19.2	21.2	23.1	27.6	21	22.7	24.2	26.6	31.5

• Item I. 3-span continuous (0.75-1-0.75) P.S.C. box deck (Cast-in-situ)

Central Span (m)	30					38					45					53					60				
Span/depth*	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33
Prestressing steel in kg/m^2 of deck-plan area	7.9	8.1	10.4	11.9	—	9.4	10.4	11.9	13.8	—	11.9	12.8	13.8	16.3	19	13.9	15.4	16.3	18.3	21.7	16.8	17.8	18.8	20.7	24.7

*for central span; same depth in outer spans.

• Item J. 4-equal spans continuous P.S.C. box deck (Cast-in-situ)

Span (m)	30					38					45					53					60				
Span/depth	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33
Prestressing steel in kg/m^2 of deck-plan area	10.6	11.9	13.3	15.8	—	12.8	14.4	15.8	18.8	—	15.4	16.8	18.8	21.5	25.6	18.2	19.8	21.7	24.6	19.3	21.2	23.2	25.1	28.1	33

• Item K. 4-span continuous (0.75-1-1-0.75) P.S.C. box deck (Cast-in-situ)

Interior Spans (m)	30					38					45					53					60				
Span/depth*	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33	20	22	25	28	33
Prestressing steel in kg/m^2 of deck-plan area	9.4	10.5	11.5	12.9	—	12.5	13	14.4	15.8	—	15.4	16.3	17.3	19.3	21.2	18.5	19.3	20.7	22.7	25	22.2	23.2	24.2	26.6	29.5

*for interior spans; same depth in outer spans.

• Item L. Simply supported structural steel girders composite with R.C. slab

Span (m)	18	21	24	27	30	36	42
Span/overall depth	14	14	14	16.5	16.5	20	20
Structural steel in kg/m^2 of deck-plan area	113	123	133	142	152	182	226

• Item M. Non-spill-through R.C. abutments on footings or piles, for spans up to about 40 m (piles excluded)

Overall Height (m)	Cantilever type		Counterfort type		Hinged to deck type		Rigid frame type (abutment leg + footing)			
	Reinfmt. *	Concrete **	Reinfmt.	Concrete	Reinfmt.	Concrete	Reinfmt.		Concrete	
							18 m span span/depth = 14.7	30 m span span/depth = 20	18 m span span/depth = 14.7	30 m span span/depth = 20
3.0	239 (58.3)	4.1	—	—	209 (83.6)	2.5	—	—	—	—
4.5	417 (78.7)	5.3	551 (62.6)	8.8	298 (82.8)	3.6	—	—	—	—
6.0	655 (80.9)	8.1	640 (65.3)	9.8	417 (74.5)	5.6	—	—	—	—
7.5	953 (75)	12.7	760 (65)	11.7	640 (70.3)	9.1	566 (72.6)	625 (60.7)	7.8	10.3
9.0	—	—	953 (67.6)	14.1	—	—	633 (68.1)	715 (55.5)	9.3	12.9
10.5	—	—	1132 (69)	16.4	—	—	700 (63.1)	790 (52.3)	11.1	15.1

NOTE 1. *'*Reinforcement*' is in kg/metre length of abutment.
2. **'*Concrete*' is in m^3/metre length of abutment.
3. *Figures within brackets* are reinforcement in kg/m^3 of concrete.
4. Cantilever returns and footings (pile caps) included in averaging.

42.8.3 Items N, O, P, Q, R and S below, are based generally on the following Design Parameters

1. design live load: 3-axle 600 KN truck, or the specified lane load, per lane, (the new loading of the Saudi Ministry of communications), 3 design lanes carriageway of 12.6 m. between kerbs.
2. design based on AASHTO specifications.
3. reinforcement bars: 4200 kg/cm^2 yield.
4. concrete: 210 and 400 kg/cm^2 in R.C. and p.s.c. respectively (28-day cylinder).
5. prestressing steel: 16600 kg/cm^2 minimum U.T.S.
6. spacing between precast p.s.c beams: 2.11 m for 15-, 20- and 25-m spans, 1.91 m for 35 m span.
7. precast p.s.c. beams and c.i.s. R.C. slab (as per table below).
8. wobble and curvature friction coefficients: 0.003/m and 0.25/rad., respectively.
9. Soil: • safe bearing capacity: 3 kg/cm^2 at founding level.
   • angle of internal friction: 30° (non cohesive).
   • angle of wall friction: 18°.
10. Seismicity: G/20 horizontal, ± G/40 vertical or wind of normal intensity.
11. Abutments and piers on R.C. footings or piles.

NOTE (i) Quantities of parapets, kerbs and wearing course excluded.
(ii) In piers and abutments, quantities of R.C. footings or pile caps are included in the averaging, but piles are excluded. Quantities of R.C. cantilever returns are included in the abutments.

Simple span (m)	Precast p.s.c. beams			R.C. slab c.i.s. over beam flanges : thickness (cm)
	depth (m)	btm. bulb width	top flange width	
15	1.10	60	2.10	22
20	1.30	60	2.10	22
25	1.50	60	2.10	22
35	2.40	70	1.00	22

- Item N. Simply supported precast p.s.c. girders with cast-in-situ R.C. slab and only (half-depth) end cross beams (no intermediate cross beams, so that where needed, the slab shutters move on trolleys that move on girder bottom bulbs, and hence a much speedier construction is possible), assuming static transverse load distribution among Girder; 3-design lanes carriageway of 12.60 m clear roadway width between kerbs:

Span (m)	Concrete		Reinforcement bars			Prestressing Steel		
	Qtty. (m^3) (one span)	m^3/m^2 of deck plan area i.e. av. thickness of deck (m)	Qtty. (T) (one span)	kg/m^3 of deck concrete	kg/m^2 of deck-plan area	Qtty. (T) (one span)	kg/m^3 of deck concrete	kg/m^2 of deck-plan area
15	117	0.619	18.8	161	100	3.0	25.7	16
20	166	0.659	27.2	164	108	5.5	33.2	22
25	215	0.683	34.8	162	111	7.8	36.3	25
35	344	0.780	54.5	159	124	15.0	43.6	34

- Item O. Concrete and reinforcement in cantilever type non-spill-through abutment of different heights (on footings or piles), for spans up to about 40 m (piles excluded):

Overall ht. of abutment above foundation level (m)	Concrete m^3/m length of abutment	Reinforcement bars	
		kg/m length of abutment	kg/m^3 of concrete
'1'	'2'	'3'	'4'
5.5	8.2	615	75
6.5	10.1	788	78
7.5	11.8	968	82
8.5	14.3	1230	86
9.5	16.7	1503	90
10.5	20.1	1870	93

- Item P. Concrete and reinforcement in cantilever type spill-through (2-columnar) abutment of different heights, on footings or piles, for spans up to about 40 m, (piles excluded); 12.6 m clear roadway width between kerbs:

Overall height of abutment above foundation level (m)	Concrete (m^3)	Reinforcement bars	
		Qtty. (kg)	kg/m^3 of concrete
'1'	'2'	'3'	'4'
9.5	191	13,500	71
11.5	240	18,200	76
13.5	298	24,138	81
15.5	364	30,940	85
16.5	399	34,314	86

NOTE (i) Table shows quantities in one Abutment for 0° skew case.
(ii) For 35° skew, quantities in columns '2' and '3' increase by about 20%.

• Item Q. Concrete and reinforcement in a 2-columnar pier of different heights, on footings or piles, for spans up to about 40 m (piles excluded); 12.6 m clear roadway width between kerbs:

Overall height of pier above foundation level (m)	Concrete (m^3)	Reinforcement bars	
		Qtty. (kg)	kg/m^3 of concrete
'1'	'2'	'3'	'4'
8	125	10,000	80
10	153	12,200	80
12	194	15,500	80
14	229	18,300	80
16	267	21,400	80
18	306	24,500	80.
20	349	28,270	81
22	397	32,158	81

NOTE (i) Table shows quantities in one pier for 0° skew.
(ii) For 35° skew, quantities in columns '2' and '3' increase by about 20%.

• Item R. Cantilever type retaining wall, retaining the roadway (surcharge effect of live load considered):

Overall height of retaining wall above foundation level (m)	Concrete m^3/m length of wall	Reinforcement bars	
		kg/m length of wall	kg/m^3 of concrete
5	3.1	216	70
7	5.9	510	87
9	9.3	890	96
11	13.8	1370	99
13	18.9	1926	102

• Item S. Cantilever type retaining wall, retaining 3H : 2V sloped embankment:

Overall height of retaining wall above foundation level (m)	Concrete m^3/m length of wall	Reinforcement bars	
		kg/m length of wall	kg/m^3 of concrete
8	10.3	1070	104
10	15.1	1650	109
12	21	2350	112

42.8.4 Item T: Prestressed Concrete 'Butterfly' Deck, i.e. Double Cantilever Deck, Constructed by Free Cantilever Method, Segment-by-Segment:

Some typical sections and quantities for a 2-lane bridge deck (7.50 m carriageway width between kerbs), with two footpaths (1.50 m each), comprising 54.5 m free cantilever arms supporting 6.0 m riding (floating) spans, designed for Indian Roads Congress Specifications and I.R.C. Class A and Class 70R loading and British (HA and 45 units of HB) loadings designed to U.K. D.O.E. Specifications. Concrete grade: 425 kg/cm^2 (28 day *cube*). Reinforcement bars: Grade 60 (4200 kg/cm^2 yield) and Grade 40 (2800 kg/cm^2 yield). Prestressing steel: 16300 kg/cm^2 minimum U.T.S., wobble and curvature friction coefficients 0.003/m and 0.25/rad., respectively. Quantities of parapets, kerbs and wearing course excluded:

I. Typical Deck Section Sizes Single cell 4.50 m overall width structural box depth: approx. 8.0 m deep at cantilever root and 2.0 m at tip. Web thickness: 250 mm at cantilever tip increasing to 350 mm at cantilever root in two steps of 50 mm at suitable intermediate sections (thickening affected on inner side). Soffit slab thickness: 200 mm at cantilever tip increasing to 600 mm at cantilever root, thickness changing linearly within a segment in whichever segments required, constant in others. Thickness details of deck slab (top slab) retained unchanged throughout.

II. Quantities of Reinforcement Bars

(i) *From consideration of transverse bending (box section)*
— *in webs:*
Grade 60 vertical bars ~
80 kg/m^3 of concrete in webs.
Grade 40 longitudinal bars ~
20 kg/m^3 of concrete in webs.
— *in top slab (including in the cantilevering footpaths):*

Grade 60 transverse bars — 235 kg/m run of bridge length.
Grade 40 longitudinals — 110 kg/m run of bridge length.

— *in soffit slab:*
Grade 60 transverse bars — 40 kg/m^3 of soffit slab concrete.
Grade 40 longitudinals — 20 kg/m^3 of soffit slab concrete.

(ii) *Additional reinforcement against shear and torsion (longitudinal design):*
Grade 60 — approx. 50 kg/m^3 of concrete in webs.

(iii) *Overall, total reinforcement bars in terms of Grade 60 steel*
120 kg/m^3 of total structural deck concrete, for preliminary estimate.

III. For the same grade of concrete and same average deck thickness, the initial prestressing force required at root of cantilever

(i) 7500 T if designed to Indian (IRC) loadings stated above (62 Nos. 12/0.5 inch Freyssi multistrands), and
(ii) 9200 T if designed to British loadings stated above (56 Nos. 12/0.6 inch Freyssi multistrands).
(iii) Corresponding *quantities of prestressing strand steel are 55 kg/m^3 of concrete in one double cantilever butterfly deck for the Indian loading and 74 kg/m^3 for the British loading.*

IV. Total dead weight of one butterfly (i.e. double cantilever) deck; including wearing course, parapets, kerbs, and lighting posts

2500 T approx.

V. Quantity of structural concrete (425 kg/cm^2 28-day cube strength):

— in one butterfly (i.e. double-arm) deck including footpaths: 765 m^3 approx.
— in its soffit slabs alone 235 m^3
— in its webs alone 250 m^3

VI. Average thickness of deck i.e. quantity of deck concrete in m^3/m^2 of deck-plan area:

— based on 7.50 m 'clear roadway width' as the plan width, ... 94 cm.
— based on approximately 10.8-m 'overall plan-width of deck', ... 64 cm.

42.8.5 Item U: A Normal River Crossing Bridge with Caisson Foundations, Designed to Indian Roads Congress Specifications:

Particulars

- 2-lane 7.5-m clear carriageway, with 1.5-m wide footpaths on either side.
- design live load: (2 lanes of) Class A or (1 lane of) Class 70-R Indian Roads Congress (I.R.C) Loading, and 400 kg/m^2 footpath loading (footpath itself capable of withstanding an isolated point load of 4T on 30 cm diameter contact area).
- structural design to the I.R.C. Specifications.
- soffit of deck: about 6 m above average river-bed level.
- seismicity: G/20 horizontal coefficient, and ± G/40 vertical coefficient.
- wind: of normal intensity.
- flood velocity: 4 m/sec. (max. mean).
- flood depth: 16 m (to maximum scour level).
- abutments and piers on caisson foundations.
- caisson (under pier) taken to about 7 m below max. scour level.
- safe bearing capacity of soil at founding level: 40 T/m^2.
- angle of internal friction of soil below maximum scour level: 30° (and skin friction ignored).
- reinforcement bars: 4200 kg/cm^2 (yield).
- prestressing steel: 16,000 kg/cm^2 ultimate tensile stress.
- wobble and curvature friction coefficients: 0.003/m and 0.25/rad., respectively.
- 28-day cube strength of concrete:
  — in deck 'longitudinally' or 'longitudinally and transversely' prestressed ... 350 kg/cm2.
  — in deck entirely in R.C. 210 kg/cm^2
  — in R.C. piers 250 kg/cm^2.
  — in caissons ... ranges from a minimum of 110 kg/cm^2 in plain concrete case (still having a certain minimum amount of reinforcement) to 160 kg/cm^2 in R.C. case.

Approximate Quantity Trends

I. Average 'thickness' of concrete in deck (based on 'overall plan width of deck, including footpaths)

1. 3-girder beam-and-slab, cast-in-situ, longitudinally in p.s.c., transversely in R.C. ... 48 to 60 cm for 30 to 45 m simple spans.
2. 3-webbed cast-in-situ R.C. box girder ... 45 cm for 20 to 25 m simple spans, and 48 to 56 cm for balanced cantilever spans and continuous spans of 30 to 40 m.

II. Reinforcement bars

1. in R.C. deck slab of longitudinally prestressed deck	180 kg/m^3 of deck slab concrete
2. in R.C. deck slab of longitudinally only reinforced deck (average thickness of concrete in deck is more than in p.s.c case.)	120 kg/m^3 of deck slab concrete
3. if deck is prestressed longitudinally as well as transversely	55kg/m^3 of total concrete in deck
4. in R.C. box section superstructure--for simple spans 20 to 25m	220 kg/m^3 of total concrete in deck
—for balanced cantilever and continuous spans of about 30 to 40 m range	260 kg/m^3 of total concrete in deck
5. in parapet beams (spanning between the tips of cross beams)	400 kg/m^3 of parapet beam concrete
6. in R.C. pier caps and abutment caps	230 kg/m^3 of respective cap concrete
7. in R.C. pier shafts and abutment shafts: • solid type 100 • cellular type 120	kg/m^3 of pier/abutment shaft concrete
8. in R.C. caps of caissons	90 kg/m^3 of cap concrete but varies with diameter of caisson.
9. in caisson steinings:	
• in thicker plain concrete steinings...	13 kg/m^3 of steining concrete (minimum)
• in thinner steining designed in R.C....	depends on design
10. in caisson-curbs (well-curbs)	80 kg/m^3 of curb concrete (minimum)

III. Prestressing Steel in Deck

1. For 30 to 45 m simple spans, 3-girder section, prestressed only longitudinally ... 40 to 50 kg/m^3 of total concrete in deck.
2. Prestressing steel for transversely prestressing the deck slab and cross beams ... 50 kg/m^3 of concrete in deck slab and cross beams.

IV. Structural Steel (2800 kg/cm^2 yield) in Caissons

1. For cutting edge of caisson
   - in Angle-and-Plate type assembly ... 50 to 85 kg/m length of cutting edge depending on type of substrata.
   - in V-shaped-plate assembly ... 160 kg/m length of cutting edge (used in hard-sub-strata).
2. For strake plates including angle stiffeners, etc. (armour plates on inner side of curb and for up to 2m on inner side of steining above curb, and for part-height on outer side of curb):
   - on inner surface of well-curb ... 80 kg/m^2 of that surface
   - on inner surface of well-steining ... 45 kg/m^2 of that surface
   - on outer surface of well-curb ... 50 kg/m^2 of that surface

42.8.6 Item V. Quantities of Structural Steel in Certain General Construction Items

1. For shutter form-work:	
• p.s.c. girder webs	107 kg/m^2 of surface to be shuttered.
• of deck slab	45 kg/m^2 of soffit surface area.
2. For building up staging falsework from bed-level.	11 to 14 kg/m^3 of volume of air-space under deck-casting platform.
3. For making the deck-casting platform	42 to 45 kg/m^2 of its surface area.
4. For trusses for supporting the staging falsework for casting the decking in situ, over a span of about 35m.	90 T for a 3-girder decking (2-lane deck), or about 290 kg/m^2 casting-platform plan-area.
5. For 2 travelling gantries needed for casting the box deck of a 2-lane deck by free cantilever method.	2 @ 40 = 80T approx.

CHAPTER 43

Guidelines for Professionally Preparing and Submitting 'Detailed Structural Analysis and Design Calculations' and 'Construction Drawings' for Client's Counter-checking and Record

1. After preparing the preliminary design and outline drawings and completing the necessary investigations, it is essential to carry out detailed design calculations and prepare detailed construction-drawings, both for cross-checking as well as official records prior to taking up the actual construction work.
2. These calculations must be prepared in a step-by-step workmanlike manner and submitted along with fully detailed working drawings after through checking at the designer's end. The following guidelines may be followed:

A. Analysis and Design 'Calculations'

A.1 ***Give a summary write-up about:***

— the type of bridge,
— span arrangement and spans,
— cross-sections of foundations and deck,
— method of 'construction' and 'loading-sequence',
— types of bearings used and their locations,
... giving simple explanatory sketches.

A.2 ***State the design 'loading' and maximum number of 'design lanes' possible to be loaded. Also mention the following:***

— the design criteria adopted,
— the materials assumed,
— the codes of practice followed,
— the 'normal permissible' stresses adopted in various materials,
— the soil characteristics assumed for back-fill as also at the founding-level as well as of the soil mass between the maximum scour level and founding-level for each foundation (as applicable),
— differential settlement of supports considered (if any),
— wind and seismic intensities assumed,
— temperature range considered for global variation,
— differential temperature gradient assumed across deck depth (and across abutment-wall in case of framed structure), etc.

A.3 ***Give details about:***

— 'maximum' scour levels at piers and abutments (as applicable),
— normal bed (ground) levels,
— 'low water' and 'high flood' levels and afflux (LWL, HFL, AHFL),
— 'normal safe bearing capacity' of substrata at various founding levels under 100% or 50% buoyancy conditions (depending on if substrata below foundations is soil or good rock),
— In case of prestressing, state complete assumptions. e.g. type of tendon*, μ and K values (curvature and wobble friction coefficients), minimum bending radius, ultimate tensile stress, section area and ultimate tensile force per tendon, E_s value, location of cables in section at various sections, cable-profiles in plan and in elevation, etc. together with the calculations for friction, position of 'null-point', jacking pressures and or stressing-end forces in cables, extensions, slip effect, and 'initial primary prestress'' horizontal force and its moment and shear values at each section. Also give the assumptions made in the calculation of losses in prestress; what portion of which loss acts on what section and on what span-arrangement must be clearly enumerated?

'Precamber', necessary to be given (in reverse direction) at various sections, must be calculated and presented, and clearly shown on the drawings and as to with respect to what profile? This precamber information is necessary in order to counter the expected long-term deflection at various sections

* It is advisable to assume a suitable but a definite system of prestressing and prepare the "execution" drawings in full detail, rather than not assume any definite prestressing system and give only values of initial prestressing force at various sections (after slip and friction) and its profile. The contractor can still be given the choice for his "alternative" by saying "or equivalent".

under all permanent loads and final prestress and should be computed on E_c value as reduced by creep, etc.

A.4 ***If any Computer Programs are used, describe***

— their brief details and salient features including the assumptions made in the mathematical modelling,

— the input data required and the details of its printout (output),

— what idealisation necessary; for instance if a 'plane-grid' is adopted then give:

- a representative sketch showing this grid, the coordinate axes and the coordinates to each joint,
- show 'joint' and 'member' numbers in it,
- work out and state the section properties allotted to various members,
- clearly show various loading 'cases' and loads 'apportioned' to each member/joint under each loading case... this is most important!

A.5 ***Presenting all the calculations***

This should be done in a methodical step-by-step, coherent and indexed manner, giving computations for each element; e.g. deck (longitudinals, cross beams, deck slab, end-blocks, etc.), bearings, abutments (stability and design), piers (stability and design), etc. for all incumbent loading cases as also for each construction stage, including for 'one-span dislodged' condition if such dislodging is possible.

A.6 ***At the end of 'each' analysis, the values of various stress resultants***

i.e., shears, moments, thrusts, torques, reactions, etc. and in case of prestressing, the initial and final values of primary and secondary prestress moments, shears, thrusts and reactions *must be clearly tabulated before proceeding with 'design'*.

B. Detailed Drawings

Show 'general arrangement' of the 'complete structure' as well as 'individual details' of 'each' structural component in the same serial as the actual construction will require! Drawings should show various details including the following.

- the materials and types of cement and steel used,
- the concrete strengths and their values at various stages of construction,
- the proposed founding levels and caution about actual founding levels,
- 'normal safe soil bearing capacity' at the proposed founding levels, stating buoyancy conditions as applicable,
- 'cover' to reinforcement in different members,
- details of structural-welding, shear-connectors, stiffeners, etc.,
- 'type and 'location' of each bearing and values of 'vertical load' on it under various load combinations together with the 'associated horizontal forces, amount of movements and rotations in orthogonal directions;—
- 'type' and 'details' of prestressing tendons (including: profiles, E_s and ultimate tensile stress/force values, required jacking-pressures and or jacking forces in cables at their stressing-ends, extensions, slips and their 'limiting' values; whether stressing from both ends or one end only; 'sequence of prestressing' and minimum concrete strength at each stage of construction; precautions while stressing the cables (details given in the chapter on 'construction considerations' in the author's book *Concrete Bridge Practice: Construction, Maintenance and Rehabilitation*) bar-bending schedules. In detailing, take care about: shape and 'development length' of shear stirr-ups and torsion hoops, bar cut-off locations and length of laps in primary reinforcement, 'bundling' of primary reinforcement in and near webs, spacing of bars from considerations of crack control, minimum reinforcement details, special reinforcements at bearing locations and cut-outs and around prestressing anchorages, etc.),
- explain the system adopted for reinforcement-nomenclaturing,
- profile-ordinates for precamber to deck soffit,
- Design live load and codes of practice followed,
- reference to 'Special Specifications' and 'General Notes for construction'.
- any other relevant detail.

CHAPTER 44

Engineering Drawings and Working Drawings

44.1 ENGINEERING DRAWINGS

General

Engineering drawings are those prepared by the Engineer for the Owner. The engineering drawings and the structural specifications form a part of Contract Documents. Engineering drawings should contain an adequate set of notes and all other essential information in a form that can be quickly and correctly interpreted. These drawings should convey definite instructions and show reinforcing bars and welded wire fabric. Engineering and working drawings may be combined.

The responsibility of the Engineer is to furnish a clear statement of design requirements; the responsibility of the Detailer is to carry out these requirements. The Engineer, in either his specifications or drawings should not merely refer the Detailer to an applicable building code for information to use in preparing placing drawings. Instead, this information should be interpreted by the Engineer and shown by him in the form of specific design details or notes for the Detailer to follow. The Engineer should require in his specification that working drawings be submitted for his approval.

Materials

The minimum standard media for production of engineering drawings shall be pencil on tracing paper. Other media providing improved reproducibility or durability such as ink, tracing cloth, or polyester film may be utilized.

Sizes

Drawings should be made in standard sizes. All sheets in any one set of drawings should be the same size. There are well-recognized sets of standard sizes, approximating to ~

18 × 24 in.	(45 × 60 cm.)
24 × 36 in.	(60 × 90 cm.)
27 × 36 in.	(70 × 90 cm.)
34 × 44 in.	(85 × 110 cm.)

All dimensions are to cutting line outside of margin. Border lines are inside of this.

Direction

An arrow indicating the direction of north should be placed beside every plan that concerns layout.

Scales

The scales used should be indicated on all engineering drawings, preferably under the title of each view. Drawings that may be enlarged or reduced in reproduction should show a graphic scale as well as a descriptive one to aid the user.

Lettering

All lettering shall be clear, legible and for normal full-size prints, not less than 1/8 in. high. If small-scale photographic prints are to be made for field use, lettering must be correspondingly larger, and meet microfilming standards.

Additional Details

Engineering drawings and/or specifications for elements like beams, girders, columns, walls, and foundations shall show type and grade of steel, service live load, concrete strength, concrete dimensions, class of tension splice or lap lengths, concrete cover for the reinforcement, required joints, and any other information needed for the preparation of reinforcement working drawings.

Dimensions

Since the engineering drawings for highway structures usually are a combination of engineering and working drawings from which the structure will be built, all dimensions should be clearly shown. Drawings must show the exact dimensions of concrete protection for all reinforcement. Where separate working drawings are prepared, structural dimensions may be omitted following the same practice as for buildings.

Reinforcement

Combination 'engineering-cum-working' drawings must show the size, spacing, and location of the bars and welded wire fabric in the structure. The list of bars must show the number of pieces, size, length, mark of bars, and bending

details of all bend bars.

Reinforcement of larger structures is sometimes detailed, fabricated, and delivered by units, for example, footings, abutments, piers, and girders. The bar list may be similarly subdivided. If the structure is sufficiently large, a separate drawing and bar list is usually made for each unit.

Reinforcing bars for foundations, piers, abutments, wingwalls, and slabs are usually shown on the plan, section, or elevation. Cross-sections must be provided for clarification where necessary. The bar list must be a complete summary of materials required.

Stirrup Anchorage

The Engineer must show or specify by notes the sizes, spacings, location, and types of all stirrups. These types include open stirrups and closed stirrups (or stirrup-ties).

There are various permissible methods of anchorage, but the most common is to be use one of the standard stirrup-tie types.

Where stirrup support bars are required, they must be specified, by the Engineer. In designing the anchorage, allowance must be made to ensure that the ends of the stirrup hook are fully encased in concrete, as when hooks turn outward into shallow slabs.

Where the design requires closed stirrup-ties for vertical shear, then the closure may consist of overlapped standard 90-deg end hooks of one-or two-piece stirrups, or properly spliced pairs of U-stirrups. Where design requires closed ties for torsion, the closure may consist of overlapped standard 135-deg hooks of one- or two-piece ties enclosing a longitudinal bar. At least one longitudinal bar must be located at each interior corner of the stirrups or ties; the size of this bar should be equal to at least the diameter of the stirrup (12 mm. minimum). These details should be indicated by the engineer. He must also show the general arrangement of all such bars and stirrups.

Bundled Bars

When bars are placed in contact with each other in groups of two, three, or four—known as "bundledbars"—the minimum clear space provided between bundles must be equal to 1.5 times the diameter of a single round bar having an area equivalent to the area of the bundle.

Splices

General

In beams or girders that require bars longer than can be carried in stock, splices must be specified. The Engineer must show or specify by notes how the splicing is to be realized, viz. lapping, welding, or mechanical connections.

The Engineer must also show by details on engineering drawings the location of all splices. In beams or girders, splices should preferably be made where the stress in the bar is minimum, i.e., at the point of inflection. Splices where the critical design stress is tensile should be avoided by the Engineer wherever possible. Lapped bars may be either in contact or separated. The Engineer should show or note on the drawings whether splices are to be staggered or made at the same location. Bars to be spliced by noncontact lapped splices in flexural members shall not be spaced transversely more than 1/5 the length of lap nor 6 in.

Lap Splices

Since the strength of a lap splice does not increase directly with the length of lap, and varies with bar diameter, concrete strength, position of the bar, distance from other bars, and type of stress (compressive or tensile), it is necessary for the engineer to show location of all splices, and to indicate by "C" or "T" whether tension or compression controls. If tension controls, he should indicate class of splice required and whether it is "top" or "other." Preferably he should dimension each splice. Where bars of two sizes are lap spliced under the design criteria of the preceding sentence, the Detailer will use appropriate tensile lap splice for the smaller bar, unless otherwise noted.

At column bar splice points, sufficient bars (or dowels) from the lower column must extend into the upper column to provide not less than the sectional area of the bars in the upper column. These bars must extend the minimum distance required for column splices. At least four bars should be so extended. The Engineer should note that unless otherwise specified or shown on engineering drawings, the Detailer will detail the remaining bars in the lower column extending to within 3 in of the top of the floor or other member transmitting the additional load to the column. Where the tops of column bars are less than 2 m above the top of footings or pedestals, the bars should extend into the footing or pedestals. Normally, dowels will be used only if specifically noted on engineering drawings.

Dowels for lap splices at column offsets should have a cross-sectional area at least equal to that of the bars above and they must extend both above and below the splice points, as shown or specified by the Engineer.

The Engineer should also be aware that it is a standard practice in the industry when detailing column verticals to use the appropriate lap splice length for the bars in the column above. This applies regardless of differences in sizes.

For column bars at a lapped splice it should be noted that the amount of offset of the bars is greater for rectangular columns than for round columns. Column verticals to be lap spliced in square or rectangular columns, where column size does not change, are usually shop offset bent into the

column above, unless otherwise shown by the Engineer. Column verticals in round columns where column sizes do not change must be offset bent only if a maximum number of lap spliced bars is desired in the column above, but the Engineer must indicate which arrangement is desired.

Where the depth of the footing, or footing and pedestal combined, is less than the minimum length of embedment required for dowels of a certain size, the size of dowel should be decreased and the number of dowels should be increased to give equivalent area and shown on the engineering drawings. Hooks at the bottom of the bars may be desirable to resist tension, but the hook may not be considered in determining the embedment provided for compression.

Butt Splices

Full welded splices or mechanical connections may be specified or, for compression only, end bearing splices may be specified as butt splices for vertical column bars. For 40 mm (and above) bars, butt splices must be used. Special preparation of the ends of the vertical bars is usually required for butt splices. Where bars are welded, the most common practice is to provide a square-cut end at the top of the lower bar and a double-beveled end on the bottom of the upper bar. Field preparation of ends by flame cutting is satisfactory. Where a mechanical splice is used, both ends of the bar may be either square-cut or standard-shear cut, depending on the type of connection used. Since the points of splices are usually staggered between alternate vertical bars and the splice location depends upon the design requirements, the Engineer must indicate the types of splices permissible, their location, and end preparation required.

Connection Details

Rigid Frame Corners

In designing the corner connection of a rigid frame, care must be taken, particularly in providing full continuity around as large a uniform radius as possible, keeping the centre within the concrete. This point is important in splicing the top bars from the girder to the outside bars in the column. The engineer must provide complete information, showing radius of bend and location, and dimensions of lap splices. If welding or a mechanical splice is to be used, a full description must be provided.

Wall Intersections and Corners

When horizontal wall reinforcement is required by design, all the horizontal bars in one, or sometimes both, faces of a wall should be sufficiently extended past a corner or intersection to be fully developed.

The engineer must indicate which, if any, horizontal reinforcement shall be extended, how far it shall be extended, and how it shall be anchored at intersections and corners of walls and footings. In areas where the applicable building code requires earthquake-resistant design, standard practice requires anchorage of all horizontal bars.

Walls with loads that open corner intersections must be reinforced differently from walls with loads that close such intersections.

Closed Stirrups

Where the engineering drawings show closed stirrups, these stirrups may be closed by two-piece stirrups using overlapping standard 90 deg end hooks enclosing a longitudinal bar, or by properly spliced pairs of U-stirrups or a standard one-piece stirrup-tie. At least one longitudinal bar must be located at each corner of the section, the size of this bar to be at least equal to the diameter of the stirrup but no less than 12 mm. These details must be shown by the Engineer. It should be noted that the use of 90-deg hooks and lapped splices in closed stirrups is not considered effective in situations where the member is subjected to high torsional stress. Tests have shown premature failure due to spalling of the concrete cover and consequent loss of anchorage in the 90-deg hooks and lapped splices in these situations.

Avoiding Bar Congestion at Joints

It is important for the Engineer to study bar layouts carefully and give the detailer the proper information. This study will point up congestion at beam-column joints of beam bars, column bars, and joint hoops. A large-scale drawing or a model or mock-up of the joint details may be worthwhile to assure that a design can be assembled and concrete can be placed.

44.2 WORKING DRAWINGS

Definition

Working drawings are working drawings for the fabrication and placing of reinforcing steel. These drawings may comprise bar lists, schedules, bending details, placing details, and placing plans or elevations. They may be prepared entirely manually or include computer printout.

Scope

Working drawings are intended to convey the Engineer's intent as covered in the Contract Documents. The Contract Documents plus any additions thereto, such as "Addenda" issued by the Engineer (per terms agreed upon in the contract if issued after the contract is made), constitute the sole authority for information in working drawings. The working drawings must include all information necessary

for complete fabrication and placing of all reinforcing steel and bar supports.

Procedure

Working drawings are prepared by a detailer in accordance with the Engineer's instructions contained in the Contract Documents. Any necessary additional information must be supplied by the contractor concerning field conditions, field measurements, construction joints, and sequence of placing concrete. After approval by the Engineer, including necessary revisions, the drawings may be used by the fabricator and placer.

Drawing Standards

Working drawings are prepared to the same general standards as engineering drawings.

Layout

Drawings usually show a plan, elevations, sections, and details of a structure, accompanied by schedules for footings, columns, beams, and slabs. The plan should be drawn in the upper left corner of the sheet, with the elevations and details below and to the right of the plan. Schedules (and bending details) should be placed in the upper right corner of the drawing.

Unless otherwise specified, standard industry practice is followed—drawings are prepared by pencil on tracing paper.

Other media providing equal or improved reproducibility or durability may be utilized, such as ink, computer or mechanically generated graphics and printout, tracing cloth, or polyester film.

An arrow indicating the direction of north should be placed beside every plan view concerning layout.

Symbols and Notation

Standard symbols and abbreviations for working drawings are shown below:

# or Φ	To indicate size of deformed bar (High yield, Grade 60)
ϕ	Plain rounds (mild steel, Grade 40)
@	Spacing centre to centre
⇄	Direction in which bars extend
↔	Limits of area covered by bars

Pl	Plain Bar	OF	Outside Face
Bt	Bent	NF	Near Face
Str	Straight	FF	Far Face
Stir	Stirrup	EF	Each Face
Sp	Spiral	Bott	Bottom
IF	Inside Face	EW	Each Way
Cl	Clear	T	Top

Where unusual details or conditions require use of other (special) symbols or abbreviations, the drawings must provide an explanation of the notation applied.

Schedules

The reinforcement of elements and many other parts of structures can best be shown in tabular form commonly referred to as a schedule. The schedule is a compact summary of all the bars complete with the number of pieces, shape and size, lengths, marks, and bending details from which shop orders can be easily and readily written. While these schedules usually include the bending details for bent bars, separate bending detail schedules may be used.

General Requirements

Upon receipt of the engineering drawings, the fabricator takes the following steps:

1. Prepares working or shop drawings (including bending details).
2. Obtains engineer's approval, if required.
3. Prepares bar lists (shop lists) and fabricates the reinforcing bars.
4. Tags, bundles, and delivers the fabricated reinforcing bars to the job site.

Sometimes the working drawings but certainly the shop drawings must show the size, shape, and location of bars in the structure, including bar supports, if supplied by the fabricator. They also serve as the basis for preparing bar lists.

To assure proper interpretation of the engineering drawings and the contractor's requirements, the fabricator's shop drawings are usually submitted for approval to the contractor before shop fabrication is begun.

For the convenience of both contractor and fabricator, reinforcement is detailed, fabricated, and delivered by units which generally consist of footings, walls, columns, etc. A separate shop drawing and bar lists is usually made for each unit. For small structures the entire requirement may be handled as one unit. For large projects, the contractor may desire a unit, to be divided to correspond with his construction schedule. Such arrangements, between the contractor and fabricator, with the Engineer's approval, are made *before the detailing is begun*. All sections should be kept as large as practicable since it is more economical to detail and fabricate for large units, especially where there is apt to be a duplication of bars.

Unlike the customary practice in the field of reinforced concrete buildings, many highway departments prepare a combination 'engineering and working' drawing. The combination drawing includes a list of reinforcing materials from which the bar fabricator prepares his bar lists. The reinforcing bar placer uses the combination drawing to place the reinforcing bars.

Schedules

Highway structure engineering drawings most often show details of the various elements directly on the plan or elevation. Schedules are sometimes used for piers, small structures, and even retaining walls. Highway engineering drawings usually include, when completely detailed, a type of schedule that is really a bill of material, sometimes segregated by elements of a structure. These drawings are used by the reinforcing bar fabricator to prepare shop bar lists.

Dimensions

When the drawings for highway structures are combination engineering and working drawings from which the structure must be built, all dimensions must be clearly shown. The builder of a bridge should not have to compute any needed dimensions. Drawings must show the exact dimensions of concrete protection for all reinforcement. For example, they must plainly show whether the cover dimension specified on a girder is the clear distance from the main reinforcement or the clear distance from the stirrups. Where separate shop drawings are prepared, structural dimensions may be omitted.

Reinforcement

Drawings must show the size, spacing, and location of the bars in the structure. The bar schedule (combined drawing) must show the number of pieces, size, length, mark of bars, and bending details of all bent bars.

Reinforcement for larger structures is usually detailed, fabricated, and delivered by units for the convenience of both contractor and fabricator, for example, footings, abutments, piers, and girders. The bar list is then similarly subdivided. If the structure is sufficiently large, a separate drawing and bar schedule is made for each unit.

Reinforcing bars for foundations, piers, abutments, wingwalls, and slabs are usually shown on the plan, section, or elevation. Reinforcement may be shown in the simplest and clearest manner; however, the bar schedule must be a complete summary.

To be certain that all of the reinforcement is properly placed or positioned in a unit, a cross-section is frequently required in addition to the plan and elevation of the unit whereon the bars are shown.

Bar Supports

Plain metal supports are widely used as a means of securely holding reinforcement in proper position while concrete is being placed. Plastic coated or stainless legs can be specified to avoid possible rusting at points of exposure. Precast mortar/concrete blocks are also used.

Other types of proprietary supports are available and may be suitable.

Class of bar supports required should be specified in the Contract Documents.

Reinforcing steel must be accurately located in the forms, and firmly held in place before and during the placing of concrete. Adequate supports are necessary to prevent displacement during construction and to keep the steel at a proper distance from the forms. Bar supports are sometimes specified to be "sufficient in number and strength to carry properly the reinforcing steel they support." However, the detailer shall show bar supports as required.

Bar supports are not intended to and shall not be used to support runways for concrete buggies or similar loads.

Side-Form Spacers

All steel reinforcement must be firmly held in place before and during casting of concrete by means of built-in concrete blocks, metallic or plastic supports, spacer bars, wires, or other devices adequate to insure against displacement during construction and to keep the steel at the proper distance from the forms. Selection of the type of spacer has traditionally been the responsibility of the constructor. Detailing of side form spacers is not a standard requirement and is performed only when specifically required by the contract documents. The reinforcing bar shop drawings need only show, and the reinforcing bar fabricator will only be responsible to supply, those side form spacers that are equal to the standard wire bar supports.

44.3 CAUTION

It has to be clearly understood that one cannot become a good practising engineer unless one goes through the 'drawing-board', as it were, making detailed working drawings for at least a year or two before attempting a meaningful design assignment. As pointed out in another chapter, good and practical design has to go hand in hand with workman-like drawings, and it is one of those facts — perhaps unpalatable to the young engineers — which cannot be short-circuited or side-tracked if the designer desires to become workman-like and useful. Only a good hard exposure to 'detailing' on the drawing-board, in a practising design office, gives that all too important 'touch of class' that does not come by the mere ability to calculate the stresses — howsoever impressively! (Also see section 23.4 in Ch. 23 on 'Detailing'.)

CHAPTER 45

Pre-Tender Data-Questionnaire—Design-cum-Construct Bridge-Tender

- Tender for Bridge at Station ... (km ...) on ... Road near ... across River ... (as applicable).
- Collect information about the Project from the Department (i.e. the client) as well as from the site and its neighbourhood concerning various aspects of technical requirements, site-conditions, availability of various materials for construction including the leads involved, restrictions imposed and competition expected.
- The following design-data, as pertinent to the bridge site, are required in order to take up the design of the bridge.

Roadway Requirements

1. Number of traffic lanes and overall carriageway width.
2. Any hard shoulders, cycle tracks, and footpaths, if so, then their widths.
3. Road formation levels at crown of wearing course along the bridge.
4. Whether wearing course is in asphaltic or reinforced concrete.
5. Arrangement of lighting, drainage, and parapets on the bridge.

Structural Requirements

6. Live load details (including of that on hard shoulders, cycle tracks and footpaths).
7. Type of bridge generally anticipated (i.e. type of foundations and superstructure) and any particular construction method specially suitable to site.
8. Wind velocity and pressure for design (based on height above mean retarding surface).
9. Horizontal and vertical seismic coefficients (applicable on full gravity weight/reaction of mass above embedment level for working out seismic forces), if the area is in a seismic zone.
10. Any skew, plan curvature and vertical curve.
11. Whether abutments should be spill-through or non-spill-through type; and any particular type of abutment arrangement preferred.
12. Minimum overall length of the bridge (if a condition).
13. If pneumatic sinking of caisson foundations is anticipated, then type of steel-strakes and cutting-edge required.

Hydrological Requirements

14. Maximum design flood discharge expected to pass under the bridge.
15. 'Maximum mean' or 'maximum' velocity of flood flow.
16. Effective linear waterway required under the bridge (after allowing for 'average thickness' of each pier and its foundation, between high flood level and normal scour level, ignoring the earth fills in front of the abutments).
17. High flood level, afflux and low water level.
18. Vertical clearance required between the affluxed high flood level and soffit of deck from the considerations of unobstructed flow of floating debris with the flood discharge.
19. Vertical and horizontal clearances required under the bridge from any navigational requirements (if a condition).
20. Size (weight) of the floating debris expected to hit a pier and distance in which it is expected to stop after hitting the pier.
21. Size (weight) of any barge expected to hit a pier and any preferences for fender arrangement.
22. Average river-bed level and cross-section of river (or of the bridge crossing) under the bridge.
23. 'Normal' and 'design (i.e. maximum)' scour levels at piers and abutments.
24. 'Minimum' founding levels at piers and abutments from consideration of maximum scour, etc.
25. If foundations are on rock, then minimum penetration required in rock (depending on type of rock).
26. Whether approaches to the two abutments are easily

possible even during floods (from construction standpoint).

27. Whether there are any perennial river channels and whether they can be diverted during working season.
28. Any temporary islands required (in the river channels) for construction of foundations.
29. Any tidal variations at site, if so, how much and timings during various seasons.

Subsurface Soil Parameters

30. Normal safe bearing capacity of substrata at the 'minimum founding levels' under flood/saturated condition, and rate of its increase per metre depth there-below.
31. Whether the sub-soil below the founding level can be assumed 100% buoyant (50% in case of ordinary rock, lesser in case of hard sound rock) for checking the overall stability of the foundation (i.e. the floatation condition).
32. Average 'particle size' of 'soil' and the corresponding 'silt factor' of the eroding soil (required for estimating the 'normal scour depth').
33. Properties of the predominating type of soil around piers and abutments below their respective maximum scour levels, as also of the 'earth fill' (above maximum scour level) around/behind abutments. These are essentially: angles of internal friction and wall friction, cohesion value, and submerged and dry densities.
34. Any particular method 'specifically' recommended for estimating relief due to passive less active earth pressures below maximum scour level in case of caisson foundations.
35. If the subsoil below maximum scour level is saturated and cohesionless, then any increase in minimum depth of foundations as a precaution against soil liquefaction under design seismic shake.
36. Relevant details of any existing bridge in the vicinity of the proposed bridge and details of previous flood marks, discharges, surface velocities and flood-spreads in the area.
37. Type of coarse-aggregates and fine-aggregates available at site or near it. Whether the coarse aggregates are natural gravel or crushed type; and whether they are alkali-reactive (certain aggregates have some silicates and carbonates in them which react with the alkali in the cement and this leads to extensive cracking of hardened concrete later). Whether the sand needs crushed component to bring it up to acceptable fineness modulus, and whether it contains unacceptable amounts of deleterious materials (e.g. clay, silt, chlorides, sulphates, etc.) which may require extensive washing prior to actual use. Whether the aggregates are highly absorptive and contain more than acceptable amount of flaky particles, are dirty and require washing.
38. Environmental conditions, saline/marine atmosphere, prevalence of windy and hot climatic conditions (or cold weather conditions), etc.
39. Availability of good potable water that can be used for making and curing concrete.
40. Rainfall per year, and 'which and how many are the wet months' per year, its effect on working season.

CHAPTER 46

'Maintenance Management System' for Highway Bridges

46.1 INTRODUCTION

Although a vast revenue resource is consumed in building a highway network (including its structures), managing its maintenance and then actually executing the maintenance work can prove even more exacting and costly if what has been built must remain operational for the intended long-term safe use. In the past the maintenance work has traditionally been a mere ongoing process conducted many a time by those who long ceased to be technical—or almost! But never before has the need for maintenance been more pressing. This calls for a scientific assessment of the problem for a workman-like understanding and execution. This requires:

(i) a thorough examination of the detailed inventories;
(ii) carrying out detailed condition surveys and visual and hands-on inspections;
(iii) analysing the observations and structures in order to unfold the causes of structural distresses;
(iv) carrying out the structural investigative computations and, where called for, the appropriate *in situ* tests on material-samples and existing structures; and
(v) ultimately writing the prescriptions for rehabilitation and repair or outright demolition and replacement.

It can readily be realised that first Maintenance Management (i.e. a rationalised methodology for assessment of the condition of what exists and how to repair and rehabilitate it to retain its operational status . . . in order to enjoy its revenue return if not financial-return), and then the execution of Maintenance (i.e. the physical execution of the maintenance decisions taken), form a whole new dimension of the society's rightful demand on the Engineer's Services if an existing highway network system has to remain there for the purpose it was constructed in the first place! The effective aggregate of the aforementioned two exercises is what is referred to as the Maintenance Management System (MMS) in the present-day technical jargon.

It is only natural that in each country the appropriate Ministry responsible for the country's highway system should take up this MMS work on priority basis, either departmentally (which, as the experience shows, ultimately does not generally work) or by engaging the services of appropriately qualified consulting engineering firms.

46.2 ELEMENTS OF THE AGREEMENT

In general, the Maintenance Consultancy Agreement* should consist of the following elements:

I. Instructions to Bidders.
II. Submission of Proposals.
III. Information and Documents relating to the Companies submitting Proposals.
IV. Evaluation of Submissions.
V. Consulting Services Agreement 'Form'.
VI. Terms of Reference for the Highways Maintenance Management System (TOR for the HMMS).
VII. Survey Activities A, B and C for Bridges, Culverts and Tunnels, and Schedule of Unit-Rates. (See ahead for details of Activities A, B and C.)
VIII. Outline of Bridge Surveys . . . Objective, Survey Activities A, B and C, and estimation of Numbers and Types of Crew and Equipment required.
IX. Conditions of Employment of the Consultant.
X. Appendices (various).

NOTE: In subsequent discussion 'Ministry' refers to the authority that owns the Highway and Bridge Network.

46.3 INCEPTION REPORT

The Highways Maintenance Management System comprises a suite of sub-systems, some of which are independent but all of which are inter-related. Before proceeding with the inventory and detailed sub-systems development, it is necessary to prepare and submit the Inception Report which includes annotated flow charts, proposed computer files and record layouts illustrating the system as a whole and the inter-relationships of the sub-systems together with a brief commentary on the contents and objectives of each sub-system. The Inception Report shall include a description of proposed reporting formats for the sub-systems, the overall capability of the system in relation to updating and maintenance and, where relevant, proposals for testing and validation of the system and its sub-systems. It shall also

* See Reference 1.

include a general outline of the proposals for producing users' handbooks and manuals, and training of the staff to be assigned by the Ministry. After review of the inception report by the Ministry and completion of required relevant revisions prepare the revised report. On receipt of written approval by the Ministry, commence work according to the approved work plans.

The Inception Report shall include,

(i) Proposals for executing the inventory and condition survey, traffic counts and vehicle weighings; proposals for measurement of pavement quality and sampling frequencies; the classification system proposed for the condition of each highway element; the type and number of equipments proposed; and a work program and manning schedule.

(ii) Proposals for the organization, specifications, and flow charts for the data based management system, its flexibility for interaction with other systems and sub-systems and future expansion.

(iii) Description of the Pavement Management System (PMS), including:
   (a) mathematical models proposed for use with PMS;
   (b) the capability of the PMS model for addressing the various issues; and
   (c) a work plan for developing and testing the prediction sub-model.

(iv) Description of the maintenance management sub-system proposed for bridges and other structures, including:
   (a) procedures to be used for visual examinations and subsequent structural and engineering investigations; and
   (b) methodology to be used in establishing priorities for maintenance, rehabilitation or replacement of bridges and other structures, where and as applicable.

(v) Description of the methodology to be used for establishing quality standards and costs for the maintenance of non-pavement related elements.

46.4 INVENTORY AND CONDITION SURVEY OF THE ENTIRE HIGHWAY NETWORK

It is necessary to develop procedures for recording the type, location and condition of all highway elements and features required for each maintenance management sub-system. Locations shall be listed to the nearest 1/100th of a kilometre when not otherwise coincident with kilometre markers. It will be required to prepare a complete classification of features and elements that would be covered by the inventory. A partial list of such elements and features is given below:

(a) Pavements including ramps, frontage roads, acceleration and deceleration lanes and shoulders
(b) Bridges: vehicular and pedestrian
(c) Culverts
(d) Tunnels
(e) Retaining walls
(f) Slope protections
(g) Guardrails
(h) Medians
(i) Pavement markings
(j) Highway signs
(k) Intersections
(l) Fences
(m) Rest areas
(n) Sidewalks
(o) Landscaping
(p) Lighting
(q) Traffic signals

It is necessary to carry out an inventory and condition survey for the entire network through the collection of information relevant to type, location and condition of each of the elements. All such inventory data shall be entered on special forms to be designed to suit the Ministry. The Ministry may already have developed special data collection forms for related applications.

Review these forms and suggest appropriate modifications, identifying code numbers while maintaining compatibility with the existing forms in the Ministry.

In collecting data on the condition of bridges and other structures, one may concern oneself initially with detailed hands-on visual examinations of the structural elements of their substructures and superstructures to insure the discovery of any critical weaknesses in these elements.

Where weaknesses are identified, then it is necessary to undertake a detailed structural investigation to determine the cause and the appropriate corrective action, the requirements of which are detailed ahead.

As part of the visual examination one has to also identify those conditions which will require routine maintenance (e.g. painting and repair of railings, repair of bridge-approaches and guardrails).

NOTE: The discussion hereafter shall be restricted only to *'structures'* since the present discussion deals only with bridges.

46.5 MAINTENANCE AND REHABILITATION OF BRIDGES AND OTHER STRUCTURES

In developing maintenance and rehabilitation requirements for bridges and other structures and their replacement, where necessary, develop a methodology for (a) the evaluation of the structural capabilities and overall performance of these features (including the substructures and superstructures); and (b) the determination of the optimum course of action

for each bridge, culvert, tunnel, etc. Then undertake a complete detailed 'visual survey' and 'hands-on inspection' of all superstructures and substructures of every bridge, culvert, tunnel, etc. in the network, to identify and systematically record any signs of distress or deterioration. Develop a methodology whereby the condition of each structure could be rated to establish:

(i) Those elements which are in good condition without any signs of deterioration and requiring no immediate action;

(ii) Items which show signs of distress or deterioration but which, under the operation conditions of the total structure, may not advance to a serious defect and which can be easily rectified through routine maintenance;

(iii) Items which show signs of deterioration and which due to distress under operation conditions would affect the structural integrity of the elements (cracking, corrosion, etc.);

(iv) Item which show signs of progressive deterioration which could lead to their failure due to excessive or repetitive loading, etc. This may necessitate restrictions in loads or should the load category be upgraded, to replacement or strengthening of the structure; and

(v) Elements which have been underdesigned or poorly constructed which show structural cracks or deterioration. Further checks by the consultant may require complete replacement or recommendations to limit the use of such structure.

46.6 BRIDGE MAINTENANCE TEAM (BMT)

Brief guidelines are presented here in order to assist the Maintenance Department to achieve a *result-oriented performance* from its Bridge Maintenance Team (BMT) in a *workman-like manner*.

• Composition of BMT (Bridge Maintenance Team):

The BMT shall, in addition to suitable number of engineers have atleast two design engineers and one Data-Entry cum Programmer attached to it. The former will assist the BMT in carrying out the *Structural Investigative Computations* in the on-going assignments, and the latter will put on tape the details of all the highway structures for an effective and rapidly-retrievable Data-base.

• Training of BMT:

A Structures Expert should impart practice-oriented professional-type training to the BMT, covering the following aspects:

A. Bridge Features:

1. *Classification* of Bridges
2. The *Substructure*
3. The *Superstructure*
4. Bridge *Bearings* and their possible Problems
5. Bridge *Expansion-Joints* and their possible Problems
6. *Economics* and *Quantity-Trends* in Alternative Reinforced and Pre-stressed Concrete Bridges

B. Bridge Maintenance:

8. *Deterioration* and *Rehabilitation* of Concrete Bridges
9. *Maintenance* of Bridges
10. *Distresses* in various Bridge-Elements and their *Reporting* (Questionnaire)
11. Some *Construction Considerations*

C. Concrete Technology and Field Practice:

12. The Story of Concrete ~
    - o Historical
    - o Coarse Aggregate
    - o Fine Aggregate
    - o Cement
    - o Water
    - o Admixtures
    - o Control of Surface Evaporation
    - o Underwater Concreting
    - o Joints ... Construction/Movement/Cold
    - o Mixing Considerations
    - o Transporting the Mix
    - o Finishing the laid Concrete
    - o Curing
    - o Sampling and Testing Concrete
    - o Acceptance Criteria:
      - — Cylinders
      - — Cores (in case of dispute)
    - o Dense Concrete
13. Concrete *Mix Design*
14. Practical Facts about *Properties of Concrete*
15. Practical Facts about *Durability of Concrete*
16. Concrete in *HOT* and/or *MARINE Environment*

Training the Engineers of Various Road Districts:

(i) The Structures Expert of the Ministry, together with the head of BMT, shall also train the designated engineers from the various Road Districts, in different aspects of Bridge Maintenance, familiarizing them appropriately enough with the associated design and construction features as well.

(ii) This sharply *focused* training would cover:
- — *Deterioration* and *Rehabilitation* of Concrete
- — Bridge *Bearings* and their possible Problems

— Bridge *Expansion-Joints* and their possible Problems
— *Maintenance* of Bridges
— *Distresses* in various Bridge-Elements and their *Reporting* (Questionnaire)
— *Inspect-and-Explain* Field-Visits

• **Working Methodology**

The BMT should have access to all the "as-built' drawings and associated information available with the Ministry. This shall include even the individual Maintenance Contract Documents. (This is necessary since going through longer bureaucratic channels can cause time-wasting bottlenecks. However, all such borrowed Documents shall be duly returned to the concerned for preserving the records as they should be.)

The BMT, with the active assistance of their Data-Entry-cum-Programmer, shall prepare an inventory of all Bridges (indeed of all the highway structures), clearly recording the updated distresses in and the corresponding maintenance actions taken for each. The details in this inventory shall be updated annually.

The BMT shall draw up a working-strategy in such a manner as to automatically double-check through spot-reinspections as to whether the suggested maintenance actions were really and actually carried out in the field. The feed-back must then be entered on the data-base tape for the appropriate structure for future reference.

The BMT must keep an updated record of the salient features of the exact scope of work of each Maintenance Contractor (e.g. whether routine maintenance or preventive maintenance, or both, and their itemized details).

The BMT should utilize the *simplified Distress-Reporting-System* as described in Chapter 47 ahead. (This is in the format of a "Questionnaire" and can be strengthened by introducing into it a simplified Condition-Rating-System suggested in the present chapter.)

One engineer from the BMT must always participate in the "Handing Over" exercise. This will make the subsequent maintenance activity more convenient and meaningful in evaluation. Last but not least, the BMT engineers should be involved in the formulation of the Maintenance Contract Document. Their experience through direct involvement with the maintenance work can be of great help in drafting the Maintenance contract documents.

• **Scope of Work for BMT**

~ ***BMT's Obligations and Interaction:***

(i) BMT engineers *shall be obligated* to be familiar with "Design" and "Construction" *features* of Bridges and shall be responsible for:
— Thoroughness of field inspection.
— Analysis of findings and observations.
— Monitoring of distresses.
— Recommendations for correction of defects (in consultation with Experts, where necessary).
— Locating areas of structures with incipient problems, and
— Programming preventive and routine maintenance actions.

(ii) The BMT engineers *shall interact with* various Experts in the Ministry in the following fields, as and when the need arises:
— Design
— Construction
— Materials
— Soils
— New Techniques
— Plant and Equipment, and
— Emergency Repairs

~ ***Objective:***

(i) To ensure "serviceability of structures" (for safe and uninterrupted traffic flow) and for "gainful protection of investment", the BMT *'has to be able to identify the need'* for Structure-Maintenance-Rehabilitation-Replacement (as necessary), *take the appropriate 'action-decisions'* and then *ensure their timely execution in the field.*

(ii) Each component of each structure must be inspected for deficiencies-deterioration-distress, e.g.
— *Concrete components* warrant inspecting for various cracking patterns (longitudinal, diagonal, mesh pattern, etc.), deck spalling, joint spalling, and other signs of distress.
— *Steel structural components* warrant inspecting for rust and loss of section due to similar corrosive action, damage due to stress, buckles, kinks, and collision and other indications of strength-reduction and distress.
— etc.

~ ***Summary of BMT's Activities:***

— Timely *'inspection'* of structures.
— *'Condition-survey'* of structures and their *'rating'*.
— *'Maintenance Action-decisions'* and their *'execution'*.

46.7 STRUCTURE—INSPECTIONS

• **Basis**

— Original completion reports must be available for all bridges.

— These form the basis for periodic inspections.
— Data thus collected from inspection to inspection should be assessed to determine need for any remedial measures (Maintenance Actions).

• **Inspections shall be Carried Out Under 3 Categories, viz:**

(i) Routine Inspections . . . at monthly intervals:

— Not very skilled operation, but is conducted quickly every month.
— *Purpose*: to report fairly obvious deficiencies/developments.

(ii) Detailed Inspections . . . at annual intervals:

— *General Type*—one year.
— *Major Type*—the next year.

In the General Type:
- o Cover all elements against a CHECKLIST.
- o Visual and hands-on inspection, supplemented by Standard Instruments and Aids.
- o Produce a written report for each part/element.

In the Major Type:
- o Inspect more intensively than in the "General Type", and this may require setting up special access facilities.

(iii) Special Inspections . . . after special events:

— This type of inspection is needed after special events, e.g.:
  - o Heavy Flood,
  - o Heavy Earthquake,
  - o Passage of high intensity live load.
— May need the services of an experienced Designer as it may involve certain:
  - o Design Checks, and
  - o Tests.

46.8 GUIDANCE FOR BMT (. . . OUTLINES)

Establishing the Condition-Rating System

Normally there are *five categories of "condition"* to be applied, *as reiterated below:*

a) Structures containing elements which are in good (includes new) condition without any signs of deterioration and requiring no immediate action.
b) Structures containing elements which show signs of distress or deterioration, but which, under the operating conditions of the total structure, may not advance to a serious defect, and which can be easily rectified through routine maintenance.
c) Structures containing elements which show signs of deterioration and which, due to distress under operation conditions, would affect structural integrity of the elements (cracking, corrosion, deflection, unusual vibration, any other deformation).
d) Structures containing elements which show signs of progressive failure due to excessive loading, possibly necessitating load restriction, strengthening, or replacement.
e) Structures containing elements which have been underdesigned or poorly constructed or are overloaded, and show structural cracks or deterioration, possibly resulting in use-limitations or replacement.

The elements examined in the condition survey (see Activities A, B and C ahead) should be categorized and sorted *to result in a Condition—Rating*, following the above described general guideline, which can be used numerically from 5 to 0, as enumerated below, to determine the course of action:

5.. No Action
4.. Routine Maintenance
3.. Special Maintenance/Rehabilitation/Repair
2.. Reduced allowable load after special repair
1.. Further investigation, but in the interim reduce load and speed and use speed breakers and rumble strips in the approaches to the bridge.
0.. Replacement.

The Condition Rating can also be made on a scale of 9 to 0 (Equivalent Rating Conditions of the US Federal Highway Authority) as depicted in Table 46.1.

Table 46.1 *Equivalent Rating Conditions*

Description	Rating
o Not applicable	N
o New condition	9
o Good condition—no repairs needed	8
o Generally good condition—potential exists for minor maintenance	7
o Fair condition—potential exists for major maintenance	6
o Generally fair condition—potential exists for minor rehabilitation	5
o Marginal condition—potential exists for major rehabilitation	4
o Poor condition—repair or rehabilitation required immediately	3
o Critical condition—need for repair or rehabilitation urgent. Facility should be closed until the indicated repair is completed	2
o Critical condition—facility is closed. Study should determine the feasibility for repair	1
o Critical condition—facility is closed and is beyond repair.	0

Inspection Guidance

~ *"Inventory" and "Condition Survey"*

Each structure has to be:

— Inspected,
— Evaluated, *and*
— Classified

in relation to various parameters affecting its performance and structural integrity.

~ *Methodology*

I. Give identity No.: to each bridge based on the *'type of structure'*.
II. Define *'elements/components for inspection'*.
III. Give *'guidelines for inspection and the remedies/repairs'*.

Type of Structure

Once the preliminary identification of the structure and crossing-type have been established, the *identification of the type of structure* should be made. With the utilization of computer analysis in mind, coupled with an awareness of the many possible combinations of structure types, *a coding system for identification of the structure-type* is advisable. The three digit code for identification of "type of structure" which is used by FHWA, US DOT, is recommended.

An example of the utilization of the three digit code for identification of "type of structure" is given in Table 46.2.

Table 46.2 *Type of Structure*

1st Digit		2nd and 3rd Digits	
1	Reinforced concrete, determinate	01	Slab
2	Reinforced concrete, indeterminate	02	Stringer/Multi-beam of girder
3	Steel, determinate	03	Girder & floor-beam system
4	Steel, indeterminate	04	Tee beam
5	Prestressed concrete, determinate	05	Box beam or Girder-multi
6	Prestressed concrete, indeterminate	06	Box beam or Girder-single
7	Timber	07	Frame
8	Stone or masonry	08	Orthotropic
9	Aluminium	09	Truss deck-type
		10	Truss through-type
		11	Arch deck-type
		12	Arch through-type
		13	Suspension
		14	Stayed girder
		15	Movable lift-type
		16	Movable bascule-type
		17	Movable swing-type
		18	Tunnel
		19	Culverts
		20	Other

Elements to be Inspected

o *Bridge 'Superstructure' Components*

1. Deck Components:
   a. Deck slab
   b. Kerbs/walkways
   c. Bridge railing
   d. Expansion joints/expansion devices
   e. Drainage devices
2. Beams/Girders (longitudinals)
3. Diaphragms or Cross Frames
4. Truss Components:
   a. Floor beams
   b. Connections
   c. End posts
   d. Top chord
   e. Lower chord
   f. Lateral bracing
   g. Sway bracing
   h. Portals
   i. Lacing
5. Bearing devices
6. Arch Components:
   a. Arch
   b. Arch columns
   c. Supports
7. Other Components (if applicable):
   a. Spandrel walls
   b. Suspension system
   c. Hangers
   d. Suspenders
   e. Anchorage
   f. Towers
8. Paint/snowcem
9. Connections:
   a. Welds
   b. Rivets
   c. Bolts
   d. Hinges/Articulations
10. Bridge machinery (if applicable)
11. Collision damage preventers/reducers (if applicable)

o *Bridge 'Substructure' Components:*

1. Abutments:
   a. General condition
   b. Beam seats and shaft/wall/columns
   c. Backwalls/Dirt walls
   d. Wingwalls
   e. Footings/Piles, etc.
   f. Settlement

g. Scour
2. Piers
   a. General condition
   b. Beam seats and shaft/wall/columns
   c. Footings/piles, etc.
   d. Fenders and dolphins
   e. Scour
   f. Settlement
3. Paint (if applicable)
4. Debris on beam seats and bearings

o *Culverts (if applicable):*
   a. General condition
   b. Alignment
   c. Adequacy
   d. Debris accumulation
   e. Headwall or endwall condition
   f. Condition at inlet and outlet

o *Channel Condition (if applicable):*
   a. Alignment
   b. Protection (vegetation, riprap, etc.)
   c. Waterway adequacy
   d. Scour
   e. Deflection of water-course and the resulting ponding and oblique attack on some foundations

o *Approaches to Bridge or Culvert:*
   a. Alignment
   b. Grade
   c. Approach slabs
   d. Guard rail
   e. Embankment condition
   f. Relief joints

o *General Components:*
   a. Warning signs
   b. Navigational lights
   c. Utilities on bridge (pipes, cables, etc.)
   d. General appraisal and operating status.

NOTE: In addition to the "serviceability" and "functionability" of bridge components, each component must be analyzed for structural deficiencies and/or deterioration. *Concrete structural components warrant inspection for:~* various cracking patterns (longitudinal, diagonal, mesh pattern, etc.), deck spalling, joint spalling, and other signs of distress or deterioration. *Steel structural components warrant inspection for:~* rust and loss of section due to similar corrosive action, stress cracks, buckles, kinks, collision damage, and other indications of strength reduction and distress. *Timber structural components should be inspected for:~* structural decay or damage due to fungus, insects, dry rot, weather attack, chemical attack, life cycle wear, collision, and other causes of loss of structural integrity. *Stone and masonry components warrant inspection for:~* damage due to weathering, spalling, plant growth (in joints and cracks) abrasion due to wind-blown particles, seasonal expansion/contraction, and other strength reducing factors.

(c) Inspection — Guidance

The Maintenance Engineer should be familiar with:
- — Terminology and elements of bridge structure.
- — Tools, devices of specialized equipment needed for inspection and testing.
- — Methodology of inspection,
- — What to look for,
- — Types of "materials" used in the bridge and, hence, be ready for the "types of distress/deterioration" expected.
- — Possible "effects" of failure of one or more component elements due to overload or collision (assuming design and construction were satisfactory).
- — Results of foundation movement, e.g. from:
  - o Lateral movement settlements
  - o Vertical movement settlements
  - o Pile settlement
  - o Rotational or tipping movement which could be caused by:
    - * Slope slides
    - * Bearing failure
    - * Consolidation of soil
    - * Seepage/frost-action
    - * Drag-down of piles (negative skin-friction)
- — Results of damage in Waterway.

• ***What to Look for***

o *Approaches:*
- — Unevenness
- — Settlement
- — Cracks in approach slab may indicate hollows settlement beneath.
- — Examine its joint with abutment wall.
- — Conditions of shoulders, slopes, drainage approach guard rail.

o *Waterway*†

o *Piers and Abutments, including foundations/piles*
- — Foundations† scour, movements, concrete condition, reinforcement condition, etc.

o *Stringers:*
- — *Timber Stringers:*
  - * Splitting
  - * Cracking/damage
  - * Excessive deflection
  - * Insect attack/decay

† For possible problems in 'Waterways', possible problems due to various types of 'Foundation Movements', various types of cracks and deteriorations possible in concrete, corrosion problems, etc., (and their Rehabilitation). See References 2, 3 and 4.

* Crushing
* Connections: clean, tight
* Bearings/seatings

Steel Stringers:
* Cracking
* Corrosion
* Bearings/seatings
* Connections
* Misalignment: flanges, webs
* Damage

Concrete Stringers: (R.C./p.s.c.)
* Cracking†
* Corrosion†
* Damage: spalling, cold joints

o *Steel Girders:*
* Cracking and corrosion
* Corrosion particularly at contact surfaces where moisture may ingress (e.g. flange plates, gussets, splices, rivets and bolts, weldlines)
* Re-entrant corners
* Sections of section/size change
* Flanges and webs: misalignment, damage/buckling
* Web stiffeners (any evidence of buckling)
* Any unusual deflections under passing loads
* Examine hinges
* Hanger bars w.r.t. ambient temperature.

o *R.C. Members:*
* 'Spotting' by rust
* Leaching (dissolution and baring of matrix)
* Spalling—in critical zones of structure: (e.g. splash zones, water line at low tide levels, portions exposed to atmospheric attack, particularly on the windward side).
* Give special attention to: girder webs, soffit slabs (in box decks), articulations, deck slabs from soffit side, bearing areas.

o *Pre-stressed Concrete Members:*
In addition to the examinations recommended for R.C. members, examine against:
* Loss of camber
* Excessive deflection
* Distress in anchorage zones, at junctions of diaphragms, cracks in webs (sometimes due to radial cable thrust)

NOTE: Wherever any distress is found, immediately:
* Indicate it on the structure by *Highlighter*, marking its extent, width, etc., and *date it* at each inspection (for continuous monitoring).
* Note it down on paper also and report it.
* Monitor it at regular intervals.

o *Bearings:*
Examine that they are:
* Functioning properly
* Clean (and not stuck/locked-up against rotation/sliding, as required).
* Anchor bolts—that they are secured.
* In neoprene bearings—no abnormal bulging, radial cracking, horizontal cracking, etc.
* In pot bearings—seal is intact/no leakage.
* In spherical bearings and cylindrical bearings—the Teflon sheet is intact.
* Examine pedestals.

o *Expansion Joints:*
* Is space for thermal movements adequate?
* Is the joint clear of debris/surfacing material?
* Is the Neoprene uncracked/uncut? (in those of the joints that incorporate Neoprene either as a seal or in modular manner)
* Are the fingers and plates freely working (in finger-type joints)?
* Is the joint assembly securely in place? (Otherwise, if loose, they can become traffic hazard).
* Is the concrete adjacent to the joint sound and free of laminations (sound by a 2 kg. hammer)?

o *Decks:*
Concrete decks, examine for:
* Cracks—various types
* Leaching
* Scaling
* Pot-holing
* Spalling
* Other signs of deterioration
* Evaluate each item for its effect on the structure and the rehabilitative work required to restore structural integrity.
* Rusting of reinforcement
* Asphaltic and other type of w.c. may hide the deterioration in deck slab, etc. until after the damage has become excessive.
* Where "permanent forms" have been used in the deck construction, some such panels may require to be removed (if possible) to closely examine portions on them.
* etc.

Steel Decks, examine for:
* Corrosion—particularly at bearing surface
* Unsound welds

† For possible problems in 'Waterways', possible problems due to various types of 'Foundation Movements', various types of cracks and deteriorations possible in concrete, corrosion problems, etc., (and their Rehabilitation)··· See References 2 and 3.

* Rivets, bolts, stiffeners
* Splices
* etc.

Timber Decks, examine for:

* Decay at contact surfaces where the surfaces bear (on stringers, between layers of planking, etc.)
* Any looseness developed in nailing/bolting
* Any abnormal deflections under passing loads (due to looseness of connections or damage to member-sections, etc.)
* etc.

NOTE: *All decks should be examined for:*

* Slipperiness (hazard)
* Drainage (no ponding on deck or near it)
* Drains/scuppers..... open and clear
* Ensure draining water does not harm other areas of bridge and those around it.

o *Road Kerbs:*

* *Concrete Kerbs:* check as for concrete, steel.
* *Timber Kerbs:* splits and decay should be checked.
* Note the condition of any painting (visibility-purpose painting).
* Examine against any loss of height from building-up of surfacing on road surface.

o *Side-walks (foot-paths):*

Concrete sidewalks:

* Cracks
* Scaling
* Spalling
* Pot holing
* Other deteriorations (e.g. rusting of steel)
* Condition at joints

Timber sidewalks:

* Soundness of timber
* Adequacy of floor planks
* Nails/bolts... secured or working loose?
* Slipperiness

Steel sidewalks:

* Corrosion/perforations
* Connections (secured or working loose)

All sidewalks should also be examined for:

* Drainage
* Deflection

o *Parapets/Railings:*

Concrete Railings:

* Cracks
* Spalls
* Scaling
* Other deteriorations (e.g. rusting of reinforcement).
* Member connections

Metal Railings:

* Condition of paint and corrosion
* Bolts
* Rust stains where posts are set in pockets (may be necessary to remove some of the mortar/grouting from around the postbase in order to examine the severity of corrosion).

Timber Railings:

* Any insect attack and decay
* Securedness of all connections
* Breakages in timber

All handrails should also be examined for:

* Damage from traffic—pedestrian or vehicular
* Vertical and horizontal alignments (foundation settlement and deficiencies in bearings may show up through these)
* Joints—ensure that they are free and working (to accommodate deck movements)
* Free of hazardous projections

o *Crash Barriers:*

Examine for:

* Traffic damage—structural
* Traffic damage—alignment
* Traffic damage—hazardous projections
* The damages in steel—as stated elsewhere.
* The damages in concrete—as stated elsewhere.
* Anchor bolts/nuts... for securedness.

o *Wearing Course:*

Examine for:

* Surface condition—cracks, spalls, pot holes, disintegration, rideability, etc.

o *Trusses in Steel:*

* Sight along roadway railing and kerbs and along chord members for:

o Any misalignment—vertical, horizontal.

—Any deviations?

* Check each member:
  — Compn. members for: kinks, bows, washers connections.
  — Counter braces: in these the "counters" should be just optimally tight (against rattling and vibrating), more would cause abnormal stresses and less would render them useless!

NOTE: Any "looped" rod-type tension members should be checked against cracks in the loop.

* Damage from passing traffic (particularly in the "portal bracings")
* All lateral bracing members against: damage, vibration movement.
* Condition of paint/corrosion.
* Condition of "connection details": bolts, rivets, welds, gussets, packing Plates, pins.

(Particularly where any rubbish/contaminants can collect as the surface drainwater washes the deck.)

* Look for rust-generation at "bearing surface", particularly in multi-plated sections — howsoever difficult or inconvenient! This is where the trouble may begin!!!
* Cracks/breaks, etc. as stated earlier.

o *Trusses in Timber:*

Examine against:

* Some of the items pointed out above for "Steel Trusses" as are relevant here.
* Alignment against sag — may indicate partial failure in joints or improper adjustment of steel vertical rods at connections.
* Splits/decay from insect attack:
  Particularly where moisture may collect, e.g. at contact surfaces and around holes through which rod bolts are fitted.
  Also: look out for *decay* at the joints, particularly where prone to collection of dirt and debris.
* Accumulation of fire-hazards around the bridge (if any).

— General Cautions:

* Observe/inspect the bridge during passage of high flood, heavy load, and combination thereof against any excessive deflection and vibration.
* Watch out against accumulation of fire hazard. (Collection of drift-wood, debris, floating logs and trees, illegal constructions nearby or below, storage of inflammable materials nearby or below, etc.)
* Try to observe the movements and rotations at the Expansion — Joints and Bearings at extreme temperatures.
* Condition of p/c slabs and of their supporting arrangement where such slab elements exist (... any missing panels, rattling panels, broken panels... traffic hazard).
* Examine behaviour of construction joints and cold joints under passing heavy loads.

46.9 OUTLINE OF BRIDGE SURVEYS

Objective, Survey Activities A, B and C, and Estimation of Numbers and Types of Crew and Equipment Required (See References 1 and 2):

The impetus for evaluation and prediction of deterioration and defects in highway structures covers a spectrum of reasons from safety of the travelling public to the integrity of the structures itself to development of a data bank information as a base of an overall highway maintenance management program. Thus, the basic maintenance considerations and objectives of highway structures (bridges, culverts and tunnels) management system are:

(i) To preserve the investment made in existing highway facilities;
(ii) To provide continuing adequate levels of safety and convenience to the highway users;
(iii) To upgrade and improve existing facilities as may be economically practical through minor maintenance;
(iv) To conserve and protect the aesthetic and ecological features of the environment; and
(v) To ensure effective and economical utilization of resources in the accomplishment of maintenance programs.

These objectives will be accomplished through the effective management of maintenance operations and resources. In fact, an up-to-date inventory data of all bridges, culverts and tunnels provides the basis for maintenance management systems.

In developing the bridge model, the recorded information available may not be sufficient to fulfil the needs of the model.

In appreciation of above, the description of Activities A, B and C and the inspection procedures to be followed to obtain the data required are as follows:

• ***Activity A***

Activity A includes the detailed visual and hands-on inspection of all bridges and highway structures and the creation of permanent information files designed to serve the future information requirements.

The purpose of the detailed inspection procedure is to provide basic data that will satisfy the following information needs:

(i) Provide the required information for the proposed mathematical model of the highway structure system. The development of the model and the input parameters will necessarily govern the type and level of information that must be obtained and assembled during the detailed inspection. Other mathematical indicators, such as 'sufficiency' type of ratings, may also be calculated in whole or in part from the basic data collected during the detailed inspection phase.
(ii) Provide information necessary to identify minor and major items of deterioration, the identification of which will require Activities B and C, which address such deteriorated items. Activity A will provide all or part of the information necessary to evaluate these items under Activities B and C.
(iii) Provide an accurate, permanent record file for all

highway structures, including the following permanent reference files:

(a) A *standardized inventory sheet*, stored electronically, that will contain enhanced FHWA* type information and act as the data base for the mathematical model. This form must be designed to provide the necessary information for the above tasks and will, to a large extent, define the detailed inspection effort. Obtaining information in a form directly usable in succeeding evaluation steps is considered extremely important. Several highway agencies, including the State of Pennsylvania in the U.S.A., have developed enhanced FHWA-type inventory sheets which may prove useful in developing a form that is specific here and serve the information needs of the mathematical model.

(b) A *supplementary file* will be established to store information relevant to a given structure, but beyond the scope of the inventory form. Specific information that may be included in such a file includes sketches of deteriorated elements, measurements, notations and other information that is beyond the scope of the inventory form, but may prove useful in future investigations. This file would be a "hard copy" file, but electronically referenced from the inventory file.

(c) A *photographic file* showing the general features of each structure, cataloging deteriorated features, should be established. Photographs are a valuable tool that can quickly provide information not easily obtained and stored by other methods. Standard views of each structure should be photographed, along with any specific deteriorated items, or items of general interest. Photographs should be adequately labelled and stored in loose leaf, 3-ring bound, hard covers, suitable for updating data.

(d) A *permanent reference file of 'as-built' drawings* should be established for all highway structures as soon as possible. As-built drawings, or design-drawings if as-built were not completed, contain information of the as-built structures.

A concerted effort should be made to obtain, catalog and maintain a file of all available as-built drawings. Availability of as-built drawings may prove invaluable if major rehabilitations or modifications are necessary in the future.

The search for as-built drawings may have to extend to the original design consultants if drawings are not presently available. However, further passage of time will only make the search more difficult when a need for the drawings arises.

* Federal Highways Authority, Washington D.C., U.S.A.

• ***Activity B***

Activity B includes the evaluation of structural components identified in Activity A that exhibit minor deterioration, the preparation of a report and recommendations addressing such deterioration, and the preparation of sufficient details and specifications for repair. Activity B will be initiated as a result of the findings of the detailed field inspection of Activity A, if found necessary. In fact, 'bridge and other highway structures' maintenance work is not as clearly defined as highway maintenance, since the structure work is generally reported as either 'major' structure maintenance or 'minor' structure maintenance. A realistic approach to a bridge maintenance program will evaluate the deterioration and defects against appropriate codes to identify work activities under major or minor structure maintenance.

In general, defects and deteriorations in non-structural elements and in 'structural elements due to non-structural causes' could be defined as minor structural distress (except where plastic shrinkage and plastic settlement cracks are severe). Typical types of defects and deteriorations in non-structural elements are given below:

(i) Clogged drainage—scupper.
(ii) Pavement migration.
(iii) Defects in concrete posts and handrails.
(iv) Failure in expansion joints.
(v) Cracking or movement in the approach slab.
(vi) Parapet geometry.
(vii) Scour at footings.
(viii) Slope failure.

Typical defects and deteriorations in structural elements due to non-structural causes are given below:

(i) Plastic and drying shrinkage cracks.
(ii) Scaling and spalling.
(iii) Corrosion of reinforcement.
(iv) Inadequate concrete cover.
(v) Plastic settlement cracks.

In addition, minor repairs may call for routine-type bridge maintenance, such as:

(i) Regular cleaning of bridges, including hand sweeping, cleaning of pipe drains, scuppers and bridge seats, and removing debris from expansion joints.
(ii) Repair or replacement of expansion joints to restore smooth safe riding surfaces.
(iii) Replacement of expansion joint seal material of bridge expansion joints to insure water-tight seals.
(iv) Repair or replace bridge handrails
(v) Sealing of bridge decks to prevent damage from chemical attack.
(vi) Applying a coat of oil and mineral mixture as

corrosion protection membrane to concrete bridge decks and curbs.

Data will be provided during Activity B on a separate form outlining the maintenance and rehabilitation needs of the structure. A work-order should be prepared for repair actions by a contractor. The said work-order will include sufficient sketches, specifications and cost estimates to define the scope of work in order to execute it.

- *Activity C*

Activity C includes the evaluation of structural components identified in Activity A that exhibit major defects and deterioration with possible safety-related implications, the preparation of a report and recommendations addressing the situation. Activity C will be initiated as a result of the findings of the detailed field inspection of Activity A. Deteriorated or damaged bridge components that may pose an immediate safety hazard will, of course, be addressed in an appropriate time frame.

Activity C will be accomplished through the following specific steps:

(i) Obtain sufficient information during the field and inventory survey to evaluate the need for major rehabilitation of the deteriorated bridge or culvert elements. The data collected from field inspection such as problem type, location, deterioration rate, safety relation, dimensions and measurement, sketches, and photos as well as inventory data will provide sufficient information to conduct a preliminary investigation and evaluation. It will also relate the problem against the coding system as it is related to the safety of the travelling public, integrity of the structure and the anticipated amount of repair required to restore the elements to good condition. Thus, the analysis of this investigation with accurate detailing of the problem will be reported in appropriate form to initiate the Activity C operation. Typical defects and deteriorations which may initiate Activity C are given below:
   (a) Physical distress or loss of patches of concrete because of corrosion of reinforcement, scaling, spalling and structural cracks.
   (b) Repair or replacement of bridge expansion joints.
   (c) Foundation settlement.
   (d) Cracks and crushing of concrete members due to foundation movement.
   (e) Pier cap spalls.
   (f) Fractured abutment back wall.
   (g) Horizontal and vertical cracks at the end blocks of prestressed concrete girder.
   (h) Damage to strands of prestressed concrete girders.
   (i) Fatigue cracking in steel members.
   (j) Corrosion in steel structures.
   (k) Brittle fracture in steel members.

(ii) Submit request to the Ministry to initiate Activity C.

(iii) Upon notice to proceed, obtain as-built plans, design computations, standard design criteria and any other pertinent data relating to the design and construction of the structures from the authorities, as available.

(iv) Conduct an in-depth inspection of the deteriorated structural elements. The said inspection shall include sufficient measurements to analyze the condition of the elements.

In case a concrete deck is found to have one or more of the following during field condition survey:
   (a) Visible concrete spalls which have occurred in the deck riding surface and/or evidence of unsound concrete in the exposed surface of the deck slab which may indicate structural distress,
   (b) Extensive deterioration of asphalt overlay logically due to underlying concrete deterioration,
   (c) Evidence of delaminations (horizontal fracture planes) in the concrete deck and structural cracking and /or deformation,
   (d) Evidence of reinforcing steel corrosion, and
   (e) Evidence of inadequate concrete cover over the reinforcing steel, etc., and the Preliminary investigation raises questions about the structural adequacy of the concrete deck to carry current traffic loads, a request to the authorities to initiate Activity C should be submitted. Upon receiving notice to proceed, a detailed field inspection will be made to further define the inadequacies. In this case some of the following recommended test components may have to be conducted as an evaluation system:
      — Delamination detection with appropriate equipment to determine extent of internal fractures of the concrete.
      — Determination of the extent of reinforcing steel corrosion.
      — Determination of the compressive strength of the concrete.
      — Chemical analysis to determine extent of chloride contamination at the levels of principal reinforcements.

      In other cases such as steel bridges whose as-built plans are not available, field measurements shall be taken to locate and size all structural components for the purpose of completing the inventory data as well as carrying out the necessary structural computation.

(v) Prepare a report, outlining the analysis and findings

of the in-depth inspection which is concerned with the evaluation of the load carrying capacity of the structure. The report will indicate the structural investigative computation* and its basis, degree of accuracy, methods of analysis and factor of safety. If the following cases arise from the structural analysis computations, then conducting field load tests† may be needed:

(a) The findings indicate a low degree of certainty in the load-carrying capacity calculations due to existence of different types of deteriorations in many structural components.

(b) The evaluated factor of safety does not meet the code specifications.

(c) The collected measured data and results of the conducted tests as well as inventory information are not enough to calculate the load carrying capacity of the structure with different degree of accuracy.

However, field load tests will not be made unless it is absolutely necessary and the only solution to determine the load carrying capacity of the structure.

(vi) If required, and upon approval of the Ministry, perform necessary load tests and undertake analysis of the structure of determine load capacity (structural investigative computations*).

(vii) Prepare a report outlining all the results of the structural investigation. The report will include the recommendations to restore, if possible, the structure to its original loading capabilities. If rehabilitation is required the report will include work-orders consisting of plans, methods and procedures of repair, required personnel and manhours, types and number of equipments, and required quantity and types of materials. This information will be provided in sufficient detail to enable a contractor to undertake the work. If the structure cannot be rehabilitated, a recommendation will be made to demolish the structure, detailing the method to be used to accomplish the work in a safe manner, retaining elements that can be reused in the new construction.

A detailed list of all possible and anticipated defects and deterioration for all structural and non-structural elements for each group of bridges will be defined and classified. These defects and deterioration will be as identified in the guidelines earlier but should be detailed out in the inspection and procedures manual which should be developed specifically for the purpose. Some of the types of defects and deterioration in concrete bridges may be re-summarised as follows:

- ***In Structural Elements***

— Corrosion of reinforcement
— Structural fracture of concrete
— Structural cracking
— Low concrete cover
— Distress in shear
— Moisture trapped in box beams
— Spalling of concrete cover
— Fractured back wall
— Deterioration of abutment
— Failed slope wall
— Pier cap spalls
— Corrosion of roller or fixed bearing
— Movement of bearing system
— Undercutting at footing, scouring at foundations
— Movement or settlement of piers and abutments

- ***In Non-structural Elements***

— Failure of expansion joints
— Leaking of expansion joints
— Clogged drainage — scrupper
— Determination of concrete in non-structural elements
— Parapet geometry
— Horizontal or vertical movement in the approach slab
— Cracking in non-structural elements
— Rough riding surface
— Pavement migration
— Failed drainage system
— Erosion of soil slope wall
— Slope failure

In general and in most cases, the deterioration and defects in non-structural elements are minor and trigger Activity B; defects in the structural elements can be major and can trigger Activity C. However, in some cases defects in non-structural elements may be a major problem, such as serious failure of expansion joints. Also, in some other cases defects and deterioration in structural elements could be considered as a minor problem, such as low concrete cover or minor cracking due to non-structural causes. Criteria will be formally established for determining the required and

* The structural *investigative computations* refer to all structural analysis needed to determine the governing bending moments, shears, torsions, reactions, etc. at critical sections of the existing structure, in order to establish the structural stability, material stresses, and other serviceability criteria (e.g. crack widths, deflections, vibrations, etc.) for ascertaining the acceptability of the structure. All investigative computations are to be done for both substructure and superstructure, taking into account the *current* design live loads as well as other forces (e.g. floods, wind, etc.). It should also be noted that these are only investigative computations and are not the same as preparing a fresh detailed design of a new structure.

† e.g. concrete cores, reinforcement tensile strength, various load tests (against bending, shear etc.,) of slab/longitudinals, etc.

necessary activity.

Two steps may have to be established for this criteria. The first step is to develop a 'coding system' to rate the condition of each element (structural and non-structural). The inspector should use the Procedures Manual and his own judgement in rating the condition of each element and its relationship to the entire structure. The second step is to establish 'consideration criteria' which are directly related to the degree and level of deterioration or defects. Among the consideration criteria the following are considered important:

(i) The safety of the travelling public.
(ii) The integrity of the structure.
(iii) The amount of repair necessary to restore the member to good condition.

A trained engineer will analyze and evaluate the rating elements which will be provided in Activity A in their relationship to the safety of the travelling public, the integrity of the structure, the other items or components of the bridge, and the amount of repair required to restore the member to good condition. Using this procedure, the engineer will be able to decide whether Activity B or C is required for the existing condition.

46.10 INSPECTION PROCEDURES

(a) Inspection Forms

Three different forms may be developed to list information obtained in the field. The first form will be filled out for every structure inspection and will entail the condition of each element of the structure. A numerical system will be used to indicate the amount of deterioration each element exhibits. For multi-span bridges, each span's elements will be rated individually. An overall rating for the entire structure will be computed based on the individual ratings of its components. This form will correspond to the requirements of Activity A.

The second form, which will provide information as part of Activity B, will entail the listing of those structural items which require repairs. The inspectors will take sufficient measurements to get an estimate of the quantities so that a contractor will have an idea of the scope of work. This form, in addition to some drawn field sketches and cost estimates, will be detailed enough to serve as a work-order for a contractor.

For structural conditions encountered in the field which warrant major rehabilitation or replacement (Activity C), a form listing all the pertinent items should be made available for the inspectors. This complete form should provide necessary information for the initiation of an analysis of the structure.

All three forms should be simple in nature, requiring a minimum of effort to complete. Most of the input will be noted by either the checking of boxes or the information can be easily key punched, resulting in a computerized readout of data.

(b) Inspection and Procedures Manual

Prior to the initiation of the inspection operations a manual should be developed which will be used as a guide by all parties involved in the inspection operations. The said manual will contain the following features:

(i) *Procedures to be followed for the inspection of bridges, tunnels and culverts* The information will be specific, indicating what to look for when inspecting each individual element. In addition, instructions will be provided on how to rate the bridge and its elements.
(ii) *Maintenance of traffic stipulations* Details will be provided on how to safely divert traffic around an inspection site, including the placement of traffic control devices and the effective use of flagmen.
(iii) *Use of inspection form* Instructions on how to fill out the inspection forms will be provided.
(iv) *List of emergency contacts* A list will be prepared of individuals to be contacted within each district in case of emergency. Local officials, police, fire departments, medical facilities, etc. will be indicated so that if the need arises, the concerned individuals can be readily notified.
(v) *Organisation chart* A chart showing the chain of command for the entire inspection operation will be prepared so that everyone is aware of who is their immediate supervisor. This chart will also show the makeup of each inspection team.
(vi) *Form processing* This section will provide instructions on how the various inspection forms are to be processed. Information such as where the forms are to be transmitted how often, to whom, etc. will be clearly outlined.

46.11 MEANS OF ACCESS AND THE REQUIRED EQUIPMENT

Means of Access for Inspection:

1. The simplest mode of inspection, visual inspection, is not free from problems. The most vulnerable areas in concrete bridges (whether reinforced or prestressed) are obviously the deck and beam soffits. Reinforcements in main load carrying members is concentrated near the soffit surfaces mostly, and signs of cracking, corrosion, spalling and exposure are to be found there. But generally, soffit is also the most inaccessible. Without proper and safe access, no inspection is possible. The problem of accessibility gets further aggravated in long span, high rise bridges, and those crossing deep perennial rivers or across tidal estuaries. The traditional access through platforms erected on barges

or scaffolding, even from dry river beds, are time consuming, costly, and can be ineffective. These are not viable means of inspection.

2. The accessibility problem has long been successfully solved through the use of *snooper crawlers*, which are movable vehicle-mounted crane-arms, tailored to the needs of horizontal and vertical accessibility all along the span, substructure and bearings. These mobile inspection units have been in wide use in many countries. Besides inspection units have been in wide use in many countries. Besides inspection, this also provides a working platform for minor repairs which would otherwise take a long time. The major advantage of this kind of equipment is its versatility. Though the equipment is costly, yet its cost-effectiveness needs little explaining. If inspection is considered inescapable, such a mobile inspection unit is the only means available, and once procured, it can be used for a large number of bridges very effectively.
3. If the professionals mean business and are prepared to discharge the responsibility of maintenance of bridges placed upon them, they must insist on the purchase of proper tools of inspection. Even when damage has taken place, the lack of means of access stand in the way of proper assessment and the timely corrective repairs. Should we rebuild at an enormous cost, rather than spend a comparatively nominal sum in providing proper means of inspection and timely remedies?
4. For very long span and/or wide bridges with large navigational clearances, like the cable-stayed suspension system or steel girder type, permanently installed travelling gantries and lifts are a must if inspection is to be carried out meaningfully, and timely maintenance assured.
5. Access to the piers and pier-frames of large height and bearings must be provided from the deck, and must be provided for at the design stage itself, so that even in routine inspections, the inspecting engineer has easy and safe access, which is essential if inspection is not to be avoided. However, no single equipment may be able to serve all the needs of maintenance inspection. It is essential that during the design stage itself the question of accessibility should receive adequate attention, and the means of access built-in as a part of the structure, to enable easy inspection and maintenance of maximum parts of the bridge.

- ***'Equipment' for Field-assessments during 'Inspection':***

(i) Rebound hammer for in-situ assessment of the strength of concrete (Schmidt hammer).
(ii) Magnetic detector for "locating reinforcement bars",
(iii) Mechanical extensometer and "transparent templates" for reading of crack-widths from surface contact.
(iv) Hydraulic devices, pressure transducers or load cells for measurement of forces or pressures, etc.
(v) — "Platforms" erected from "ground" or on
— "Barges" ... not very speedy or convenient.
— *"Ladders"* ... limited utility and range
— *"Cherry-Pickers" moving on bed (vehicle-mounted)*: ... limited to working from ground level but not on water and its reach is only about 10 m, ... not useful for taller heights.
— *"Snooper-Crawler", moving on deck itself*: ... costly equipment, but most versatile and elegantly suited for tall and long spans (together with an EXPLORER which can go down vertically along a pier/abutment from top down):
  * It is basically a *Vehicle-mounted multiple elbowed crane-arm* which can crawl and permit snooping of otherwise inaccessible bridge elements, e.g. underside of decks, beams, cross beams, expansion joints, bearings, pier and abutment caps, etc.
  * It also has a working platform, from which repairs can be attended to.
  * Its EXPLORER permits the inspectors to go down (and up) the tall substructure vertically and carry out the inspections elegantly.

- ***Equipment and Personnel***

(a) Based on type and numbers of structures to be surveyed, and within what required time frame, it is contemplated that three different categories of structural inspection crews will need to be established. Pertinent data regarding each is as follows:

Survey-Crew 'Type'	*Equipment Proposed*	*Items Inspected*	*No. Involved*
(A)	Ladders, cherry-pickers	'(1A) Bridges' less than 10.7* metres high '(2A) Box culverts' more than 1.5 metres high '(3A) Tunnels'	x_1 no. of bridges y_1 no. of box culverts z no. of tunnels
(B)	Snoopers	'(1B) Bridges' more than 10.7* metres high	x_2 no. of bridges + those bridges in (1A) unreachable from below
(C)	Video cameras	'(1C) Box culverts' less than 1.5 metres high '(2C) Pipe culverts'	y_2 no. of box culverts y_3 no. of pipe culverts

*10.7 m is the maximum height reach of a cherry-picker vehicle if inspection has to be done from below the bridge.

The estimate of inspection time for each of the items is as follows:

Item (1A): (x_1 bridges) × ('b' spans/bridge) × 1/5 day/span = n_1 days

Item (2A): (y_1 box culverts) × ('c' barrels/culvert) × ('d' metres/barrel) × (1/200 day per m per barrel) = n_2 days.

Item (3A): (z tunnels) × ('e' metres/tunnel) × (1/100 metres/day) = n_3 days

Total days of type (A) crew = $n_1 + n_2 + n_3$ = **'L' days**

Item (1B): (x_2 bridges × { 'f' spans per bridge } × (1/2 day per span) = **'M' days**

This is the total days of Type (B) crew

Item (1C): (y_2 culverts) × ('g' barrels/culvert) × (15 metres/barrel) × (1/250 day/m per barrel) = **'N_1' days**

Item (2C): (y_3 culverts) × ('h' metres/culvert) × 1/250 day/m ..... = **'N_2' days**

Total days of culvert survey crew = $N_1 + N_2$ = **'N' days**

- The amount of inspection time available as per project—

  Precedence chart = P months/crew × 4.33 wks/mo. × 6 days/week = Q 'crew-days

- *Number of crews required for each crew-type*

$$\text{Crew Type (A)} = \frac{L \text{ days}}{Q \text{ crew-days}} = \text{'}H\text{'} \text{ (nearest higher whole number)}$$

$$\text{Crew Type (B)} = \frac{\text{M days}}{\text{Q}} = \text{'J'}$$

$$\text{Crew Type (C)} = \frac{N \text{ days}}{Q} = \text{'K'}$$

$$\text{Total} = (\text{H} + \text{J} + \text{K} =) \text{ 'V' no. of crews}$$

The typical inspection crew will be composed of four full-time members and a part-time administrator with skill levels as follows:

Grade C: Trained Structural Engineer, field crew leader in charge of Activities A and B and preliminary assessment for disposition of a structure into Activity C.

Grade D: Trained Junior Civil Engineer responsible for inventory and condition measurements and filling-in of forms.

Grade E: Assistant, equipment operator if applicable, or traffic controller.

Administrator: Part-time worker, responsible for logistics support and coordination with local authorities.

Inspection crews will be supervised by a Senior Structural Engineer (Grade B) who will be responsible for three field teams at a time, preparation of reports, and will have the governing opinion for disposition of a structure into Activity C.

(b) *A partial list of required equipment for the 'V' no. of crews is given below*:

			Not required
• For bridges...	(1B)... 'Bronto' 4-lane Skylift (Snooper) (See Note 1 ahead)	—	decide
• For bridges...	(1A)... Cherry-picker 4-WD	—	decide
• For bridges, culverts, tunnels (traffic control)	Two cabin pickup 4-WD with flashing arrow panel	—	decide
• For culverts	(1C) & (2C) Culvert video Inspection	—	's' (See Note 2 ahead)
• Non-destructive testing pundit		—	—
• Destructive and redesign check	1. Concrete core drilling 2. Full-scale load testing 3. Bearing capacity check	—	Lease as needed
• Bridge deck testing	Ground Penetrating Radar (GPR)	—	—
	(See Note 3 ahead)	—	—
• Traffic warning signs	Survey crew	—	—
	Right/left lane closed-1/2 mile	—	—
	Pavement width transition	—	—
	End of work zone	—	—
	Traffic cones	—	—
• Miscellaneous	35 mm camera/flash	—	—
	Portable computers	—	—
	30′ ladder	—	—
	15′ ladder	—	—
	50′ pocket tape	—	—
	6′ folding ladder		—
	100′ tape		—
	Chipping hammer		—
	Calipers		—
	Scraper		—
	Small magnifying glass		—
	Flash lights		—
	Hard hats		—
	Marking tools		—
	Safety belts		—
	Crack-template gauges		—
	Drag chain		—
	Swiss hammer		—
	Pachometer*		—
	Red flags		—
	Rubber boots		—
	Rain suits		—
	Wire brushes		—
	First aid kits		—
	Steel toe shoes		—
	Goggles		—
	Gloves		—
	Walkie-Talkie		—

decide

decide

*For detecting reinforcement (and rust).

Notes

1. The ('Bronto' Skylift) Snooper provides access to the underside of the superstructure from the existing roadway surface for those structures which are excessively high or which due to the terrain cannot be viewed from the ground below via ladders or cherry-pickers. The only alternative to the use of this equipment is to construct a scaffolding system hanging from the underside of the bridge. This system, which would have to be constructed and dismantled in each span would be costly and very time consuming. In addition, the bridge condition may not permit hanging scaffolding system from the structures point of view.
2. The videos provide a method of inspecting those box culverts which could not be inspected by reasonably easily walking through them. Assume there exist approximately T_1 culverts which are 1.5 metres of less in height (which, by virtue of their span of 6 metres — distance between outside walls — will need to be inspected as part of the contract) and T_2 no. of pipe culverts. The amount of effort required to inspect these structures has to be determined. Assume that the T_1 box culverts and T_2 pipe culverts add up to approximately q metres length. In an available inspection time of Q days/crew, at an inspection rate of 250 metres/day, the total crews required will be $(q/Q = 250) = s$, and thus the number of video cameras necessary is likewise s.
3. For bridges having a concrete deck with an asphalt wearing surface, the Ground Penetrating Radar (GPR) is the best device and is recommended for purchase to detect deterioration, defects and any delamination problem in the concrete deck under the asphaltic cover. This device will help in determining the deck condition without removing the asphalt cover and in estimating the amount and type of rehabilitation required especially in Activity C. It will provide the necessary data required to help the decision maker to define the amount and type of action to restore the concrete deck to good condition. For instance the Virginia Department of Highways and Transportation in the U.S.A. has adopted the GPR technique for bridge inspection and has prepared a manual to assist engineers in the use of the radar technique.

46.12 SCHEDULE OF UNIT RATES FOR INVESTIGATIVE STRUCTURAL COMPUTATIONS FOR BRIDGES AND CULVERTS (... to be sought from the consultant if job let out on contract)

Type of Structure	Per structure ↓ Statically Determinate	Statically Indeterminate
I. Bridge Superstructure[1]		
(a) Slab type Solid, voided, reinf. conc./prest. conc.	...	...
(b) Stringer/multi-beam girder*	...	...
(c) Tee beam*	...	...
(d) Box beam/girder–multiple*	...	...
(e) Box beam/girder–single spread*	...	...
(f) Girder and floor beam system Reinf. conc. slab supported on transversely placed floor beams which rest on main longitudinal steel girders.	...	...
(g) Frame		
(i) Single bay	NA	...
(ii) Multiple bays	NA	...
(h) Orthotropic deck	...	...
(i) Truss-deck type	...	...
(j) Truss-through type	...	...
(k) Arch-deck type	...	...
(l) Arch-through type	...	...
(m) Suspension	...	...
(n) Stayed girder	...	...
(o) Movable — lift type	...	...
(p) Movable — bascule type	...	...
(q) Movable — swing type	...	...
II. Culverts[1]		
(a) Single span single cell, reinf. conc. box section	...	...
(b) Multi-span multiple cell, reinf. conc. box section		

NOTE: Consultants should note that in a group of structures whose superstructures (at least) are essentially identical, the fee for the investigative structural computation work will be paid only for one such structure out of the group owing to the advantage of repeat usage of the said computational work.

46.13 SCHEDULE OF UNIT RATES FOR 'STATIC LOAD' AND 'MATERIAL' TESTS (... to be sought from the consultant if job let out on contract)

I. Conducting equivalent static load test as per relevant standards in order to ascertain the allowable service load capacity.

A. *On deck-slab (in beam- and slab-type or box-type superstructure)*

(i) against punching shear at one critical section in a span ...

[1]May be skewed.

**Section:* beam and slab type (beams may be individual solid webs or individual boxes or a single/multi cell box); *Construction:* wholly cast *in situ* or partly precast and partly cast *in situ: Material:* reinf. conc. or prest. and reinf. conc. or steel beams with reinf. conc. slab (composite/non-composite).

(ii) each repeat of A (i) above in the same span or in any other span of the same structure ...
(iii) against flexure at one critical section in a span ...
(iv) each repeat of A (iii) above in same span or in any other span of the same structure ...

B. On solid-slab superstructure

(i) against flexure at a critical section in a span ...
(ii) each repeat of B (i) above in the same span or in any other span of the same bridge ...

C. On a longitudinal 'beam-member' of the superstructure

(i) against flexure at one critical section in a span ...
(ii) each repeat of C(i) above in the same span or in any other span of the same bridge ...
(iii) against transverse shear at one critical section in a span ...
(iv) each repeat of C (iii) above in the some span or in any other span of the same bridge ...

II. Conducting as per the relevant A.S.T.M. the compressive strength tests on drilled-out concrete cores (including the cost of drilling, etc., all inclusive) and tensile strength tests on test lengths of reinforcement bars/structural steel elements taken out as representative samples from the structures in question (including cost of obtaining the sample, making good the structure. etc., all inclusive):
(i) per acceptable concrete core test ...
(ii) per acceptable reinforcement sample test ...
(iii) per acceptable structural steel sample test ...

III. Conducting non-destructive testing of concrete using ultrasonic technique: per complete set of tests to ascertain the concrete quality and strength in the entire superstructure per span of approximately overall plan dimensions 30 m × 17 m. (This rate shall be applied pro rata for different plan dimensions.) ...

46.14 STRUCTURE-MAINTENANCE 'REPORTS' AND 'RECORDS'

Function

One of the important functions of bridge maintenance is to prepare a complete, accurate and current record of each bridge on the highway system. Much of the usefulness of the information obtained from field investigations depends upon its availability in a permanent and concise record. The records must be maintained in an orderly system and be readily available to any interested authorized person. Records should provide a full history of the structure including all recommendations for strengthening and repair along with the actions which have been taken on these recommendations. The file will provide data on the capacity of the structure, including the calculations substantiating reduced-load-limits. Complete record information in a good usable form is vital to the function of providing a safe, smooth riding highway.

Reports

- The basic records shall contain the general description, history, dimensions, and condition of the structure, as well as any recommendations which may have been necessary at and since the time of the first investigation. In making a report, keep in mind that money may be allocated or repairs designed, based on this information. Furthermore, it is a legal record which may form an important element in some future litigation. *The language used in reports should be clear and concise and, in the interest of uniformity, the same phraseology should be used as far as possible to avoid ambiguity of meaning.* The information contained in reports is obtained from field investigations, supplemented by reference to "as-built"or "field checked" plans. The source of all information contained in a report should be clearly evident.
- A report should be made for each bridge-investigation, even though it may only be a cursory investigation. Such short inspections are made many times for the purpose of checking some specific item where a problem or change may be anticipated. Even though no changes are evident in this inspection and the condition seems relatively unimportant, documenting this information would be valuable in the future.
  At least two photographs of each bridge investigated, one showing a roadway view and one a side elevation view, should be included *as part of the original bridge report.* Other photos necessary to show major defects or other important special features, should also be included. A photo showing utilities on the structure is desirable.
- Necessary stress calculations to determine the safe load limit based on field measurements or checked plans, should be available for quick and easy reference.
- A 'paint record' for each steel structure should be a part of the bridge report file. Subsequent paint inspections and painting work performed will be a part of the

record, thus keeping it current and providing information which will be valuable in programming future painting requirements.

Original Bridge Report (OBR)

Original bridge reports should show a minimum of the following information:

(i) *Bridge Number:* The official number assigned to structure.

(ii) *Date of Investigation:* Date on which the field investigation was made.

(iii) *Name:* The full name of the bridge. Other common names by which it is known may be placed in parentheses following the official name.

(iv) *Location:* Location of the bridge must be sufficiently described so that it can be readily spotted on a map or found in the field. It will normally be located by route number, district, and log-kilometre.

(v) *Description:* Briefly give all pertinent data concerning the type of structure. The type of superstructure will generally be given first followed by the type of piers and type of abutments along with their foundation. If the bridge is on piles, the type of piles should be stated. If data is available, indicate the type of soil upon which footings are founded, maximum bearing pressures, pile capacities, etc.

(vi) *Skew:* Normally, the skew angle will be taken from the plans and it is to be recorded to the nearest degree. If no plans are available, the angle is to be measured. If the skew angle is 0 degree, it should be so stated. The skew angle is the angle between the centre-line of a pier and a line normal to the roadway centre-line.

(vii) *Spans:* The number of spans and the span lengths are to be listed. These shall be listed in the same direction as the log-kilometre. Span lengths shall be recorded centre to centre (c/c) or clear open distance (clr) between piers, bents, or abutments. Measurements shall be along centre-line of the roadway.

(viii) *Total Length:* This shall be the overall length and shall be the length of the roadway which is supported on the bridge structure. This will normally be the length from paving notch to paving notch, or between front faces of Abutment-backwalls (dirt walls) measured along centre-line.

(ix) *Roadway Width:* This shall be the minimum clear width between kerbs. On divided roadways such as found on freeways under overcrossings, the roadway width will be taken as the travelled way between shoulders. Also, the shoulders and median width will be given.

(x) *Surfacing:* Type of wearing surface and its thickness.

(xi) *Sidewalks:* If only one is present, the side shall be noted thus: '1 at the rate of 1.5 m (east)'. Sidewalks on both sides are noted thus: '2 at the rate of 1.5 m'. If there are no sidewalks, note 'none'.

(xii) *Railing:* The type and material of the railing is to be listed.

(xiii) *Alignment:* Note whether the bridge is tangent or on a curve. If the bridge is on a curve, state the radius of the curve if plans are available for this information. On the older roads and bridges, a comparison of the alignment with the general alignment of the road should be made.

(xiv) *Traffic Lanes:* State the number of 'actual' traffic-lanes and number of 'design' traffic-lanes.

(xv) *Design Live Loading:* The live loading for which the bridge was designed will be stated if it is known. A structure widened or otherwise altered so that different portions have different live load designs, is to have each live loading specified.

(xvi) *Waterway:* May be classed as: 'not a factor', 'excessive', 'sufficient', 'barely sufficient', or 'insufficient'. The velocity of the stream should be classed with reference to its scouring probabilities, such as: 'normally high velocity', 'normally medium velocity'. A statement also should be made describing the material making up the stream bed. In cases where the cross-section is uniform, a statement may be made of the average clear height, and the profile usually submitted may be dispensed with.

(xvii) *Other Features Crossed:* List facilities over which the structure crosses in addition to the main obstacle.

(xviii) *Clearances:* A clearance diagram should be made for each structure which restricts the vertical clearance under the deck, such as overcrossings, underpasses, through-truss bridges, channels, etc. The minimum number of vertical measurements shown on the diagram will be at each edge of the travelled way and the minimum vertical clearance within the travelled way.

(xix) *Date Built:* Date of the completion. If the bridge has been widened since the time of the original construction, the widening date should also be shown thus: 'widened-(date)'. A similar notation should be made for any major reconstruction.

(xx) *Plans:* State what plans are available, where they are filed, and if they are "as-built".

(xxi) *Plans and Dimensions:* When plans are not on file, sufficient data shall be included under this heading to permit an adequate stress analysis of the entire structure, when needed.

(xxii) *Condition of Structure:* All signs of distress, failure, or defects worthy of mention, as well as descriptions of condition and appraisal, shall be noted with sufficient accuracy so that another man at a future date can easily make a comparison of condition or rate of disintegration/distress.

Photographs and sketches should be used freely as needed to illustrate and clarify conditions of structural elements. Good diagrams are very helpful at future investigations in determining progression of cracks, spalls, and various other defects, and to help determine changes and their magnitude.

(xxiii) *Restrictions:* Note any load, speed or traffic restrictions in force on the bridge and if known, record date of establishment and identification of agency who put the restriction or restrictions in force.

(xxiv) *Miscellaneous:* Include information on high water marks, unusual loadings or conditions, and such general statements as cannot be readily incorporated into the other headings.

(xxv) *Stress Analysis:* Some structures will require a stress analysis to determine their operating and inventory ratings or, where necessary, their load limits for posting. A general statement of the results of the stress analysis with a note on which members were found to be weak, what stresses were used, and any other modifying factors which were assumed in the analysis, should be given.

(xxvi) *Recommendations:* Under this heading, all directions for repair and maintenance, posting, rescinding of posting, or other instructions regarding the bridge, are included.

(xxvii) *Signature:* Reports are to be signed and dated by the person conducting the investigation.

(xxviii) *Channel Profile:* A sheet showing the channel profile at the upstream side of a bridge over a waterway should be a part of the original bridge report. The sketch should show the foundation of the structure and where available, description of material upon which footings are founded, elevation of the pile tips and/or of the footings of piers and abutments. This information is valuable for reference in anticipating possible scour problems through yearly observations and especially useful to detect serious conditions during period of heavy flow. Vertical measurements should be made or referenced to a part of the structure such as the top of kerb or top of railing which is readily accessible during high water.

Soundings, in addition to the single line channel profile are necessary at some river piers to provide adequate information on scour conditions and how the piers may be affected. Such requirements will vary with stream velocity and general channel stability. The necessity of additional soundings must be determined. These soundings will normally be limited to an area within a radius of 30 m from a pier.

(xxix) *Encroachment:* An encroachment sheet should be submitted with the original sheet of the original bridge report when there is one or more utility on the structure. A utility in the immediate area, though not fastened to the bridge, should also be included, such as a sewer line, etc.

(xxx) *Environmental Conditions:* Any unusual environment which may have an effect on the structure, such as salt spray, industrial gases, etc., should be noted in the report.

(xxxi) *Average Daily Traffic:* State the ADT along with the date or record. This information should be updated at intervals of approximately 5 years.

Revised Original Bridge Report (ROBR)

When a bridge is significantly altered by widening, lengthening or by some other manner which extensively modifies the structure, a new original bridge report should be written. The new report should be filled out completely using the new dimensions and description as necessary. Many of the items will be unchanged and remain the same as in the old original report.

The revised original report should clearly indicate that it supersedes the existing report. However, the old original report is still an important record for the structures and should be retained in the file.

Supplementary Bridge Report (SBR)

Reports on a structure are to be made for investigations made subsequent to the date of the original bridge report. Following are items that should be included in the *supplementary report:*

(i) *Bridge Number:*

(ii) *Date:* Date upon which the supplementary field investigation was made.

(iii) *Bridge Name and Location:* Same as for the original report.

(iv) *Work Done:* All work that has been done to the bridge since the last inspection should be listed.

(v) *Revised Dimensions:* This information is to be given when some maintenance or improvement work has

altered the dimensions of the structure. New bed profiles for unstable channels should be mentioned and plotted on the profile record sheet.

(vi) *Condition of Structure:* Similar as for original report.

(vii) *Restrictions:* Note any changes in load, speed, or traffic restrictions in force on the bridge from that recorded in the original bridge report or in a previous supplementary bridge report. Record the date of establishment and identification of agency who put the restriction or restrictions in force, if known.

(viii) *Revised Stress Analysis:* This information is to be given when maintenance or improvement work or reduced strength of members has altered the condition of the structure, thus making another stress analysis necessary.

(ix) *Recommendations:* All instructions for work, postings, or other items requiring acting from others will be included here in the same manner as was done in the original report.

(x) *Signature:* The report must be signed and dated by the person responsible for the report.

Records — Check List

— ***Function***

— ***Reports:***

1. *General*
2. *Original Bridge Report:*
   - o Bridge Number
   - o Date of Investigation
   - o Name
   - o Location
   - o Description
   - o Skew
   - o Spans
   - o Total Length
   - o Roadway Width
   - o Surfacing
   - o Sidewalks
   - o Railing
   - o Alignment
   - o Traffic Lanes
   - o Design Live Loading
   - o Waterway
   - o Other Features Crossed
   - o Clearances
   - o Date Built
   - o Plans
   - o Plans and Dimensions
   - o Condition of Structure
   - o Restrictions
   - o Miscellaneous
   - o Stress Analysis
   - o Recommendations
   - o *Signature*
   - o Channel Profile
   - o Encroachments
   - o Environmental Conditions
   - o Average Daily Traffic
3. *Revised Original Bridge Report*
4. *Supplementary Bridge Report:*
   - o Bridge Number
   - o Date
   - o Bridge Name and Location
   - o Work Done
   - o Revised Dimensions
   - o Conditions of Structure
   - o Restrictions
   - o Revised Stress Analysis
   - o Recommendations
   - o *Signature*

46.15 REHABILITATION WORK

This will include guidelines for repairing various types of distresses. (See References 2 and 3.)

REFERENCES

1. Raina, V.K., *"Consultancy and Construction Agreements for Bridges, including Field Investigations,"* Tata McGraw-Hill Publishers, New Delhi.
2. Raina, V.K., *"Concrete Bridge Practice—Construction, Maintenance, and Rehabilitation*, Tata McGraw-Hill Publishers, New Delhi.
3. Raina, V.K., *"Concrete for Construction—Facts and Practice*, Tata McGraw-Hill Publishers, New Delhi.
4. Raina, V.K., ***"Inspection, Repair, Strengthening, Testing and Load Capacity Evaluation of Concrete Bridges,"*** Tata McGraw-Hill, New Delhi.

CHAPTER 47

'Bridge-Distress' Reporting — A Workman-like Approach

NOTE: Also refer to Chapter 5 in the author's other book: *Inspection, Repair, Strengthening, Testing and Load Capacity Evaluation of Concrete Bridges* for a more detailed presentation of the subject.

47.1 INTRODUCTION

- In order to receive 'accurate-enough' feedback from the field-engineers regarding any distress in various component-elements of bridges, it is a good idea to train them in filling a simplified 'Report-Form' and ensure that they understand its details (remembering that their levels of technical understanding may be rather humble, for various reasons).
- For this purpose a simplified Questionnaire-type Report-Form is presented in 47.2 below, which should be read in accompaniment with its ATTACHMENT 'A'. An 'example' of how to fill this Report-Form for a particular bridge (hypothetical) is then presented in ATTACHMENT 'B' in order to orchestrate the Bridge-Distress Reporting methodology.
- Sometimes extra vigilance is required in respect of 'Special Bridges'. In such a case it becomes necessary to identify the Special Bridges. A bridge may be simple rather than complicated in terms of its structural details but yet it may well qualify as a Special Bridge simply because it may lie in an urban (congested) area where detouring may be very difficult, or that it may lie in an important artery which cannot be afforded to be closed, or that it may be so high that detouring through the bed so much below it may not be possible. A bridge may also qualify as a Special Bridge owing to possibilities of its becoming unpassable because of chances of:
  - — failure by instability,
  - — failure due to fatigue,
  - — failure of its components,
  - — failure of its systems,
  - — partial failure due to severe exposure conditions
  - and — difficulty in maintenance owing to complicated structural arrangements or features.

For this purpose 'Definition of a Special Bridge' has been briefly indicated in ATTACHMENT 'C'. This may be of assistance while deciding as to which bridges are 'special'.

47.2 BRIDGE-DISTRESS REPORTING (IN A QUESTIONNAIRE FORMAT). (FOR APPORTIONING APPROPRIATE CONDITION-RATINGS . . . SEE THE DISCUSSION ON THIS SUBJECT IN CHAPTER 46.)

QUESTIONNAIRE:

- NOTES *for the field-engineer reporting the distress:*

1. For definition of "Special Bridge", see enclosed Attachment 'C'.
2. From your records and information about each "special bridge" in your jurisdiction, please indicate what problems, if any, were encountered (or are likely to be encountered) during its service-life* that required (are likely to require) special efforts for Maintenance and Repair.

   The problems could be in Foundations, Abutments, Piers, Bearings, Expansion Joints, Superstructure, etc.

   Please also indicate any of the Construction-Stage problems, information about which may be important during Maintenance phase.
3. For each Special Bridge with problematic Maintenance, the necessary information should be given briefly in the format given below. Please also describe any relevant information that may not have been covered in this Questionnaire.
4. *Assistance in pin-pointing some of the items that may call for special vigilance and Maintenance:*

   Purely for your assistance, some possible Problems that can call for Special Maintenance-effort have already been pointed out and 'serial numbered' in the enclosed Attachment 'A'. Please go through these carefully.
5. Where you find that some details and Problems of your Bridges can be appropriately expressed by the descriptions given in the Attachment 'A', there the 'corresponding Serial Numbers' should be inserted in the relevant boxes in this Questionnaire. This will save time and retain uniformity of expression and adequacy of detail. This will help in the overall evaluation task ultimately. (Attachment 'B' shows an Example of how to do the distress Reporting.)

• REPORTING:

(a) Bridge's Name: .....
(b) At Stn. .....
(c) Location .....
(d) Type .....
(e) Longitudinal Spans and Arrangement (Sketch) . .
.....
.....
(f) Cross-section of Superstructure (Sketch) ...
.....
.....
(g) Date Constructed .....
(h) Construction Contractor's Name .....
(i) Construction Supervised by .....
(j) Design Consultant .....
(k) Maintenance Contractor's Name .....
(l) Why do you think it is a Special Bridge (if it is a Special Bridge): .....

1. FOUNDATIONS:

Type: [ ], ...

Problems: [ ], [ ], ...

*Condition Rating** [ ], [ ], [ ], ...

• (element by element)

• • *Hence decide if Bridge useable 'globally', subject to what Maintenance-Actions,* [*or*] *'Close Bridge till remedial measure taken',* [*or*] *'Replacement (and temporary auxiliary supports) necessary, meanwhile close the Bridge.*

Brief descriptions with explanatory sketches:
.....
.....

2. ABUTMENTS & PIERS:

Type: [ ], [ ], [ ], ...

Problems: [ ], [ ], ...

Condition Rating: [ ], [ ], [ ], ...

• (element by element)

• • *Hence decide if Bridge useable 'globally', subject to what Maintenance Actions,* [*or*] *'Close Bridge till remedial measures taken,'* [*or*] *the Replacement (and temporary auxilliary supports) necessary, but meanwhile close the Bridge.*

Brief descriptions with explanatory sketches:
.....
.....

* For explanation, see discussion in Chapter 46.

3. BEARINGS:

Type: [ ], ...

Problems: [ ], [ ], ...

Condition Rating: [ ], [ ], ...
(each)

Brief descriptions with explanatory sketches:
.....
.....

4. EXPANSION JOINTS:

Type: [ ], ...

Problems: [ ], [ ], ...

Condition Rating: [ ], [ ],
(each)

Brief descriptions with explanatory sketches:

5. SUPERSTRUCTURE:

Type: [ ], ...

Problems: [ ], [ ], ...

Condition Rating: [ ], [ ], [ ], ...

• (element by element)

• • *Hence decide if Superstructure useable 'globally' subject to what Maintenance-Actions,* [*or*] *'Close Bridge till remedial measures taken',* [*or*] *Superstructure must be replaced immediately (meanwhile close the bridge.)*

Brief descriptions with explanatory sketches:
.....
.....

6. DRAINAGE:

Problem: [ ], ...

Condition Rating: [ ], [ ], ...

Brief descriptions with explanatory sketches:
.....
.....

7. CORROSION IN STRUCTURAL-STEEL

Type: [ ], ...

Problem: [ ], [ ], ...

Condition Rating: [ ], [ ], ...
(each element)

Brief descriptions with explanatory sketches:
.....
.....

ATTACHMENT 'A' to BRIDGE 'QUESTIONNAIRE'

1 - FOUNDATIONS (symbol F)

- *Types:*
  - # **F**.1 footings (open foundations)
  - # **F**.2 piles
  - # **F**.3 caissons
  - # **F**.4 other (explain)
- *Some Problems may arise from:*
  - # **F**.a - undermining due to scour,
  - # **F**.b - dislodging of Gabions/protection works,
  - # **F**.c - exposure of piles,
  - # **F**.d - concrete cracking (from numerous causes*)
  - # **F**.e - loosening of anchor-holds,
  - # **F**.f - settlement (differential, overall),
  - # **F**.g - movement/tilting,
  - # **F**.h - overload/under-design**
  - # **F**.i - other (explain)

---

* *these could be:*

- plastic shrinkage of concrete,
- plastic settlement of concrete,
- early thermal and shrinkage movements and lack of adequately 'spaced' and 'functioning' movement-joints,
- drying shrinkage of concrete (long term),
- rusting of steel under moist conditions, due to:
  - — oxidation of steel,
  - — carbonation of concrete,
  - — chloride-ion attack on steel (chloride salts from surrounding soil and water or from aggregates themselves if contaminated),
- sulphate attack on cement in concrete (sulphate salts may ingress from surrounding soil and water or may also be present as impurities in the aggregates),
- alkali-aggregate reaction (where aggregates contain reactive silicates and carbonates),
- Temperature:
  - — variation
  - — gradients
- weathering and disintegration of concrete,
- crazing in concrete,
- fire and/or explosion

etc.

(see REFERENCES 1, 2 and 3.)

** *resulting in distress due to:*

- excessive flexure, shear, and/or torsion
- excessive direct force (axial compression/tension)
- excessive bearing pressure
- restraint against designed deformation
- inadequate soil-bearing conditions, etc.

2 - ABUTMENTS & PIERS: (Symbols **A** *and* **P***)*

- *Types:*
  - # **A**.1 —open-type abutments
  - # **A**.2 —closed-type abutments
  - # **A**.3 —other (explain)
  - # **P**.1 —solid wall type pier
  - # **P**.2 —frame type pier
  - # **P**.3 —single shaft type pier, solid
  - # **P**.4 —single shaft type pier, hollow
  - # **P**.5 —other (explain)
- *Some Problems may arise from:*
  - # **AP**.a — tendency to move/slide/rotate, which may also result in change in plumb and line and consequent misalignment/displacement in the Bearings (with all the consequent effects),
  - # **AP**.b — consequences of:
    - —foundation scour
    - —foundation settlements
  - # **AP**.c — impact hits from passing vehicles (and from barges and floating debris in case of bridges in water bodies),
  - # **AP**.d — height more than about 10 m, so that unusual and special equipment may be needed for inspecting and carrying out the Maintenance Work in inaccessible zones,
  - # **AP**.e — Concrete cracking (from numerous causes*)
  - # **AP**.f — overload/under-design
  - # **AP**.g — other (explain)

3 - BEARINGS (Symbol **B***)*

- *Types:*
  - # **B**.1 — 'Elastomeric' type:
    - · generally used for relatively short-span and medium-span bridges.
  - # **B**.2 — 'Pot' type:
    - · generally used for medium-span bridges.
  - # **B**.3 — 'Spherical' and 'Cylindrical' (teflon coated) type:
    - · generally used where deck-rotation and/or loads are large.
  - # **B**.4 — other (explain)
- *Some Problems may arise from:*
  - # **B**.a — In the **Elastomeric Bearings** (which may be provided with a bonded PTFE layer and a stainless steel sliding plate when large movements are to be catered for), problems may arise from:
    - (i) compression bulging of sides,

(ii) unequal vertical deformation among a row of bearings in one line, owing to failure of one or more bearings due to excessive load,
(iii) unequal vertical deformation owing to improper seating surfaces between the bearing and the deck-soffit and/or the pedestal underneath,
(iv) 'surface' cracking,
(v) 'radial' cracking in the bulges,
(vi) excessive shear-deformation (due either to under-design or manufacturing defect).

B.b — In the **Pot Bearings** (of 'fixed', 'free to slide in any direction' or 'guided to slide in a particular direction' types), problems may arise from:
(i) excessive rotation causing excessive deformation in the potted elastomeric disc and in the surrounding seal. (The temporary clamping bolts should either have been removed after installation or should have sheared subsequently.)
(ii) In the case of sliding types, if the PTFE bonded to the top of the middle-plate is outstretchingly visible, then the top plate is not long enough in the direction of principal-movement. Such a bearing may need to be replaced (since such naked PTFE, open to ingress of dirt, may soon cease to function as designed).

B.c — In the **Spherical and Cylindrical Steel Bearings** (former capable of rotation in any direction and the latter capable of rotation in only one direction), with mating convex and concave (coated) steel surfaces, problems may arise that are somewhat similar to those in the case of Pot Bearings described earlier.

B.d — In the case of **crude forms of bearings** (such as felt or tar-paper layers), problems, may arise in the deck above and in the concrete under the bearings from cracking due to 'constraint against free rotation and sliding'.

B.e — Other (explain)

4 - EXPANSION JOINTS (Symbol **EJ***)*

- *Types:*

EJ.1 — Buried-type (These are ancient type. In these a joint-filler is filled in the movement-accommodating-gap of the joint, and the wearing course is either carried across or butts with it from either side),

EJ.2 — Elastomeric-type (These are most commonly used in the modern-day bridges. They are waterproofed and modular in build-up, making transporting, handling and installing easy. Can take large movements of superstructure.)

EJ.3 — Finger-type (Generally these are steel plates cut out into intermeshing finger arrangement, bolted to the edges of the adjacent decks.)

EJ.4 — Other (explain)

- *Some Problems may arise from:*

EJ.a — In the **Buried-type** joints:
(i) swelling-up of wearing course,
(ii) weeping out of the filler,
(iii) consequent thudding by the passing wheels and riding-discomfort,
(iv) cracking at edges.

EJ.b — In the **Elastomeric-type** joints:
(i) Tearing of elastomer (and consequent exposure of their reinforcing steel plates where such plates exist). This may be due to manufacturing defect or due to breaking of concrete in the supporting deck-edges,
(ii) loosening of anchor bolts (and obvious consequences).

EJ.c — In the **Finger-Type** joints:
(i) loosening of anchor bolts,
(ii) fracture of fingers (owing to combined effect of high impact pounding from passing wheels and fatigue from highly repetitive nature of such loading, the effect becoming severer as the anchor bolts loosen.)

EJ.d — Other (explain)

5 - SUPERSTRUCTURE (Symbol **S***)*

- *Types:*
  - *The* **span-arrangement** *may be:*

S.1 · Simply supported spans

S.2 · Balanced-Cantilever and segmentally built Free-Cantilever Spans
S.3 · Semicontinuous spans (i.e. simply supported precast p.s.c. girders with continuous r.c.deck slab over piers, making them continuous for subsequent loads)
S.4 · Continuous spans (either cast as such, or built by stage-construction, or built by segmental cantilever method)
S.5 · Frame (single or multi-span)
S.6 · Cable-Stayed spans
S.7 · Other (explain)

- *The* **construction** *may be in:*
  - reinforced concrete
  - reinforced and prestressed concrete
  - composite or non-composite structural steel and reinforced concrete
  - structural steel (plate girder or Truss)
  - Other (explain)
- *The* **superstructure cross-section** *may be:*
  - (i) · 'r.c. solid slab'-type (cast-in-situ, c.i.s.),
  - (ii) · 'r.c. voided slab'-type (c.i.s.),
  - (iii) · 'r.c. box section'-type (c.i.s.),
  - (iv) · 'r.c. beams-and-slab'-type (c.i.s.),
  - (v) · 'precast r.c. beams and cast-in-situ slab'-type,
  - (vi) · 'precast p.s.c. beams and cast-in-situ slab'-type,
  - (vii) · 'p.s.c. voided slab'-type (c.i.s.),
  - (viii) · 'p.s.c. box'-type (c.i.s. or precast segmental),
  - (ix) · 'precast box segments prestressed with precast concrete web, diaphragm and slab elements'-type,
  - (x) · 'steel beams and concrete slab' (composite or non-composite)-type,
  - (xi) · other (explain)
- *Some Problems may arise from:*

S.a · concrete cracking (from numerous causes*)
S.b · settlement of foundations (differential, overall)
S.c · movement/tilting of foundations
S.d · overload/under-design**
S.e · accidental hit from passing traffic
S.f · accidental fire
S.g · distress in Articulations (Halving Joints)
S.h · Shear lag effect (particularly during the construction stages) whereby the applied prestress is absorbed by only part of the assumed deck-section, leading to cracking in unprestressed regions.
S.i · inaccessibility for inspection (because ... piers taller than about 10 m. so that crane and ladder arrangement not enough; etc.)
S.j · other (explain)

*, * * : See footnotes on page 649.

6 - DRAINAGE (Symbol D*)*

D. Inefficient drainage of water from and around the bridge can be hazardous for the traffic and can also lead to unsightly staining of concrete surfaces. When such ponded water gets mixed with any wind-blown sand and surrounding soil containing sulphate and chloride salts, the resulting attacks on concrete and steel can lead to harmful cracking of concrete and rusting of steel and continuous disintegration of the effected reinforced (and prestressed) concrete components of the bridge. Footings, piles and pile-caps, buried and boxed portions of substructure, caps and seating beams, abutment-walls, boxed-decks, etc., can all fall prey to such distress and damage.

7 - CORROSION IN STRUCTURAL-STEEL (Symbol C*)*

C. Steel beams, boxes and trusses, used in superstructures for example, are susceptible to rusting and corrosion-distress due to oxidation and chloride-attack under moist conditions. Hence the anti-rust painting must always be ensured as a rust-protection means. Detection of paint-failure requires close hands-on examination. Where rust-inhibiting steel (CORTEN) is used without any protective-coating, the effectiveness of the rust-inhibiting iron-oxide coating should be checked for distress (e.g. signs of unevenness and flaking of the surface protecting layer). The most likely areas of rust-formation are: external faces of outer girders, all welds/rivets/bolts, site joints, and gusset connections. Old steel bearings also need a special examination to ensure whether rust has not incapacitated them from performing the intended rocking/rolling/sliding, as the case may be. (See References 1 and 2.)

REFERENCES

1. Raina, V.K., *"Concrete for Construction—Facts and Practice,"* Tata McGraw-Hill, New Delhi.
2. Raina, V.K., *"Concrete Bridge Practice—Construction, Maintenance and Rehabilitation,"* Tata McGraw-Hill, New Delhi.
3. Raina, V.K., *"Concrete Bridges—Inspection, Repair, Strengthening, Testing and Load Capacity Evaluation,"* Tata McGraw-Hill, New Delhi.

ATTACHMENT 'B' (EXAMPLE)

An 'EXAMPLE' of how to fill the BRIDGE 'QUESTIONNAIRE' if the information given in the ATTACHMENT 'B' has to be conveyed:

NOTE : **Condition Ratings have not been indicated here but in an actual case the appropriate RATINGS should be shown, based on the Ratings discussed in Chapter 46.**

PROCEED as follows:

— items (a) to (l) ... fill-in as relevant to the particular bridge in question.

— ***FOUNDATIONS:***

- If FOUNDATION is, say, of 'footing' type, then insert [F.1] in the box in front of 'Type'.
- If the Problems are, say, 'undermining due to scour' and 'settlement', then insert [F.a] and [F.f] in the boxes in front of 'Problems'.
- Also, give brief descriptions with simple free-hand sketches as to 'which foundations', 'extents of scour and settlement', etc.

— ***ABUTMENTS & PIERS:***

- If ABUTMENTS are, say, of 'open' type and PIERS are 'solid wall' type, insert [A.1] and [P.1] in the boxes in front of 'Type'.
- If the Problem is, say, 'foundation scour' only in a particular Pier, then insert [P.b (scour)] in one box in front of 'Problems'.
- Also, give a brief description with simple free-hand sketches as to 'which pier', 'problem caused', etc.

— ***BEARINGS:***

- If BEARINGS are, say, of 'Elastomeric' type, then insert [B.1] in the box in front of 'Type'.
- If the problem is, say, 'compression bulging of sides', then insert [B.a. (i)] in one box in front of 'Problems'.
- Also, give a brief description as to 'which Bearing', 'extent of bulging/damage', etc.

— ***EXPANSION JOINTS:***

If it is, say, an 'Elastomeric' type Joint and the elastomer has torn off, and 'breaking of the supporting concrete edge of deck' has been the cause, then insert [EJ.2] & [EJ.b.(i) (due to breaking of the supporting concrete edge)] in the boxes in front of 'Type' and 'Problems', respectively.

Also, give necessary descriptions with explanatory sketches.

— ***SUPERSTRUCTURE:***

- If it is, say, 'p.s.c. continuous spans' of 'box' section, built using 'precast segments', and the problem is, say, damage due to an 'accidental hit from an over-height speeding truck', then insert [S.4 (viii), precast segmental] and [S.e] in the boxes in front of 'Type' and 'Problems', respectively.
- Also give brief description as to 'location' and 'extent', and explanatory sketches.

NOTE:

No other problems in this bridge.

ATTACHMENT 'C' to BRIDGE QUESTIONNAIRE

Definition of a 'Special' Bridge Structure

1. General:

Highway structures may be considered Special when their failure, partial or total, causes disfunction of the highway system. The more serious the disruption, the more Special the Structure in terms of maintenance requirements. The gravity of a failure may be assessed in terms of potential loss of life and property, capital cost of rehabilitation work or the disruption to traffic with resultant loss in trade, communication and strategic planning. Special Bridge Structures can thus be categorised in different ways. A system using failure mechanism is used to describe Special Structures here.

2. Failure by Instability:

Structural failure by instability is sudden and total, and often involves loss of life and property. Such Special Structures are:

a) Bridges in escarpments whose foundations may be undermined by precipitation run off during the rainy season.
b) Wadi-Bridges whose foundations may be undermined by scour.
c) Bridges in unstable geological formation which may be damaged by landslides.

3. Fatigue Failure:

Like failure by instability, fatigue failures are sudden and often catastrophic. Some examples of fatigue failure are:

a) Failure of tie-pins in suspended spans of steel bridges.
b) Unbonded external prestressing strands.
c) Hangers of suspension bridges.
d) Stay anchorages of stay bridges.

4. Component Failure:

Component failure is generally not catastrophic but unless corrected, will lead to structural failure. The most common types of such failures are:

a) Bridge bearings such as elastomeric, pot, spherical and steel bearings of various types.
b) Bridge expansion joints, including nosings.

5. Systems Failure:

Failure of electro-mechanical systems may make structures inoperable. Examples of such systems are:

a) Electro-mechanical units of moveable bridges e.g. swing, bascule and lift bridges.
b) Ventilation of long tunnels.
c) Drainage and Pumping Systems of Underpasses.

6. Partial Failure From Severe Exposure Conditions:

Bridges in severe environment such as marine and coastal, are particularly susceptible to partial failure from material deterioration. To this category must be added Structures where the ground contains high concentration of salts such as in 'Sabkha' areas.

7. Special Structures by Scale, Design and Construction:

In addition to the Special Structures categorised above by failure types, there are Structures which are complex both by design and construction technique. There are also Structures which are Special by virtue of their scale. The maintenance requirements of such Structures must take into account the special features of design, construction and scale.

a— Such a bridge could even be a short single span type if, for instance, it is a frame or has a curved or skewed deck. On the contrary, a Special Bridge does not necessarily have to be long. For example, a multi-span bridge, comprising of a number of relatively short simply supported spans with superstructure comprising of say cast-in-situ reinforced concrete 'slab' or 'beam-and-slab' arrangement, resting through Neoprene Pads on a mundane substructure of not too tall piers on shallow open-foundations (not subjected to any significant scour), may qualify as a Special Bridge only if it is strategically located.

b— On the other hand, if a bridge has significantly tall piers (in excess of about 10 m.), involves deep foundations (that are also likely to be subjected to significant scour), and has a superstructure that is longitudinally continuous with or without halving-joints (possibly also curved and skewed in plan), involves prestressing in one or both directions, has special (modular or finger type) expansion joints to cater for large deck movements, etc., then, such a bridge is a Special Structure. It calls for a precise understanding of the function and performance of its foundations, the superstructure, and of the special expansion joints and bearings. It also calls for special equipment for carrying out inspection and attending to the maintenance work.

c— In addition to these special features, following are some examples of Bridges whose design as well as construction are Special in themselves, and they consequently call for Special effort both for understanding their structural mechanics as well as in their monitoring and maintenance:

—Frame, single or multi-span
—Orthotropic deck
—Continuous Truss—deck type
—through type
—Cable-Stayed bridge
—Suspension bridge
—Movable Superstructure—LIFT type
—SWING type
—BASCULE type

8. Examples of Some of the SPECIAL Features of Bridges (which on their own may or may not qualify the Bridge to be a Special Bridge):

— Abutments and Piers being taller than about 10m, (requiring Special Plant and Equipment for inspecting and repair work).
— Foundations subjected to significant Scour and possible undermining.
— Special Bearings (of Elastomeric type, Pot type, Spherical type etc.) subjected to high loads and large movements and rotations.
— **Superstructures** of the following types:
  o Statically determinate superstructures (simply supported spans, or balanced cantilevers), involving spans in excess of about 25 to 30m. These may comprise of 'p/c psc beams and in-situ top slab' or 'box-sections', etc.
  o Statically indeterminate superstructures, involving

individual spans in excess of about 10 to 15 m. (It could be a voided slab, or 'simple-span beams rendered semicontinuous by in-situ continuous r.c.slab' or 'box-section'.)

— The 'Halving-Joints' (i.e. the 'Articulations') in the Superstructures.

— The Expansion Joints, particularly those that cater for movements of an 'aggregate superstructure-length in excess of about 30 to 40m' from the Fixed Bearing point (or Zero-Movement-Point).

CHAPTER 48

Bridge Engineering—Some Topical Reflections

48.1 INTRODUCTION

Bridge design and construction have come a long way during the past few centuries and 'much water has flowed down' particularly in the past hundred years or so during which more bridges have been built than in all the previous centuries. In fact large number of today's existing bridges have been built only during the past fifty years or so and the rate of design and construction has been the highest essentially during the past twentyfive years.

This rather recent rapid rate of growth in the comprehension of the science of analysis and design and the art of building bridges has led to numerous developmental experiences. Secret of success being constancy of purpose, a workman like introspection leads to certain 'reflections', some of which perhaps are topical. The topics chosen here are:

- the 'bridge culture' (a brief historical note),
- rational approach to structural design,
- 'new' codes of practice (split load-factors),
- design-education,
- cure against cracking of concrete—not by calculations alone!
- life care of road bridges,
- concrete structure... deterioration and rehabilitation.
- appropriate technology for developing countries.
- blue-print for guiding technical development in the field of civil engineering public works in developing countries.
- inspection check-list (... field work).

While conservatism may have a place in some fields of activity, a robustly practical engineer has to be constantly on the run, merely to keep up. This would not be so bad if the engineering activity had only a short life span before it went obsolete (e.g. in electronics, space technology and weaponry). Unfortunately for the poor bridge engineer, whose bridges must last upwards of half a century at least, long range forecasting of the changes in social organization, which influence the quality and quantity of traffic, becomes a predicament. This calls for an amalgam of intuition, introspection, and reflection, by the initiated restless minds.

48.2 THE 'BRIDGE CULTURE' (A brief historical note)

1. The primitive bridge building must have started with a trial and error process. In some misty morning of prehistoric past, a human trying to cross a stream, probably saw a fallen tree across it. When he tried to clamber over, it might have broken and dropped him in the drink. So he thought real hard and felled a bigger tree and this took him across. The first primitive single span wooden beam bridge was thus designed and built that day. In reality the modern bridge hardly conceal their ancestry. Vine ropes secured on two towering trees on the opposite banks of a deep gorge, with a bamboo or brushwood deck or without one, were the earliest forerunners of the suspension bridges. The first arch bridge might have similarly been built about four thousand years back in the Euphrates valley.
2. ***The man who creates technology, the engineer with the aid of the scientist, is the society's most radical revolutionary. He is the fundamental agent of all social changes.***
3. To trace this glorious history of bridges from the beginning, we should perhaps divide it into two periods, as it is customary to talk about history in terms of periods. The first period would be before concrete—the stone to steel ages and second in which we are living, after concrete. Stone and timber were the traditional building materials since the start of civilisation. Stone arches used mostly in early bridges date back to the days of Babylon. Romans learned the art of building arches from the Ettruscans. These arches were semi-circular. Flat circular arches appeared in full glory during the renaissance period. Although the first theory of arches came to be established as late as 1695 and was used in practical design only in 1729, the optimum profile of the stone arch had been found very early by artist builders intuitively and has changed very little since. The proven durability of material and the long experience in intuitive proportioning made masonry arch bridges the most popular form of construction even in the early days of railways until iron bridges took over. Arch did continue to be built in iron, steel and concrete

because of its unmatched aesthetical qualities, besides structural efficiency.

4. While stone and timber remained the common building materials for bridges, the mid-nineteenth century demanded stronger and bigger bridges over large rivers for railways. So around 1840 the transition period from timber to steel began. In this period cast iron (and later wrought iron) was tried out by bridge builders. The first recorded Western use of iron in bridges was a chain bridge built in 1734 by the German army across the Oder river in Prussia. However, cast iron was not found very suitable for building large bridges. A combination of cast iron for compression members and wrought iron for tension members was first used in truss bridges from 1840 onwards, specially for railway bridges.
5. In 1856, Bessemer patented a process for making large quantities of steel economically. In 1861 Siemens and Martin introduced the open hearth process. In the last part of the 19th century the new material steel caught the imagination of bridge builders. By 1890 the advantages of the steel were undisputed and the use of structural steel spread with speed to all countries. Wrought iron was completely expelled from all new bridges and by the turn of the century steel brought in spans unlimited, built for the heavy loads of railway traffic. The Firth of Forth cantilever bridge of 520m span or Brooklyn bridge, Roebling's suspension bridge of 490m span, were a few of the famous achievements of the 19th century to mark the beginning of modern era of bridge engineering. By the turn of the century, the growing use and availability of structural steel and greater skills in design analysis paved the way for larger span bridges. Multiple span girder bridges, arch bridges and cantilever bridges in steel reached up to maximum spans of 500m or more with comparative ease. Howrah bridge (1943), a steel cantilever bridge with a main span of 150 m, is a typical example of the large crop of outstanding bridges built in steel in the earlier years of this century.
6. A very substantial advance in the understanding and theoretical analysis of the load response of the structural system of suspension bridges was made by the twenties of this century and many elegant bridges were built, like Lindenthal's Manhattan bridge (1909) with 450m span, followed by Steinman's 340m span Florianopolis bridge and the Delaware river bridge of 530m span in 1926, to name a few at random. The big leap forward came in 1931 with the construction of George Washington bridge, which has been acclaimed by Stussi as a "great and most important step in the evolution of the art of bridge engineering". Le Corbusier was enchanted with this bridge and in his 1937 publication "When Cathedrals Were White" said: "The George Washington Bridge is the most beautiful bridge in the world... It is blessed, the seat of grace." The bridge broke the 1000m span barrier and its span length of 1060m was double that of Delaware bridge, the then longest span in the world. It also proved, by the successful construction of an eight-lane major roadway without stiffening girders, an important fact sensed by Navier and intuitively made use of by Roebling. The bridge was built by Amman, a Swiss engineer who migrated to America in 1904. Three decades later, this 'foremost bridge builder in the world' contributed his last great masterpiece to the land of his adoption. Amman's Verrazano Narrows Bridge in New York was opened in 1964, ten months before he died. Standing with a main span of 1300m, the longest realised until now in U.S.A., the bridge is the highest landmark in the history of long span suspensions bridge building, which has flourished since the thirties, leading to the construction of economical, gracefully slender and ambitious structures in large numbers in all parts of the world. The lessons in ensuring aerodynamic stability were learnt in a hard way by the tragedy of the 855m Tacoma Narrows Bridge which fluttered and perished in 1940 in a 64 km/h wind. The progress has gone unabated with innovations, like the stiffened deck of Severn bridge, to replace the deep stiffening trusses and the future may reveal yet newer directions of development.
7. In 1820, Navier in opting for the classical suspension system, condemned the cable stayed solution as unsuitable. It remained so until in 1938 Dischinger developed a suspension system which was actually a combination of cable stayed and classical suspension bridge types. The actual development of the modern cable stayed bridge started with the construction of Stromsund bridge in Sweden around 1953 by DEMAG. Then in an unprecedented spurt of development followed the famous Rhine family of cable stayed bridges, which at present number about a dozen with spans up to 300m and more.
8. Leonhardt hypothecated in his 1972 IABSE paper that for spans of 750m to 1500m the cable stayed system was technically and economically superior to the classical suspension bridge especially with regard to aerodynamic stability. The use of A-form pylons with corresponding inclination of planes of cables, fan shaped smaller diameter cables at closer spacing etc., are the new trends in the design to realise the large spans. High cable stresses to ensure favourable stiffness ratios and flat aerodynamically shaped beam sections with sharp wind noses are being used. The cable-stayed system with a steel or concrete deck will surely find

increasing use in many future bridges, from the modest foot-bridge to the mighty vehicular bridges, not only for the slim attractive looks which are no mean virtues, but also as the economically viable alternative to other conventional bridge types.

9. The demands of the bridge builder and his reliance on scientific knowledge led to the birth of structural mechanics. Timoshenko in his 'History of Strength of materials' points out: "The construction of the first railroads greatly affected the development of strength of materials by presenting a series of new problems especially in bridge engineering which had to be solved". So the midpoint of nineteenth century marked a turning point in the history of bridge building by a change to scientific method of design of bridges and consequently for all structural design. Stussi remarked in the IABSE Symposium on 'Concepts of Safety of Structures' (London, 1969) that the scientific period of design "was introduced by Louis Navier (1785-1836) who provided the transition to scientifically based construction with his principal work; "Resume des Lecons" (1826) and thereby fundamentally created the applied science of statics". Navier gave these lessons on 'the application of mechanics in the establishment of construction and of machines' in the Royal School of Bridges and Roads (L'Ecole Royale des Ponts et Chaussees), Paris, and some of his students were to emerge as eminent engineers in future years. After Navier, the development of engineering design was fostered by very many pioneers like Carl Culmann, Otto Mohr, Friedrich Engesser, to name only a few of the giants.

   This complete break from the past attitudes, and the zeal infused by the scientific thinking, was directly responsible for the scale of achievements of the engineer in the nineteenth century which appears to be massive indeed when compared to those of the past centuries all taken together. Even the great advances made by the Romans in bridge building had led to maximum span of a mere 35m. From the time of the Romans till the end of the eighteenth century advances made were irregular until the bridge engineer of the nineteenth century reached for the sky with his new scientific thinking. An environment of progress unlimited, where the engineer walks hand in hand with the researcher-scientist has been the keynote of the engineer's work since. Bridge building has thus become a highly intellectualised activity.

10. The use of concrete, an artificial conglomerate of gravel or broken stone, sand, and lime or cement, is of great antiquity. Concrete made with 'surkhi' has been used in India traditionally. And the glorious history of Roman concrete, used to build such famous structures as the Pantheon, is more well-known. But the art almost went into oblivion until its grand revival in the recent times marked by the entry of portland cement into the scene.

11. With the progress of portland cement concrete, it came to be used in bridges of substitute stone. Concrete was first used for a bridge of 13.5m span across a canal in France in 1840.

12. However, concrete never emerged as the 'many splendoured thing' until the art of reinforcing, fully developed by the turn of this century, made it so. This bold engineering concept of 'reinforcing the weak tensile zone of concrete by stronger iron inserts' was really established in the mid-nineteenth century itself. Of course, like many inventions in civil engineering, the story can always be traced to hoary past. The use of iron reinforced masonry may date back to first century B.C. by the Romans or even earlier in the Indian subcontinent. But we have to come back to more recent times which heralded its general structural applications. The Frenchman Lambot's famous rowboat 3.30m long 'made by plastering Roquefort cement on a skeleton network of iron and wire' was followed by his English patent in 1855. Coigent, another Frenchman, took out a patent at about the same time on the structural use of reinforced concrete. In 1855, Wilkinson secured in England a patent for a concrete arch floor, reinforced with tie bars, which had been discussed by Fairbirn in his book (1864). Moiner's first patent was taken three years later. Many others took about this time patents on reinforced concrete in one form or other, in various lands. Thus in the nineteenth century, reinforced concrete was still in the empirical period of patents and specialists.

13. The early twentieth century saw a dramatic change in attitudes to reinforced concrete. Improved concrete materials, mass production of good quality cement and better knowledge of proportioning, made attainable quality standards of concrete much higher than before. What threw the doors of progress wide open was the development of a design theory to interpret adequately the 'mechanics of the structural behaviour of reinforced concrete'. The search for a basis of rational concrete 'structure' design began in the last part of the nineteenth century itself. Thaddeus Hyatt is credited to be the first to establish the basis of analysis of stress in reinforced concrete by stating 'the principle of bond' and that 'the reinforcement must be able to resist sufficient tensile stresses to balance the compressive stresses in concrete'. Hyatt was an American lawyer by education but inventor by nature and was later to take patents on deformed bars. He published his 28-page book on 'the

use of portland cement concrete combined with iron' in 1877 which may have antedated the design principles to emerge by good two decades. Koennen, a government architect in Berlin, was commissioned to deduce the methods of computation of reinforced concrete sections and he published his design rules in 1886. Coignet's ideas on elastic design were printed two years later. Inelastic theories of design, which were to be rejected rather irrationally later, also appeared in 1898.

14. So in the early years of the 20th century, the stage was set for reinforced concrete to play the role destined for it. Professional societies flourished and research results were eagerly devoured for translation into applications. The familiar infrastructure of a national and generalised design and construction practice was born.
15. The days of the patents and specialists were ended. Specifications of reinforced concrete materials and design rules were published by professional societies in all advanced countries. The Prussian regulations of 1907 were reported to be most complete set of design rules at that time. In 1909, the joint code of ACI, ASCE, and other professional societies of America, interested in reinforced concrete, made its grand entry. The French Commission on reinforced concrete had formulated its design rules in 1906.
16. Reinforced concrete won the hearts of bridge builders so readily because of economy. So a 'comparison of cost with iron/steel or masonry' was invariably reported while talking about a reinforced concrete bridge in the early years of this century. Here is an example quoted from the 1904 proceedings of Institution of Civil Engineers (extract of a paper from La Genie Civil, Paris) about a bridge just constructed, spanning the river Aisne at Soissons made throughout in reinforced concrete' costing £ 7,700. "For a similar bridge in masonry, £ 1,875 would have to be added (to the cost figure), while if the bridge were in steel, an extra cost of £ 1,460 would be entailed." These were very telling figures to guide one's options. But direct saving in construction cost was not all that mattered. The outstanding virtue of reinforced concrete that made the difference was 'its durability and minimal need for maintenance care'. Doubts about corrosion of reinforcement embedded in the cracked tensile zone and exposed to the elements could have thrown cold water on the bridge engineers' enthusiasm for reinforced concrete and stunted its progress. Fortunately they were too pragmatic to be so inhibited.
17. Post-war years saw the fast emergence of reinforced concrete in its full glory. The massive rebuilding of bridges and buildings in war ravaged Europe brought new pressures on the pace of development of reinforced concrete. Higher strengths with higher quality of materials were the obvious avenues open. The history of a century of development of reinforced concrete materials and methods of making good and better concrete is itself a fascinating 'subject'. The fundamentals of making good concrete were established firmly by the twenties. Abrams, giving his well-known correlation of water-cement ratio with concrete strength, wrote in 1919; 'use the smallest quantity of mixing water that will produce a plastic or workable concrete'. With careful control of water and proportioning, attention was focused on 'workability, placing, and compaction'. Freyssinet proposed 'compaction by vibration in 1917. Alongside came the improvement in composition and fineness of grinding of cement, and the strength of concrete increased.
18. For the bridge builder, a companion development of reinforcing steel to higher strengths, specially in post-war years in Europe, made a big impact. Mild steel bars became the universal form of reinforcement since the 1900s. In Europe, they used it plain; Americans preferred it deformed. The designers had chained themselves universally to a low permissible stress of 125 MPa and the bogey of cracking and corrosion made higher strength reinforcement a taboo. The visions of economy, possible with high strength reinforcement, made Europeans conquer these irrational apprehensions, egged on by the pressure of post-war years.
19. The big break in the art of reinforcing the concrete came with the introduction of 'prestressing'. The idea of prestressing, conceived and put to practice in the 20th century in the single happening of most significance in the entire history of the art and technology of construction. Man must have practised prestressing for centuries. In some distant past, an unknown genius might have used iron rings in wooden barrels or iron bands on wooden cart's wheels, exploiting the concept of 'precompression'. The man who first gave the form and content to this concept, the originator of prestressed concrete, was none other than Eugene Freyssinet, a great builder of our times.
20. A lesser man would have been content with the fame and fortune he had in 1930. But Freyssinet was a man with a mission in his heart. His life carries an obvious message. His bitter struggles for seven long years for the cause of prestressed concrete made history. If history has to have its heroes, here was one for the history of bridges.
21. Freyssinet took patents in 1928. He was out to sell the history's most exciting building material, but with no buyers around. He had reportedly told his friend Le Corbusier: "I reached my goal. Now I am looking

around to see what I can use this discovery for. And in my opinion modern society needs housing parks and highways." Le Corbusier was so touched by the sentiments of his friend as to conclude: "into that one short sentence he has crammed a vast wealth of poetry, lyricism, of solidarity, of concern for mankind and the hearts of men".

22. The big boom in prestressed concrete came only after the war. Although France and Belgium led initially, the development and use of prestressed concrete flourished in all countries in Europe and soon spread to all parts of the world. Prestressed concrete came to be used most in bridge construction. Of about the 500 bridges built in Germany during 1949-53, seventy per cent were in prestressed concrete. Prestressed concrete was introduced in USA in 1949 by Magnel's Walnut Bridge and at about the same time in India by the Coleroon bridge, using Freyssinet system. Bridge builders revelled in prestressed concrete as it helped them to realise larger and yet larger spans in concrete. As more liked to use steel where it was possible to use concrete, prestressed concrete had to face demands of increasingly larger spans, with improved techniques of design, construction and materials.
23. In 1960 beam bridges had reached spans up to 160m and Morandi's bridge across the Lake of Maracaibo was under construction with spans of 235m by the help of stay cables. In 1970, the longest span of beams reached 230m in Japan, and for cable-stayed concrete bridges, designs had been made with about 300m spans.
24. The achievement in building long-span bridges in prestressed concrete are too close to out times to afford a historical perspective. The development of new landmarks in span, form, and construction technology, is growing at a dizzy pace. The panorama is so vast as to prohibit any attempt to survey all its features in this limited space.
25. Hamana bridge in Japan also claims attention as one of the best among a growing family of long-span prestressed concrete bridges erected by cantilever construction method. The Hamana bridge has a 240m main span with a central hinge. The originator of this type of construction, double cantilever box beam superstructures built without the use of falsework, which has revolutionised the building of prestressed concrete bridges, is Ulrish Finsterwalder—one of the greatest bridge builders of our times. A later modification of this method is the cantilever construction with precast segments introduced in the 1963 construction of Choisy-le-Roi bridge in Paris by Enterprises Compenon Bernard. Since then the precast segmental construction in cantilever has gained worldwide acceptance, mainly for the fast erection. In recent European projects, a fantastic rate of erection of urban viaducts of two or three spans of 35m to 45m in a single week has been reported. The span range of economical application of box beam bridges erected by cantilever construction is believed to lie normally between 50 to 200m. Hamana bridge seems to extend this limit by a good 40m. Urado Bay bridge in Japan is a 270m span cantilever type bridge. Use of lightweight aggregates in future may increase the span range of these elegant bridge types further.
26. In brief, the present development of prestressed concrete proved to the hilt that Freyssinet's seven years of ordeal did not go in vain. Continuing efforts to expand its scope of application has led to growingly imaginative forms of construction. These structures carry a central message that the man-made environment of structures in these last two decades of the century can be built not only economically, but also with elegance and dignity. Amongst these prestressed concrete structures, bridges occupy pride of place. Of course, prestressed concrete has been used and developed in all kinds of structures. Yet the idea of prestressing arose out of bridges and in bridges do appear the most impressive forms of engineering applications of prestressed concrete.
27. ***It is an undeniable professional responsibility on the part of a bridge engineer to have the cultural depth of knowing the history of his profession.***

48.3 RATIONAL APPROACH TO STRUCTURAL DESIGN

1. The basic principles underlying the philosophy of structural design are that (i) the structure should have a predetermined useful life depending on whether it is to be a monumental structure, permanent structure or just a temporary structure, (ii) the load, causing failure of a structure, should be sufficiently greater than the working loads, so that the probability of the failure of a structure during its life-time is less than a limit specified and the required safety is in-built into it, (iii) the deformations, vibrations and deflections of the structure under the applied working loads are not large enough to impair the safety or serviceability of the structure during its life time, and (iv) economic considerations with regard to design, construction and maintenance are duly recognized. In other words, the aim is to design a structure with maximum safety at a minimum cost. In so doing, methodologies like the elastic analysis followed by the more recent plastic or limit state analyses are in vogue currently, the 'limit state approach' being the latest trend.
2. The traditional method of design was based on

the concept of 'allowable stresses' and associated conventionally with theory of elasticity. This required a comparison between maximum stress under the working loads and the fracture stress of the material (or the stress level at which the material failed) to respond elastically. The factor of safety determining the allowable stresses, catered for a margin of safety. The factor of safety, therefore, revealed a subjective striving on the part of the designer to have a measure of safety, fully accounting for limitations of knowledge and arbitrariness of assumptions involved. Subjective ignorance regarding the resistance of materials at the limiting state of failure was thus added to the objective uncertainty associated with imperfection of human observation.

The safety factor being fixed rather arbitrarily based on subjective arguments, the most refined design by elastic theory and allowable stress method could have doubtful merits. Its very validity remained to be ascertained in absence of identification of objective conditions obtaining at failure. Perfection of design method demanded removal of the element of subjective ignorance. The expedient assumption of linear elasticity at all load levels was not true. 'Linearity of the stress-strain diagram of a material is a desirable property, only so far as it creates the theoretical conditions for the application of the relatively simple methods of classical elastic theory. It is the very slight deviation from elasticity, being the expression of the ability of the material to relieve by limited yielding and localised excessive stresses, that creates the practical conditions for the application of the elastic theory in design of engineering structures. Recognition of inelastic behaviour of materials under certain conditions led to the use of theory of design for plasticity to be adopted in structural design.

3. The plastic theory is based on the failure concept and recognition of the formation of requisite number of plastic hinges necessary for making a structure geometrically and compatibly deformable. The moment of resistance at the plastic hinge is assumed to be constant and independent of angular changes. Further, the plastic moment of resistance is proportional to the yield stress of the material; the latter, for purposes of design, being assumed to be constant. However, researchers have found that for complete sections of beams, the yield stress varies, thus making the assessment of moments at plastic hinges uncertain. The load factors built in the plastic design are more to account for the unforeseen contingencies such as weaknesses and deterioration of material, defects of fabrication and construction, errors in design and detailing, rather than only load variations. The plastic behaviour of a beam is simple but that of a beam-column complicated and uncertain, the latter being affected by angular rotations of the ends of the columns. Aspect like lateral instability, a complex phenomenon in the elastic range itself, becomes all the more complicated in the plastic stage, making its assessment possible only by approximations. The applicability of the plastic theory has been practised mostly for continuously acting loads and there are deficiencies in the approach when intermittent live loads also act. Thus, even the plastic approach suffers from uncertainty with regard to material properties, residual stress and yield stress, lack of adequate knowledge in respect of plastic behaviour of beam-column and empiricism and inadequacy of provisions relating to lateral instability and action of live loads.
4. With the design method correlated to failure state, it is necessary to introduce basic modifications to the method of application of factor of safety. The conventional definition of a factor of safety as the ratio between the two stress levels now losses its meaning, as the basic assumption of stress ratio being identical to load ratio is no longer held valid. Safety can, therefore, be rationally defined in terms of load ratios only. So factor of safety is introduced by way of comparison between increased value of load or load effects with relevant resistance of structure. The revised method of design by this rational way of introducing the factor of safety is commonly known as the limit state design method. Of course, every rational design is necessarily a limit design. It is the specification of the 'limiting conditions' that may differ for different approaches to design. The limit state design method compels the designer to make an objective assessment of the mechanics of resistance of structure relevant to the limit state investigated. The design method is no longer chained to elastic theory. The element of subjective ignorance associated with allowable stress design is removed with the perfections in design analysis, obtained by continual theoretical and experimental investigation. The redefined factor of safety reflects the measure of inescapable uncertainty due to inherent imperfections of human observations. For a rationalised application of the method, a unique factor of safety is often substituted by 'different and partial safety factors' valid for different limiting states of ultimate failure and serviceability limit states of deformation, local damage, etc. Apart from load increment factors, reduction factors to cater for uncertainties in quality of materials are also specified separately.
5. The recent trends in development of rational concepts of structural safety have been marked by the introduction of a new philosophy of design. The structural engineer today generally uses a deterministic analysis of safety

problems, presuming a complete knowledge of load and structural response, both considered as a function of non-random parameters. So he would strive for absolute safety which is no more than convenient fiction. The design approach, to be rational, has to be a probabilistic one, where basic parameters are considered as random. In the words of Freudenthal, one of the pioneers of probabilistic design philosophy, the laws of structural design must be considered as a combination of functional and statistical relationships, functional so far as the laws of theory of structures are concerned and statistical to the extent that real physical properties appear as parameters of functional relations'. The probabilistic approach to safety, the first attempts towards which could be traced back to the late thirties, has been discussed by many devotees and sceptics alike for a long time. The principles of the probabilistic design philosophy have today found general acceptance. The problem now being investigated and sought to be solved relates to the direct applicability of principles to practical design and computation. A few major difficulties faced in the application of the probabilistic approach to practical design rules are: (1) choice of appropriate probability model, (2) introduction of subjective elements into probabilistic structure (no amount of observable data, it is apprehended, may be sufficient to make entirely objective decisions in a civil engineering design), and (3) interpretation of probabilistic information in a form which leads to rational decision.

6. In view of the above, the need for performance studies of structures and a continual monitoring of data to improve upon the various parameters involved in design cannot be over-emphasised..
7. Structural failures are often on account of bad workmanship, human error, incorrect approach, etc., and to cover such contingencies, often the design loads are arbitrarily increased, resulting in the present design loads being hypothetical. The factors attributing to failure cannot be clubbed under one common factor like safety or increase in loads or reduction in structural strength.

48.4 NEW CODES OF PRACTICE (SPLIT 'LOAD-FACTORS'!)

1. The good old working stress design philosophy, sometimes supplemented by the so-called ultimate strength or load factor design, is abandoned. The new format adopted is, of course, limit state design combined with a semi-probabilistic approach of safety. Looking back, the death of the conventional code format has been a historical process spreading over quite a few decades, with a slow but steady progress from the East to the West before the twain could meet. It was the Russians who set the ball rolling. As far back as 1938, they chose to break away from working stress design though it continued to enjoy wide patronage in the rest of the world. The limit state design, method was developed in U.S.S.R. in the period 1947–49 and approved in 1955. The Russian Standard on reinforced concrete NITU 123–55 was re-elaborated in 1962 through SNIP II-B. 1.62 and the design method was adopted by virtue of the recommendations of COMECON in East Europe also, in 1963. In 1964, CEB brought out its first edition of a model code adopting semi-probabilistic limit state design method followed by the joint FIP/CEB second edition in 1970. The design principles or the code format adopted by CEB and FIP are now commonly advocated by other international organisations such as ISO (International Standards Organisation), CIB (Conscil International du Baitment) and CECM (Convention Europeene de la Construction Metallique). A new era of international agreement and unification in "structure" design philosophy has dawned. Many national codes have, as a result, changed directly or indirectly reflecting the influence of CEB Code format. The British Unified Code appearing in late 1972 has a new limit state design format, following CEB model code closely. So, the replacement of the traditional code format by the new one is almost the inevitable natural process like shedding of old clothes and taking on new ones.
2. The single distinguishing feature of the new design philosophy or code format has been claimed to be the 'rationalisation of the concepts of safety'. The new philosophy is meant to open the eyes of the engineers and designers to the non-deterministic character of safety. Structural safety is not a physical measurable property. It is a demand of the society for guarding against any danger to life and property, and, therefore, depends on the social attitudes and environment. One cannot help recalling that oft-quoted passage from the Code of Hammurabi, King of Babylon (2500 B.C.)—"If a builder builds a house for a man and the house which he has built collapses and causes the death of the owner of the house, the builder shall be put to death". The Babylonian builder had of course aimed for absolute safety. He could hardly afford to think or talk of any probability or risk of failure. Has the pre-historic tradition changed? The talk of finite probabilities of failure remains as much a taboo among most of the lay public and even of the engineering profession, though heads may not literally roll today! This irrational attitude has also shaped the 'structure' design codes. Traditional code format identifies loads, strengths of materials, structural response and dimensions by nominal deterministic values. It effectively hides implicit uncertainties involved

and any inkling about the probability distribution of loads, structural characteristics, consequences of failures, or errors in formulae used in analysis and design, etc. This dogmatic approach, implying the non-existent, absolute, risk-free decision is considered highly unethical and unhealthy in the scientific environment of today. It has to be explicitly recognised that associated with every human socio-economic activity is 'a risk which depends on various losses possible and the probability with which they are incurred'. The object of structural design is to reduce the risk of failure to an 'optimum' level. As money for building a structure that the society can afford to pay is 'not unlimited', the safety level decision can only be obtained in terms of 'optimisation'. The main purpose of a code should be maximisation of utility from the standpoint of society. So, the question 'how safe is safe enough' involves interdisciplinary socio-economic decision-making.

3. The last decade or two has seen an explosion of rational thinking on structural safety. IABSE, in close cooperation with other international agencies, has played a laudable role in projecting these developments through various publications and symposia on the subject. The London Symposium (1969) on "Concepts of Safety of Structures and Methods of Design" will, perhaps, be readily recalled. In the ASCE-IABSE Conference on Tall Buildings (Lehigh University, 1972) also, the state-of-art reports of various technical committees like "Structural Safety and probabilistic Methods" (TC 10), "Load Factor (Limit States) Design" (TC 19). "Commentary on Structural Standards" (TC 20) and "Limit State Design" (TC 26), provided a most interesting exposition of the subject viewed from various angles of principles and practice. Literature on Structural Safety linked to probabilistic and optimisation approaches has been steadily piling up. The results of theoretical research now promise development of a safety theory 'based on probability and economic considerations...to replace previous obscure fancies'. As a result of this continuing and intensive scientific discussion, there is general agreement that time has come for writing code specifications that handle the non-deterministic character of safety and uncertainty involved in an explicit manner.

   Divergent opinions have, however, been expressed as to the form and complexity and scope of such code formats. The ideal form of specifying safety, it has been pointed out, is through 'reliability optimisation'. But even the impatient do not find it rational to implement it immediately!

4. Dr. Blakey, presiding over the discussions on limit state design philosophy (Proceedings: Australian and New Zealand Conference on Tall Buildings, Sydney, 1973) narrated the following interesting anecdote in his opening remarks: "I am remembering one of my colleagues who was enthused with the limit state concept to the extent that he circularised many interested parties with the intention of gaining their support for the inclusion of these techniques in building codes. He received a variety of replies and he summed them up as saying—"*This is a great idea. P.S. what does it mean.*" The little postscript, though looking rather naive, asks a very relevant question which deserves to be answered fully by the code makers. Paraphrased a bit, it raises the vital issue whether *the split safety factors* of the new code format, the definition of limit states, etc., represent any significant departure from or improvements upon the deterministic conventional load factors and the design approach. It is not difficult to sell the 'great idea' of the new principle as the first step on the road to full freedom from deterministic approach. But has the first step been taken at all in reality? If the 'characteristic' loads are nothing but the old 'nominal' values, and safety factors are just intuitively fished out, the end result of the design process can hardly be expected to be improved merely *by the magic jargons of limit states, characteristic values, etc.* When new code format goes little beyond split-safety-factors, progress towards improvement in design process, or the end result, depends entirely on how rationally they are calibrated. So, if the calibration is not done on the basis of scientific theories, and experimental data not statistically interpreted, the difference between the traditional and the new method is completely drowned and only the high sounding jargon sticks out. Dr. Neville, explaining the background of fixing limit state load factors of the British Unified Code relating to the primary combination of dead load and live load concedes the following in a frank discussion: "The values of these partial (safety) factors were chosen so as to produce designs which are essentially the same as the existing designs. I remember in the Committee we asked the Cement and Concrete Association in England to run a number of designs with various figures and the argument was that if one designs this sort of a room for this sort of loading, the new British Code method should produce the same size of slabs, the same size of beams, the same size of columns because we know that these things work and nobody was prepared to stick his neck out and sign his name to a document which will allow us to design something which is significantly smaller in size. I might just as well let the cat out of the bag and say that the factors 1.4 and 1.6 are there by the grace of God. There was a period of 5 months when the values were 1.5 for dead and 1.5 for live. The committee agreed to that in response to the argument from the designers who said it

is far too much work for us to use 1.4 on dead and 1.6 on live. We agreed to it until about five meetings later, when two most outspoken consultants were away and the others managed to regain their position.'' (Neville: Discussion Session 4, Limit State Philosophy Loads, *Proceedings Australian New Zealand Conference on Tall Buildings*, 1973). The semi-probabilistic basis of new split-safety factors thus comes down in a heap, like a house of cards! Some food for thought — this.

48.5 DESIGN EDUCATION

1. What makes 'structure' design-education a special concern is its creative design content. An education for creative design may be the professed aim of any course in structure design, but achieving it by a systemised process to yield the best results is not that easy. Creativity is something which cannot perhaps be taught. It is an experience one must live through to learn. So it is futile to imagine that textbooks or classroom lectures, however scholarly they may be, can make a designer. ''Can design be taught?'' The question has been examined rather thoroughly in a paper bearing the same title by Harris (Procs. Instn. Civ. Engrs., Part 1, 1980, Aug.). Practising engineers are sometimes prone to look down upon the entire proposition as fictive. According to these pessimists, the educator and the practitioner live in two separate worlds, with communication almost shut off. They simply do not talk the same language at all. So the educator can teach design theory alone and nothing more. ***Any attempt to teach the students some design by learned professors who may themselves lack their moorings in 'actual competitive' design practice, could do more harm than good.*** Unlearning the wrong lessons may take longer than learning first hand closer to the scene of action.
2. Such pessimism about design-education seems to be an over-reaction. But with a little introspection, it is quite feasible to modify it for the right kind of design-education. Complete absence of design-education course content, and exposure to theory alone, could injure the young minds. Real-life design practice lies far from the crowd of design theories. Lack of appreciation of this basic fact of life can ail the system.

   In examining the question what can be taught and how much can be taught, Harris defines the elements of a design process, and weighs off each separately. His conclusions, quoted below, are interesting:

   ''If the various operations be examined as to which 'can be taught' and which 'can be taught about' (those in which the student can be exercised effectively and those about which he can be warned and informed, ready for closer acquaintance in practice), one would have a list like this:

   (a) Appreciation: Can be exercised, even in the absence of a real client.
   (b) Conception: Exercise is possible and desirable.
   (c) Appraisal: Self-criticism can be developed.
   (d) Decision: No replacement for reality.
   (e) Checking and Elaboration: Present education concentrates on the former; as for the latter, to sense its importance is about all that can be contrived.''
3. A systematic attempt towards developing a design-education model, which concentrates on every element of the design process, has been described by Cowan in a recent paper (Procs. Instn. of Civ. Engrs., Part 1, 1981, Nov.). On the basis of the findings on the application of the model, Cowan agrees with Harris fully that design 'can be and should be taught'. His recommendations are quoted below:

   ''A programme of design-education, which systematically explores each element in an expressed statement of the design process, is both feasible and worthwhile and should be included as a part of the overall curriculum in engineering education.''
4. The problem 'how' design should be taught has engaged attention of other authors. For example, William et al.* has identified five types of basic approach to design education. These are:

   (1) ***Classical classroom approach***: A concept is explained in a lecture, and a single answer problem that involves the concept is assigned as homework.
   (2) ***Socratic approach***: By his questions, the instructor leads the students through the problem-solving situation.
   (3) ***Guided design approach***: A small group of students is given an open-ended problem. They are guided through the solution of the problem by printed instructions that ask them to perform a step in decision-making process and then provide feedback on what would have been accomplished. The student must learn new material to solve the problem.
   (4) ***Case studies approach***: The instructor chooses an open-ended problem that requires the student to work at or a little above his level of development. The student may be required to learn some new material before he can solve the problem. His solution is compared with actual solution.
   (5) ***Professional approach or authentic involvement***: Here the problem is directly encountered. There are no clues to the method of solution, none of the tools required in the solution are identified and there is no certainty that an accepted solution exists''.

* (Present and future directions in design-education, Engineering issues, ASCE, January 1975)

5. Obviously, the efficacy of the approaches increases as we go down the list. The preferences of a professional approach over the classical classroom or Socratic approach would hardly need a comment. However, the problem that stares us in face is how to translate these preferences into action within the framework of the engineering education system that prevails.
6. The impact of computers in structure design is another aspect of design-education which may be of critical concern to the profession. Emphasis on mathematical computation has to be reviewed critically in this context. While the analyst-designer must have adequate knowledge of the mathematics of 'structure' analysis, ***he need not be a deft expert in computing moments, shears, and stresses in structure elements. The job may be taken over and done better by computer people. It would be more important than ever for the student to understand almost instinctively how the structure really works, how to proportioning of its various elements affects its load response, and what detailing it takes to make the structure practical and tick. The good designer can and should be taught to 'think' more than 'compute'.***
7. The young graduate may be well versed in all sophisticated analytical techniques, but his pride and self-confidence may get a rude jolt when he is unable to read a plan, let alone really design a bridge or a structure in all its 'practical detail'.
8. ***The fault lies with the system which forgot to teach him the fundamental fact that design is mostly 'detailing' which is made on drawing boards, not on the calculation sheets, nor in the computer. The only way out of this impasse is to open the doors of universities wide enough to bring more of the design-office into the classroom.*** The mechanics of a stable, effective, planned, and rightly motivated 'educator-practitioner' interaction must be evolved with a national priority.

48.6 CURE AGAINST CRACKING OF CONCRETE—NOT BY CALCULATIONS ALONE!

1. Cracking and crackwidth-prediction-and-control have been much in the limelight in recent years. The problem has engaged growing attention of researchers, code-makers, designers and constructors, to such an extent that it almost selects itself as one of the most discussed topics of the last decade, among people dealing with concrete.
2. All this talk about crackwidth-computation may lead one to believe that the panacea of all ills lies in accurate prediction of flexural crackwidth or shear crackwidth. Nothing could be farther from truth. Computing crackwidth is an important design task but a designer with overreliance on crackwidth-computation for crack control may soon find it disappointing.
3. Cracking of concrete occurs due to tensile stresses. But the occurrence of such tensile stresses may be traced to various sources other than flexural tension. Had flexural stresses been the one and only reason for tensile stresses in concrete, 'full' prestressing would have easily solved the problem!
4. There are other kinds of tensile stresses. It is, therefore, to be expected that the concrete structure remains unimpressed by the most sophisticated prediction of flexural crackwidth and may crack due to these other tensile stresses.
5. One of the commonest causes of cracking in structures is the lack of reinforcing at locations where significant tensile stresses occur under predictable or unpredictable combinations of effects. Another kind comprises avoidable 'deficiencies in construction quality of concrete resulting in so-called plastic cracking'.
6. In concrete, seeds of most cracking distress are sown before hardening and application of loading. Deficiencies and consequent damages that have been done to concrete at birth or at a very young age cannot be undone after it matures and is put into service. If visible cracking has not appearing already in the concrete when poorly made, it remains very vulnerable due to unhealed scars. Reinforcement does not help, and computations go haywire!
7. To guard against such cracking, one must understand one's concrete better. It is a multiphase, inherently heterogeneous, complex material. In engineering use, we interpret the behaviour of concrete at phenomenon level, which can be deemed as little better than prescientific. The status of knowledge on microstructure of concrete and associated theories of crack propagation and fracture of concrete have grown fast in recent years. It is now clear that microcracks, lying hidden from view, hold the key to the cracking behaviour of concrete like all other properties pertaining to strength and deformation.
8. Microcracks are minute flaws or discontinuities inherent to heterogeneous nature of concrete. They appear in concrete, not yet loaded, or lightly loaded while lying in the forms, waiting to gain its mechanical strength. The cracks are too narrow to falsify the use of elastic theory or the assumption of a homogeneous isotropic material by the structural designer. (Some microcracks may also get healed partly or completely with the crystals precipitating during the continued hydration of cement. But this autogeneous healing does not solve all problems.) They nevertheless create, through their

geometry, zones of potential fracture. So it is but expected that any excessive micro-cracking that occurs before hardening of concrete may prove to be the major source of cracking distress in the service life of the structure.

9. Restraint to free deformation while concrete hardens (plastic shrinkage, plastic settlement, etc.) is the primary cause of random non-structural cracking. Cracking can occur in otherwise sound concrete when still fresh due to the tendency of concrete to contract, caused by loss of moisture or change in temperature. Other reasons are differential settlement over obstructions, such as large pieces of aggregates or steel, separation of aggregates and matrix due to settlement of latter with local bleeding under coarse aggregate. Segregation and faulty compaction result in voids and honeycombs.
10. Even when deficient quality of construction induces excessive microcracking which soon develops into visible cracks, these may lie undetected for long despite their early appearance. These are usually obscured at early age of concrete by water bleeding initially to the surface of concrete. All looks well until the curing is over or the concrete is put into service. Unfavourable ambient conditions soon magnify the damages for all to see and ponder over. But post-mortem analysis often ignores the original sin.
11. The tensile strength of concrete develops very slowly and the concrete obviously remains very vulnerable to cracking at a young age. Deficiencies in composition of concrete, placing and curing, etc., and more significantly the adverse environmental conditions (wind, heat and cold) are agents which magnify the distress. Even if visible cracking is not very evident, due to severe microcracking, the tensile strength of the hardened concrete may be much smaller than anticipated and severe cracking distress under subsequent load-induced tensile stresses can occur.
12. As mentioned earlier, cracking in concrete yet to harden, can be attributed basically to (1) temperature change, and (2) drying shrinkage. The mechanism of cracking due to temperature change is simple enough to appreciate. Cement liberates heat during hydration. While temperature rises, the plastic concrete hardens as it cools. A temperature differential can thus be set up between the outer and inner portions of the concrete mass during this temperature change. This can be aggravated by environmental conditions. Restraint to free deformation results in tensile stress (not necessarily flexural).
13. Similarly, drying shrinkage generates tensile stresses due to restraint of deformation. Drying is largest at the exposed surface which tends to shrink more than the interior, yielding a restraint to the deformation of interior layers.
14. Conscious efforts to reduce these restraint stresses and the consequent risks of cracking in plastic concrete may involve some elementary care in making good concrete. For reducing the cracking caused by temperature change, the temperature in the interior of concrete must be kept down and there are many ways of doing it. The most obvious one is to strike at the grass-roots. Cement is the source of heat of hydration. So the less the cement, less is the temperature change. Of course, cement is also the primary source of strength. So an optimum quantity, just enough to gain the desired strength, should be used. Non-optimal use of cement may make the concrete costlier or even stronger than desired but never better! Merely richer mix for the same target strength can in the limit be poorer in quality. It has been reported from a survey of a number of mass concrete structures in USA that the only concretes free from cracking were the 'leanest'. Perhaps temperature differences alone cannot explain the observed better cracking resistance of leaner mixes. Lower modulus of elasticity and lower drying shrinkage characteristics may also contribute equally.
15. The above elementary rule of making better concrete with less cement may be too well-known to be restated. Yet it remains a fact of life that misconceived reliance on liberal addition to cement content to compensate for lack of controls on construction quality continues. Accent should be on durability and bond more than on extra strength alone. This calls for coarser gradation, cleaner dust-free aggregates of good quality, tight water-cement ratio, casting in low-wind low-heat low-cold weather, with adequate curing. It is worse when people add extra cement hoping it will 'take care' of strength when the worker quietly adds more water arbitrarily to permit him an easier workability locally. This in fact leads later to all sorts of non-load-induced random cracks that subsequently accentuate, which ultimately destroys the integrity of the structure.
16. Technical Report No: 22 of the Concrete Society indicates various non-load induced cracks that can plague the concrete in a structure. It shows their most common locations, their primary and secondary causes, remedies and approximate times of appearance, and is therefore a good general guide. More details may be found in the report itself.
17. A designer doing voluminous computations of crack-width should perhaps first ensure that the mix of the concrete going into the structure is properly designed and followed because excess of cement alone can lead to loss of quality in respect of cracking behaviour.
18. Caution against using too finely ground cement for

not so high grades of concrete is also called for. Modern high-strength cements are ground much finer than earlier. They liberate heat of hydration much faster and increase the risk of setting up a thermal gradient in a thick member. So, where cracking behaviour is a critical consideration, use of a very finely ground high-strength cement may not be favoured.

19. For preventing excessive drying shrinkage of concrete, it is important to protect the concrete surface by continued curing. Members with large thickness, or those cast in adverse ambient condition, will require greater care. Premature exposure by removal of forms also can aggravate the risk. In control of drying shrinkage, the effect of water content and mix proportions is most important. Choosing the lowest water-cement ratio practicable is necessary from the viewpoint of cracking resistance also.
20. 'Settlement cracking' also appears in concrete before hardening. This type of cracking can be seen in poorly laid concrete on upper surface of slab following approximately the top reinforcing pattern. The obstruction to the downward movement of solid particles by the reinforcement grid is the obvious cause. The incompatibility of the concrete mix, placing, and compacting techniques with the reinforcement arrangement contribute to the distress. This can be controlled by slightly revibrating the concrete before its initial setting. Plastic shrinkage cracks (which start at top and descend downwards, sometimes following the obstruction-creating path of the reinforcement grid and sometimes appearing normal to wind direction) can be controlled by robust timely trowelling. They are caused when the rate of surface evaporation is faster than the rate of internal bleeding (rising upward of water in fresh concrete).
21. The assumption of uniform stress distribution over the total 'effective' compression flange width only brushes aside a considerable deviation of stresses at the flange-edges. But such inaccuracies do not spell any trouble nor do they signify any gross error. Gross errors occur when the designer fails to subjectively account for the possible significant impact of the inherent inaccuracies of his analysis. *In spite of the large sophistication in the tools of mathematical analysis of a structure, a 'structure' design can hardly live by computation alone. Unless design decisions are supplemented by subjective engineering judgement of the actual structural behaviour, the finest efforts may end in disaster. This lesson has been learnt the hard way many times in this century. All the brilliant mathematics of the first Tacoma Narrows bridge design did not save it from the collapse due to aerodynamic instability. The small geometric inaccuracies ignored in analysis brought about startling disasters of mighty steel bridges. Added to all this are the unavoidable differential temperature stresses even whose prediction is inaccurate.*
22. In some of the recent concrete bridges some unsightly cracking has appeared due to tensile stresses ignored in common design practice. The principal agent in these cases has been identified as temperature variation. Leonhardt has reported instances of severe unanticipated cracking of beam webs due to non-uniform temperature distribution and consequent stresses. Priestley has talked about thermal-stress-induced cracking of prestressed concrete box girders of a major urban motorway viaduct in New Zealand. All this cracking was found to be strongly correlated to the ambient temperature and solar radiation. Ignoring these stresses, as we have been doing in earlier design practice, cannot but be termed as a gross error. *While these stresses may be pre-calculated, cracking of concrete due to them cannot be avoided unless sections are thin. Best bet is to provide closely spaced reinforcement bars of small diameter along the skin in crack-critical zones.* This will allow more cracks but of narrower width. Here again quality control exercised in making the concrete will greatly help in limiting the total cracking.

48.7 LIFE-CARE OF ROAD BRIDGES

1. We cannot afford to forget that there is a hidden price on every investment made today—maintenance and repair costs of a much longer tomorrow. The bill is usually delayed but inevitably becomes due and must be paid. The price for neglect of maintenance care can be staggering, and costly national investment may be imperilled.
2. The learned designer and the expert builder may turn back and retort that all this is too much ado about nothing. The designer today is better equipped and so is the builder. Sophisticated structural analysis and rational considerations of safety have resulted in more economic and safer structures. Materials and construction techniques have improved. Bridges are designed better, built better, to last their lifetime.
3. There can be no disputing these claims. But the fact remains that engineer's job hardly ends with the bridge being built and put into operation. It only starts a new phase of activities and this is no less challenging even if less glamourous. The benefits promised to the society by the planner and designer are now directly measured by the real life performance of the bridge.
4. A long true vigil of the engineer starts. He must care for the minutest details to ensure peak performance of

the facility for the longest period possible. Interest and involvement of the engineer in maintenance flow from the central philosophy of the profession as serving the use and convenience of man.

5. An associated technical activity of equal importance is the 'performance measurement' to serve as feedback to future planning, designing and construction activities. No human ever did anything perfect. Bridge builders have always a lot to learn from past mistakes, if correctly monitored. Continuing improvement in engineering judgement could result from such performance measurement and thus gain tangible benefits.
6. Maintenance measures for road bridges suffer as the inventory of bridges in service grows in years. Older bridges do not receive the attention they need until too late when irreparable damages may occur. Bridges constructed and put into operation get left to themselves. Even routine maintenance inspection and reporting go by default. 'Pockets' that do not drain, expansion joints that rattle, or rollers clogged in dirt that do not roll, remain mute witnesses to lack of even cursory care. Records of bridge details and their current condition may not always be available. Even systematic documentation of design, construction data measurements and completion plans may be missing. However, the overall picture is not as gloomy. Most of the bridge structures are generally in good enough shape. But dark patches like these can blemish the general scene.
7. Signs of complacency can creep in. In bridge maintenance practice what may prevail is an unplanned maintenance. A damage detected by design or accident calls for corrective operations. If the results are satisfactory, these operations end with a finality. There may be little feedback for future planned preventive care for either the affected structure or for similar vulnerable ones. Adequate maintenance-planning and organisation are the obvious needs.
8. Planned maintenance is the obvious option. This may be defined as 'work organised with forethought, control and records.' It comprises two essential elements:
   (a) Planned preventive maintenance
   (b) Planned corrective maintenance

   Both of these must be organised with 'forethought, control and records.' They must be mutually balanced and go hand in hand.
9. The main objectives of maintenance planning of bridges may be restated as:
   (a) To preserve the investment;
   (b) To provide adequate level of safety of user;
   (c) To provide adequate level of comfort and convenience of vehicular travel;
   (d) To ensure economy in expenditure and resources.
10. These general statements apply to overall highway maintenance as well. However, the special requirements and considerations of bridge maintenance are easily discernible. The technical priorities often become confused because of the intrinsic difference in user reaction. The travelling public may be quite vocal about the inconvenience caused by pavement potholes, but serious signs of distress of a bridge structure may escape notice and criticism.

    If priorities in maintenance schedule are motivated by public reaction, urgent bridge maintenance operations may get deferred due to constraints of money and manpower limitation. Such deferments may prove costlier and more catastrophic than imagined. Planned maintenance can correct these pitfalls.
11. Bridge maintenance has a special character and requires special skills. The surface defects may be cosmetic and easily restored. But the more important factors to which the objectives are principally oriented are prevention of the 'loss of property'. Hazards to human lives which cannot be measured in economic terms are to be guarded against. Maintenance operations are basically aimed to prevent premature loss of facility and furnish possible forewarning.
12. Unfortunately if often happens that attention to maintenance requirements of bridges is aroused by an accident. The example readily recalled is the Silver Bridge collapse in U.S.A. on December 15, 1967. The tragedy took the toll of 16 lives of men, women and children. In its wake, deep public concern and apprehension were created over maintenance responsibilities and performances. Committees deliberated; U.S. Congress held hearings; Federal and State legislations were enacted. AASHTO Bridge Committee in cooperation with Bureau of Public Roads undertook the development of a comprehensive Manual for Bridge Maintenance Inspections. The concluding remarks of the 'President's Task Force on Bridge Safety Committee 3—National Study to Assure Bridge Safety', quoted below, indicate the extent of feverish activities that followed:

    "Federal, state, and local government agencies having responsibility for bridge inspection and maintenance, are either presently intensifying existing programs or developing new improved programs... The concern for the safety of the bridge user has been greatly aroused. New inspection equipment is being purchased, sophisticated devices are being employed for inspection of bridge members and determination of scour at bridge piers, bridges are being reappraised

as to load carrying capacities, hundreds of consultant engineering contracts are being let to inspection services... This concern by the public is reflected in the legislation passed by the 90th Congress which makes bridge inspection on Federal-Aid highway systems 'mandatory' and further provides for the establishment of Inspection Standards and a training program for bridge inspectors''.

13. A bridge built is to fail as surely as a man born dies. It is designed and built for a finite life and finite risk of failure. An engineer, therefore, needs to be motivated only by the basic ethical objective of maximisation of utility. Incident-generated responses often vitiate rational decision-making and ethical optimisation. In some countries one has seen design seismic coefficients shoot up immediately following a strong tremor and perhaps change again with time! Maintenance is a long-term activity. It is important to avoid these arbitrary lows and highs, sudden spurts in awareness, followed by the spells of complacency in planning bridge maintenance.
14. Monitoring the quality of performance of the bridge and assessing whether the specified level is being maintained is the controlling function of maintenance management. The monitoring that may be decided upon must be a method of regular observation and testing. Routine maintenance inspection by fully qualified and trained bridge inspectors is an essential part of the monitoring system. But the scope of such inspection as well as supplemental procedures necessary must vary to suit special requirements. Observation and testing should be done with instrumentation as and where needed.
15. ''The approach of an engineer to a problem of maintenance should be identical to the approach of a doctor to his patient. It involves 'detection', 'diagnosis' and 'cure' with an overriding requirement for 'prevention'. In order to proceed, the engineer must have the knowledge of the various forms of deterioration which are known to occur (the symptoms), the cause or causes of various types of decay (the disease) and how to correct the condition (cure).'' 'Expertise' needed for maintenance is therefore too important to be ignored.
16. Last but not least is the 'performance measurement'. Of course, this is not an activity completely isolated from maintenance. It is, however, identified by the more specialised purpose of feedback and is, therefore, more selective in operation.
17. Organised scientific enquiries into performance characteristics of bridges in service are needed to find the right answers. For example, possible large deflections due to concrete creep, likely to stabilise after say 3 to 5 years of service, effects of restraint stresses due to temperature gradients, vibration, and the serviceability/distortion problems of large span thin-walled box bridge beams, etc. Monitoring 'performance data' by planned instrumentation could serve as an important aid to the solution of these problems.
18. Acquisition of 'loading history data' of bridges is another basic area of performance measurement. Studies on intensity, frequency of truck traffic loading and trends are associated with basic parameters of bridge design and linked to the optimisation of the entire transportation system cost.

48.8 CONCRETE STRUCTURE—DETERIORATION AND REHABILITATION

(... for details refer to the author's books referred to on p: 652

(A) Conceptual Understanding

Structure	Element	Causes of damage	Investigations	Types of damage	Rehabilitation requirements	Methods of rehabilitation
Super-structure	1. Asphalt layer 2. Concrete slab 3. Parapet 4. R.C. girders 5. P.S.C. girders 6. Box type structures 7. Articulations	See (B) ahead	See (C) ahead	See (D) ahead	After due inspection, decide between: * Replacement * Repair * Strengthening and * Protection	See (E) & (F) ahead
Substructure	1. Pier cap 2. Piers, abutmts 3. Pile caps, footings 4. Piles					
Others	1. Retaining walls 2. Approach slabs 3. Bearings 4. Expansion joints					

(B) Causes of Damage

1. *Quality of Constituents of Concrete Itself*:
   1. Cement
   2. Aggregates
   3. Water
   4. Additives
2. *Environmental Effects*:
   1. Corrosion/chloride attack/sulphate attack
   2. Alkali-aggregate reaction
   3. Marine conditions/sea environment
   4. Weathering.
      * Thermal variations
      * Sand abrasion
      * Waves
      * Rain/humidity
      * Wind
3. *Foundation Problems*:
   1. Settlements/Differential settlements
   2. Variation in subsoil water level
   3. Ground movements
4. *Construction Errors*:
   1. Poor quality concrete/construction practice.
   2. Form work/false work problems
   3. Inadequate compaction
   4. Incorrect detailing of reinforcement
   5. Inadequate and delayed curing
   6. Excessive construction loads
5. *Design Errors*:
   1. Design load underestimated
   2. Under-design/strength overestimated
   3. Omission of appropriate joints
      * Construction joints
      * Movement joints
   4. Unacceptable cold joints
   5. Unaccounted internal stresses:
      * Thermal
      * Shrinkage
      * Creep
      * Unforeseen load conditions
6. *Unexpected or Unprecedented Load Conditions*:
   1. Illegal overloading/speeding
   2. Unusual floods
   3. Unusual earthquakes

(C) Investigations

Type	Analysis
(A) *Routine Examinations*:	
1. Compressive strength	'Schmidt' Hammer REBOUND
2. Crack measurements	Crack microscope or equivalent
3. Crack monitoring	Mechanical gauge/tell-tales
4. Concrete cover	Magnetic cover meter
5. Carbonation	Phenolphthalein test
6. Corrosion detection	Colebrand pathfinder
7. Corrosion determination	Corrosion test
8. Moisture contents	Moisture detector
9. 'Joint' monitoring	Crack monitor
10. Chloride concentration	Drilling cores/powder analysis to plot the chloride penetration profile
11. 'Mix' composition	Valuation of components for analysis of internal faults.
(B) *Special Examination*:	
1. Concrete strain	Load testing, use of strain gauges
2. Structural deformation	Levelling instruments/survey
3. Soil pressure	Capsule load cells
4. Load measurements	Load cells
(C) *Special Equipments*:	
1. Electric Power supply	Stand-by generator, etc.
2. Core drilling	Rotary cutting tool
3. Pick and knock devices	Hammers (ordinary/electrical)
4. Temperature measurement	Thermometers/Thermocouples
5. Humidity measurement	Hygrometers
6. Tests on fresh concrete	Properties, according to standards
7. Load application rigs	Hydraulic jacks, frames, anchors, etc.
8. Laboratory testing	Various laboratory facilities

(D) Types of Damage

1. *Spalling*
2. *Swelling/bulging*
3. *Crumbling*
4. *Cracking and spalling due to corrosion of Reinforcement*
   * Corrosion of steel can be due to its exposure to moisture and atmospheric oxygen and corrosive chemicals and fumes.
   * Corrosion of steel can also be due to ingress or presence of chlorides in combination with moisture. This is an electro-chemical process where electric currents are set up between points in reinforcements located in weaker and stronger chloride solutions, leading to eating-up of reinforcement due to formation of iron oxide (rust).
   * Corrosion of steel can also be caused by Carbonation (increase in acidity in the moisture in concrete owing to reduction in its pH value because of formation of calcium carbonate from calcium hydroxide (released by hydration of cement) and the atmospheric CO_2)
   * Corrosion in each of the above cases leads to the 'formation of Iron-Oxide' (rust) which occupies higher volume and hence causes radial expansion of the bars and consequent bursting—pressure that causes cracking along the line of reinforcement. Soon rust-stains weep out of these cracks, rust increases, cracks widen, and ultimately the surrounding concrete spalls!
5. *Cracking due to sulphate attack*:
   * Ingress of sulphates from surrounding soil, water, or atmosphere, or sulphates that may be present in the aggregates themselves and/or in the mix-water or curing-water, chemically react with C_3 A in cement in the presence of moisture, forming the sulpho-aluminate crystals which occupy more volume and hence exert expansive pressures that cause cracking.
6. *Cracking due to alkali-aggregate reaction*
   * Alkali in cement reacts with certain reactive carbonates and silicates found in certain aggregates, causing the formation of silicate-gel which exerts expansive pressures causing extensive cracking.
7. Surface erosion/abrasion.
8. Crazing (surface cracking generally within laitance thickness).
9. Plastic shrinkage and plastic settlement cracks (within the first 3 to 4 hrs.)
10. Early thermal movement cracks (after 1 day but generally within 3 to 4 weeks of casting; causing by:
    — lack of movement joints
    — presence of constraints to free movement
11. Drying shrinkage cracks
12. Cracks due to overstressing of steel and concrete.

NOTE Cracks can be 'live' or 'dormant', depending on whether they continue to open or do not open further.

(E) Methods of Rehabilitation

(1) *Outright Replacement*
   — Demolish and reconstruct

(2) *Repair* First take necessary precautions of closing traffic if required, and temporarily supporting the structure carefully and then:
   (a) *Remove unsound concrete, using*:
      • Hammers
      • Pneumatically operated tools/compressor
      • Very high pressure water jet
   (b) *Clean reinforcement steel all around, using*:
      • Wire brushes
      • Sand blasting
      • High pressure water jet
   (c) *Restore removed concrete, by*:
      • Shotcreting (by layers)
      or • Guniting (by layers)
      or • Epoxy-mortar
      or • Epoxy concrete
      or • Normal (Higher strength) concrete
   (d) *Seal cracks*: (after suitably cleaning the cracks/pressure cleaning):
      * *for dormant cracks*:
      — Seal by pressure injecting a suitable epoxy formulation (unless low-pressure cement grouting can dependably do the job).
      * for live cracks:
      — These are like "movement joints". Cut a chase or a groove (a recess) along the crack (dimensions depending on type of crack), then seal with a suitable Flexible filler (mastics, thermo-plastics, elastomers) and, where called for, use an additional covering strip of flexible pre-formed rubberised or tar-bonded strip.

3. *Strengthening*
   (a) Bolts, anchors, additional structural steel
      * Consult Trade Representatives (e.g. HILTI, CIBA-GEIGI, etc.)
   (b) Externally bonded steel plates
      * Consult Trade Representatives (e.g. HILTI, CIBA-GEIGI, etc.)
   (c) External prestressing
      * Consult Specialist Agencies (e.g. V.S.L., Freyssinet, B.B.R., etc.)

4. *Protection*:
   (a) *Protection Against Corrosion of Steel (Chloride Attack and Carbonation Effect)*:
      * Although many Trade-products may be available from the Trade (e.g. CIBA-GEIGI) which can be used even subsequently, but their long term efficacy is doubtful. It is best to use dense concrete:
      * Increase minimum cover suitably (by about 25%)
      * Increase minimum cement content (to about 350 to 450 kg/m^3, depending on the conditions)
      * Reduce w/c ratio to about 0.37 to 0.40
      * Use plasticiser (additive) to gain on workability
      * Compact concrete by thorough vibration
      * Adopt slightly more fines in the total aggregate content.
      * Adopt early and adequate curing.
      * Paint concrete surface by high-pressure hot bitumen, if not objected to.
      * Preferably use Portland Blast Furnace slag cement containing about 65 to 70% of the slag in the cement (by weight). This gives a very dense paste which inhibits diffusion of chloride ions (and even alkali ions), thus inhibiting attacks from chlorides and sulphates and even the alkali-aggregate reaction.
   (b) *Protection Against Sulphate Attack*:
      * Use sulphate-resisting cement (Type V cement: ASTM) which has low C_3A content, and dense concrete as in (a) above.
   (c) *Protection Against Alkali-aggregate Reaction*:
      • Use low alkali cement
      • If such aggregates must be used, then at least replace part by non-reactive aggregates even if long haulage is involved.
      • Use Portland Blast Furnace slag cement as explained above.
5. *Special methods*:
   (a) Replacement of expansion-joints/bearings
   (b) Replacement Operations involving heavy-lifting and massive sliding/moving.
   (c) Ground and rock anchoring to stabilise unstable abutments, piers, walls.
   (d) Strengthening of foundations (various methods)
   (e) Strengthening of articulations locally

NOTE For all these five items it is advisable to consult specialist agencies like: V.S.L., Freyssinet. B.B.R.. etc.

(F) Crack Repair Illustrations

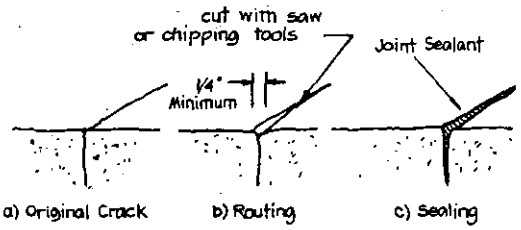

Fig. 48.1 *Repair of crack by routing and sealing is a method suitable for cracks that are dormant and not structurally significant. Routing and cleaning before installing the sealant add significantly to life of the repair.*

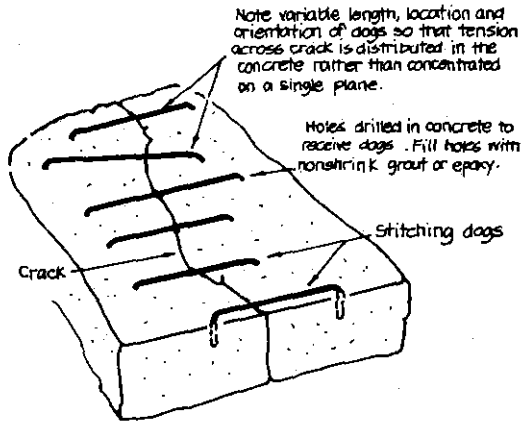

Fig. 48.2 *Crack repair by stitching restores tensile strength across major cracks. Where there is a water problem, the crack should be made water tight first to protect the stitching dogs from corrosion.*

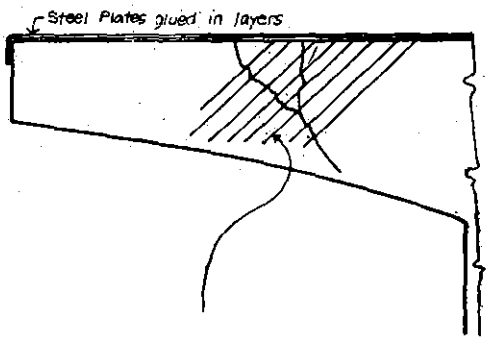

Fig. 48.3 *Added reinforcement installed to strengthen repair. Holes are drilled at right angles to the crack then filled with epoxy before bars are inserted.*

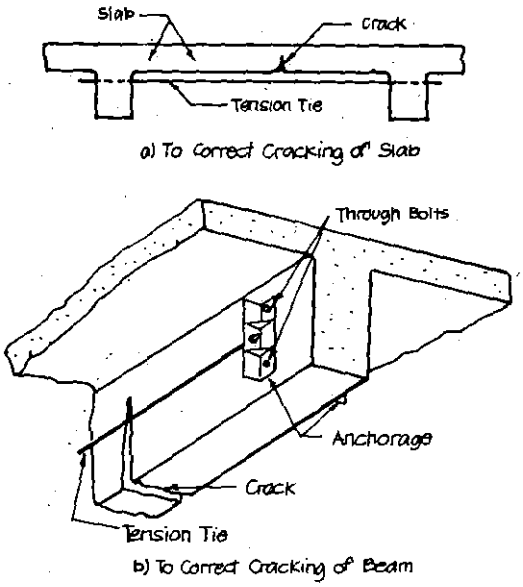

Fig. 48.4 *External prestressing can close cracks and restore structural strength. Careful analysis of the effects of the tensioning force must be made or the crack may migrate to another position.*

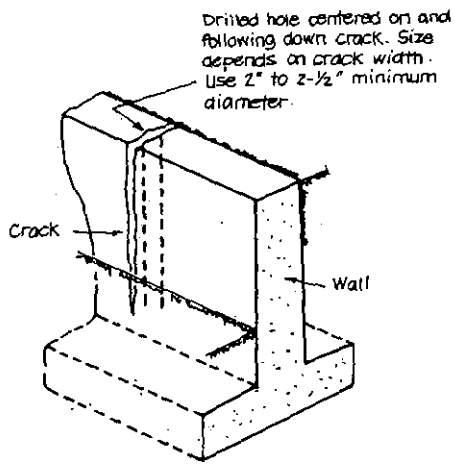

Fig. 48.5 *Drilling and plugging is a repair method well suited to vertical cracks in retaining walls. The repair material becomes a structural key to resist loads and prevent leakage through the crack.*

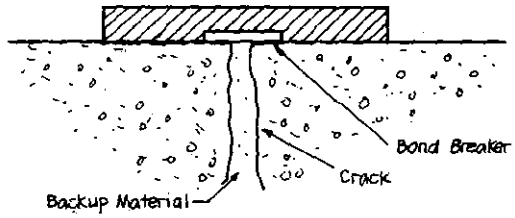

Fig. 48.6 *Flexible surface sealant can be used over narrow cracks subject to movement, if appearance is not a consideration. Note bond breaker over the crack itself.*

48.9 APPROPRIATE TECHNOLOGY FOR DEVELOPING COUNTRIES

Introduction

A lesser developed country (LDC) would obviously like to catch up with the modern technological advancement on the one hand and create more employment on the other. LDC technology needs are enormous compared to their relatively meagre resources and therefore most of the linkage problems (e.g. industry-university links) should better be matters of urgent policy design rather than just matters of occasional reminding.

Assessment of the 'appropriateness' of technology is not a new idea. Workers revolt against machines during the industrial revolution, the clinical testing of pharmaceutical products, or even the cost-benefit analysis of development projects, all involve technology assessment in one form or another.

It requires people with talent, tact, discretion and imaginative analysis to assess the appropriateness of technology for a given LDC keeping in view the limitations of prevailing standards of education, experience and society. Suppose in a poor country with dense population a low cost housing scheme has to be executed. It should obviously be a very simple type of building, fully made of locally available materials using local skill, and all the work should be done by the local hand. Water, sewerage and electricity are of course a must. Now suppose such a housing scheme has also set up in an oil rich country where materials may be unavailable locally (even the water may be brackish) and there is little skilled labour available. Therefore a good part of the materials and staff and labour have to be imported. These imported people have to live and that means providing for them temporary housing, transportation, recreation, schools for their children, medical care and leave arrangement. Also, materials have to be transported, which might mean waiting in congested harbours, traffic jams, overloads, etc. There may also be insufficient communication due to possible

lack of Telex and telephone facilities. Consequence of these considerations should obviously be a design which asks for minimum manpower and short construction time. The lesser the man-hours, the lesser are the problems. Production method may well have to be extremely advanced and sophisticated, equipment heavy and expensive, materials imported and of first class quality—and not necessarily cheap. In the above two cases obviously the technology which is appropriate for one is not so for the other—and yet both technologies are appropriate for the individual LDCs.

All developing countries lack to some extent the economic infrastructure, the social infrastructure and the productive assets. This unfortunate phenomenon was probably accentuated due to the dominating powers using them as sources of labour and raw materials and as markets for finished goods.

With objectives like international free trade, infant industry protection, and employment creation, the expression 'appropriate technology' becomes difficult to use. International trade between developed and less developed countries may cause dependence through ownership of technology.

In many instances where the shift from very labour intensive to intermediate class of technology appears an economically attractive proposition, the non-availability of the required capital can act as dominant constraint.

Phenomena

A technology is appropriate if it involves simplicity of production, use and maintenance, and cheapness and independence, without being wholly mystified by experts. It has a much closer relationship to life and can be a means of raising consciousness.

The cities exploit the countryside and themselves are sacrificed to the requirements of international commerce. No area lives for itself alone. Self-sufficiency and self-reliance are most important so that:

- the economy can proceed at stable pace,
- the people can define their own goals,
- the system is not too dependent on alien processes,
- the people are reasonably protected against commercial exploitation.

Intermediate technology, appropriate technology, or convenient technology—which one? We want the correct technology for the environment prevailing at a particular time in the particular part of the world under question. This correct technology is best described as appropriate technology. Appropriate technology must be subject to the prevailing local conditions such as the indigenously available raw materials, manpower, plant, power, etc.

Developmental process transforms the traditional into modern. It is the technology-gap that really creates the income-gap. After three decades of development efforts, however, unemployment and the compromise between labour intensive versus mechanised techniques have emerged as the most critical problems of today in many developing countries.

The value of manpower training programmes for professional practice and workman like production has been underestimated, whereas the importance of research has been overestimated in the past. It is more important to extend research into practice than only make more research which is not put to use. Manpower training is capital saving process while research is capital using process, and though the latter should never be stopped altogether, the former definitely deserves more attention in the present situation.

It is essential to improve the knowledge and skills in the developing countries for better utilisation of technology. This demands building the necessary infrastructure in the LDCs which includes educational, scientific and technical disciplines. There is no shame in first copying and then creating one's own.

Different types of alternative technologies are best identifiable in terms of the communities that have to practise them. An attempt should be made to cast the goals into a set of rules which the technologists might follow. The latter should not be left to judge intuitively as to whether a given line of development meets the goals. Some typical goals are:

- Regional self-sufficiency
- Resources conservation
- Direct control by consumers
- Absence of exploitation
- Ecological equilibrium

The LDC's first problem generally is to find employment for large reserves of manpower by investing in industries which may even be of somewhat doubtful profitability. Sometimes the engineering firms commissioned to make the techno-economic studies are connected with plant manufacturers or process holders and therefore may give a slant in their report towards the choice of certain types of plants and processes. The choice is between less advanced technology in order to absorb available manpower or more advanced technology which could be export-oriented. LDCs many times are not properly equipped to face the choice of technology. The engineering firms justifiably keep their secrets jealously and are not always prepared to pass them on the others. ***The 'tied' aid is not the ideal background for choosing the right technology.***

Development

Economists can elegantly make projections and the scientists, being no less, may matchingly project the consequences of the developments in science. It may be desirable to subordinate science to economy in the early

stages since the R&D facilities and resources may be scarce.

The economy of LDCs requires certain capital intensive industries which require a great deal of investment. The examples are steel, cement, aluminium, paper, certain essential chemicals and fertilizers and, most of all, the road and rail building as also the ports. Many such industries can give birth to ancillary small-scale industries and in the process create employment necessitating local technology. Many of these, however, require rather long periods of gestation that cannot be helped.

The success of construction investment depends on the suitability of the overall concept of a project. Designs must not only be suitable for local conditions but appropriate use of indigenous materials, local skills and manpower should be tied in. The total cost of the project including maintenance should be in proportion to the financial resources of the state.

Development can be categorised into three heads: ***Economic***, ***Social*** and ***Industrial***:

(i) ***Economic*** For developing agricultural and industrial production the major bottleneck is transportation. Owing to lack of it, a commodity may be sold at a certain time at a very high cost in big cities while farmers in far flung rural areas may not find a buyer for it because they may be unable to transport it owing to bad roads. Another type of bottleneck in the transport sector can be the lack of port facilities and there are instances when the waiting time for loading and unloading ships reached as high as three to five months at certain times! Hence the development of roads and ports is of basic importance for improving the economic infrastructure. The urgent need in transport sector is for more roads of all types, ranging from national highways to feeder rural roads, coupled with coordinated development of port facilities. The main problem here is the choice of optimum standards to meet the present and the projected future needs, the choice of the appropriate construction methods and the balance between labour intensive and the capital intensive techniques.

(ii) ***Social*** This requires developing low cost housing, schools, hospitals, administrative offices, community recreation centres, water supply, sewerage, rural electrification, etc.

(iii) ***Industrial*** This includes industries, power stations, irrigation structures, canal networks, etc.

When the financial inputs are scarce and labour and skills are relatively easily available, then whether labour intensive methods should be used (even when they may be more costly) in order to relieve unemployment, is a question for debate. Political environment has a lot to do with this debate and it is difficult to generalise on this as there are so many shades of it. There are dictatorships of left and right, powerful monarchies, various brands of democracy, influential aristocracies and straight military government. Any study which fails to take careful account of the political pressure and constraints is liable to prove a waste of time.

Feasibility

To handle the heavy work-load including the preliminary and detailed engineering, substantial staff is needed and it is here that the locally available engineers, technicians, draughtsmen and foremen can be trained in order that the host country strengthens her own infrastructure. The design standards, for example in the case of roads, particularly with regard to the road widths, radii and drainage, should be very carefully considered. Requirement of the width of the carriageway should be extremely carefully studied as any extra over the optimum will lead to a considerable loss of resources and any shortfall may lead to traffic congestion and bottlenecks. Where necessary, the minimum radius of a curve and the gradient might have to be suitably altered in order to avoid long and deep cuts and fills so as to reduce the danger of slides during heavy rains. This is particularly true for tropical countries and obviously detailed soil investigation is of paramount importance in the design stage. The roads may be generally paved with lighter pavement (such as double surface treatment) and in certain cases the roads are even left as simply gravel roads. This, sometimes, can lead to increased cost of maintenance, not to talk of the danger of accidents due to development of corrugations. Similarly in the design of structures, an account has to be taken of the availability of the locally available materials and skills. For example in some LDCs whether the bridge superstructure should be in steel or in concrete will obviously depend on what is easily available and the skill required for the same. By and large, many LDCs now possess cement works and produce steel reinforcing bars and light steel sections and therefore a concrete superstructure might well be more appropriate.

A typical road might begin as an earth track and then gradually be surfaced and re-constructed. In LDCs where the financial resources are scarce but labour and skills are available cheaply, obviously the choice is for labour intensive techniques and on the face of it there would be nothing wrong if materials for fill work are taken from the nearby pits along the route, loaded in the cane baskets and head-carried to the site and dumped, without calling for much earth moving machinery. But inherent fault may be the lack of proper compaction, which, in due course, may call for heavy maintenance cost for repairs and running. The question of using primitive construction methods compared to at least partially mechanised operations should therefore not be made purely a matter of 'minimum first cost'. A road has to be built to a certain standard and does require a certain

minimum of mechanised operations which should not be substituted for by manual labour alone. The appropriateness of the technology will lie in the indepth study of the variables and preferably should not be sacrificed for the available technology alone.

Assessment of feasibility is a comparison between two or more alternative future situations, a systematic comparison of relative merits and demerits from technical, financial, economical, social, environmental, administrative and political points of view.

Financially the feasibility assessment would mean: 'what is the return of revenue for a certain financial input'. Economical feasibility assessment would mean to go further and study not just the revenue return for the financial input but also investigate the good it might do to the general public at large. Some projects that are not bright from the financial assessment angle, may still be economically desirable. Projects like railways, roads and ports will generally be profitable even from the financial angle but projects like hospitals, motorways, flood protection works, public health projects and community development centres may be economically viable even if not very much financially.

Most engineering projects have a long life and are undertaken more in the interest of future generations. Since the final decisions generally are not taken by those who conduct the study but by politicians, it is incumbent that political implications will at some stage influence the decision. In some LDCs the political importance of a new road/port/airport/bridge or an industrial estate may be more overpowering than any of the other criteria in the total feasibility analysis. The aim of a feasibility report should not be so much as to recommend a decision but to provide information that can lead to it.

Methodology

LDCs must endeavour to develop their own engineering capabilities in order to break their dependence on DC markets. Even technical assistance, unless restricted, pruned and evaluated regularly, reinforces relations of dependence.

Technology aid should be conceived not merely for filling LDC resource gaps, but as an attempt at developing the capability of LDC people. Unfortunately, many a time, technological issues in LDCs are discussed by non-technologist, and despite their pious intentions, this does not always help.

The DCs should make information available to LDCs and the U.N. should play a big role in this field. But this does not mean it would be possible for the U.N. to establish a data bank to give information on all available technical designs and processes. Every LDC should establish a technical information service to provide appropriate technical information. Advice and funds may be obtained through multilateral and bilateral aid sources. A considerable part of this information may be obtained for a small fee. However, know how and detailed information can still not be available without high cost.

DCs should be invited to aid the LDCs and to make an effort to translate their techniques into the ones that match the economic and labour conditions and the in-situ environments of the latter.

Detailed information about appropriate technologies should be collected, subjected to field trials and documented in suitable form for communication to the developing countries.

Environmental care is of importance not only to DCs but also to LDCs though the latter may not always afford its priority. The damage to environment from insufficiently planned development activities can in fact result in decrease rather than increase in the happiness. Mistakes made in the earlier years of the development of the DCs need not be repeated all over again in the LDCs.

After the projects are constructed, many times the element of maintenance is given secondary importance and soon the lack of it take its toll. The maintenance is of paramount importance and this requires training of manpower where not available.

Suggestions

All human beings, and engineers are no exception, are strongly linked to their own limits of experience. Extrapolation concerning change of size, type of loading, climate, construction materials, the levels of efficiency, feed-back, feed-forward and above all the way of life, should be taken up very carefully.

Planning and construction of a country's road network is generally placed under the jurisdiction of the PWDs working under the jurisdiction of its Ministry of Transport. These PWDs, particularly in LDCs, unfortunately may not be fully conversant with the latest and appropriate technical developments in modern methods of design and construction, and, for certain reasons cannot keep pace with the furious standards of workmanship and efficiency of the consultants and constractors. It is therefore necessary in such a case that the PWD concentrates on the overall planning and maintenance, leaving the engineering and construction to the consultants and contractors (of course typing them up with necessary contractual obligations).

Security lies in a technology that follows the natural-balance cycles. It is of great significance to bend technologies to suit the renewable sources of energy like wind, sun, water, vegetable-fuels, plant and animal products, soil, stone and above all the human labour.

It is not a good solution to use second-hand machinery,

because, even if it may require less investment, it may well lead to an increase in production-cost and thereby widen the already existing technological gap between the DCs and LDCs. ***While some LDCs may have the finance (like the oil wealth) and therefore may be capable of calling for the most advanced technological expertise, they generally are handicapped in receiving technology. They may even suffer on account of the non-availability of their own skilled staff for taking over and, in the process, may spend far too much without fully utilising it. On the other hand the poorer LDCs may have most of the manpower already available but the lack of finance may compel them to search for self-reliance and compromise technology.*** A technology service should be established at the international level which could help the developing countries to set up technology cells and train skilled personnel.

Some capital intensive and high technology industries will still be needed even when the general policy may be to encourage small-scale industries. A certain measure of foreign investment and technical collaboration may still be required. It is necessary not to confuse self-reliance here and to accept the foreign investment and collaboration in such zones not from a sense of inadequacy but from a sense confidence essential to such sectoral development.

So far as possible, the whole of design should be carried out locally in the LDC even though in some cases it may even increase the cost. This kind of cost increment should be treated as a welcome investment. To get the entire package from a foreign country will only make the concerned LDC more dependent in future.

Practical difficulties arise in LDCs because the level of dependable information is usually low and sometimes statistical information available may be unreliable and therefore one should not incorporate more detail in the analysis than the data permits. The poorer the data, the simpler and cruder will be the analysis. Therefore in some of the LDCs it may be better to simplify the methodology since data may not be good enough. Their priorities may be different. This may require lower standards to be accepted for items like transport, electricity and water, in order to realise resources for basic housing, sewerage, health and education.

The continued and timely supply of various modern farm equipments and inputs such as factors, power-tillers, fertilisers, pesticides, etc., is very important for the agricultural base and the latter forms the back-bone of the infrastructure needed for technology development. To ensure this, indirect taxes should be lowered on these farm equipments and inputs. Agriculture is a high-risk industry, where, in case of a failure, the farmer loses all. Therefore it might be prudent to tax the farmer (as opposed to those owning farm holdings on big scale) on the output he sells than on the input he has to buy.

Certain industries suffer from lack of adequate demand to what they are capable of producing even within the existing limitations. Others face not so much the lack of demand as the inability to produce enough owing to a variety of reasons as for example: power shortage and transportation bottle-necks. These difficulties are magnified sometimes owing to their tie up with agricultural production. These problems require improvement in efficiency, better management and re-scheduling the priorities. Support and assistance through economic and fiscal policies is called for.

Between the two extremes of 'computers for people' approach on the one hand and 'spreading information as to how to block the operation of computer systems' on the other, lies the approach of compromise technology which is essentially labour based.

Although the case of each individual LDC has to be considered on its own merits, a very careful study will require to be made regarding whether labour intensive techniques, capital intensive techniques or a partial combination of the two should be adopted. Certain LDCs have some skilled and ample unskilled manpower available but not enough capital, and some other have the necessary capital but not enough manpower. ***If a time bound programme has to be followed it is essential to take up manpower training on a massive scale and this is where the U.N.'s D.E.S.D. comes in.***

48.10 BLUE-PRINT FOR GUIDING TECHNICAL DEVELOPMENT IN THE FIELD OF CIVIL ENGINEERING PUBLIC WORKS IN DEVELOPING COUNTRIES

(A) Synopsis

Even knowledge, unless indexed properly, cannot be quickly tapped when faced with a time-bound problem. By the same token it is essential that any technical development programme be at least guided, if not regimented, for a time-bound progress. This calls for first understanding the problem, then gearing up the means to tackle it and then marshalling a blue-print approach for solving it.

The presentation here indentifies some of the relevant civil engineering public works together with some of their details and possible problems, and reviews the working system of the Public Works Departments.

Then, in an attempt to rectify the rut and guide the technical development process, the author suggests a more relevant system of manpower-training and technical education.

After describing various phases that are incumbent in a civil engineering public work, the possible U.N. Technical Advisory assistance is recommended. The importance of

applied research against pure research and the element of competition are stressed.

(B) Introduction

1. ***'The first object is to 'know'—within limits of our ability to do so, and the second is to 'apply'—without going beyond what we know."***
2. If a time bound progress has to be achieved in technical development, it is essential to evaluate the problem and decide upon the solution philosophy in a workman-like manner.
3. Talented people have to tactfully use their discretion in preparing, appraising, planning and executing the technical development programmes, although actual implementation is not always easy. The political influence in any country, more so in a developing country, can thwart such activity unless tact and discretion are used.
4. Even after three-and-a-half decades of development efforts, the end-product less than matches the needs of the multiplying multitude of masses. Technical development can be achieved by massive manpower training as relevant to indigenous conditions and conversion of techniques to suit the local availability of materials, skills and environment.
5. This subject assumes all the more importance as the developing countries focus more attention on the mechanisms for achieving collective self-reliance. There is no alternative for the developing countries but essentially to search from within to strengthen their own infrastructures as the major burden of their development must finally be borne by them only. Even though many of these countries in the past have been physically conquered, politically subjugated, culturally aggressed and economically exploited, there is no point in wasting further time in this rhetoric as otherwise history will only repeat itself. We, as today's well-wishers of humankind, have an urgent job to do and must remember that the only thing dark about these countries is our own ignorance about them.
6. Whether it is the 'wealthy poor', the 'well-to-do poor', the 'less poor' or the 'rock-bottom' developing countries, they constitute a world of their own.
7. The aim basically has to be to bend the methods and technologies, to suit the indigenous availability of manpower, materials and equipment and simultaneously strengthen the infrastructure by a massive programme of technical training. The present paper attempts at putting forth a workman-like plan for achieving just this.

(C) Some Relevant Civil Engineering Public Works

1. Let us first identify the more relevant civil engineering public works and then identify some of their details briefly and possible problems.
2. ***The more relevant civil engineering public works are:***
   (a) Roads and Railways,
   (b) Bridges,
   (c) Buildings, e.g. Low-cost housing, Administrative Offices, Schools, Hospitals, Community Development Centres, etc.
   (d) Water supply, drainage and sewerage works,
   (e) Canal networks and flood-protection schemes, and
   (f) Ports and Harbours.
3. ***Some details given very briefly***

(a) *Roads and Railways* The need of the project and terms of reference; reconnaissance survey, route location and alignment; rate of progress for laying out the alignment; minimum vertical and horizontal curves, gradients and radii; drainage; possibility of aerial photography for locating stretches in difficult locations: topographical and geological features; future development of areas fed by the road/rail; 'formation width' in relation to the contemplated future traffic; lands to be appropriated and acquired; structural works involved (e.g. bridges, culverts, causeways, drainage arrangements, retaining walls, rest houses, etc.); sources of materials (and leads involved), labour and plant available locally; how much labour-intensive and how much mechanised techniques to be adopted; plant and equipment requirement; quality of structure, possible causes of disintegration of road surface; subsoil drainage and control, method of rolling; size and grading of metal; design loads, traffic and speed; maintenance; preparation of preliminary estimates; agency for execution; project report.

(b) *Bridges* Need of the bridge; approaches to the bridge; appropriate location of the bridge; effective linear waterway requirement in keeping with the catchment area, drainage run-off, afflux, bed scour, etc.; vertical clearance requirement; maximum scour depth at foundations; minimum founding depths, right or skew bridge crossing; bridge width (i.e. single-lane, double-lane or multi-lane); requirement of footpaths—if any; design loadings; type of foundation (wells, piles, open foundations, etc.); spill-through or non-spill-through type of abutments; subsoil investigations and soil parameters; high flood level and low water level; wind intensity; earthquake intensity; maximum mean flood velocity; temperature variation; type of bridge, method of construction and optimum span length; possibility of ground movements and unequal settlement of foundations; whether concrete or steel superstructure; type of bearings and expansion joints; permissible stresses during construction and during service conditions, considering the appropriate load combinations; river training works (wired boulder crates and mattresses; guidebunds, spurs, etc.); preliminary design, drawings, quantities and estimate;

agency for execution; project report.

(c) *Buildings, e.g. low cost housing, administrative offices, schools, hospitals, community development centres, etc.* Need of the particular project at a particular location and terms of reference; functional requirements, i.e. estimation of the type and volume of service to cater for and correspondingly designation of the size, number of floors and the allied services involved; architectural requirements; structural requirements; preparation of preliminary architectural plans; preliminary structural designs, drawings, bills of quantities and estimates; agency for execution; project report.

(d) *Water supply, drainage and sewerage projects*

(i) ***Water supply***: Reasons necessitating the scheme/additional water supply; quality and quantity of existing supply and its sources; possible sources of additional supply and arrangements for its intake; discharge from wells, tests of yield from wells and construction of tube-wells and open wells; well linings, number of people, number of animals and the volume of industry to be catered for; estimated daily requirement and how calculated; purification, distribution and pumping; impurities in water, hard and soft waters and particular chemical processes required; storage and service reservoirs; settling basins and sedimentation tanks; coagulation, sterilisation and chlorination; colour and taste removal; domestic storage tanks; elevated tanks; pressurised supply; leakage and wastage of water; different methods of distribution and design of the system; design of mains and supply pressures at different points and taps; economical velocities in distribution pipes; whether supply will be metered; piping-material and jointing systems; type of pumps and power for working them; the total cost of the scheme, the annual recurring cost and the depreciation of the plant and equipment; cost estimates; executing agency; project report and terms of reference.

(ii) ***Drainage and sewerage***: Statement of problem; surface drainage; rainfall runoff from catchment, time of concentration and flood drainage, permeability of soil; combined and separate systems of drainage; open drains; underground sewers; size of sewers for different systems; volume of sewage for design, storm overflow; self-cleansing velocity in sewer lines; flushing of drains and sewers; section shape of sewers; choice of materials for making sewers; problems in setting out sewer-lines, and excavation; laying and jointing pipes and sewer lines, sewerage crossing; manholes, ventilation and maintenance; drainage traps and inspection chambers; testing against leakage; plumbing and fixtures; standards for public sanitary convenience; house disposal systems; disposal of sullage from towns; location of disposal works; preliminary treatment of sewerage; chemical precipitation of sewerage, filtration; sludge digestion and activated process; treatment of industrial wastes; methods of sewage disposal; cost estimates; executing agency; project report and terms of reference.

(e) *Canal networks and flood protection schemes* Need for a particular project; suitability of the soil for construction of embankments; stabilising side-slopes against slip failure; alignment of river and flood embankments; hydraulic gradient; closing of breaches; pitching and revetting materials for core-wall; cut-off trenches; construction of spurs; design and layout of distributories; water courses; service roads; top widths and boundary ditches; lining of canals; canal cross-sections; seepage and evaporation losses in canals and reservoirs; suitability of water for irrigation and of soil for reclamation; water logging; soil erosion; silting up in canals and reservoirs; storage of rain water run off; desilting and maintenance of canals; eradiction of water weeds; cross drainage works; aquaducts and syphons; problems in the design of weir floors; aprons and cut-off walls; non-silting velocity in canals; flood escape channels; cost estimates; executing agency; project report and terms of references.

(f) *Ports and harbours* Need for a particular project and terms of reference; cost-benefit forecast; effect of the project on the hinterland and its development; functional requirements; structural requirements; collection of data on wind, waves, currents, seismic intensity and sea bed contours; soundings; subsoil characteristics, scour patterns; alternative locations and alignments; recommended location and alignment and type of berthing structure with reasons; the overall integrated scheme; marine movements and road transportation of passengers and cargo; particulars of the soil investigation to be conducted and its possible effect on the overall cost; location of suitable entrance approach channel and availability of the turning radius for the ships; fixing the levels of the top of the berths and of the dredged floor on the basis of the estimated astronomical highest and lowest tides; extent of dredging and the requirements of the tug boats/turning dolphins; requirement of accessories on the berthing structures; buoys and entrance channel marks; types of fenders and recommendations; type of piles and foundation to be adopted; design criteria, e.g. loading, current forces, wave force, wind force, seismic force, scour of sea bed, impact force from docking of ships and hydrodynamic effect during docking; mooring-line forces, etc.; type of mooring dolphins and considerations for designs; preparation of preliminary designs, drawings, quantities and cost estimates; expected time required for construction; executing agency; project report.

4. ***Possible problems***

Problems can also raise from inadequacy and vague wording of the contract clauses and therefore it is necessary that the contract documents are prepared thoroughly and handled by those who are conversant with this subject and its legality. Examples of possible troublesome clauses are clauses pertaining to the amount of bond, the amount of third party insurance, time for completion of the works, amount of liquidated damage, length of maintenance period after handing over, the percentage and limit of retention money, the period within which the payment is to be made after the certificates have been issued, upward revision in cost owing to escalation in labour wages and material prices (incident to inflation), any change in scope of work necessitated either due to change in specifications and/or physical site conditions, accident or injury to workmen and the consequent compensation, interference with traffic and adjoining properties, access to site, expected risks and agreement about nomination of arbitrator, etc.

(D) Public Works System of Working

1. Before we proceed further, it will be appropriate to briefly touch upon the ***public works system of working***.
2. The ***Ministry in charge of Public Works***, under its Secretary (who is answerable to the Hon'ble Minister) has generally a Director General, the Additional Directors General (one for each discipline) and the Zonal Chief Engineers working under the Secretary. Each Chief Engineer has his compliment of Superintending Engineers, Executive Engineers, Assistant Engineers and technicians together with the supporting Administrative and Accounts staff.
3. While the Hon'ble Minister and the political party in power may change, the rest of the staff generally remains. ***Unfortunately the pulls and pushes of bureaucracy and obligation to hierarchy and political influence in most developing countries sometimes frustrate some of these engineers into becoming mere bureaucrats which prevents them from dabbling in the delights of bold, decisive and independent planning, design and construction***. This has to be taken explicit note of in understanding the system of the state run public works departments (P.W.D.s). These departmental engineers may therefore generally not be fully conversant with the latest and with the appropriate technical developments in modern methods of design, construction and management, and for understandable reasons, cannot keep pace with the furious standards of result and profit oriented workmanship and efficiency of the independent professional consultants and contractors. It is therefore necessary in such a case that the P.W.D. concentrates on overall general planning and maintenance leaving the engineering and construction to the acknowledged Consultants and Contractors, of course tying them to necessary contractual obligations.
4. Most engineering projects have a long life and are undertaken more in the interest of future generations. Since the final decisions generally are not taken by those who conduct the feasibility and cost-benefit studies but by politicians, it is incumbent that ***political implications will at some stage influence the decision***. In some developing countries the political importance of a new project may be more overpowering than any of the other criteria in its feasibility analysis.
5. ***Cost of works***

   (a) The total cost of a works is not only the final cost of its construction but also includes the expenditure to be incurred for its maintenance and the interest lost on the capital investment.

   (b) Cost of civil engineering public works are sometimes paid in instalments. Equal periodical payments which are allowed to accumulate, each earning its own interest, are called annuities. This sum of money is set aside annually to accumulate compound interest and form what is called a sinking fund in order to clear a debt.

   (c) In estimating operating expenses of engineering works, an allowance is made for depreciation. In calculating this, a certain life-span is first assumed for the work/plant, and then annuity, at an assumed rate of compound interest, for the assumed life span, is estimated so as to equal the cost of the work/plant and this provides a fund by means of which the work/plant may be replaced/replenished when needed.
6. If a particular job has to be done and delivered by a privately run contracting agency, its total cost will not only include the working cost of materials, labour, plant, engineering time and overheads, but also profits and provision for promotional expenses. The cost of engineering time will have to include not just the gross salaries but also certain employee benefits (e.g. provident fund, gratuity, leave travel, medical benefits, insurance and compensation, leave salary, etc.) and a bonus as an incentive for straining to stay alive in the competitive cut-throat market. Against this, the P.W.D. system hardly provokes competition in-house, and the result can be an out-of-focus design and costing and less than workman-like manner of working. (If a private firm worked with laxity, it would soon fold up owing to incisive competition from rivals.) Their overheads and costs mount owing to an excessively hierarchical manner of working and lack of independence and practising professional touch.

agency for executis not uncommon to see more engineers D. overseeing a job than the number of (c) *Buildings, e.g.* ne Consultant/Contractor actually doing *schools, hospitals* ***art of doing a work with ones own*** Need of the part ***thus become a casualty. This endemic*** and terms of re ***ting of many of the state-run P.W.D.s*** estimation of the ***e drastically altered for a meaningful*** for and correspon ***opment programme to flourish.*** of floors and the requirements; str **Technical Education** preliminary archi designs, drawings, indeed necessary, to tilt the technical for execution; proj to an applied direction than leave it mical.

(d) *Water supply, ce*

(i) ***Water supply:*** students successfully clearing the 12-ditional water chooling, a large majority should be supply and its dustrial Training Institutes for shoplevel supply and a 3 year vocational training courses in from wells, tesical fields. This cadre would form the of tube-wells astructure at the grass-root level.

of people, nu the interested school leaving students, industry to be er intellect and better bent of mind, and how calced to the standard 5-year engineering pumping; imput the Institutes of Technology. Here and particular could be a rigorous blend of academic service reservoing. These would-be engineers should tanks; coagulatih exposure to the views of practising and taste removited from industry) by forging the tanks; pressuriustry-university link-ups. In course of water; different engineers who will percolate into of the system; f the infrastructure, providing pivotal at different poiat may be termed as battalion and distribution pipme of them of course will have to man piping-material ater in their lives.

power for work of these graduate engineers should the annual recu 'specialisation' on the basis of their plant and equipt. This specialisation has to be really project report ang over a period of about 3 years: about

(ii) ***Drainage and si*** s postgraduate study in the particular drainage; rainfaon at the University level, followed by concentration analf years' professional training, partly combined and ly in construction, by rotation. Some drains; undergrong engineers may of course go in for systems; volume without the practical training back-self-cleansing ve

and sewers; sectivity in the field of civil engineering for making sewelves not only design and construction and excavation; rative and budgetary control, it is lines, sewerage subjects of public-administration and maintenance; draluded in the under-graduate studies.

testing against lea he technicians (who have come out of for public sanitaring Institutes) and the engineers (who have come out of the Institutes of Technology) are put through refresher courses at periodic intervals during the course of their service in order to keep them in touch and updated.

(F) Relating Experience to Level of Responsibility

1. This relationship should be interpreted very carefully, particularly in regard to minimum qualifications, experience, duties required to be performed, and the requirements of supervision, decision-making and commitment. The level of responsibility pertains more to a position than to person. Promotions should not be based purely on 'age' and 'number' of years of experience, because some people may show considerable promise at a relatively early age and unless they are picked up and nurtured, the resulting frustration can be degenerative. An individual's position will have a lot to do with the merit of his background and experience, his efficiency in performance and of course the prevailing market value.
2. However, it is always a good idea to make lateral recruitment at higher levels from time to time for injecting fresh blood into the system. Some experts in individual fields should also be brought in on a term-appointment basis in order to get the benefit of their expertise for obvious advantage.

(G) Phases

1. Construction of a civil engineering public work project should be looked at in ***five phases***, namely feasibility investigation, engineering investigation, design, construction, and maintenance.

   (a) In the ***feasibility investigation phase*** broad studies are undertaken including the questions of viability, and consequently experts of many disciplines may have to be involved. Net product is a report upon which the Clients, together with their financial advisors, will decide if the scheme has to progress further.

   (b) In the ***engineering investigation phase*** the study enters more deeply into the engineering aspects and detailed site surveys and soil investigations, etc. Special investigations may be necessary in order to facilitate the study of alternative engineering solutions.

   (c) In the ***design phase*** detailed engineering design and drawings are prepared together with the contract documents, specifications and Bills of Quantities, in order to enable the Client to invite tenders for evaluation, acceptance and taking up construction.

   (d) In the ***construction phase*** the authorities have to see that the construction proceeds in accordance with the drawings and specifications under appropriate supervision on site. The appointment of specialist Inspectors and Expeditors may be called for. The

authorities have to check the progress and the quality of the work in relation to the programme and terms of the contract.

(e) In the ***maintenance phase*** the authorities may have to devise the maintenance procedures and effect their implementation lest lack of maintenance should nullify the entire effort.

(H) U.N. Assistance

1. ***The U.N. and its affiliates should strengthen their technical Advisory Teams which can assist on the following lines:***

   (a) Review the need for different public works projects in individual countries.

   (b) Assess the techno-economic feasibility of the assistance-requests from the individual countries for various public work projects.

   (c) Detail out the terms for field inspections and project reporting.

   (d) Assist in evaluating the appropriateness of the relevant experts who may be drafted in for individual assignments.

   (e) Review the write-ups of specifications and contract documents.

   (f) Assist in scrutinising the technical proposals and bids, and in making recommendations for the award of contracts.

   (g) Oversee and analyse the field reports and review the implementation of projects periodically.

   (h) Assist in-site supervision and quality control checks, maintenance checks, and manpower-training programmes through table-top discussions and refresher courses, clearly distinguishing between 'theory' and 'professional-practice'.

   (i) Study the existing work methods of the various Public Works Departments by working from within them for short periods of time, and then suggest ways of improving the same in an attempt to make them competitively self-reliant.
2. However, the U.N. cannot establish a data-bank to give information on all available technical designs and processes. It is the developed countries that should establish technical information services to provide appropriate technical information. Advice and funds may be obtained through multilateral and bilateral aid sources.
3. An international appropriate technology service should be established at the U.N. level which could help the developing countries to set up technology cells and train the skilled personnel.

(I) R and D

1. Much depends on how result-oriented is the technical training imparted to its recipients. ***Manpower training is a capital-saving process while research is capital-using process, and though the latter should never be stopped altogether, the former definitely deserves more attention in the present context.***
2. The less developed countries many times are not properly equipped to face the choice of technology. ***It may be desirable to subordinate science to economy, at least in the early stages***, since the R and D facilities and resources may be scarce and the results desired faster. ***It has to be remembered that for the purpose of development, time cannot be stilled and, therefore, good use has to be made of local skills, manpower and equipment even as we go along the developmental process transforming the traditional into the modern, without waiting too much for research.***

(J) Competition

Healthy but stiff competition alone leads to improvements. Therefore while some designs and techniques must be standardised for the obvious dual aim of simplifying procedures and cutting out avoidable multiplication of effort and time, the element of competition for better and bolder designs and construction methods should be religiously encouraged. ***This will lead to better designers and constructors which finally are the barometer of the quality of technical development.***

48.11 INSPECTION CHECKLIST (... field work)

An inspection checklist should be developed by the safety inspector or safety engineer with the help of executives, superintendents, and foremen, tailored to the requirements of the individual operations of the work. General checklists which apply to most of the problems, likely to be encountered will be easily developed. In addition, 'special checklists' for each individual job will be necessary, as peculiar problems are bound to occur. These will include, but will not be limited to, soil conditions, access roads, storage areas, power and utility lines, special equipment, camp housing, and other features.

The following list is offered only as a general guide, and is not intended to be comprehensive, nor should it be followed in this form if job requirements would be better served otherwise.

General Checklist

1. ***Accident Prevention Organization***
   a. Schedule for posting safety material.
   b. Hard hat requirements.

c. Safety meetings scheduled and posted.

2. ***Housekeeping and Sanitation***
   a. General neatness of working areas.
   b. Regular disposal of waste and trash.
   c. Passageways and walkways clear.
   d. Adequate lighting.
   e. Projecting nails removed.
   f. Oil and grease removed.
   g. Waste containers provided and used.
   h. Sanitary facilities adequate and clean.
   i. Drinking water tested and approved.
   j. Adequate supply of water.
   k. Salt tablets.
   l. Drinking cups or sterilized bubblers.
3. ***First Aid***
   a. First aid station.
   b. First aid supplies.
   c. First aid instruction on the job.
   d. Telephone numbers and locations of nearby physicians.
   e. Telephone number and location of nearest hospital.
   f. Injuries reported promptly to proper persons.
4. ***Fire Prevention***
   a. First instructions to personnel.
   b. Fire extinguishers identified, checked, lighted.
   c. Phone number of fire department posted.
   d. Hydrants clear, access to public thoroughfare open.
   e. Good housekeeping.
   f. NO SMOKING posted and enforced where needed.
5. ***Electrical Installations***
   a. Adequate wiring, well insulated.
   b. Fuses provided.
   c. fire hazards checked.
   d. Electrical dangers posted.
   e. Proper fire extinguishers provided.
6. ***Hand Tools***
   a. Proper tool being used for each job.
   b. Neat storage, safe carrying.
   c. Inspection and maintenance.
   d. Damaged tools repaired or replaced promptly.
7. ***Power Tools***
   a. Good housekeeping where tools are used.
   b. Tools and chords in good condition.
   c. Proper grounding.
   d. Proper instruction in use.
   e. All mechanical safeguards in use.
   f. Tools neatly stored when not in use.
   g. Right tool being used for the job at hand.
   h. Wiring properly installed.
   i. Enough men used to handle material.
8. ***Powder Actuated Tools***
   a. Local laws and ordinances complied with.
   b. All operators qualified.
   c. Tools and charges protected from unauthorized use.
   d. Competent instruction and supervision.
   e. Tools checked and in good working order.
   f. Tools not used on any but recommended materials.
   g. Safety goggles or face shields.
   h. Flying hazard checked by backing up, removal of personnel, or use of captive stud tool.
9. ***Ladders***
   a. Stock ladders inspected and in good condition.
   b. Stock ladders not spliced.
   c. Properly secured, top and bottom.
   d. Siderails on fixed ladders extend above top landing.
   e. Built-up ladders constructed of sound materials.
   f. Rungs not over 12 inches on centre (preferably)
   g. Stepladders fully open when in use.
   h. Metal ladders not used around electrical hazards.
   i. Proper maintenance and storage.
10. ***Scaffolding***
   a. Erection under proper supervision.
   b. All structural members adequate for use.
   c. All connections adequate.
   d. Safe tie-in to structure.
   e. Ladders and working areas free of debris, grease.
   f. Proper footings provided.
   g. Passersby protected from falling objects.
   h. Supports plumb, adequate crossbracing provided.
   i. Guard rails and toeboards in place.
   j. Scaffold machines in working order.
   k. Ropes and cables in good condition.
   l. Frequent inspection.
11. ***Hoists, Cranes and Derricks***
   a. Inspect cables and sheaves.
   b. Check slings and chains, hooks and eyes.
   c. Equipment firmly supported.
   d. Outriggers used if needed (stability).
   e. Power lines inactivated, removed or at safe distance.
   f. Proper loading for capacity at lifting radius.
   g. All equipment properly lubricated and maintained.
   h. Signalmen where needed.
   i. Signals understood and observed.
12. ***Heavy Equipment***
   a. Regular inspection and maintenance.
   b. Lubrication and repair of moving parts.
   c. Lights, brakes, warning signals operative.
   d. Wheels checked when necessary, tyre pressures checked.
   e. Haul roads well maintained and laid out properly.
   f. Protection when equipment is not in use.
13. ***Motor Vehicles***
   a. Regular inspection and maintenance.
   b. Qualified operators.

c. Local and state vehicle laws and regulations observed.
d. Brakes, lights, warning devices operative.
e. Weight limits and load sizes controlled.
f. Personnel carried in a safe manner.

14. Conveyers and Cableways
a. Proper inspection and maintenance.
b. Good housekeeping.
c. Screens and other protection where needed.
d. Adequate inspection, maintenance ladders, and walkways.
e. Lighting.

15. Marine Equipment
a. Waterways rules and regulations observed.
b. Life jackets.
c. Life preservers.
d. Fire protection.
e. Engine and machinery room safety.
f. Floating pipelines, walkways, and handrails.
g. Handrails aboard vessels.
h. Decks clear, free of oil and grease.
i. Lights.
j. Cable.

16. Equipment Maintenance
a. Planned maintenance and inspection programme.
b. Adequate equipment records.
c. Proper oils, fuels, lubricants used.

17. Garages and Repair Shops
a. Fire hazards.
b. Dispensing of fuels and lubricants.
c. Good housekeeping.
d. Lighting.
e. Carbon monoxide dangers.

18. Barricades
a. Floor openings planked over or barricaded.
b. Roadways and sidewalks effectively protected.
c. Adequate lighting provided.
d. Traffic controlled.

19. Handling and Storage of Materials
a. Neat storage area, clear passageways.
b. Materials neatly stacked.
c. Stacks on firm footings, not too high.
d. Proper number of men for each operation.
e. Men picking up loads correctly.
f. Materials protected from heat and moisture.
g. Protection against falling into hoppers and bins.
h. Dust protection observed.
i. Extinguishers and other fire protection.
j. Traffic routing and control.

20. Excavation and Shoring
a. Shoring of adjacent structures.
b. Shoring and sheathing as needed for soil and depth.
c. Public roads and sidewalks supported and protected.
d. Materials not too close to edge of excavations.
e. Lighting at night.
f. 'Water' controlled.
g. Equipment at safe distance from edge.
h. Ladders or stairs provided where needed.
i. Equipment ramps adequate, slope not too great.
j. Frequent inspection.

21. Pipelines
a. Shoring and bracing as needed.
b. Equipment in working order.
c. Proper access to deep trenches.

22. Demolition
a. Operations planned ahead.
b. Shoring of adjacent structures.
c. Material chutes.
d. Sidewalk and other public protection.
e. Clear operating space for trucks and other vehicles.
f. Adequate access ladders or stairs.
g. Regular supervision.

23. Pile Driving
a. Proper storage procedures.
b. Unloading only by properly instructed and experienced workmen.
c. Steam lines, slings, etc., in operating condition.
d. Piledriving rigs properly supported.
e. Ladders on frames.
f. Care observed by 'top' man.
g. Cofferdams maintained and inspected.
h. Adequate pumping available.
i. Proper bracing and ties in current.

24. Tunnels
a. Adequate ventilation.
b. Adequate lighting.
c. Good housekeeping.
d. Tunnel supports.
e. Electrical lines.
f. Operation of hauling equipment.
g. Proper personal protection.
h. Gas detectors used.
i. Proper transportation of personnel.
j. Control of blasting operations.
k. Dust protection.
l. Drilling safety observed.

25. Shafts
a. Ladders, stairs adequate and safe.
b. Top of shaft barricaded, toeboards provided.
c. Adequate lighting.
d. Ventilation.
e. Inspection and maintenance of elevators and hoists.
f. Signals being used.
g. Shoring and bracing.

26. ***Compressed Air Work***
   a. Time limits for pressure and exertion observed.
   b. Decompression chamber provided.
   c. Gauges operating.
   d. Exhaust valves.
   e. Signals and telephone connection.
   f. Ventilation.
   g. Sanitary facilities.
   h. Medical Doctor at site.
27. ***Boilers***
   a. State and local regulations and codes observed.
   b. Pressure gauges checked and operating.
   c. Equipment in good working order.
28. ***Pressure Vessels***
   a. Gauges operating and checked regularly.
   b. Equipment in sound working condition.
   c. Connections properly made.
29. ***Explosives***
   a. Qualified operators.
   b. Proper transport vehicles.
   c. Local laws and regulations observed.
   d. Storage magazines constructed per regulations or as recommended.
   e. Experienced personnel handling explosives at all times.
   f. Cases opened with wooden tools only.
   g. NO SMOKING posted and observed where appropriate.
   h. Detonators tested before each shot.
   i. All personnel familiar with signals, and signals properly used at all times.
   j. Inspection after each shot.
   k. Proper protection and accounting for all explosives at all times.
   l. Proper disposition of wrappings, waste, and scrap.
   m. Advise residents nearby of blasting cap danger.
   n. Check radio frequency hazards.
30. ***Inflammable Gases and Liquids***
   a. All containers clearly identified.
   b. Proper storage practices observed.
   c. Fire hazards checked.
   d. Proper storage temperatures and protection.
   e. Proper types and number of extinguishers nearby.
   f. Carts for moving cylinders.
31. ***Welding and Cutting***
   a. Qualified operators.
   b. Screens and shields.
   c. Goggles, gloves, clothing.
   d. Equipment in operating condition.
   e. Electrical equipment 'grounded'.
   f. Power cables protected and in good repair.
   g. Fire extinguishers of proper type nearby.
   h. Inspection for fire hazards.
   i. Inflammable materials protected.
   j. Gas cylinders chained upright.
   k. Gas lines protected and in good condition.
32. ***Steel Erection***
   a. Safety nets.
   b. Hard hats, safety shoes, gloves.
   c. Taglines for tools.
   d. Fire hazards at rivet forge and welding operations.
   e. Floor opening covered and barricaded.
   f. Ladders, stairs, or other access provided.
   g. Hoisting apparatus checked.
33. ***Concrete Construction***
   a. Forms properly installed and braced.
   b. Adequate shoring, plumbed and crossbraced.
   c. Shoring remains in place until strength is attained.
   d. Proper curing period and procedures.
   e. Check heating devices/cooling devices (adverse weather)
   f. Mixing and transport equipment supported, and traffic planned and routed.
   g. Adequate runways.
   h. Protection from cement dust.
   i. Hardhats, safety shoes, shirts covering skin.
   j. Nails and stripped-form material removed from area.
34. ***Masonry***
   a. Proper scaffolding.
   b. Masonry saws properly equipped, dust protection provided.
   c. Safe hoisting equipment.
35. ***Highway Construction***
   a. Laws and ordinances observed.
   b. Competent flagmen properly dressed, instructed, posted.
   c. Adequate warning signs and markers.
   d. Equipment not blocking right-of-way.
   e. Traffic control through construction site.
   f. Adequate marking and maintenance of detours.
   g. Dust control.
   h. Adequate lighting.
36. ***Quarries and Gravel Pits***
   a. Proper signals and safety procedures for blasting.
   b. Screening plants operated safely.
   c. Traffic routed and controlled.
37. ***Prestressing***
   a. Approved execution drawings and details of anchorages, cables, tendons, ducting, groutpipes, tendon profiles, type of stressing jacks, limiting values of Jacking Forces, tendon extensions, expected slip at anchors (pull-in), adjustment of extensions with variation in Modulus of Elasticity value, required concrete strengths at various stages

of prestressing etc.

b. Instructions: 'what to do' if acceptable extensions or jack pressures not attained at appropriate conjoint values.

c. Check on long term slips (24 hr)

d. Grouting instructions (cleaning, mix, pressure procedure).

e. Protection against heating, rusting, pitting, oiling, etc.

f. Adequately experienced crew, conversant with all procedures and cautions in prestress application.

g. Maintenance of jacks, oil baths, gauges, pressure hoses, joints and connections in lines, master wedges, including periodical calibration of pressure-gauges.

h. Proper storage of tendons, ducts, anchorages and equipment.

NOTE: Also see the relevant details in the chapter on "Construction considerations" in the author's other book: *"Concrete Bridge Practice ... Construction, Maintenance and Rehabilitation"*.

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224R-80	Control of Cracking in Concrete Structures
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309.2R-82	Identification and Control of Consolidation-Related Surface Defects in Formed Concrete
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345-82	Standard Practice for Concrete Highway Bridge Deck Construction
347-78	(Reaffirmed 1984) Recommended Practice for Concrete Formwork
350R-83	Concrete Sanitary Engineering Structures
503R-80	Use of Epoxy Compounds with Concrete
503.3-79	Standard Specification for Producing a Skid-Resistant Surface on Concrete by the Use of Multi-Component Epoxy System
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C 881-78 Standard Specifications for Epoxy-Resin-Base Bonding Systems for Concrete

23. ***American Association of State Highway and Transportation Officials (AASHTO)***

... Various Design Specifications

APPENDIX 1

Metric Units and Conversion Factors

Subject	Imperial Unit	S.I. Unit	Symbol	Conversion factor
Length	mile	kilometre	km	1 mile = 1.609 km
	yard	metre	m	1 yd = 0.914 m
	foot	metre or millimetre	m or mm	1 ft = 0.305 m = 304.8 mm
	inch	millimetre	mm	1 in = 25.400 mm
Area	square mile	square kilometre	km^2	1 $mile^2$ = 2.590 km^2
	acre	square kilometre or hectare	km^2 or ha	1 acre = 0.004 km^2 = 0.405 ha
	square yard	*square metre*	*m^2*	*1 yd^2 = 0.836 m^2*
	square foot	square metre	m^2	1 ft^2 = 0.093 m^2
	square inch	square millimetre	mm^2	1 in^2 = 645.16 mm^2
Volume	cubic yard	cubic metre	m	1 yd^3 = 0.765 m^3
	cubic foot	cubic metre	m^3	1 ft^3 = 0.0283 m^3
	cubic inch	cubic millimetre	mm^3	1 in^3 = 16.387.1 mm^3
Capacity	gallon	litre	litre	1 gal = 4.546 litre
Mass of materials	*ton*	*tonne*	*tonne*	*1 ton = 1.016 tonne*
	hundred weight	kilogramme	kg	1 cwt = 50.802 kg
	pound	kilogramme	kg	1 lb = 0.454 kg
	ounce	gramme	g	1 oz = 28.350 g
Density	pounce per cubic foot	kilogramme per cubic metre	kg/m	1 lb/ft^3 = 16.019 kg/m^3
	pound per cubic yard			*1 lb/yd^3 = 0.593 kg/m^3*
Force	pound force	newton	N	1 lbf = 4.448 N
	ton force	kilonewton	kN	1 tonf = 9.964 kN
	pound force per foot	newton per metre	N/m	1 lb/ft = 14.591 N/m
	ton force per foot	kilonewton per metre	kN/m	1 ton/ft = 32.690 kN/m

Subject	Imperial Unit	S.I. Unit	Symbol	Conversion factor
Pressure	pound force per square foot	newton per square metre	N/m^2	1 lbf/ft^2 = 47.880 N/m^2
	pound force per square inch	newton per square millimetre	N/mm^2	1 lbf/in^2 = 0.00689 N/mm^2
	ton force per square foot	kilonewton per square metre	kN/m^2	1 ton/ft^2 = 107.250 kN/m^2
	ton force per square inch	kilonewton per square millimetre	kN/mm^2	1 $tonf/in^2$ = 0.0154 kN/mm^2
Stress	pound force per square inch	newton per square millimetre	N/mm^2	1 lbf/in^2 = 0.00689 N/mm^2
	ton force per square inch	newton per square millimetre	N/mm^2	1 $tonf/in^2$ = 15.444 N/mm^2
	ton force per square foot	kilonewton per square metre	kN/m^2	1 $tonf/ft^2$ = 107.250 kN/m^2
Modulus of elasticity	pound force per square inch	newton per square millimetre	N/mm^2	1 lbf/in^2 = 0.006.89 N/mm^2
Bending	pound force inch	newton millimetre or newton metre	N mm or N m	1 lbf in = 112.985 N mm = 0.1113 N m
	pound force foot	newton metre	N m	1 lbf ft = 1.356 N m
	ton force foot	kilonewton metre	kN m	1 tonf ft = 3 037 kN m
Section modulus	in^3	mm^3	mm^3	1 in^3 = 16 386 mm^3 —
Second moment of area	in^4	mm^4	mm^4	1 in^4 = 416 210 mm^4

DECIMALS OF AN INCH
For each 64th of an inch
With Millimetre Equivalents

Fraction	1/64ths	Decimals	Millimetres (Approx.)	Fraction	1/64ths	Decimals	Millimetres (Approx.)
...	1	.015625	0.397	...	33	.515625	13.097
1/32	2	.03125	0.794	17/32	34	.53125	13.494
...	3	.046875	1.191	...	35	.546875	13.891
1/16	4	.0625	1.588	9/16	36	.5625	14.288
...	5	.078125	1.984	...	37	.578125	14.684
3/32	6	.09375	2.381	19/32	38	.59375	15.081
...	7	.109375	2.778	...	39	.609375	15.478
1/8	8	.125	3.175	5/8	40	.625	15.875
...	9	.140625	3.572	...	41	.640625	16.272
5/32	10	.15625	3.969	21/32	42	.65625	16.669
...	11	.171875	4.366	...	43	.671875	17.066
3/16	12	.1875	4.763	11/16	44	.6875	17.463
...	13	.203125	5.159	...	45	.703125	17.859
7/32	14	.21875	5.556	23/32	46	.71875	18.256
...	15	.234375	5.953	...	47	.734375	18.653
1/4	16	.250	6.350	3/4	48	.750	19.050
...	17	.265625	6.747	...	49	.765625	19.447
9/32	18	.28125	7.144	25/32	50	.78125	19.844
...	19	.296875	7.541	...	51	.796875	20.241
5/16	20	.3215	7.938	13/16	52	.8125	20.638
...	21	.328125	8.334	...	53	.828125	21.034
11/32	22	.34375	8.731	27/32	54	.84375	21.431
...	23	.359375	9.128	...	55	.859375	21.828
3/8	24	.375	9.525	7/8	56	.875	22.225
...	25	.390625	9.922	...	57	.890625	22.622
13/32	26	.40625	10.319	29/32	58	.90625	23.019
...	27	.421875	10.716	...	59	.921875	23.416
7/16	28	.4375	11.113	15/16	60	.9375	23.813
...	29	.453125	11.509	...	61	.953125	24.209
15/32	30	.46875	11.906	31/32	62	.96875	24.606
...	31	.484375	12.303	...	63	.984375	25.003
1/2	32	.500	12.700	1	64	1.000	25.400

APPENDIX 2

Some General Data

SI PREFIXES										
Multiplication Factor									Prefix	Symbol
1	000	000	000	000	000	000	=	10^{18}	exa	E
	1	000	000	000	000	000	=	10^{15}	peta	P
		1	000	000	000	000	=	10^{12}	tera	T
			1	000	000	000	=	10^{9}	giga	G
				1	000	000	=	10^{6}	mega	M
					1	000	=	10^{3}	kilo	k
						100	=	10^{2}	hecto	h
						10	=	10^{1}	deka	da
						0.1	=	10^{-1}	deci	d
						0.01	=	10^{-2}	centi	c
						0.001	=	10^{-3}	milli	m
					0.000	001	=	10^{-6}	micro	μ
				0.000	000	001	=	10^{-9}	nano	n
			0.000	000	000	001	=	10^{-12}	pico	p
		0.000	000	000	000	001	=	10^{-15}	femto	f
	0.000	000	000	000	000	001	=	10^{-18}	atto	a

Temperature Conversion Tables

$$\text{deg C} = \frac{\text{deg F} - 32}{1.8}, \text{ deg F} = 1.8 \text{ deg C} + 32$$

deg C	deg F		deg F	deg C
−40	−40.0		−40	−40.0
−35	−31.0		−30	−34.4
−30	−22.0		−20	−28.8
−25	−13.0		−10	−23.3
−20	−4.0		0	−17.7
−15	+5.0		+10	−12.2
−10	14.0		20	−6.6
−5	23.0		30	−1.1
0	32.0		40	+4.4
+5	41.0		50	10.0
10	50.0		60	15.5
15	59.0		70	21.1
20	68.0		80	26.6
25	77.0		90	32.2
30	86.0		100	37.7
35	95.0		110	43.2
40	104.0		120	48.8
45	113.0		130	54.4
50	122.0		140	60.0

(Contd.)

deg C	deg F		deg F	deg C
55	131.0		150	65.5
60	140.0		160	71.1
65	149.0		170	76.6
70	158.0		180	82.2
75	167.0		190	87.7
80	176.0		200	93.3
			210	98.8
90	194.0		220	104.4
100	212.0			
120	248.0		240	115.5
140	284.0		260	126.6
160	320.0		280	137.7
180	356.0		300	148.9
200	392.0		400	204.4
300	572.0		500	260.0
400	752.0		600	315.6
500	932.0		800	426.7

An *approximate* method of converting temperatures which is correct to within 5 deg F, between 0 and 100 deg F, is—

$$\text{deg C} = \frac{\text{deg F} - 30}{2}$$

Shrinkage of Concrete

The shrinkage of concrete made with quartz aggregate due to ageing is approximately—

Time	*Shrinkage (per cent)*
After 28 days	0.025
After 3 months	0.035
After 12 months	0.050

Conversion Factors

Note that it is strict continental practice to use lower case letters always for unit unless the unit derives from a name, e.g. Watt—W, Fahrenheit—F, and kilowatt—kW.

To convert	Into	Multiply by	Reciprocal
Linear			
Inches	centimetres	2.54	0.3972
Inches	metres	0.0254	39.3701
Feet	centimetres	30.4799	0.0328
Yards	metres	0.9144	1.0936
Miles (5,280 ft)	kilometres	1.6093	0.62137
Chains	miles	0.0215	80.0
Furlongs	miles	0.125	8.0
Area			
Acres	square chains	10.0	0.1
Acres	hectares	0.4047	2.471
Acres	square miles	0.0016	640.0
Acres	square yards	4,840.0	0.0002
Square inches	square centimetres	6.4516	0.155
Square feet	square decimetres	9.2903	0.1076
Square yards	square metres	0.8361	1.196
Square chains	square metres	404.7	0.0025
Square chains	square yards	484.0	0.0021

(Contd.)

To convert	Into	Multiply by	Reciprocal
Square feet	square metres	0.0929	10.764
Square yards	hectares	0.00008	11,960.0
Square inches	square metres	0.0000645	1,550.0
Volume			
Cubic inches	cubic centimetres	16.387	0.061
Cubic feet	cubic metres	0.02832	35.32
Cubic feet	litres	28.317	0.0353
Cubic yards	cubic metres	0.7645	1.308
Cubic inches	cubic decimetres	0.01639	61.024
Miscellaneous			
British Thermal Units	calories	252.0	0.004
British Thermal Units	kilocalories	0.252	3.968
Btu per square foot	kilocalories per square metre	2.713	0.369
Atmospheres	pounds per square inch	14.7	0.0679
Feet of water	pounds per square inch	0.4335	2.3067
Metres of water	pounds per square inch	1.418	0.7031
Inches of mercury	pounds per square inch	0.4912	2.036
Millimetres of mercury	pounds per square inch	0.0193	51.71
Kilowatts	horsepower	1.34	0.746
Miles per hour	feet per second	1.4667	0.6818
Weight			
Ounce (Avoirdupois)	drams	16	0.0625
Ounce (Av.)	grammes	28.35	0.0353
Ounce (Troy)	grammes	31.1035	0.03215
Pound (16 oz)	kilogrammes	0.45359	2.2046
Hundred weights (112 lb)	kilogrammes	50.8	0.0197
Tons (2,240 lb)	tonnes (= 1,000 kg)	1.016	0.9842
Hundred weights	quintal (= 100 kg)	0.508	1.968
Pounds	tons	0.0004	2,240.0
Fluid			
Cubic feet	litres	28.317	0.0353
Gallons (Imp.)	litres	4.546	0.22
Gallons	cubic feet	0.1605	6.232
Litres	cubic centimetres	1,000.0	0.001
Pints	litres	1.7598	0.568
Gallons (Imp.)	gallons (U.S.)	1.2	0.8327
Circular			
Degrees	radians	0.0175	57.29
Compound			
lb/lineal foot	kg/lineal metre	1.488	0.6711
ton/lineal foot	kg/lineal metre	3,333.0	0.0003
lb/in^2	kg/cm^2	0.0703	14.223
lb/in^2	kg/mm^2	1.575	0.635
lb/ft^2	kg/m^2	4.883	0.2048
lb/in^2	ton/ft^2	0.0643	15.55
lb/ft^2	kg/m^3	16.019	0.9625
lb/yd^3	kg/m^3	0.5917	1.686
lb/gal	kg/l	0.0998	10.0166
ft-lb	kg-l	0.1382	7.233
ft-ton	tonne-m	0.3096	3.229

(Contd.)

To convert	Into	Multiply by	Reciprocal
Horsepower	Force de Cheval	1.0139	0.9863
ft^3/sec	m^3/hr	101.94	0.098
ft^3/hr	l/sec	0.0787	127.2
lb/hp	kg/Force de Cheval	0.447	2.237
Grain/gal	g/l	0.0143	70.15

Figures Relating to Water

1 cubic foot of fresh water weighs 62.4 lb
1 cubic foot of sea water weighs 64.0 lb
1 British gallon of fresh water weighs 10.0 lb
1 United States gallon = 0.83 British gallon
1 litre of fresh water = 0.22 British gallon
1 cubic foot of fresh water = 6.24 British gallons
Head of fresh water in feet × 0.433 = pressure (lb/in.^{2})
Head of sea water in feet × 0.444 = pressure (lb/in.^{2})

Weights of Constructional Materials

Miscellaneous materials					N/m^2	lb/ft^2
				Clay floor tiles	575	12
				Pavement lights	1,200	25
		kN/m^3	lb/ft^3	Damp-proof course	48	1
	Tarmacadam	22.6	144		N/m^2 per mm thickness	lb/ft^2 per in thickness
	Macadam (waterbound)	25.1	160			
	Snow: compact	2.4 to 8.0	15 to 50	Felt (insulating)	1.9	1
	loose	0.8 to 1.9	5 to 12			
				Paving slabs (stone)	26.4	14
	Vermiculite (aggregate)	0.8	5	Granite sets	28.3	15
	Terracotta	20.8	132	Asphalt	22.6	12
	Glass	26.7	170	Rubber paving	15.1	8
	Cork: granular	1.2	7½	Polyvinylchloride	19 (av.)	10 (av.)
	compressed	3.8	24	Glass-fibre (forms)	1.9	1
Timber	General	7.9(av.)	50(av.)	Wooden boarding & blocks	N/m^2 per mm	lb/ft^2 per in
	Douglas fir	4.7	30	Softwood	4.7	2½
	Yellow pine, spruce	4.7	30	Hardwood	7.5	4
	Pitch pine	6.6	42	Hardboard	10.4	5½
	Larch, elm	5.5	35	Chipboard	7.5	4
	Oak (English)	7.1 to 9.4	45 to 60	Plywood	6.1	3¼
	Teak	6.3 to 8.6	40 to 55	Blockboard	4.7	2½
	Jarrah	9.4	60	Fibreboard	2.8	1½
	Greenheart	10.2 to 11.8	65 to 75	Wood-wool	5.7	3
	Quebracho	12.6	80	Plasterboard	9.4	5
				Weather boarding	3.8	2
Stone and other materials	Natural stone (solid)				kN/m^3	lb/ft^3
	Granite	25.1 to 28.7	160 to 183	Stone rubble (packed)	22.0	140
	Limestone: Bath stone	20.4	130	Quarry waste	14.1	90
	Marble	26.7	170	Hardcore (consolidated)	18.9	120
	Portland stone	22.0	140	All-in aggregate	19.6	125
	Sandstone	22.0 to 23.6	140 to 150			
	Slate	28.3	180			

(Contd.)

Metals, steel constructions, etc.		kN/m³	lb/ft³			
	Iron: cast	70.7	450	Structural steelwork	Net weight of member	
	wrought	75.4	480	rivetted	+ 10% for cleats, rivets, bolts, etc.	
	ore: general	23.6	150			
	(crushed) Swedish	36.1	230	welded	+ 1 1/4% to 2 1/2% for welds, etc.	
	Steel (see also below)	77.0	490	Rolled sections: beams	+ 2 1/2%	
	Copper: cast	85.6	545	stanchions	+ 5% (extra for caps & bases)	
	wrought	87.7	558	Plate-webgirders	+ 10% for rivets or welds; stiffeners, etc.	
	Brass	83.3	530			
	Bronze	87.7	558			
	Aluminium	27.2	173			
	Lead	111.0	707			
	Zinc (rolled)	70.0	446			
				Steel stairs: industrial	N/m	lb/ft
		g/mm²	lb/in²	type 1m or 3ft wide	820	56
		per metre	per foot	Steel tubes:		
				50mm or 2 in bore	45 to 60	3 to 4
	Steel bars	7.85		Gas piping:		
		(7.85T/m³)	3.4	20mm or 3/4 in	18	1 1/4
	Rail tracks				kN/m of track	lb/ft of track
	standard gauge main lines	Bull-head rails, chairs, transverse timber (softwood) sleepers, etc.			2.4	165
		Flat-bottom rails, transverse prestressed concrete sleepers, etc.			4.1	280
		Add for electric third rail			0.5	35
		Add for crushed stone ballast			25.5	1,750
					kN/m²	lb/ft²
		Overall average weight: rails, connections, sleepers, ballast, etc.			7.2	150
					kN/m of rail	lb/ft of rail
		Bridge rails, longitudinal timber sleepers, etc.			1.1	75

To convert values in kN to values in kg multiply by 102.

Normal Concrete:	kN/m³	kg/m³
—plain	22	2250
—reinforced	24	2450
—prestressed reinforced	25	2550

COEFFICIENTS OF EXPANSION (α per 100°)

The coefficient of linear expansion (α) is the change in length, per unit of length, for a change of one degree of temperature. The coefficient of surface expansion is approximately two times the linear coefficient, and the coefficient of volume expansion, for solids, is approximately three times the linear coefficient.

A bar, free to move, will increase in length with an increase in temperature and will decrease in length with a decrease in temperature. The change in length will be $\alpha t l$ where α is the coefficient of linear expansion, t the change in temperature, and l the length. If the ends of a bar are fixed, a change in the temperature (t) will cause a change in the unit stress of $E\alpha t$, and in the total stress of $AE\alpha t$, where A is the cross-sectional area of the bar and E the modulus of elasticity.

The following table gives the coefficient of linear expansion for *100°*, or *100 times the value indicated above.*

EXAMPLE: A piece of medium steel is exactly 40 feet long at 60°F. Find the length at 90°F assuming the ends free to move.

$$\text{Change of length} = \alpha t l = \frac{.00065 \times 30 \times 40}{100} = .0078 \text{ ft}$$

The length at 90°F is 40.0078 ft.

EXAMPLE: A piece of medium steel is exactly 40 feet long and the ends are fixed. If the temperature increases 30°F. What is the resulting change in the unit stress?

Change in unit stress = $E\alpha t$ =

$$\frac{29,000,000 \times .00065 \times 30}{100} = \begin{array}{c} 5655 \text{ lb. per sq. in.} \\ (397.6 \text{ kg./cm}^2) \end{array}$$

COEFFICIENTS OF EXPANSION FOR 100 DEGREES = 100α

Materials	Linear expansion		Materials	Linear expansion	
	Centigrade	Fahrenheit		Centigrade	Fahrenheit
METALS AND ALLOYS			STONE AND MASONRY		
Aluminium, wrought	.00231	.00128	Ashlar masonry	.00063	.00035
Brass	.00188	.00104	Brick masonry	.00061	.00034
Bronze	.00181	.00101	Cement, Portland	.00126	.00070
Copper	.00168	.00093	Concrete	.00099	.00055
Iron, cast, gray	.00106	.00059	Granite	.00080	.00044
Iron, wrought	.00120	.00067	Limestone	.00076	.00042
Iron, wire	.00124	.00069	Marble	.00081	.00045
Lead	.00286	.00159	Plaster	.00166	.00092
Magnesium, various alloys	.0029	.0016	Rubble masonry	.00063	.00035
Nickel	.00126	.00070	Sandstone	.00097	.00054
Steel mild	.00117	.00065	Slate	.00080	.00044
Steel, stainless, 18–8	.00178	.00099			
Zinc, rolled	.00311	.00173			
TIMBER			TIMBER		
Fir (Parallel to fibre)	.00037	.00021	Fir (perpendicular to fibre)	.0058	.00032
Maple (Parallel to fibre)	.00064	.00036	Maple (perpendicular to fibre)	.0048	.00027
Oak (Parallel to fibre)	.00049	.00027	Oak (perpendicular to fibre)	.0054	.00030
Pine (Parallel to fibre)	.00054	.00030	Pine (perpendicular to fibre)	.0034	.00019

EXPANSION OF WATER

If Maximum Density = 1 unit at 4°C, then

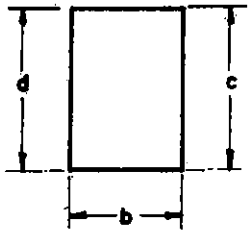
C°	Volume	C°	Volume	C°	Volume	C°	Volume	C°	Volume	C°	Volume
0	1.000126	10	1.000257	30	1.004234	50	1.011877	70	1.022384	90	1.035829
4	1.000000	20	1.001732	40	1.007627	60	1.016954	80	1.029003	100	1.043116

APPENDIX 3

Properties of Geometric Sections and Shapes

PROPERTIES OF GEOMETRIC SECTIONS

RECTANGLE
Axis of moments on base



$$A = bd$$

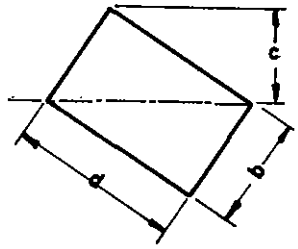
$$c = d$$

$$I = \frac{bd^3}{3}$$

$$S = \frac{bd^2}{3}$$

$$r = \frac{d}{\sqrt{3}} = .577350\ d$$

RECTANGLE
Axis of moments on diagonal



$$A = bd$$

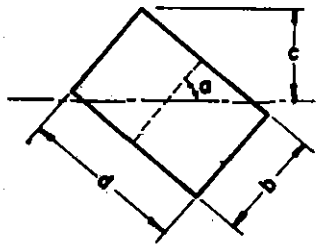
$$c = \frac{bd}{\sqrt{b^2 + d^2}}$$

$$I = \frac{b^3 d^3}{6(b^2 + d^2)}$$

$$S = \frac{b^2 d^2}{6\sqrt{b^2 + d^2}}$$

$$r = \frac{bd}{\sqrt{6(b^2 + d^2)}}$$

RECTANGLE
Axis of moments any line through centre of gravity



$$A = bd$$

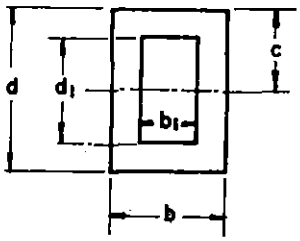
$$c = \frac{b \sin a + d \cos a}{2}$$

$$I = \frac{bd(b^2 \sin^2 a + d^2 \cos^2 a)}{12}$$

$$S = \frac{bd(b^2 \sin^2 a + d^2 \cos^2 a)}{6(b \sin a + d \cos a)}$$

$$r = \sqrt{\frac{b^2 \sin^2 a + d^2 \cos^2 a}{12}}$$

HOLLOW RECTANGLE
Axis of moments through centre



$$A = bd - b_1 d_1$$

$$c = \frac{d}{2}$$

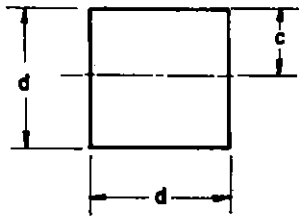
$$I = \frac{bd^3 - b_1 d_1^3}{12}$$

$$S = \frac{bd^3 - b_1 d_1^3}{6d}$$

$$r = \sqrt{\frac{bd^3 - b_1 d_1^3}{12A}}$$

$$Z = \frac{bd^2}{4} - \frac{b_1 d_1^2}{4}$$

SQUARE
Axis of moments through centre



$$A = d^2$$

$$c = \frac{d}{2}$$

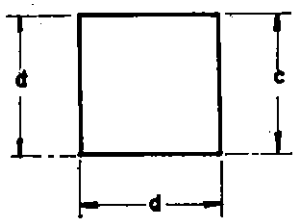
$$I = \frac{d^4}{12}$$

$$S = \frac{d^3}{6}$$

$$r = \frac{d}{\sqrt{12}} = .288675\ d$$

$$Z = \frac{d^3}{4}$$

SQUARE
Axis of moments on base



$$A = d^2$$

$$c = d$$

$$I = \frac{d^4}{3}$$

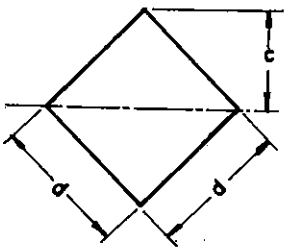
$$S = \frac{d^3}{3}$$

$$r = \frac{d}{\sqrt{3}} = .577350\ d$$

PROPERTIES OF GEOMETRIC SECTIONS *(Contd.)*

SQUARE

Axis of moments on diagonal



$$A = d^2$$

$$c = \frac{d}{\sqrt{2}} = .707107\ d$$

$$I = \frac{d^4}{12}$$

$$S = \frac{d^3}{6\sqrt{2}} = .117851\ d^3$$

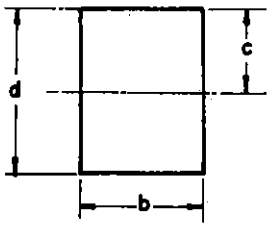
$$r = \frac{d}{\sqrt{12}} = .288675\ d$$

$$Z = \frac{2c^3}{3} = \frac{d^3}{3\sqrt{2}}$$

$$= .235702\ d^3$$

RECTANGLE

Axis of moments through centre



$$A = bd$$

$$c = \frac{d}{2}$$

$$I = \frac{bd^3}{12}$$

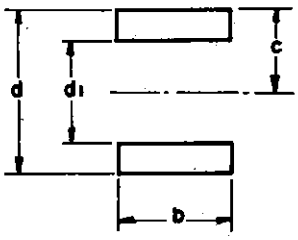
$$S = \frac{bd^2}{6}$$

$$r = \frac{d}{\sqrt{12}} = .288675\ d$$

$$Z = \frac{bd^2}{4}$$

EQUAL RECTANGLES

Axis of moments through centre of gravity



$$A = b(d - d_1)$$

$$c = \frac{d}{2}$$

$$I = \frac{b(d^3 - d_1^3)}{12}$$

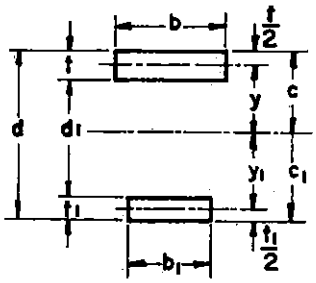
$$S = \frac{b(d^3 - d_1^3)}{6d}$$

$$r = \sqrt{\frac{d^3 - d_1^3}{12(d - d_1)}}$$

$$Z = \frac{b}{4}(d^2 - d_1^2)$$

UNEQUAL RECTANGLES

Axis of moments through centre of gravity



$$A = bt + b_1t_1$$

$$c = \frac{1/2bt^2 + b_1t_1(d - 1/2t_1)}{A}$$

$$I = \frac{bt^3}{12} + bty^2 + \frac{b_1t_1^3}{12} + b_1t_1y_1^2$$

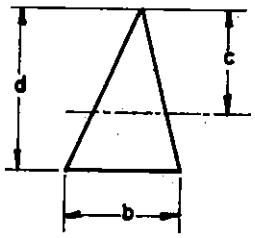
$$S = \frac{I}{c} \quad S_1 = \frac{I}{c_1}$$

$$r = \sqrt{\frac{I}{A}}$$

$$Z = \frac{A}{2}\left[d - \left(\frac{t + t_1}{2}\right)\right]$$

TRIANGLE

Axis of moments through centre of gravity



$$A = \frac{bd}{2}$$

$$c = \frac{2d}{3}$$

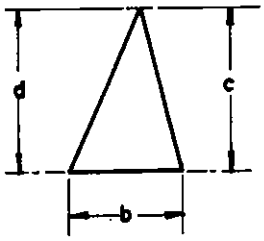
$$I = \frac{bd^3}{36}$$

$$S = \frac{bd^2}{24}$$

$$r = \frac{d}{\sqrt{18}} = .235702\ d$$

TRIANGLE

Axis of moments on base



$$A = \frac{bd}{2}$$

$$c = d$$

$$I = \frac{bd^3}{12}$$

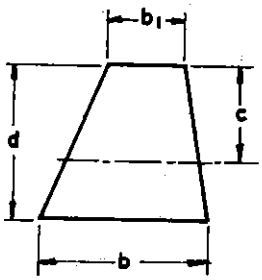
$$S = \frac{bd^2}{12}$$

$$r = \frac{d}{\sqrt{6}} = .408248\ d$$

PROPERTIES OF GEOMETRIC SECTIONS *(Contd.)*

TRAPEZOID

Axis of moments through centre of gravity



$$A = \frac{d(b + b_1)}{2}$$

$$c = \frac{d(2b + b_1)}{3(b + b_1)}$$

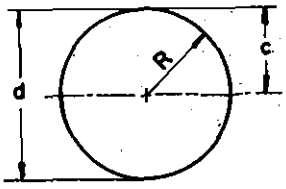
$$I = \frac{d^3(b^2 + 4bb_1 + b_1^2)}{36(b + b_1)}$$

$$S = \frac{d^2(b^2 + 4bb_1 + b_1^2)}{12(2b + b_1)}$$

$$r = \frac{d}{6(b + b_1)} \sqrt{2(b^2 + 4bb_1 + b_1^2)}$$

CIRCLE

Axis of moments through centre



$$A = \frac{\pi d^2}{4} = \pi R^2$$

$$= .785398\ d^2$$

$$= 3.141593 R^2$$

$$c = \frac{d}{2} = R$$

$$I = \frac{\pi d^4}{64} = \frac{\pi R^4}{4}$$

$$= .049087\ d^4$$

$$= .785398\ R^4$$

$$S = \frac{\pi d^3}{32} = \frac{\pi R^3}{4}$$

$$= .098175\ d^3$$

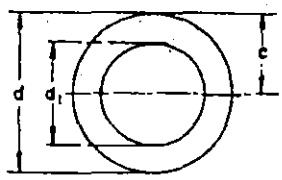
$$= .785398\ R^3$$

$$r = \frac{d}{4} = \frac{R}{2}$$

$$Z = \frac{d^3}{6}$$

HOLLOW CIRCLE

Axis of moments through centre



$$A = \frac{\pi(d^2 - d_1^2)}{4} = .785398\ (d^2 - d_1^2)$$

$$c = \frac{d}{2}$$

$$I = \frac{\pi(d^4 - d_1^4)}{64} = .049087\ (d^4 - d_1^4)$$

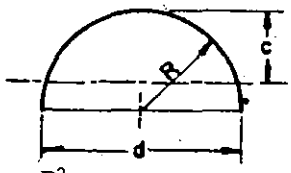
$$S = \frac{\pi(d^4 - d_1^4)}{32d} = .098175\ \frac{d^4 - d_1^4}{d}$$

$$r = \frac{\sqrt{d^2 + d_1^2}}{4}$$

$$Z = \frac{d^3}{6} - \frac{d_1^3}{6}$$

HALF CIRCLE

Axis of moments through centre of gravity



$$A = \frac{\pi R^2}{2} = .1.570796\ R^2$$

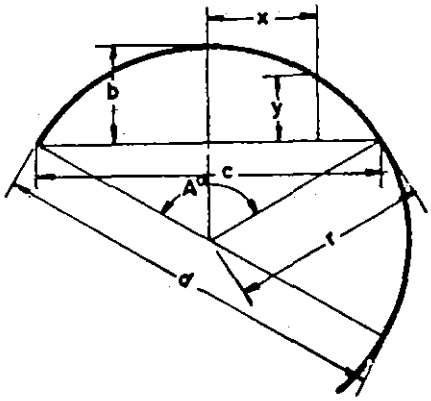
$$c = R\left(1 - \frac{4}{3\pi}\right) = .575587\ R$$

$$I = R^4\left(\frac{\pi}{8} - \frac{8}{9\pi}\right) = .109757\ R^4$$

$$S = \frac{R^3}{24}\frac{(9\pi^2 - 64)}{(3\pi - 4)} = .190687\ R^3$$

$$r = R\ \frac{\sqrt{9\pi^2 - 64}}{6\pi} = .264336\ R$$

PROPERTIES OF THE CIRCLE



PROPERTIES OF THE CIRCLE (*Contd.*)

Circumference = 6.28318 r = 3.14159 d

Diameter = 0.31831 circumference

Area = 3.14159 r^2

Arc $a = \dfrac{\pi r A^\circ}{180^\circ} = 0.017453\, rA^\circ$

Angle $A^\circ = \dfrac{180^\circ a}{\pi r} = 57.29578 \dfrac{a}{r}$

Radius $r = \dfrac{4b^2 + c^2}{8b}$

Chord $c = 2\sqrt{2br - b^2} = 2r \sin \dfrac{A}{2}$

Rise $b = r - 1/2\sqrt{4r^2 - c^2} = \dfrac{c}{2} \tan \dfrac{A}{4}$

$= 2r \sin^2 \dfrac{A}{4} = r + y - \sqrt{r^2 - x^2}$

$y = b - r + \sqrt{r^2 - x^2}$

$x = \sqrt{r^2 - (r + y - b)^2}$

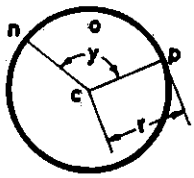
Diameter of circle of equal periphery as square = 1.27324 side of square

Side of square of equal periphery as circle = 0.78540 diameter of circle

Diameter of circle circumscribed about square = 1.41421 side of square

Side of square inscribed in circle = 0.70711 diameter of circle

CIRCULAR SECTOR



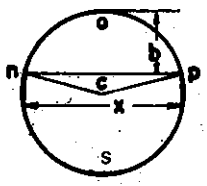
r = radius of circle y = angle ncp in degrees

Area of Sector $ncpo$ = 1/2 (length of arc $nop \times r$)

$= \text{Area of circle} \times \dfrac{y}{360}$

$= 0.0087266 \times r^2 \times y$

CIRCULAR SEGMENT



r = radius of circle x = chord b = rise

Area of segment nop = Area of sector $ncpo$

− Area of triangle ncp

$= \dfrac{(\text{Length of arc } nop \times r) - x(r - b)}{2}$

Area of Segment nsp = Area of Circle

− Area of segment nop

VALUES FOR FUNCTIONS OF π

$\pi = 3.14159265359, \log = 0.4971499$

$\pi^2 = 9.8696044, \log = 0.9942997$

$\dfrac{1}{\pi} = 0.3183099, \ \log = \bar{1}.5028501$

$\sqrt{\dfrac{1}{\pi}} = 0.5641896, \log = \bar{1}.7514251$

$\pi^3 = 31.0062767, \log = 1.4914496$

$\dfrac{1}{\pi^2} = 0.1013212, \ \log = \bar{1}.0057003$

$\dfrac{\pi}{180} = 0.0174533, \log = \bar{2}.2418774$

$\sqrt{\pi} = 1.7724539, \log = 0.2485749$

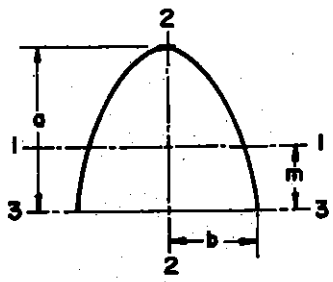
$\dfrac{1}{\pi^3} = 0.0322515, \ \log = \bar{2}.5085504$

$\dfrac{180}{\pi} = 57.2957795, \log = 1.7581226$

NOTE: Logs of fractions such as $\bar{1}.5028501$ and $\bar{2}.5085500$ may also be written 9.5028501−10 and 8.5085500−10 respectively.

PROPERTIES OF GEOMETRIC SECTIONS (*Contd.*)

PARABOLA



$A = \dfrac{4}{3}ab$

$m = \dfrac{2}{5}a$

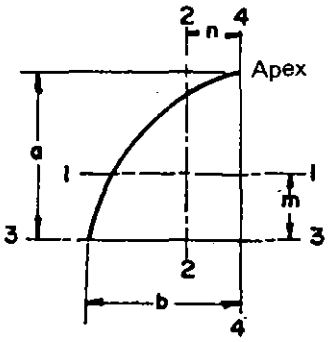
$I_1 = \dfrac{16}{175} \times a^3 b$

$I_2 = \dfrac{4}{15} \times ab^3$

$I_3 = \dfrac{32}{105} \times a^3 b$

PROPERTIES OF GEOMETRIC SECTIONS (*Contd.*)

HALF PARABOLA



$$A = \frac{2}{3}ab$$

$$m = \frac{2}{5}a$$

$$n = \frac{3}{8}b$$

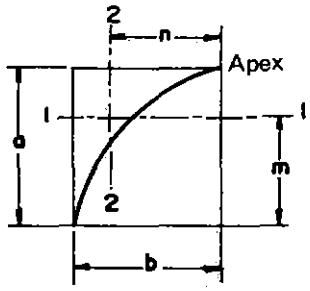
$$I_1 = \frac{8}{175} \times a^3b$$

$$I_2 = \frac{19}{480} \times a^3b$$

$$I_3 = \frac{16}{105} \times a^3b$$

$$I_4 = \frac{2}{15} \times ab^3$$

COMPLEMENT OF HALF PARABOLA



$$A = \frac{1}{3}ab$$

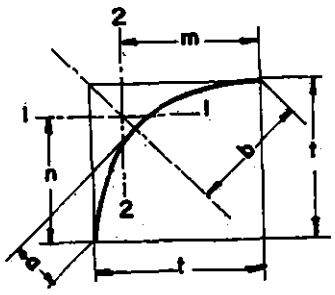
$$m = \frac{7}{10}a$$

$$n = \frac{3}{4}b$$

$$I_1 = \frac{37}{2100}a^3b$$

$$I_2 = \frac{1}{80}ab^3$$

PARABOLIC FILLET IN RIGHT ANGLE



$$a = \frac{t}{2\sqrt{2}}$$

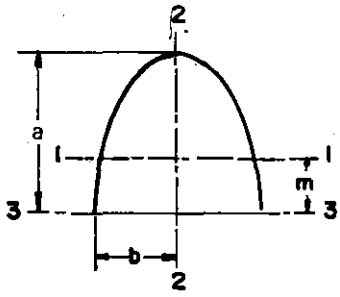
$$b = \frac{t}{\sqrt{2}}$$

$$A = \frac{1}{6}t^2$$

$$m = n = \frac{4}{5}t$$

$$I_1 = I_2 = \frac{11}{2100}t^4$$

* HALF ELLIPSE



$$A = \frac{1}{2}\pi ab$$

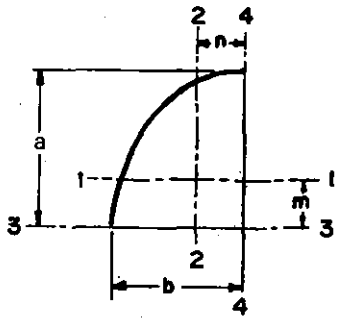
$$m = \frac{4a}{3\pi}$$

$$I_1 = a^3b\left(\frac{\pi}{8} - \frac{8}{9\pi}\right)$$

$$I_2 = \frac{1}{8}\pi ab^3$$

$$I_3 = \frac{1}{8}\pi a^3b$$

* QUARTER ELLIPSE



$$A = \frac{1}{4}\pi ab$$

$$m = \frac{4a}{3\pi}$$

$$n = \frac{4b}{3\pi}$$

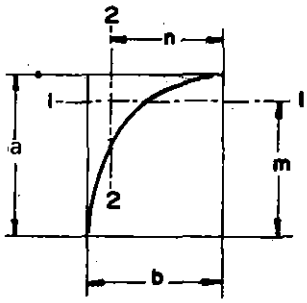
$$I_1 = a^3b\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$$

$$I_2 = ab^3\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$$

$$I_3 = \frac{1}{16}\pi a^3b$$

$$I_4 = \frac{1}{16}\pi ab^3$$

* ELLIPTIC COMPLEMENT



$$A = ab\left(1 - \frac{\pi}{4}\right)$$

$$m = \frac{a}{6\left(1 - \frac{\pi}{4}\right)}$$

$$n = \frac{b}{6\left(1 - \frac{\pi}{4}\right)}$$

(*Contd.*)

* To obtain properties of half circle, quarter circle and circular complement substitute $a = b = R$.

PROPERTIES OF GEOMETRIC SECTIONS (*Contd.*)

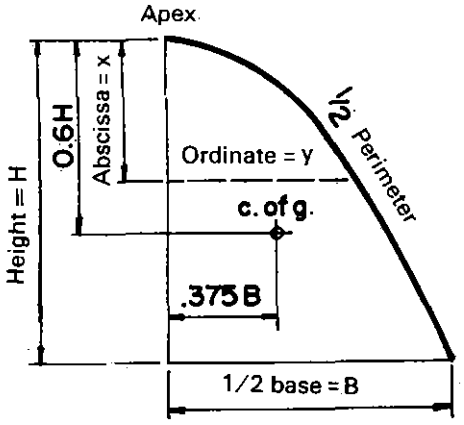
$$I_1 = a^3 b \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36(1 - \frac{\pi}{4})} \right)$$

$$I_2 = ab^3 \left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36(1 - \frac{\pi}{4})} \right)$$

PROPERTIES OF PARABOLA AND ELLIPSE

PARABOLA

When $H \div B = 0.1$ or less, approximate 1/2 perimeter $= \sqrt{B^2 + \frac{4}{3} \cdot H^2}$ or use formulae for circular arcs.

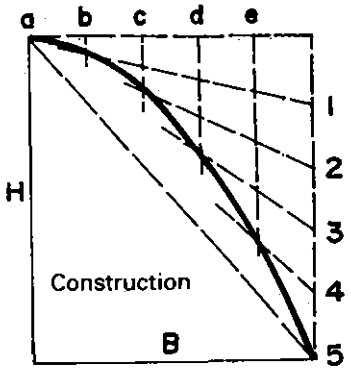


Area = 2/3HB

Perimeter P = $B^2 \div H$

$x = y^2 \div P$

$y = \sqrt{x P}$



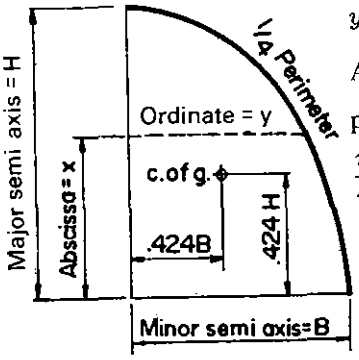
PROPERTIES OF PARABOLA AND ELLIPSE (*Contd.*)

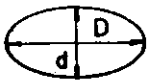
ELLIPSE

$(x^2 \div H^2) + (y^2 \div B^2) = 1$ $\quad x = (H \div B)\sqrt{B^2 - y^2}$

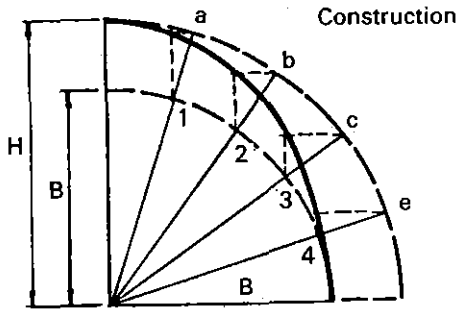
$y = (B \div H)\sqrt{H^2 - x^2}$

Approximate 1/4 perimeter = $\frac{\pi}{4}\sqrt{2(H^2 + B^2)}$

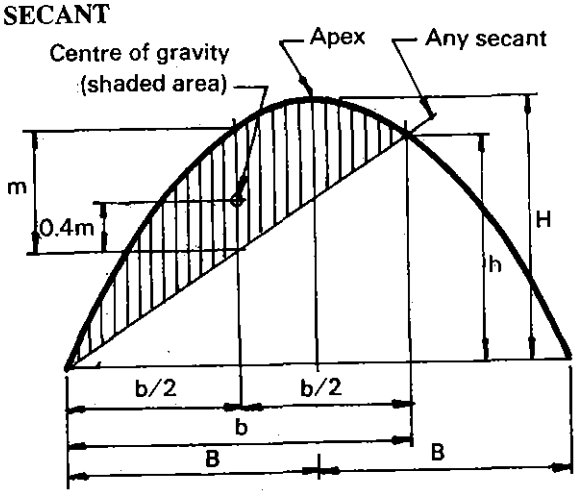




Area = .7854 Dd



AREA BETWEEN PARABOLIC CURVE AND SECANT



Length *b* may vary from O to 2B.

(Contd).

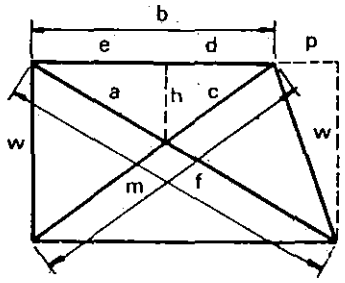
$$h = Hb\left(\frac{2B-b}{B^2}\right)$$

$$m = \frac{Hb^2}{4B^2}$$

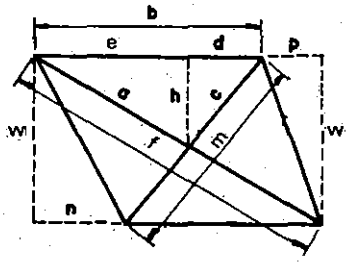
$$\text{Shaded area} = \frac{2}{3}bm$$

$$= \frac{Hb^3}{6B^2}$$

BRACING FORMULAS



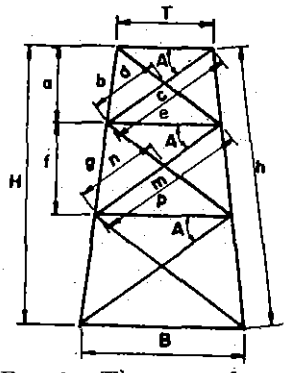
Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bw	m	$\sqrt{b^2+w^2}$
bp	d	$b^2 \div (2b+p)$
bp	e	$b(b+p) \div (2b+p)$
bfp	a	$bf \div (2b+p)$
bmp	c	$bm \div (2b+p)$
bpw	h	$bw \div (2b+p)$
afw	h	$aw \div f$
cmw	h	$cw \div m$



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bnw	m	$\sqrt{(b-n)^2+w^2}$
bnp	d	$b(b-n) \div (2b+p-n)$
bnp	e	$b(b+p) \div (2b+p-n)$
$bfnp$	a	$bf \div (2b+p-n)$
$bmnp$	c	$bm \div (2b+p-n)$
$bnpw$	h	$bw \div (2b+p-n)$
afw	h	$aw \div f$
cmw	h	$cw \div m$

Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bkv	m	$\sqrt{(b+k)^2+v^2}$
$bkpvw$	d	$bw(b+k) \div [v(b+p)+w(b+k)]$
$bkpvw$	e	$bv(b+p) \div [v(b+p)+w(b+k)]$
$bfkpvw$	a	$fbv \div [v(b+p)+w(b+k)]$
$bkmpvw$	c	$bmw \div [v(b+p)+w(b+k)]$
$bkpvw$	h	$bvw \div [v(b+p)+w(b+k)]$
afw	h	$aw \div f$
cmv	h	$cv \div m$

PARALLEL BRACING



$k = (\log B - \log T) \div$ no. of panels. Constant k plus the logarithm of any line equals the log of the corresponding line in the next panel below.

$$a = TH \div (T+e+p)$$

$$b = Th \div (T+e+p)$$

$$c = \sqrt{(1/2T + 1/2e)^2 + a^2}$$

$$d = ce \div (T+e)$$

$$\log e = k + \log T$$

$$\log f = k + \log a$$

$$\log g = k + \log b$$

$$\log m = k + \log c$$

$$\log n = k + \log d$$

$$\log p = k + \log e$$

The above method can be used for any number of panels.

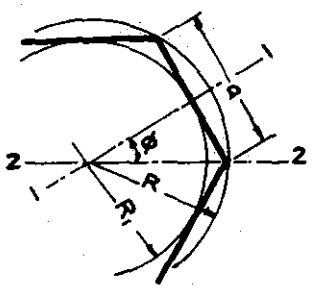
(Contd.)

In the formulas for "a" and "b" the sum in parentheses, which in the case shown is $(T+e+p)$, is always composed of all the horizontal distances except the base.

PROPERTIES OF GEOMETRIC SECTIONS AND STRUCTURAL SHAPES

REGULAR POLYGON

Axis of moments through centre



n = Number of sides

$$\phi = \frac{180°}{n}$$

$$a = 2\sqrt{R^2 - R_1^2}$$

$$R = \frac{a}{2\sin\phi}$$

$$R_1 = \frac{a}{2\tan\phi}$$

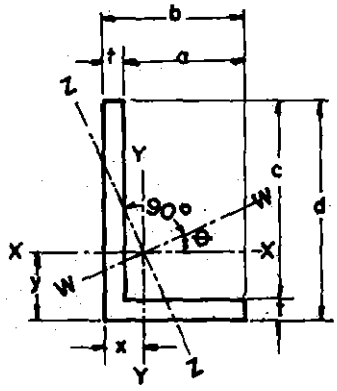
$$A = \frac{1}{4}na^2\cot\phi = \frac{1}{2}nR^2\sin 2\phi = nR_1^2\tan\phi$$

$$I_1 = I_2 = \frac{A(6R^2 - a^2)}{24} = \frac{A(12R_1^2 + a^2)}{48}$$

$$r_1 = r_2 = \sqrt{\frac{6R^2 - a^2}{24}} = \sqrt{\frac{12R_1^2 + a^2}{48}}$$

ANGLE

Axis of moments through centre of gravity



Z-Z is axis of minimum I

$$\tan 2\theta = \frac{2K}{I_y - I_x}$$

$$A = t(b+c), x = \frac{b^2 + ct}{2(b+c)}, y = \frac{d^2 + at}{2(b+c)}$$

K = Product of Inertia about X–X and Y–Y

$$= \mp\frac{abcdt}{4(b+c)}$$

$$I_x = \frac{1}{3}[t(d-y)^3 + by^3 - a(y-t)^3]$$

$$I_y = \frac{1}{3}[t(b-x)^3 + dx^3 - c(x-t)^3]$$

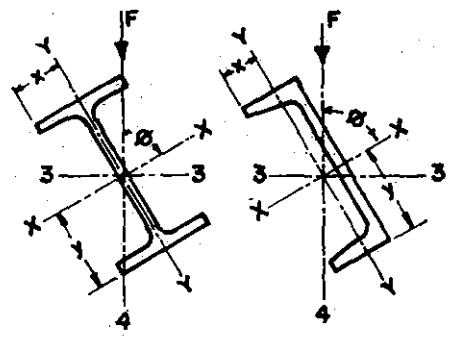
$$I_z = I_x\sin^2\theta + I_y\cos^2\theta + K\sin 2\theta$$

$$I_w = I_x\cos^2\theta + I_y\sin^2\theta - K\sin 2\theta$$

K is negative when heel of angle, with respect to c.g., is in 1st or 3rd quadrant, positive when in 2nd or 4th quadrant.

BEAMS AND CHANNELS

Transverse force oblique through centre of gravity



$$I_3 = I_x\sin^2\phi + I_y\cos^2\phi$$

$$I_4 = I_x\cos^2\phi + I_y\sin^2\phi$$

$$f_b = M\left(\frac{y}{I_x}\sin\phi + \frac{x}{I_y}\cos\phi\right)$$

where M is bending moment due to force F.

APPENDIX 4

Mathematical Data

$\pi = \frac{355}{113}$ (approx.) $= \frac{22}{7}$ (approx.) $= 3.141592654$ (approx.)

One radian $= \frac{180°}{\pi} = 57.3°$ (approx.) $=$ $57.2957795°$ (approx.)

Length of arc subtended by an angle of one radian = radius of arc.

One degree Fahrenheit = 5/9 degree Centigrade or Celsius.

Temperature of t°F $= 5/9(t - 32)$°C.

Temperature of t°C $= (1.8t + 32)$°F.

Base of Naperian logarithms, $e = \frac{193}{71}$ (approx.) $=$

$\frac{2721}{1001}$ (approx.) $= 2.718281828$ (approx.)

To convert common into Naperian logarithms, multiply by $\frac{76}{33}$ (approx.) $= \frac{3919}{1702}$ (approx.) $= 2.302585093$ (approx.).

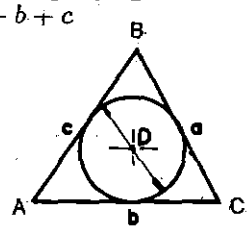
Nominal value of $g = 9.80665$ m./sec^2 $= 32.174$ ft/sec^2.

Diameter of inscribed circle of a triangle:

$$D = \frac{2b\sqrt{a^2 - \left(\frac{a^2 + b^2 - c^2}{2b}\right)^2}}{a + b + c}$$

For isosceles triangle, $a = c$:

$$D = \frac{b\sqrt{4a^2 - b^2}}{2a + b}$$



Solution of Triangles

Applicable to any triangle ABC in which $AB = c$;

$BC = a$; $AC = b$; $\frac{\sin A}{a} = \frac{\sin B}{b} = \frac{\sin C}{c}$

Area $= \frac{bc \sin A}{2} = \frac{ac \sin B}{2} = \frac{ab \sin C}{2} =$

$\sqrt{s(s-a)(s-b)(s-c)}$, where $s = 1/2(a + b + c)$.

$\sin \frac{A}{2} = \sqrt{\frac{(s-b)(s-c)}{bc}}$ $\quad \cos A = \frac{b^2 + c^2 - a^2}{2bc}$

THE GREEK ALPHABETS

Letters from the Greek alphabet are frequently used as symbols in engineering problems.

Alpha	A	α	Iota	I	ι	Rho	P	ρ
Beta	B	β	Kappa	K	κ	Sigma	Σ	σ
Gamma	Γ	γ	Lambda	Λ	λ	Tau	T	τ
Delta	Δ	δ	Mu	M	μ	Upsilon	Υ	υ
Epsilon	E	ϵ	Nu	N	ν	Phi	Φ	ϕ
Zeta	Z	ζ	Xi	Ξ	ξ	Chi	X	χ
Eta	H	η	Omicron	O	o	Psi	Ψ	ψ
Theta	Θ	θ	Pi	Π	π	Omega	Ω	ω

ALGEBRAICAL FORMULAE

Products and Factors

$$(x + a)(x + b) = x^2 + (a + b)x + ab$$
$$(x + a)(x - a) = x^2 - a^2$$
$$(x \pm a)(x^2 \pm ax + a^2) = x^3 \pm a^3$$
$$(x \pm a)^2 = x^2 \pm 2ax + a^2$$
$$(x \pm a)^3 = x^3 \pm 3x^2a + 3xa^2 \pm a^3$$

General Solution to a Quadratic Equation

If $\quad ax^2 + bx + c = 0$

$$x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

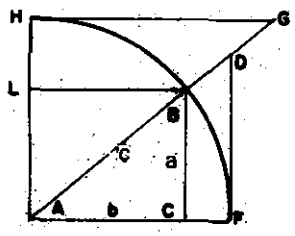
Indices

$$a^m \times a^n = a^{m+n}$$
$$a^m \div a^n = a^{m-n}$$
$$(a^m)^n = a^{mn}$$
$$\log(ab) = \log a + \log b$$
$$\log(a/b) = \log a - \log b$$
$$\log(a^n) = n \log a$$
$$\log \sqrt[n]{a} = \log a \times 1/n$$

STANDARD FORMS OF DIFFERENTIALS AND INTEGRALS

Differential $\frac{dy}{dx}$	Function $\longleftarrow y \longrightarrow$	Integral $\int y dx$
1	x	$\frac{1}{2}x^2 + C$
0	a	$ax + C$
1	$x \pm a$	$\frac{1}{2}x^2 \pm ax + C$
$2x$	x^2	$\frac{1}{3}x^3 + C$
nx^{n-1}	x^n	$\frac{1}{n+1}x^{n+1} + C$ (except when $n = -1$)
$-x^{-2}$	x^{-1}	$\log_e x + C$
$\frac{du}{dx} \pm \frac{dv}{dx} \pm \frac{dw}{dx}$	$u \pm v \pm w$	$\int u dx \pm \int v dx \pm \int w dx$
$u\frac{dv}{dx} + v\frac{du}{dx}$	uv	No general form
$\frac{v\frac{du}{dx} - u\frac{dv}{dx}}{v^2}$	u/v	No general form
$\frac{du}{dx}$	u	$ux - \int x du + C$
e^x	e^x	$e^x + C$
x^{-1}	$\log_e x$	$x(\log_e x - 1) + C$
$0.4343x^{-1}$	$\log_{10} x$	$0.4343x(\log_e x - 1) + C$
$a^x \log_e a$	a^x	$\frac{a^x}{\log_e a} + C$
$\cos x$	$\sin x$	$-\cos x + C$
$-\sin x$	$\cos x$	$\sin x + C$
$\sec^2 x$	$\tan x$	$-\log_e \cos x + C$
$\cosh x$	$\sinh x$	$\cosh x + C$
$\sinh x$	$\cosh x$	$\sinh x + C$
$\text{sech}^2 x$	$\tanh x$	$\log_e \cosh x + C$
$-\frac{1}{(x+a)^2}$	$\frac{1}{x+a}$	$\log_e(x+a) + C$

TRIGONOMETRIC FUNCTIONS



Radius $AF = 1$

$= \sin^2 A + \cos^2 A = \sin A \operatorname{cosec} A$

$= \cos A \sec A = \tan A \cot A$

Sine $A = \frac{\cos A}{\cot A} = \frac{1}{\operatorname{cosec} A} = \cos A \tan A$

$= \sqrt{1 - \cos^2 A} = BC$

Cosine $A = \frac{\sin A}{\tan A} = \frac{1}{\sec A} = \sin A \cot A$

$= \sqrt{1 - \sin^2 A} = AC$

Tangent $A = \frac{\sin A}{\cos A} = \frac{1}{\cot A} = \sin A \sec A = FD$

Cotangent $A = \frac{\cos A}{\sin A} = \frac{1}{\tan A} = \cos A \operatorname{cosec} A = HG$

Secant $A = \frac{\tan A}{\sin A} = \frac{1}{\cos A} = AD$

Cosecant $A = \frac{\cot A}{\cos A} = \frac{1}{\sin A} = AG$

$\sin^2\theta + \cos^2\theta = 1 \quad \sec^2\theta - \tan^2\theta = 1$

$\operatorname{cosec}^2\theta - \cot^2\theta = 1$

$$\sin(\theta + \phi) = \sin\theta\cos\phi + \cos\theta\sin\phi$$
$$\cos(\theta + \phi) = \cos\theta\cos\phi - \sin\theta\sin\phi$$
$$\sin(\theta - \phi) = \sin\theta\cos\phi - \cos\theta\sin\phi$$
$$\cos(\theta - \phi) = \cos\theta\cos\phi + \sin\theta\sin\phi$$

$$\tan(\theta + \phi) = \frac{\tan\theta + \tan\phi}{1 - \tan\theta\tan\phi} \quad \tan(\theta - \phi) = \frac{\tan\theta - \tan\phi}{1 + \tan\theta\tan\phi}$$

$\sin\theta + \sin\phi = 2\sin 1/2(\theta + \phi)\cos 1/2(\theta - \phi)$

$\sin\theta - \sin\phi = 2\cos 1/2(\theta + \phi)\sin 1/2(\theta - \phi)$

$\cos\theta + \cos\phi = 2\cos 1/2(\theta + \phi)\cos 1/2(\theta - \phi)$

$\cos\theta - \cos\phi = -2\sin 1/2(\theta + \phi)\sin 1/2(\theta - \phi)$

$\sin\theta\sin\phi = 1/2[\cos(\theta - \phi) - \cos(\theta + \phi)]$

$\sin\theta\cos\phi = 1/2[\sin(\theta + \phi) + \sin(\theta - \phi)]$

$\cos\theta\cos\phi = 1/2[\cos(\theta + \phi) + \cos(\theta - \phi)]$

$\text{Sin}\, A = \cos A, \quad \tan A = 2\sin\frac{A}{2} \cdot \cos\frac{A}{2}$

$= \sqrt{1/2(1 - \cos 2A)} = \sqrt{1 - \cos^2 A}$

$\text{Cos}\, A = \frac{\sin A}{\tan A} = 2\cos^2\frac{A}{2} - 1 = 1 - 2\sin^2\frac{A}{2}$

$= \sqrt{1 - \sin^2 A}$

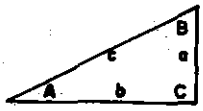
$\text{Tan}\, A = \frac{\sin A}{\cos A} = \frac{\sin 2A}{1 + \cos 2A} = \sqrt{\frac{1}{\cos^2 A} - 1}$

$\text{Sin}\frac{A}{2} = \sqrt{\frac{1 - \cos A}{2}} \qquad \text{Sin}\, 2A = 2\sin A \cdot \cos A$

$\text{Cos}\frac{A}{2} = \sqrt{\frac{1 + \cos A}{2}} \qquad \text{Cos}\, 2A = 2\cos^2 A - 1$

$\text{Tan}\frac{A}{2} = \frac{1 - \cos A}{\sin A} \qquad \text{Tan}\, 2A = \frac{2\tan A}{1 - \tan^2 A}$

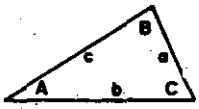
RIGHT ANGLED TRIANGLES



$$a^2 = c^2 - b^2$$
$$b^2 = c^2 - a^2$$
$$c^2 = a^2 + b^2$$

Known	Required					
	A	B	a	b	c	Area
a, b	$\tan A = \frac{a}{b}$	$\tan B = \frac{b}{a}$			$\sqrt{a^2 + b^2}$	$\frac{ab}{2}$
a, c	$\sin A = \frac{a}{c}$	$\cos B = \frac{a}{c}$		$\sqrt{c^2 - a^2}$		$\frac{a\sqrt{c^2 - a^2}}{2}$
A, a		$90° - A$		$a \cot A$	$\frac{a}{\sin A}$	$\frac{a^2 \cot A}{2}$
A, b		$90° - A$	$b \tan A$		$\frac{b}{\cos A}$	$\frac{b^2 \tan A}{2}$
A, c		$90° - A$	$c \sin A$	$c \cos A$		$\frac{c^2 \sin 2A}{4}$

OBLIQUE ANGLED TRIANGLES



$$s = \frac{a + b + c}{2}$$

$$K = \sqrt{\frac{(s-a)(s-b)(s-c)}{s}}$$

$$a^2 = b^2 + c^2 - 2bc \cos A$$
$$b^2 = a^2 + c^2 - 2ac \cos B$$
$$c^2 = a^2 + b^2 - 2ab \cos C$$

Known	Required					
	A	B	C	b	c	Area
a, b, c	$\tan \frac{1}{2}A = \frac{K}{s-a}$	$\tan \frac{1}{2}B = \frac{K}{s-b}$	$\tan \frac{1}{2}C = \frac{K}{s-c}$			$\sqrt{s(s-a)(s-b)(s-c)}$
a, A, B			$180° - (A + B)$	$\frac{a \sin B}{\sin A}$	$\frac{a \sin C}{\sin A}$	
a, b, A		$\sin B = \frac{b \sin A}{a}$			$\frac{b \sin C}{\sin B}$	
a, b, C	$\tan A = \frac{a \sin C}{b - a \cos C}$				$\sqrt{a^2 + b^2 - 2ab \cos C}$	$\frac{ab \sin C}{2}$

TRIGONOMETRICAL RATIOS

Angle Degrees	Sine	Tangent	Cotangent	Cosine	—
1	0.0175	0.0175	57.2900	0.9998	89
2	0.0349	0.0349	28.6363	0.9994	88
3	0.0523	0.0524	19.0811	0.9986	87
4	0.0698	0.0699	14.3007	0.9976	86
5	0.0872	0.0875	11.4301	0.9962	85
6	0.1045	0.1051	9.5144	0.9945	84
7	0.1219	0.1228	8.1443	0.9925	83
8	0.1392	0.1405	7.1154	0.9903	82
9	0.1564	0.1584	6.3138	0.9877	81
10	0.1736	0.1763	5.6713	0.9848	80
11	0.1908	0.1944	5.1446	0.9816	79
12	0.2079	0.2126	4.7046	0.9781	78
13	0.2250	0.2309	4.3315	0.9744	77
14	0.2419	0.2493	4.0108	0.9703	76
15	0.2588	0.2679	3.7321	0.9659	75
16	0.2756	0.2867	3.4874	0.9613	74
17	0.2924	0.3057	3.2709	0.9563	73
18	0.3090	0.3249	3.0777	0.9511	72
19	0.3256	0.3443	2.9042	0.9455	71
20	0.3420	0.3640	2.7475	0.9397	70
21	0.3584	0.3839	2.6051	0.9336	69
22	0.3746	0.4040	2.4751	0.9272	68
23	0.3907	0.4245	2.3559	0.9205	67
24	0.4067	0.4452	2.2460	0.9135	66
25	0.4226	0.4663	2.1445	0.9063	65
26	0.4384	0.4877	2.0503	0.8988	64
27	0.4540	0.5095	1.9626	0.8910	63
28	0.4695	0.5317	1.8807	0.8829	62
29	0.4848	0.5543	1.8040	0.8746	61
30	0.5000	0.5774	1.7321	0.8660	60
31	0.5150	0.6009	1.6643	0.8572	59
32	0.5299	0.6249	1.6003	0.8480	58
33	0.5446	0.6494	1.5399	0.8387	57
34	0.5592	0.6745	1.4826	0.8290	56
35	0.5736	0.7002	1.4281	0.8192	55
36	0.5878	0.7265	1.3764	0.8090	54
37	0.6018	0.7536	1.3270	0.7986	53
38	0.6157	0.7813	1.2799	0.7880	52
39	0.6293	0.8098	1.2349	0.7771	51
40	0.6428	0.8391	1.1918	0.7660	50
41	0.6561	0.8693	1.1504	0.7547	49
42	0.6691	0.9004	1.1106	0.7431	48
43	0.6820	0.9325	1.0724	0.7314	47
44	0.6947	0.9657	1.0355	0.7193	46
45	0.7071	1.0000	1.0000	0.7071	45
—	COSINE	COTANGENT	TANGENT	SINE	ANGLE DEGREES

APPENDIX 5

Estimation of Bursting Tensile Stress in 'Caisson-Steining' under 'Pneumatic-Sinking' Conditions

Where it is anticipated to sink the caisson by means of 'pneumatic-sinking', it is necessary that the steining (and the well-curb) are made with concrete of not less than 160 to 200 kg/cm^2 28 days cube strength. This concrete should be easily workable and well compacted so as to make it adequately impermeable, lest the compressed air should leak out. Obviously any injury to the concrete, resulting from explosive charges used in the process of well-sinking, should be avoided. Generally no special hoop reinforcement is required either in the steining or in the curb (the caisson-shoe) for 'sinking-depths' to about 30m. (This corresponds to about 30 T/m^2 or 3 kg/cm^2, i.e., 3 atmospheres pressure, and is the upper limit of human endurance.) Unless the steining is academically thin and the nominal hoop reinforcement in the steining and the total reinforcement in the well-curb are less than the minima stipulated in the appropriate codes, for instance the Indian Roads Congress-'Bridge Design Stipulations' (for more details see the chapter on 'substructure' in this book), no special reinforcement is required. Nevertheless a resolute and quick workman-like check is necessary at the time the quantities have to be decided and when design and drawings are to be finalised.

The following simplified assumptions and analysis may be adopted:

1. Assume that the corbel (i.e. the air-lock stool) will be laid at the 'top' of the total anticipated caisson-height.
2. Ignore the relief against bursting from the surrounding soil mass (for various practical reasons).
3. Assume low-water-level condition exists, and that the caisson is at the stage of 'sinking completed'.
4. Internal pneumatic air pressure is at least 15% higher than the maximum water pressure from outside (for obvious reasons).
5. Check the greatest bursting tensile stress in the steining (this will be in the top unit-meter depth of the steining) on its inner as well as outer faces and, only if this exceeds the section-capacity, check this stress at lower depths too in order to detail the steining section down the depth. It is advisable that the maximum tensile stress be limited to less than the permissible tensile stress in concrete (uncracked section basis), since, otherwise, should the concrete have to crack in order to transfer the hoop-tension to the hoop reinforcement, the resulting cracking can cause leakage of air pressure.
   *See explanatory note ahead**.
6. Work out the 'average' tensile stress in the concrete in the curb-section (= average bursting pressure within its height × mean radius of curb × curb-height, divided by section area of curb) and limit it to the permissible tensile stress in concrete, ignoring the nominal reinforcement in the curb.

EXPLANATORY NOTE ON ITEM 5

—Refer to Fig. A5.1 on p: 709, which is self explanatory

—Steining is a thick-shell (of known inner and outer radii r_i and r_0), hence Lame's formula can be used to estimate f_{t_i} and f_{t_0}, the inner and outer peripheral hoop stresses, from

$$f_t = \frac{p \cdot r_i^2}{(r_0^2 - r_i^2)} \left\{ 1 + \frac{r_0^2}{r^2} \right\} \qquad \text{(A 5.1)}$$

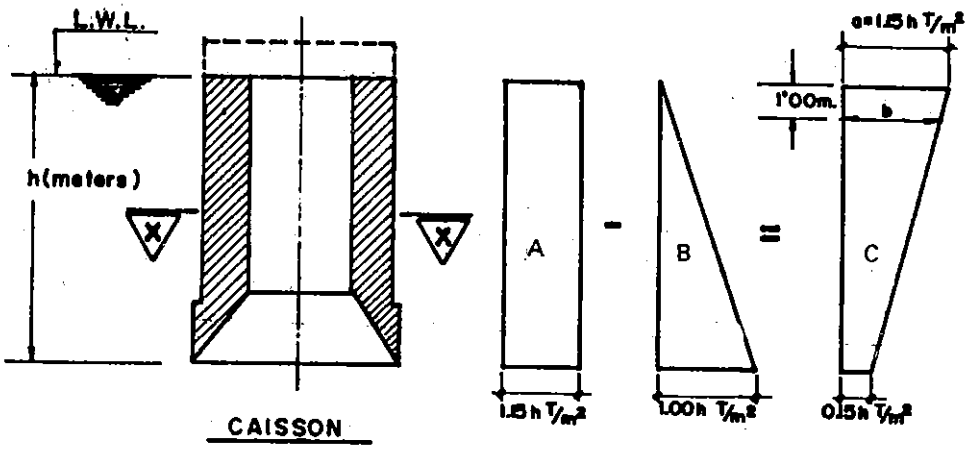
where $r = r_i$ for f_t on inner surface,

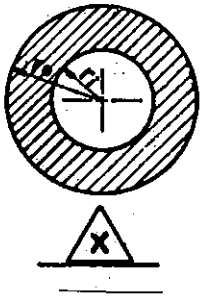
and

$r = r_0$ for f_t on outer surface,

p being the 'average' hoop stress.

Hence f_{t_i} and f_{t_0} can be found from Eq. (A 5.1) in terms of p. 'Average' hoop tension T in the top 1-metre depth of steining of thickness $(r_0 - r_i)$, is $T = \left\{ \frac{f_{t_0} + f_{t_i}}{2} \times 1.00 \times (r_0 - r_t) \right\}$ in tonne units, with f_{t_0} and f_{t_i} (and p) in T/m^2 units, and, r_0 and r_i in metre units. Equating the T to the maximum bursting force which exists in the top 1-metre depth of steining (which is = $\left(\frac{a+b}{2}\right) r_i \times 1.00$ tonnes, a and b as in the figure), obtain the value for p. Hence f_{t_i} and f_{t_o} can be quickly evaluated, and the greater of these, i.e. f_{t_i}, ensured against the permissible tensile strength of concrete and either hoop reinforcement designed as needed, or alternatively, concrete section and/or strength modified to suit.





A = OUTWARD BURSTING PRESSURE DUE TO COMPRESSED AIR.
B = INWARD (RELIEVING) HYDROSTATIC PRESSURE
C = NET BURSTING PRESSURE

Fig. A 5.1

APPENDIX 6

Extracts from the A.C.I. Committee Report 343 and the AASHTO Design Specifications

Some extracts are reproduced below with acknowledgement and courtesy purely as a ready reference 'companion reading' (However, reference must be made to the original texts for details).

... 'NOTATIONS' explained at the end of this Appendix.

NOTE: For Load Factor Design (R.C.), See Sections 7.3, etc., ahead.

(A) SERVICE-LOAD DESIGN

8.7 Permissible Stresses for Prestressed Flexural Members at Service Conditions

8.7.1 Flexural stresses in concrete immediately after transfer, before losses due to creep and shrinkage, should not exceed the following:

(a)	Compression: Pre-tensioned	$0.60f'_{ci}$
	Post-tensioned	$0.55f'_{ci}$
(b)	Tension in member without bonded auxiliary reinforcement (unprestressed or prestressed) in the tension zone	$3\sqrt{f'_{ci}}$

Where the calculated tensile stress exceeds this value, bonded reinforcement should be provided to resist the total tensile force in the concrete computed on the assumption of an uncracked section.

8.7.2 Flexural stresses in concrete at service loads, after allowance for all prestress losses, should not exceed the following:

(a)	Compression	$0.40f'_c$
(b)	Tension in precompressed tension zone:	
	Members with bonded auxiliary reinforcement to control cracking	$6\sqrt{f'_c}$
	Members with bonded auxiliary reinforcement to control cracking, but exposed to corrosive environments or severe exposure conditions	$3\sqrt{f'_c}$
	Members without bonded auxiliary reinforcement	0

8.7.3 The permissible stresses recommended in Sections 8.7.1 and 8.7.2 may be exceeded if it is shown by tests or analysis that performance will not be impaired.

8.7.4 Stress in prestressing steel should not exceed the following:

(a)	Due to temporary jacking force, but not greater than the maximum value recommended by the manufacturer of the steel or of the anchorages	$0.80f_{pu}$
(b)	Pre-tensioning tendons immediately after transfer or post-tensioning tendons immediately after anchoring	$0.70f_{pu}$

8.8 Service Load Designing (R.C.)

8.8.1 *General*

Nonprestressed members may be designed with reference to service loads and allowable stresses. The stresses in concrete should not exceed the values given in Table A6–8.8.1 the tensile stress in reinforcement, f_s, should not exceed the following:

Grade 40 or Grade 50 reinforcement	20,000 psi
Grade 60 reinforcement	24,000 psi

8.8.2 *Development of Reinforcement*

The calculated tension or compression of the reinforcement at each section must be developed on each side of that section by proper embedment length, end anchorage or, for tension only, hooks. Development of reinforcement should be computed by procedures recommended in Chapter 13.

8.8.3 *Flexure*

For investigation of service load stresses the straight-line theory of stress and strain in flexure should be used and the assumptions of Section 8.1 should apply.

8.8.4 *Compression Members*

The combined axial load and moment capacity of compression members should be taken as 35 per cent of that computed in accordance with the recommendations of Section 7.3.5. Slenderness effects should be included according to the recommendations of Section 7.3.6. The term P_u in Eq. (7.20) should be replaced by 2.85 times the design axial load.

8.8.5 *Shear*

8.8.5.1 The design shear stress should be computed by:

$$v = V/b_w d$$

where b_w should be taken as the width of web and d should be taken as the distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement. For a circular section, b_w should be taken as the diameter and d need not be taken less than the distance from the extreme compression fibre to the centroid of the longitudinal reinforcement in the opposite half of the member.

8.8.5.2 When the reaction in the direction of the applied shear, introduces compression into the end region of the member, sections located less than distance d from the face of the support may be designed for the same v as that at a distance d.

8.8.5.3 The shear stress carried by the concrete, v_c, should not exceed $0.95\sqrt{f'_c}$ when v exceeds v_c, shear reinforcement should be provided.

8.8.5.4 Shear reinforcement should conform to the general requirements of Section 7.3.7.1. When shear reinforcement perpendicular to the axis of the member is used, the required area should be computed by:

$$A_v = \frac{(v - v_c)b_w s}{f_s}$$

8.8.5.5 When $(v - v_c)$ exceeds $2f'_c$, the maximum spacings given in Section 7.3.7.1.6 should be reduced by one-half. The maximum shear carried by concrete plus shear reinforcement should not exceed $5\sqrt{f'_c}$.

8.8.6 *Shear Stress in Slabs and Footings*

8.8.6.1 The shear capacity of slabs and footings in the vicinity of concentrated loads or reactions should be governed by the more severe of two conditions:

(a) The slab or footing acting as a wide beam, with a critical section extending in a plane across the entire width

Table A6-8.8.1 *Allowable Stresses*

Description		Basic value	Allowable Stresses For strength of concrete shown		
			$f'_c =$ 3000 psi	$f'_c =$ 4000 psi	$f'_c =$ 5000 psi
Modular ratio, $n = \frac{E_s}{E_c}$ For normal weight concrete	n	$\frac{29,000,000}{57,000\sqrt{f'_c}}$	9	8	7
Flexure					
Extreme fibre stress in compression	f_c	$0.4f'_c$	1200	1600	2000
Extreme fibre stress in tension (plain concrete)*	f_t	$1.6\sqrt{f'_c}$	88	102	113
Modulus of rupture*	f_r	$7.5\sqrt{f'_c}$	410	475	530
Shear					
Beams:					
Shear carried by concrete*	v_c	$0.95\sqrt{f'_c}$	52	60	67
Maximum shear carried by concrete plus shear reinforcement	v	$5\sqrt{f'_c}$	274	316	354
Slabs and footings:					
Shear carried by concrete*	v_c	$1.8\sqrt{f'_c}$	99	114	127
Maximum shear carried by concrete plus shear reinforcement	v	$3\sqrt{f'_c}$	165	190	212
Bearing on loaded area†	f_b	$0.3f'_c$	900	1200	1500
Bearing on loaded area subjected to high edge stresses due to deflection or eccentric loading†	f_b	$0.225f'_c$	675	900	1125

* When lightweight aggregate concretes are used, the allowable stresses should be multiplied by 0.75 for "all-lightweight" concrete, and 0.35 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be increased by $\sqrt{A_2/A_1}$, but not more than 2. When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

and located at a distance d from the face of the concentrated load or reaction area. For this condition the slab or footing should be designed in accordance with Section 8.8.5.

(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the slab and located so that its periphery is a minimum and approaches no closer than $d/2$ to the periphery of the concentrated load or reaction area. For this condition the slab or footing should be designed in accordance with Sections 8.8.6.2 and 8.8.6.3.

8.8.6.2 The peripheral shear stress should be computed by:

$$v = V/b_0 d$$

in which V and b_0 are taken at the critical section defined in Section 8.8.6.1 (b). The peripheral shear stress should not exceed the shear stress carried by the concrete $v_r = 1.8\sqrt{f'_c}$ unless shear reinforcement is provided in accordance with Section 8.8.6.3, in which case v_u should not exceed $3\sqrt{f'_c}$.

8.8.6.3 Shear reinforcement consisting of bars or wires may be provided. For design of such shear reinforcement, shear stresses should be investigated at the critical section defined in Section 8.8.6.1(b) and at successive sections more distant from the support; and the shear stress v_c, carried by the concrete at any section should not exceed $0.9\sqrt{f'_c}$, where v exceeds v_c, shear reinforcement should be provided according to Section 8.8.5.4.

8.4 Distribution of Reinforcement in Flexural Members

8.4.1 Only deformed reinforcement should be considered effective for principal reinforcement, except that plain bars or smooth wire may be used as spirals or reinforcement ties for confinement. All tension reinforcement should be well distributed in zones of maximum tension.

When the design yield strength, f_y, for tension reinforcement exceeds 40,000 psi, cross-sections of maximum tension stress should be so proportioned that the calculated stress in the reinforcement at service load, f_s, in kips per square inch, does not exceed the value computed by:

$$f_s = \frac{z}{\sqrt[3]{d_c A}} \qquad (8.1)$$

but f_s should not be greater than $0.6 f_y$,

where A = effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as the reinforcement, divided by the number of bars, sq. in. When the main reinforcement consists of several bar sizes, the number of bars should be computed as the total steel area divided by the area of the largest bar used.

d_c = thickness of concrete cover measured from the extreme tension fibre to the centre of the bar located closest thereto.

8.4.2 The quantity z in Eq. (8.1) should not exceed 170 kips per in. for members in moderate exposure conditions and 130 kips per in. for members in severe exposure conditions. Where members are exposed to very aggressive exposure or corrosive environments, such as deicer chemicals, the denseness and nonporosity of the protecting concrete should be considered, or other protection, such as a waterproof protecting system, should be provided in addition to satisfying Eq. (8.1).

8.4.3 Where girder flanges are in tension, the tension reinforcement should be distributed over an effective width as defined in Section 7.2.11.2 or 7.2.12.2, or a width equal to one-tenth of the girder span, whichever is smaller. If the effective slab width exceeds one-tenth of the span, additional longitudinal reinforcement should be provided in the outer portions of the slab.

8.4.4 If the depth of the side face of a flexural member exceeds 2 ft. longitudinal reinforcement having a total area not less than 10 per cent of the principal tension reinforcement should be placed, one-half near each side face of the member, and distributed in the zone of flexural tension. The spacing of such reinforcement should not exceed 12 in. or the width of the web, whichever is less. Such reinforcement may be included in computing the flexural capacity only if a stress and strain compatibility analysis is made to determine stresses in the individual bars.

(B) LOAD-FACTOR DESIGN (R.C.)

7.3 STRENGTH REQUIREMENTS

7.3.1 *Required Strength*

7.3.1.1 Bridge structures and structural members should be designed to have strength at all sections at least equal to the structural effects of the load groups which represent various combinations of loads and forces to which the structure may be subjected. Each part of such structure should be proportioned for the group loads that are applicable, and the maximum design required should be used.

7.3.1.2 The serviceability requirements of Chapter 8 should also be satisfied to insure adequate performance at service load levels.

7.3.2 *Strength*

The strength of a member or cross-section in terms of load, moment, shear, or stress should be taken as the strength computed in accordance with the recommendations and assumptions of this Section 7.3 modified by a capacity

reduction factor, ϕ.*. The following values of ϕ should be used:

For flexure	$\phi = 0.90$
For shear and torsion	$\phi = 0.85$
For spirally reinforced compression members	$\phi = 0.75$
For tied compression members	$\phi = 0.70$
For bearing on concrete	$\phi = 0.70$

The value of ϕ may be increased linearly from the value for compression members to the value for flexure as the axial design load, P_u, decreases from $0.10 f'_c A_g$ or P_b, whichever is smaller, to zero.

For precast prestressed units produced in plants having an acceptable quality control program, the capacity reduction factors in Section 9.2 may be applied. Development lengths specified in Chapter 13, do not require a ϕ factor.

7.3.3 *Design Assumptions*

The strength design of members for flexure and axial loads should be based on the assumptions given in this section, and on satisfaction of the applicable conditions of equilibrium and compatibility of strains.

(a) *Strain in the reinforcing steel and concrete should* be assumed directly proportional to the distance from the neutral axis.

(b) The maximum usable strain at the extreme concrete compression fibre should be assumed equal to 0.003.

(c) *Stress in reinforcement below the specified yield* strength, f_y, for the grade of steel used should be taken as E_s times the steel strain. For strains greater than that corresponding to f_y, the stress in the reinforcement should be considered independent of strain and equal to f_y.

(d) Tensile strength of the concrete should be neglected in flexural calculations of reinforced concrete.

(e) The relationship between the concrete compressive stress distribution and the concrete strain may be assumed to be a rectangle, trapezoid, parabola, or any shape which results in prediction of strength in substantial agreement with the results of comprehensive tests.

(f) The recommendations of Section 7.3.3(e) may be considered satisfied by an equivalent rectangular concrete stress distribution for concrete having a compressive strength f'_c of 8000 psi or less. This stress block is defined as follows: A concrete stress of 0.85 f'_c should be assumed uniformly distributed over an equivalent compression zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain. The distance c from the fibre of maximum strain to the neutral axis is measured in a direction perpendicular to that axis. The factor β_1 should be taken as 0.85 for strengths, up to and including 4,000 psi. For strengths above 4,000 psi β_1 should be reduced continuously at a rate of 0.05 for each 1,000 psi of strength in excess of 4,000 psi, but β_1 should not be taken less than 0.65.

7.3.4 *Flexure*

7.3.4.1 *Minimum Reinforcement of Nonprestressed Flexural Members*

7.3.4.1.1 At any section of a flexural member where tension reinforcement is required by analysis, the reinforcement provided should be adequate to develop a moment strength at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture of the concrete. The minimum reinforcement ratio, ρ_{min} required for this condition may be obtained from the following approximate expressions.

Rectangular Sections:

$$\rho_{min} = \frac{2\sqrt{f'_c}}{f_y}$$

T-Sections: $\rho_{min} = 0.4\left(1 + \frac{b_w}{b}\right)\frac{\sqrt{f'_c}}{f_y}$

I-Sections: $\rho_{min} = 2\left[1 - (1 - b_w/b)(1 - 2h_f/h)^3\right]\frac{\sqrt{f'_c}}{f_y}$ (7.1)

7.3.4.1.2 The minimum reinforcement recommendations may be waived if the area of reinforcement provided at a section is at least one-third greater than that required by analysis.

7.3.4.2 *Maximum Reinforcement of Nonprestressed Flexural Members*

7.3.4.2.1 For flexural members, the reinforcement ratio provided should not exceed 0.75 of that ratio ρ_b, which would produce balanced conditions for the section under flexure.

7.3.4.2.2 Balanced conditions exist at a cross-section when the tension reinforcement reaches its yield strength, f_y, just as the concrete in compression reaches its assumed ultimate strain of 0.003.

7.3.4.3 *Rectangular Sections with Nonprestressed Tension Reinforcement Only*

For rectangular sections, the design moment strength may be computed by:

$$M_u = \phi\left[A_s f_y d\left(1 - 0.6\rho\frac{f_y}{f'_c}\right)\right] \qquad (7.2)$$

* *The coefficient provides for the possibility that small adverse variations* in material strengths, workmanship, and dimensions, while individually *within acceptable tolerances and limits of good practice, may combine* to result in under capacity. The capacity reduction factors given here are *intended to apply to reinforced concrete, excluding prestress concrete.* All ϕ factors given in this chapter agree with those given in ACI318-71. *Those given in Chapter 9 for prestressed concrete differ with those* given for prestressed concrete in AASHTO Standard Specifications for Highway Bridges and in ACI 318-71.

$$= \phi\left[A_s f_y\left(d - \frac{a}{2}\right)\right] \tag{7.3}$$

where

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

The balanced reinforcement ratio, ρ_b, for rectangular sections with tension reinforcement only is given by:

$$\rho_b = \frac{0.85\beta_1 f_c'}{f_y}\frac{87,000}{87,000 + f_y} \tag{7.4}$$

7.3.4.4 *I- and T-Sections with Tension Reinforcement Only*

7.3.4.4.1 When the compression flange thickness is equal to or greater than the depth to the neutral axis, a/β_1, the design moment strength may be computed by the equations given in Section 7.3.4.3.

7.3.4.4.2 When the compression flange thickness is less than a/β_1, the design moment strength may be computed by:

$$M_u = \phi\left[(A_s - A_{sf})\, f_y\left(d - \frac{a}{2}\right) + A_{sf} f_y (d - 0.5h_f)\right] \tag{7.5}$$

where

$$A_{sf} = \frac{0.85 f_c'(b - b_w)h_f}{f_y}$$

$$a = \frac{(A_s - A_{sf})f_y}{0.85 f_c' b_w}$$

The balanced reinforcement ratio, ρb, for I- and T-sections with tension reinforcement only is given by:

$$\rho b = \left[\frac{b_w}{b}\frac{0.85\beta_1 f_c'}{f_y}\frac{87,000}{87,000 + f_y} + \rho_f\right] \tag{7.6}$$

where

$$\rho f = \frac{A_{sf}}{b_w d}$$

7.3.4.5 *Rectangular Sections with Compression Reinforcement*

For rectangular sections and flanged sections in which the neutral axis lies within the flange, the design moment strength may be computed by:

$$M_u = \phi\left[(A_s - A_s')\, f_y\left(d - \frac{a}{2}\right) + A_s' f_y (d - d')\right] \tag{7.7}$$

where

$$a = \frac{(A_s - A_s')f_y}{0.85 f_c' b}$$

and the following condition should be satisfied:

$$\frac{(A_s - A_s')}{bd} \geq 0.85\beta_1 \frac{f_c' d'}{f_y d}\frac{87,000}{87,000 - f_y} \tag{7.8}$$

When the value of $(A_s - A_s')/bd$ is less than the value given by Eq. (7.8), so that the stress in the compression reinforcement is less than the yield strength, or when effects of compression reinforcement are neglected, the moment strength may be computed by the equations in Section 7.3.4.3 except when a general analysis is made based on stress and strain compatibility using the assumptions given in Section 7.3.3.

The balanced reinforcement ratio, ρb, for rectangular sections with compression reinforcement is given by:

$$\rho b = \frac{0.85\beta_1 f_c'}{f_y}\frac{87,000}{87,000 + f_y} + \rho'\frac{f_s' b}{f_y} \tag{7.9}$$

where

f_{sb}' = stress in compression reinforcement at balanced conditions

$$= 87,000\left(1 - \frac{d'}{d}\frac{87,000 + f_y}{87,000}\right) \leq f_y$$

7.3.4.6 *Other Nonprestressed Cross-sections*

For other cross-sections and for conditions of nonsymmetrical bending, the design moment strength, $M_u = \phi M_t$, should be computed by a general analysis based on stress and strain compatibility using the assumptions given in Section 7.3.3. The requirements of Section 7.3.4.2 should also be satisfied.

7.3.4.7 *Prestressed Concrete Members*

7.3.4.7.1 The design moment strength for prestressed concrete members may be computed by the same strength design procedures and equations recommended for nonprestressed members. For prestressing steel, f_{ps} should be substituted for f_y.

7.3.4.7.2 In lieu of a more precise determination of f_{ps} based on strain compatibility, and provided that f_{se} is not less than 0.5 f_{pu}, the following approximate values should be used:

Bonded members

$$f_{ps} = f_{pu}\left(1 - 0.5\rho_p \frac{f_{pu}}{f_c'}\right) \tag{7.10}$$

Unbonded members

$$f_{ps} = f_{se} + 10,000 + \frac{f'_c}{100\rho_p} \qquad (7.11)$$

but f_{ps} should not be taken more than $f_{se} + 60,000$, nor the yield strength of the prestressing tendons.

7.3.4.7.3 Nonprestressed reinforcement, when used in combination with prestressed steel, may be considered to contribute to the tensile force in a member at design moment strength an amount equal to its area times its yield strength.

7.3.4.7.4 The amount of prestressed and nonprestressed steel used for calculation of the design moment strength should be such that

$$A_{ps}\frac{f_{ps}}{bdf'_c}$$

or

$$A_s\frac{f_y}{bdf'_c} + A_{ps}\frac{f_{ps}}{bdf'_c} - A'_s\frac{f_y}{bdf'_c}$$

is not greater than 0.30 For flanged sections, b should be taken as the web width and the steel area should be that required to develop the compressive strength of the webs only.

7.3.4.7.5 When a steel ratio in excess of 0.30 is used, the design moment strength should be based on the compression portion of the internal resisting moment couple. The following expressions satisfy the intent of this recommendation:

For rectangular sections, or flanged sections in which the neutral axis is within the flange:

$$M_u = \phi(0.25f'_c bd^2) \qquad (7.12)$$

For flanged sections in which the neutral axis falls outside the flange:

$$M_u = \phi[0.25f'_c b_w d^2 + 0.85f_c(b - b_w)]\, h_f(d - 0.5h_f) \qquad (7.13)$$

7.3.4.7.6 The total amount of prestressed and nonprestressed reinforcement provided should be adequate to develop a moment strength at least 1.2 times the cracking moment calculated on the basis of the modulus of rupture of the concrete.

7.3.5 *Nonprestressed Compression Members with or without Flexure*

7.3.5.1 *General Requirements*

7.3.5.1.1 The design of cross-sections subject to combined flexure and axial load should be based on stress and strain compatibility using the assumptions given in Section 7.3.3. For prestressed members refer to Chapter 9. Slenderness effects should be included according to the recommendations of Section 7.3.6.

7.3.5.1.2 All members subjected to a compression load should be designed for an eccentricity equal to the greatest of

(a) that corresponding to the maximum design moment which accompanies this compression load, or
(b) 0.05 h for spirally reinforced compression members, or 0.10 h for tied compression members, about either axis, or
(c) 1 in. about either axis.

7.3.5.2 *Minimum Reinforcement for Compression Members*

7.3.5.2.1 Longitudinal reinforcement for compression members should be not less than 0.01 nor more than 0.08 times the gross area of the section. The minimum number of longitudinal reinforcing bars should be six for bars in a circular arrangement and four for bars in a rectangular arrangement. For other shapes, one bar should be placed at each corner. The minimum size of bar should be # 5.

7.3.5.2.2 In a compression member which has a larger cross-section than required by considerations of loading, a reduced effective area may be used for determining minimum longitudinal reinforcement, provided that in no case should less longitudinal reinforcement be used than that required by the minimum section designed with 1% of longitudinal reinforcement.

7.3.5.3 *Compression Member Strength*

The following provisions may be used as a guide to define the range of the load-moment interaction relationship for members subjected to combined flexure and axial load.

7.3.5.3.1 Pure compression. The axial design load strength of a cross-section in pure compression, P_0, may be computed by:

$$P_0 = \phi[0.85f'_c(A_g - A_{si}) + A_{st}f_y] \qquad (7.14)$$

Concentric loading is a hypothetical loading condition since all members subjected to a compression load should be designed for eccentricities not less than the values recommended in Section 7.3.5.1.2.

7.3.5.3.2 Pure flexure. The assumptions given in Section 7.3.3 or the applicable equations for flexure given in Section 7.3.4 may be used to compute the design moment strength, M_u, for the condition of pure flexure.

7.3.5.3.3 Balanced conditions. Balanced conditions for a cross-section are defined in Section 7.3.4.2. For a rectangular section with reinforcement in one or two faces and located at approximately the same distance from the axis of bending, the balanced load, P_b, and balanced moment, M_b, may be computed by:

$$P_b = \phi[0.85f'_c ba_b + A'_s f'_s - A_s f_y] \qquad (7.15)$$

and

$$M_b = P_b e_b = \phi \left[0.85 f_c' b a_b \left(d - d'' - \frac{a_b}{2}\right) + A_s' f_s' (d - d' - d'') + A_s f_y d''\right] \quad (7.16)$$

where

$$a_b = \left(\frac{87,000}{87,000 + f_y}\right) \beta_1 d$$

$$f_s' = 87,000 \left(1 - \frac{d'}{d} \frac{87,000 + f_y}{87,000}\right) \leq f_y$$

7.3.5.3.4 Combined flexure and axial load. The design strength under combined flexure and axial load should be based on stress and strain compatibility using the assumptions given in Section 7.3.3. The strength of a cross-section is controlled by tension when the axial design load strength, P_w, is less than P_b (or e is greater than e_b). The strength of a cross-section is controlled by compression when the axial design load strength, P_w is greater than P_b (or e is less than e_b).

The combined axial load and moment strength should be multiplied by the appropriate capacity reduction factor, ϕ, for spirally reinforced or tied compression members. The value of ϕ may be increased linearly from the value for compression members to the value for flexure as the axial design load strength, P_w, decreases from $0.10 f_c' A_g$ or P_b, whichever is smaller, to zero.

7.3.5.4 *Biaxial Loading*

In lieu of a general section analysis based on stress and strain compatibility for a loading condition of biaxial bending, the design strength of noncircular members subjected to biaxial bending may be computed by the following approximate expressions:

$$P_{uxy} = \frac{1}{(1/P_{ux}) + (1/P_{uy}) - (1/P_0)} \quad (7.17)$$

when the applied axial design load,

$$P_u \geq 0.1 f_c' A_g$$

or

$$\frac{M_x}{M_{ux}} + \frac{M_y}{M_{uy}} \leq 1 \quad (7.18)$$

when the applied axial design load,

$$P_u < 0.1 f_c' A_g$$

For disproportionate rectangular sections, Eq. (7.17) may yield unconservative results. In such cases, an analysis should be made based on stress and strain compatibility using the assumptions given in Section 7.3.3.

7.3.6 *Strength Reduction for Length of Compression Members*

7.3.6.1 Wherever possible the design of compression members should be based on a comprehensive analysis of the structure. Such analysis should be a second-order (Ref. 7.1) analysis, taking into account the deformation of the structure and the duration of the loads. When axial loads are of sufficient magnitude to reduce stiffness or increase fixed-end moments, such effects should be included. When a second-order analysis is made, additional correction of moments for length effects are not required.

In lieu of a second-order analysis, the design of compression members may be based on the approximate procedure recommended in Section 7.3.6.2.

7.3.6.2 *Approximate Method for Length Effects for Compression Members*

7.3.6.2.1 For purposes of determining the limiting dimensions of compression members, the unsupported length l_u should be taken as the clear distance between lateral supports, except as recommended in Subsections (a), (b), (c), and (d) below.

(a) In pile bent construction, l_u for the pile should be the clear distance between the lowest lateral support described in Section 7.3.6.2.1 (c) and the lower extremity at which the pile may be assumed to be fixed, depending on the soil conditions.

(b) For compression members supported on spread or pile footings, l_u should be the clear distance between the footing and the underside of the deeper flexural member framing into the compression member in the direction of potential translation at the next higher level.

(c) For members restrained laterally by struts, ties or beams, l_u, should be the clear distance between consecutive struts in each vertical plane: provided that, to be an adequate support two such struts should meet the compression member at approximately the same level, and the angle between vertical planes through the struts should not vary more than 15 degrees from a right angle. Such struts should be of adequate dimensions and have sufficient anchorage to restrain the member against lateral deflection.

(d) For compression members restrained laterally by struts or beams with brackets used at the junction, l_u should be the clear distance between the lower support or point of fixity of the base, and the lower edge of the bracket, provided that the bracket width equals that of the beam or strut and is at least half that of the compression member.

7.3.6.2.2 The length which produces the greatest ratio of length to radius of gyration of the section compatible with the loading and support conditions should be considered.

7.3.6.2.3 The radius of gyration may be taken equal to 0.30 times the overall depth in the direction of bending for rectangular members, and 0.25 times the diameter for

circular members. For other shapes, the radius of gyration may be computed for the gross concrete section.

7.3.6.2.4 The effective length of compression members should be taken as kl_u, where the effective length factor, k, should not be less than that recommended in Subsections (a) and (b) below, Ref. 7.2.

(a) Lateral or sidesway instability should be considered in defining the effective length, kl_u, for all structures in which the total translational stiffness of the bracing elements is less than six times the sum of the combined translational stiffnesses of all of the compression members in the level under consideration. All other structures may be assumed to be restrained against lateral instability. For structures subject to sidesway, the value of kl_u should be determined with due consideration of cracking and reinforcement on relative stiffness. When determining the stiffness ratio of beams and columns to obtain k, the beam stiffness should be that of the cracked section, and column stiffness should be obtained using Eq. (7.22) with β_d taken as zero. In no case should k be taken as less than 1.0.

(b) In structures restrained against translation, the value of k should be taken equal to 1.0 unless an analysis shows that a smaller value may be used.

7.3.6.2.5 For compression members braced against sidesway, the effects of slenderness may be neglected when kl_u/r is less than $34-12\ M_1/M_2$. For compression members not braced against sidesway, the effects of slenderness may be neglected when kl_u/r is less than 22. For all compression members with kl_u/r greater than 100, a second-order analysis should be made. M_1 is the smaller end moment on the compression member calculated from a conventional elastic analysis, positive if member is bent in single curvature, negative if bent in double curvature. M_2 is the larger end moment on the compression member calculated from a conventional elastic analysis, always positive.

7.3.6.2.6 Compression members should be designed using the applied design axial load, P_u, from a conventional elastic analysis and a magnified moment M_c defined by

$$M_c = \delta M_2 \tag{7.19}$$

where

$$\delta = \frac{C_m}{1 - P_u/\phi P_c} \geq 1.0 \tag{7.20}$$

and

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} \tag{7.21}$$

In lieu of a more precise calculation, EI, in Eq. (7.21) may be taken either as

$$EI = \frac{(E_c I_g/5) + E_s I_s}{1 + \beta_d} \tag{7.22}$$

or alternatively

$$EI = \frac{E_c I_g/2.5}{1 + \beta_d} \tag{7.23}$$

where β_d is the ratio of maximum design dead load moment to maximum design total load moment, always positive.

For members braced against sidesway and without transverse loads between supports C_m may be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \tag{7.24}$$

but not less than 0.4.

For members braced against sidesway and with transverse loads between supports in the plane of loading, C_m may be determined by analysis. For all other cases, C_m should be taken as 1.0.

7.3.6.2.7 When design of compression members is governed by the minimum eccentricities recommended in 7.3.5.1.2 M_2 should be based on the minimum eccentricity with conditions of curvature determined by either of the following:

(a) When the actual computed eccentricities are less than the minimum, the computed end moments may be used to evaluate the conditions of curvature.

(b) If computations show that there is no eccentricity at both ends of the member, conditions of curvature should be based on a ratio of M_1/M_2 equal to one.

7.3.6.2.8 When compression members are subject to bending about both principal axes, the moment about each axis should be amplified by δ, computed from the corresponding conditions of restraint about that axis.

7.3.6.2.9 In structures which are not braced against sidesway, the flexural members should be designed for the total magnified end moments of the compression members at the joint.

7.3.6.2.10 When a group of compression members on one level comprise a bent, or when they are connected integrally to the same superstructure, and collectively resist the sidesway of the structure, the value of δ should be computed for the member group, P_u and P_c should be taken as the ΣP_u and ΣP_c for all members in the group. In designing each member in the group, δ should be taken as the larger of (a) the value computed for the group as a whole, or (b) the value computed for the individual compression member assuming its ends to be braced against sidesway.

7.3.7 *Shear*

7.3.7.1 *Shear Reinforcement—General Requirements*

7.3.7.1.1 A minimum area of shear reinforcement should be provided in all flexural members, except slabs and footings, where the design shear stress is greater than one-half the shear stress carried by the concrete.

7.3.7.1.2 Where shear reinforcement is required the area provided should be not less than

$$A_v = \frac{50 b_w s}{f_y} \tag{7.25}$$

where b_w and s are in inches.

7.3.7.1.3 Shear reinforcement may consist of:

(a) Stirrups perpendicular to the axis of the member or making an angle of 45 degree or more with the longitudinal tension reinforcement.

(b) Welded wire fabric with wires located perpendicular to the axis of the member.

(c) Longitudinal bars with a bent portion making an angle of 30 degrees or more with the longitudinal tension bars.

(d) Combinations of stirrups and bent bars.

(e) Spirals.

7.3.7.1.4 Web reinforcement should extend to a distance d from the extreme compression fibre and should be anchored at both ends to develop the design yield strength of the reinforcement.

7.3.7.1.5 Spacing of web reinforcement. Where web reinforcement is required and is placed perpendicular to the axis of the member, it should be spaced not further apart than $0.50d$ in nonprestressed concrete and $0.75h$ in prestressed concrete, but not more than 24 in. Inclined stirrups and bent bars should be so spaced that every 45 degree line, extending toward the reaction from the middepth of the member, $0.50d$, to the longitudinal tension bars, should be crossed by at least one line of shear reinforcement.

7.3.7.1.6 The design yield strength of nonprestressed shear and torsion reinforcement should not exceed 60,000 psi.

7.3.7.2 *Shear Strength*

7.3.7.2.1 The design shear stress v_u should be computed by:

$$v_u = \frac{V_u}{\phi b_w d} \tag{7.26}$$

where b_w should be taken as the width of web and d should be taken as the distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement. For prestressed concrete members d need not be taken less than $0.80h$. For a circular section, d need not be taken less than the distance from the extreme compression fibre to the centroid of the longitudinal reinforcement in the opposite half of the member.

7.3.7.2.2 When the reaction in the direction of the applied shear introduces compression into the end region of the member, sections located less than a distance d from the face of the support may be designed for the same v_u as that computed at a distance d. For prestressed concrete members, sections located at a distance less than $h/2$ may be designed for the shear computed at $h/2$.

7.3.7.2.3 When the design shear stress exceeds the shear stress carried by the concrete, shear reinforcement should be provided. Whenever applicable, the effects of torsion according to the recommendations of Section 7.3.8 should be added.

7.3.7.3 *Shear Stress Carried by Concrete for Nonprestressed Members*

7.3.7.3.1 The shear stress carried by the concrete should not exceed:

$$v_c = 2\sqrt{f'_c} \tag{7.27}$$

unless a more detailed analysis is made in accordance with Sections 7.3.7.3.2 or 7.3.7.3.3.

For members subjected to axial tension or torsion, the shear stress carried by the concrete should not exceed the values given in Sections 7.3.7.3.4 or 7.3.7.3.5.

7.3.7.3.2 The shear stress carried by the concrete may be computed by:

$$v_c = 1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u} \tag{7.28}$$

but v_c should not exceed $3.5\sqrt{f'_c}$. The quantity $V_u d/M_u$ should not be taken greater than 1.0, where M_u is the bending moment occurring simultaneously with V_u at the section considered.

7.3.7.3.3 For members subjected to axial compression, v_c may be computed by:

$$v_c = 2\left(1 + 0.0005\frac{N_u}{A_g}\right)\sqrt{f'_c} \tag{7.29}$$

The quantity N_u/A_g should be expressed in pounds per square inch.

7.3.7.3.4 For members subjected to significant axial tension, shear reinforcement should be designed to carry the total shear, unless a more detailed analysis is made using

$$v_c = 2\left(1 + 0.002\frac{N_u}{A_g}\right)\sqrt{f'_c} \tag{7.30}$$

where N_u is negative for tension, the quantity N_u/A_g should be expressed in pounds per square inch.

7.3.7.3.5 At cross-sections subject to a design torsion stress exceeding $1.5f'_c$, in accordance with the recommendations of Section 7.3.8, the shear stress carried

by the concrete should not exceed:

$$v_c = \frac{2\sqrt{f'_c}}{\sqrt{\left(1 + \frac{v_{tu}}{1.2v_u}\right)^2}} \qquad (7.31)$$

7.3.7.4 *Shear Stress Carried by Concrete for Prestressed Members*

7.3.7.4.1 For members having an effective prestress force at least equal to 40% of the tensile strength of the flexural reinforcement, the shear stress carried by the concrete should not exceed:

$$v_c = 0.6\sqrt{f'_c} + 700\frac{V_u d}{M_u} \qquad (7.32)$$

but v_c need not be taken less than $2f'_c$ nor should v_c be greater than $5f'_c$ unless a more detailed analysis is made in accordance with Section 7.3.7.4.2. When applying Eq. (7.32), M_u is the bending moment occurring simultaneously with V_u at the section considered and d should be the distance from the extreme compression fibre to the centroid of the prestressing tendons.

7.3.7.4.2 The shear stress carried by the concrete may be computed as the lesser of v_{ci} or v_{cw}.

$$v_{ci} = 0.6\sqrt{f'_c} + \frac{\left(V_d + \frac{V_i M_{cr}}{M_{max}}\right)}{b_w d} \qquad (7.33)$$

but need not be taken less than $1.7\sqrt{f'_c}$, where

$$M_{cr} = (I/y_t)(6\sqrt{f'_c} + f_{pc} - f_d)$$
$$v_{cw} = 3.5\sqrt{f'_c} + 0.3f'_{pc} + V_p/b_d w \qquad (7.34)$$

Alternatively, v_{cw} may be taken as the shear stress corresponding to a multiple of deadload plus live load which results in a computed principal tensile stress of $4\sqrt{f'_c}$ at the centroidal axis of the member, or at the intersection of the flange and the web when the centroidal axis is in the flange. In a composite member, the principal tensile stress should be computed using the cross-section which resists live load.

When applying Eqs (7.33) and (7.34), d should be the distance from the extreme compression fibre to the centroid of the prestressing tendons or $0.8h$, whichever is greater. The values of M_{max} and V_i in Eq. (7.33) should be computed from the load distribution causing maximum moment to occur at a section.

In a pretensioned member in which the section at a distance $h/2$ from the face of the support is closer to the end of the beam than the transfer length of the tendons, the reduced prestress should be considered when calculating v_{cw}. This value of v_{cw} should also be taken as the maximum limit for Eq. (7.32). The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 70 diameters for strand and 100 diameters for single wires.

7.3.7.5 The recommendations for shear stress carried by the concrete apply to normal weight concrete. When lightweight aggregate concretes are used, one of the following modifications should apply:

(a) When f_{ct} is determined, the stress, v_c or v_{tc} should be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_c}$ but the value of $f_{ct}/6.7$ used should not exceed $\sqrt{f'_c}$.

(b) When f_{ct} is not determined, all values of $\sqrt{f'_c}$ affecting v_c, v_{tc}, or M_{cr} should be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

7.3.7.6 *Design of Shear Reinforcement*

7.3.7.6.1 Shear reinforcement should conform to the general recommendations of Section 7.3.7.1. When shear reinforcement perpendicular to the axis of the member is used, the required area should be computed by:

$$A_v = \frac{(v_u - v_c)b_w s}{f_y} \qquad (7.35)$$

7.3.7.6.2 When inclined stirrups or bent bars are used as shear reinforcement, the following applies:

(a) When inclined stirrups are used, the required area should be computed by:

$$A_v = \frac{(v_u - v_c)b_w s}{f_y(\sin\alpha + \cos\alpha)} \qquad (7.36)$$

(b) When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support, the required area should be computed by:

$$A_v = \frac{(v_u - v_c)b_w d}{f_y \sin\alpha} \qquad (7.37)$$

in which $(v_u - v_c)$ should not exceed $3\sqrt{f'_c}$.

(c) When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, the required area should be computed by Eq. (7.36).

(d) Only the centre three-fourths of the inclined portion of any longitudinal bar that is bent should be considered effective for shear reinforcement.

7.3.7.6.3 Where more than one type of shear reinforcement is used to reinforce the same portion of the

member, the required area should be computed as the sum for the various types separately.

7.3.7.6.4 When $(v_u - v_c)$ exceeds $4\sqrt{f'_c}$, the maximum spacings recommended in Section 7.3.7.1 should be reduced by one-half.

7.3.7.6.5 The value of $(v_u - v_c)$ should not exceed $8\sqrt{f'_c}$.

7.3.8 *Combined Torsion and Shear for Nonprestressed Members*

7.3.8.1 Torsion effects should be included for shear and bending whenever the design torsion stress exceeds $1.5\sqrt{f'_c}$ For members with rectangular or flanged sections, the design torsion stress should be computed by:

$$v_{tu} = \frac{3T_u}{\phi \Sigma x^2 y} \tag{7.38}$$

where $\Sigma x^2 y$ should be taken for the component rectangles of the section, but the overhanging flange width used in design should not exceed three times the thickness of the flange. The rectangles may be so formed as to provide the largest value of $\Sigma x^2 y$.

7.3.8.2 Rectangular box sections may be taken as solid sections, provided that the wall thickness, h, is at least $x/4$. For box sections with a wall thickness less than $x/4$, but greater than $x/10, \Sigma x^2 y$ should be multiplied by $4h/x$. When h is less than $x/10$, the stiffness of the wall should be considered.

7.3.8.3 Hollow box sections with a wall thickness less than $x/4$, in which the longitudinal torsion reinforcement consists of less than eight bars distributed around the section perimeter, should have at each interior corner a fillet with minimum leg size of $x/6$. When the longitudinal torsion reinforcement consists of eight or more bars distributed around the section perimeter, fillets should be provided at each interior corner, with a minimum leg size of $x/12$ but preferably not more than 4 in.

7.3.8.4 Sections located less than a distance d from the face of the support may be designed for the same torsion as that computed at a distance d.

7.3.8.5 The torsion stress carried by the concrete should not exceed:

$$v_{tc} = \frac{2.4\sqrt{f'_c}}{\sqrt{1 + \left(1.2\frac{v_u}{v_{tu}}\right)^2}} \tag{7.39}$$

7.3.8.6 When the design torsion stress v_{tu} exceeds the torsion stress carried by the concrete v_{tc}, torsion reinforcement should be provided.

7.3.8.7 For members subjected to significant axial tension, torsion reinforcement should be designed to carry the total torque, unless a more detailed analysis is made in which v_{tc} given by Eq. (7.39), and v_c given by Eq. (7.31) are multiplied by $(1 + 0.002N_u/A_g)$, where N_u is negative for tension.

7.3.8.8 For noncompact open cross-sections, warping torsional resistance may be taken into account based on an analysis (7–3).

7.3.8.9 *Design of Torsion Reinforcement*

7.3.8.9.1 Torsion reinforcement, where required, should be provided in addition to reinforcement required to resist shear, flexure, and axial forces. The reinforcement required for torsion may be combined with that required for other forces, provided the area furnished is the sum of the individually required areas and the most restrictive requirements for spacing and placement are met.

7.3.8.9.2 The required area of closed stirrups should be computed by:

$$A_t = \frac{(v_{tu} - v_{tc}) s \Sigma x^2 y}{3\alpha_t x_1 y_1 (f_y)} \tag{7.40}$$

where $\alpha_t = [0.66 + 0.33(y_1/x_1)]$, but not more than 1.50.

7.3.8.9.3 The spacing of closed stirrups should not exceed $(x_1 + y_1)/4$, or 12 in., whichever is the smaller.

7.3.8.9.4 The required area of longitudinal bars should be computed by:

$$A_l = 2A_t \frac{x_1 + y_1}{s} \tag{7.41}$$

or by

$$A_l = \left[\frac{400xs}{f_y}\left(\frac{v_{tu}}{v_{tu} + v_u}\right) - 2A_t\right]\left(\frac{x_1 + y_1}{s}\right) \tag{7.42}$$

whichever is the greater. The value of A_l computed by Eq. (7.42) need not exceed that obtained by substituting

$$\frac{50 b_w s}{f_y} \text{ for } 2A_t$$

7.3.8.9.5 The spacing of longitudinal bars, distributed around the perimeter of the stirrups, should not exceed 12 in. At least one longitudinal bar should be placed in each corner of the stirrups.

7.3.8.9.6 Torsion reinforcement should be provided at least a distance $(d + b)$ beyond the point theoretically required.

7.3.8.9.7 For T-, I-, or L-sections, the arrangement of the torsion reinforcement should reflect the choice of the component rectangles.

7.3.8.9.8 The design torsion stress, v_{tu}, should not exceed:

$$v_{tu} = \frac{12\sqrt{f'_c}}{\sqrt{1 + \left(1.2\frac{v_u}{v_{tu}}\right)^2}} \tag{7.43}$$

7.3.9 *Shear Friction*

7.3.9.1 These provisions apply where it is inappropriate to consider shear as a measure of diagonal tension, and particularly in design of reinforcing details for precast concrete structural elements.

7.3.9.2 A crack should be assumed to occur along the shear path. Relative displacement should be considered resisted by friction maintained by shear-friction reinforcement across the crack. This reinforcement should be approximately perpendicular to the assumed crack.

7.3.9.3 The shear stress v_u should not exceed $0.2f'_c$ nor 800 psi.

7.3.9.4 The required area of reinforcement should be computed by:

$$A_{vf} = \frac{V_u}{\phi f_y \mu} \tag{7.44}$$

The coefficient of friction, μ, should be 1.4 for concrete cast monolithically, 1.0 for concrete placed against hardened concrete, and 0.7 for concrete placed against as-rolled structural steel.

7.3.9.5 Direct tension across the assumed crack should be provided for by additional reinforcement.

7.3.9.6 The shear-friction reinforcement should be well distributed across the assumed crack and should be adequately anchored on both sides by embedment, hooks, or welding to special devices.

7.3.9.7 When shear is transferred between concrete placed against hardened concrete, the interface should be rough, clean, and free of laitance with a full amplitude of approximately 1/4 in. When shear is transferred between as-rolled steel and concrete, the steel should be clean and without paint.

7.3.10 *Horizontal Shear Design for Composite Concrete Flexural Members*

7.3.10.1 In a composite member, full transfer of the shear forces should be assured at the interfaces of the separate components.

7.3.10.2 Full transfer of horizontal shear forces may be assumed when all of the following are satisfied:

(a) the contact surfaces are clean and intentionally roughened

(b) minimum ties are provided in accordance with Section 7.3.10.6

(c) web members are designed to resist the entire vertical shear, and

(d) all shear reinforcement is anchored into all intersecting components. When all of the above are not satisfied, horizontal shear should be fully investigated.

7.3.10.3 The design horizontal shear stress, v_{dh}, may be computed at any cross-section as

$$v_{dh} = \frac{V_u}{\phi b_v d}$$

in which d is for the entire composite section. Alternatively, in any segment not exceeding one-tenth of the span, the actual change in compressive or tensile force to be transferred may be computed, and provisions made to transfer that force as horizontal shear to the supporting element.

7.3.10.4 The horizontal shear may be transferred at contact surfaces using the permissible horizontal shear stress, v_h, stated below.

(a) When ties are not provided, but the contact surfaces are clean and intentionally roughened, permissible v_h = 80 psi.

(b) When the minimum tie requirements of Section 7.3.10.6 are provided and the contact surfaces are clean but not intentionally roughened, permissible v_h = 80 psi.

(c) When the minimum tie requirements of Section 7.3.10.6 are provided and the contact surfaces are clean and intentionally roughened, permissible v_h = 350 psi.

(d) When v_{dh} exceeds 350 psi, design for horizontal shear should be made in accordance with Section 7.3.9.

7.3.10.5 When tension exists perpendicular to any surface, shear transfer by contact should be assumed only when the minimum tie requirements of Section 7.3.10.6 are satisfied.

7.3.10.6 *Ties for Horizontal Shear*

7.3.10.6.1 When vertical bars or extended stirrups are used to transfer horizontal shear, the tie area should not be less than that required by Section 7.3.7.1.1 and the spacing should not exceed four times the least dimension of the supported element nor 24 in.

7.3.10.6.2 Ties for horizontal shear may consist of single bars, multiple leg stirrups, or the vertical legs of welded wire fabric. All ties should be adequately anchored into the components by embedment or hooks.

7.3.10.7 *Measure of Roughness*

Internal roughness should be assumed only when the contact surface is roughened, clean, and free of laitance. Roughness should have a full amplitude of approximately 1/4 in.

7.3.11 *Special Recommendations for Brackets and Short Cantilevers*

7.3.11.1 These recommendations apply to brackets and short cantilevers having a shear-span-to-depth ratio, a/d;

of unity or less. The distance d should be measured at a section adjacent to the face of the support, but should not be taken greater than twice the depth of the member at the outside edge of the bearing area.

7.3.11.2 When special provisions are made to avoid tension due to restrained shrinkage and creep, so that the member is subject to shear and moment only, the design shear stress should not exceed:

$$v_u = 6.5\left(1 - 0.5\frac{\alpha}{d}\right)(1 + 64\rho_v)\sqrt{f'_c} \qquad (7.45)$$

where $$\rho_v = \frac{A_s + A_h}{bd}$$

but not greater than

$$0.20\frac{f'_c}{f_y}$$

and A_h should not exceed A_s.

7.3.11.3 For members subject to direct tension, the design shear stress should not exceed:

$$v_u = \left[6.5 - 5.1\sqrt{\frac{N_u}{V_u}}\right]\left[1 - 0.5\frac{\alpha}{d}x\right]$$
$$\left\{1 + \left[64 + 160\sqrt{\frac{N_u^3}{V_u}}\right]\rho\right\}\sqrt{f'_c} \qquad (7.46)$$

where ρ should not exceed 0.13 f'_c/f_y and N_u/V_u should not be taken less than 0.20. The tensile force N_u should be regarded as a live load even when it results from creep, shrinkage, or temperature change.

7.3.11.4 Closed stirrups or ties parallel to the main tension reinforcement having a total cross-sectional area A_h not less than 0.50 A_s should be uniformly distributed within two-thirds of the effective depth adjacent to the main tension reinforcement.

7.3.11.5 The ratio $\rho = A_s/bd$ should not be less than 0.04 (f'_c/f_y).

7.3.12 *Special Recommendations for Deep Flexural Members*

7.3.12.1 These recommendations apply to members with a span-to-depth ratio, l_v/d, less than five and the members are loaded at the top or compression face.

7.3.12.2 The shear stress carried by the concrete should not exceed:

$$v_c = 2\sqrt{f'_v} \qquad (7.47)$$

unless a more detailed analysis is made using

$$v_c = \left(3.5 - 2.5\frac{M_u}{V_u d}\right)\left(1.9\sqrt{f'_c} + 2500\rho_w\frac{V_u d}{M_u}\right) \qquad (7.48)$$

where the term $\left(3.5 - 2.5\frac{M_u}{V_u d}\right)$ should not exceed 2.5, and v_c should not exceed $6\sqrt{f'_c}$ · M_u and V_u are the bending moment and shear occurring simultaneously at the critical section defined by Section 7.3.12.3.

7.3.12.3 The critical section for shear measured from the face of the support should be taken at 0.15 l_n for uniformly loaded flexural members and 0.50 for members with concentrated loads, but not greater than d. Shear reinforcement required at the critical section should be used throughout the span.

7.3.12.4 The design shear stress should not exceed $8\sqrt{f'_c}$ when l_n/d is less than 2. When l_n/d is between 2 and 5, the design shear stress should not exceed:

$$v_u = \frac{2}{3}\left(10 + \frac{l_n}{d}\right)\sqrt{f'_c} \qquad (7.49)$$

7.3.12.5 The area of shear reinforcement should be computed from:

$$\frac{A_v}{s}\frac{\left(1 + \frac{l_n}{d}\right)}{12} + \frac{A_{vh}}{s_2}\frac{\left(11 - \frac{l_n}{d}\right)}{12} = \frac{(v_u - v_c)b_w}{f_y} \qquad (7.50)$$

7.3.12.6 The area of shear reinforcement A_v perpendicular to the main reinforcement should not be less than 0.0015 bs, and s should not exceed $d/5$ or 18 in. The area of shear reinforcement A_{vh} parallel to the main reinforcement should not be less than 0.0025 bs_2 and s_2 should not exceed $d/3$ or 18 in.

7.3.13 *Special Recommendations for Slabs and Footings*

7.3.13.1 The shear strength of slabs and footings in the vicinity of concentrated loads or reactions should be governed by the more severe of two conditions:

(a) The slab or footing acting as a wide beam, with a critical section extending in a plane across the entire width and located at a distance d from the face of the concentrated load or reaction area. For this condition the slab or footing should be designed in accordance with the recommendations in Sections 7.3.7.1 through 7.3.7.6.

(b) Two-way action for the slab or footing, with a critical section perpendicular to the plane of the slab and located so that its periphery is a minimum and approaches no closer than $d/2$ to the periphery of the concentrated load or reaction area. For this condition the slab or footing should be designed in accordance with Sections 7.3.13.2 and 7.3.13.3.

7.3.13.2 The peripheral shear stress should be computed by:

$$v_u = \frac{V_u}{\phi b_0 d} \qquad (7.51)$$

in which V_u and b_0 are taken at the critical section defined

in Section 7.3.13.1(b). The peripheral shear stress should not exceed the shear stress carried by the concrete $v_c = 4\sqrt{f'_c}$ unless shear reinforcement is provided. If the ratio of the short side to the long side of the concentrated load or reaction area, B_c, is less than 0.5, v_u should not exceed the larger of $v_c = (2 + 4B_c)\sqrt{f'_c}$ or $2\sqrt{f'_c}$, unless shear reinforcement is provided. If shear reinforcement is provided in accordance with Section 7.3.13.3, v_u should not exceed $6\sqrt{f'_c}$.

7.3.13.3 Shear reinforcement consisting of bars or wires may be provided. For design of such shear reinforcement, shear stresses should be investigated at the critical section defined in Section 7.3.13.1(b) and at successive sections more distant from the support; and the shear stress carried by the concrete at any section should not exceed $2\sqrt{f'_c}$. Where the peripheral shear stress exceeds the shear stress carried by the concrete, shear reinforcement should be provided.

7.3.13.4 Shear reinforcement consisting of steel I- or channel shapes should be designed in accordance with the following provisions, which do not apply where shear is transferred to a column from an edge of a slab.*

7.3.13.4.1 Each shearhead should consist of steel shapes so fabricated by welding that each pair of the four arms is continuous through the column. The ends of shearheads may be cut at angles up to 30 degrees with the horizontal, provided that the plastic moment capacity of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead. The ratio α_v, between the EI-value for each shearhead arm and that for the surrounding composite cracked slab section of width $(c_2 + d)$ should not be less than 0.15. All compressive flanges of the steel shapes should be located within $0.3d$ of the compressive surface of the concrete slab. The steel shapes should not be deeper than 70 times their web thickness.

7.3.13.4.2 The full plastic moment of resistance, M_p, required for each arm of the shearhead should be computed by:

$$M_p = \frac{V_u}{\phi 8}\left[h_v + \alpha_v\left(l_v \frac{c_1}{2}\right)\right] \qquad (7.52)$$

where ϕ is the capacity reduction factor for flexure and l_v is the minimum length of each shearhead arm required to comply with the requirements of Sections 7.3.13.4.3 and 7.3.13.4.4.

7.3.13.4.3 The critical slab section should be perpendicular to the plane of the slab. The section should cross each shearhead arm at points three-quarters of the distance

$$\left(l_v - \frac{c_1}{2}\right)$$

from the column face to the end of the shearhead, and it should be so located that its periphery is a minimum. However, the critical section need not approach closer than $d/2$ to the periphery of the column.

7.3.13.4.4 The shear stress, v_u, should not exceed $4\sqrt{f'_c}$ on the critical section defined in Section 7.3.13.4.3.

7.3.13.4.5 The shearhead may be assumed to contribute a resisting moment, M_v, to each column strip of the slab computed by:

$$M_v = \frac{\phi \alpha_v V_u}{8}\left(l_v - \frac{c_1}{2}\right) \qquad (7.53)$$

where ϕ is the capacity reduction factor for flexure, and l_v is the length of each shearhead arm actually provided. However, M_v should exceed neither 30% of the total moment resistance required for each column strip of the slab, nor the change in column strip moment over the length l_v, nor the value of M_p given by Eq. (7.52).

7.3.14 *Transfer of Moments to Compression Members*

7.3.14.1 Except where a compression member is restrained on three or four sides by flexural members of approximately equal depth, the shear force, V_u, acting on the compression member within the depth of the larger flexural member should be taken as the ultimate force exerted by the tensile reinforcement framing directly into the compression member from that flexural member. The shear strength should be calculated in accordance with Sections 7.3.7.2 through 7.3.7.6. Minimum shear reinforcement should be provided in accordance with Section 7.3.7.1.1.

7.3.14.2 When unbalanced gravity load, wind, or earthquake causes transfer of bending moment between compression and flexural member, the per cent of the moment given by

$$\frac{100}{\left[1 + \frac{2}{3}\sqrt{\frac{c_1}{c_2}}\right]}$$

should be considered transferred by flexure across the periphery of the critical section defined in Section 7.3.13.1 (b), and the remainder by eccentricity of the shear about the centroid of the critical section. Shear stresses should be taken as varying linearly about the centroid of the critical section and the maximum shear stress should not exceed $4\sqrt{f'_c}$.

7.3.15 *Bearing*

7.3.15.1 Bearing stresses should not exceed $0.85\phi f_c$, except as provided below.

7.3.15.2 When the supporting surface is wider on all sides than the loaded area, the permissible bearing stress on the loaded area may be multiplied by $\sqrt{A_2/A_1}$, but not more than 2.

* Tests indicate that, due to torsional effects and other peculiarities, the behaviour of shearheads located at a slab edge differs substantially from that at other locations.

7.3.15.3 When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustrum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.

7.3.15.4 When the loaded area is subjected to high edge stresses due to deflection or eccentric loading, the permissible bearing stress on the loaded area should be multiplied by a factor of 0.75.

REFERENCES

7.1 Anderson, A.R., and S.E. Moustafa, "Ultimate Strength of Prestressed Concrete Piles and Columns," ACI Journal, *Proceedings V. 67*, No. 8, August 1970, pp. 620–635.

7.2 MacGregor, J.G., J.E. Breen, and E.O. Pfrang, "Design of Slender Concrete Columns," ACI Journal, *Proceedings V. 67*, No. 1, January 1970, pp. 6–28.

7.3 Kollbrunner, C.F., and F. Basier, *Torsion in Structures*. Springer Verlag, New York, Inc., New York, 1969.

7.4 *CEB Information Bulletin # 83*, pp. 65–66.

DETAILING OF REINFORCEMENT

13.2 Development and Splices of Reinforcement

13.2.1 *Development Requirements, General*

13.2.1.1 The calculated tension or compression in the reinforcement at each section should be developed on each side of that section by embedment length or end anchorage or a combination thereof. For reinforcement bars in tension, hooks may be used in developing the reinforcement bars.

13.2.1.2 Tension reinforcement may be anchored by bending it across the web and making it continuous with the reinforcement on the opposite face of the member, or anchoring it there.

13.2.1.3 The critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent.

13.2.1.4 Reinforcement should extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member, 12 bar diameters, or one-sixteenth of the clear span, whichever is greater, except at supports of simple spans and at the free end of cantilevers.

13.2.1.5 Continuing reinforcement should have an embedment length not less than the development length l_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

13.2.1.6 Flexural reinforcement should not be terminated in a tension zone unless one of the following conditions is satisfied:

(a) The shear at the cutoff point does not exceed two-thirds that permitted, including the shear strength of furnished web reinforcement.

(b) Stirrup area in excess of that required for shear is provided along the terminated reinforcement over a distance from the termination point equal to three-fourths the effective depth of the member. The excess stirrup-area should not be less than 60 $b_w s/f_y$. The resulting spacing should not exceed $(d/8)\beta_b$ where β_b is the ratio of the area cut off to the total area of reinforcement at the section.

(c) For # 11 and smaller bars, the continuing bars provide double the area required for flexure at the cut-off point and the shear does not exceed three-fourths that permitted.

13.2.2 *Positive Moment Reinforcement*

13.2.2.1 At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members should extend along the same face of the member into the support. In beams, the length of this extension should be at least 6 in. When a flexural member is part of the lateral load resisting system, the positive reinforcement required to be extended into the support should be anchored to develop the full f_y in tension at the face of the support.

13.2.2.2 At simple supports and at points of inflection, positive moment tension reinforcement should be limited to a diameter such that l_d computed for f_y does not exceed

$$M/V + l_a$$

where M is the computed moment capacity assuming all positive moment tension reinforcement at the section to be fully stressed. V is the maximum applied design shear at the section. l_a at a support should be the sum of the embedment length beyond the centre of the support and the equivalent embedment length of any furnished hook or mechanical anchorage, l_a at a point of inflection should be limited to the effective depth of the member or $12d_b$, whichever is greater. The value M/V in the development length limitation may be increased 30% when the ends of the reinforcement are confined by a compressive reaction.

13.2.3 *Negative Moment Reinforcement*

13.2.3.1 Tension reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, should be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

13.2.3.2 At least one-third the total reinforcement provided for negative moment at the support should have an embedment length beyond the point of inflection not less than the effective depth of the member, 12 bar diameters, or one-sixteenth of the clear span, whichever is greater.

13.2.4 *Special Members*

Adequate end anchorage snould be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footing; brackets; deep flexural members, or members in which the tension reinforcement is not parallel to the compression face.

13.2.5 *Development Length of Deformed Bars and Deformed Wire in Tension*

The development length l_d, in inches, of deformed bars and deformed wire in tension should be computed as the product of the basic development length of (a) and the applicable modification factor or factors of (b), (c) and (d), but l_d should be not less than recommended in (e).

(a) The basic development length should be
For # 11 or smaller bars
(Note 1) ..... $0.04A_b f_y/\sqrt{f'_c}$
but not less than
(Note 2) ..... $0.004d_b f_y$
For # 14 bars
(Note 3) ..... $0.085f_y/\sqrt{f'_c}$
For # 18 bars
(Note 3) ..... $0.11f_y/\sqrt{f'_c}$
For deformed
wire ..... $0.03d_b f_y/\sqrt{f'_c}$

(b) The basic development length should be multiplied by the applicable factor or factors for:
Top reinforcement (Note 4) 1.4
Reinforcement bars with f_y greater than 60,000 psi ..... $2-60,000/f_y$

(c) When lightweight aggregate concrete is used, the basic development lengths should be multiplied by 1.33 for "all lightweight" concrete and 1.18 for "sand-lightweight" concrete with linear interpolation when partial sand replacement is used, or the basic development length may be multiplied by 6.7 $\sqrt{f'_c/f_{ct}}$, but not less than 1.0, when f_{ct} is determined.

(d) The basic development length may be multiplied by the applicable factor or factors for:
Reinforcement being developed in the length under consideration and spaced laterally at least 6 in. on centre and at least 3 in. from the side face of the member to the edge bar, measured in the direction of the spacing ..... 0.8. Where anchorage or development for f_y is not specifically required, reinforcement in flexural members in excess of that required ..... (A_s required)/(A_s provided) Reinforcement bars enclosed within a spiral which is no less than 1/4 in. diameter and no more than 4 in. pitch ..... 0.75.

(e) The development length, l_d, should be taken as not less than 12 in. except in the computation of lap splices and anchorage of web reinforcement.

NOTES:

1. The constant carries the unit of 1/in.
2. The constant carries the unit of in. 2/lb.
3. The constant carries the unit of in.
4. Top reinforcement is horizontal reinforcement so placed that more than 12 in. of concrete is cast in the member below the bar as part of the same concrete placement that encases the bar.

13.2.6 *Development Length of Deformed Bars in Compression*

The development length l_d for bars in compression should be computed as $0.02f_y d_b/\sqrt{f'_c}$ but should not be less than $0.0003f_y d_b$ or 8 in. Where excess bar area is provided, the l_d length may be reduced by the ratio of area required to area provided. The development length may be reduced 25% when the reinforcement is enclosed by spirals not less than 1/4 in. in diameter and not more than 4 in. pitch.

13.2.11 *Development Length of Prestressing Strand*

13.2.11.1 Three or seven-wire pretensioning strand should be bonded beyond the critical section for a development length, in inches, not less than

$$(f_{ps} - 2/3f_{se})d_b$$

when d_b is the nominal diameter in inches, f_{ps} and f_{se} are expressed in kips per square inch and the expression in the parentheses is used as a constant without units. Investigation may be limited to those cross-sections nearest each end of the member which are required to develop their full strength under the specified design load.

13.2.11.2 Where bonding of the strand does not extend to the end of the member the bonded development length recommended in Section 13.2.11.1 should be doubled.

13.2.12 *Mechanical Anchorage*

Any mechanical device capable of developing the strength of the reinforcement without damage to the concrete may be used as anchorage.

13.2.13 *Anchorage of Web Reinforcement*

13.2.13.1 Web reinforcement should be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other steel will permit, and in any case the ends of single leg, single U-, or multiple U-stirrup should be anchored by one of the following means:

(a) A standard hook plus an effective embedment of 0.5 l_d. The effective embedment of a stirrup leg should be taken as the distance between the mid-depth of the member $d/2$ and the start of the hook (point of tangency).

(b) Embedment above or below the middepth, $d/2$, of the beam on the compression side for a full development length l_d but not less than 24 bar diameters or, for deformed bars or deformed wire, 12 in.

(c) For # 5 or smaller bars bending around the longitudinal reinforcement through at least 135 degrees, the effective embedment of a stirrup degree should be taken as the distance between the mid-depth of the member $d/2$ and the start of the hook (point of tangency).

NOTATION

a = depth of equivalent rectangular stress block, in. [Article 7.3.3 (f)]

a_b = depth of equivalent rectangular stress block for balanced conditions, in.

a_v = ratio of stiffness of shearhead arm to surrounding composite slab section

A = effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, sq. in. When the main reinforcement consists of several bar sizes the number of bars shall be computed as the total steel area divided by the area of the largest bar used.

A = axial load deformation (Chapter 5)

A = maximum expected acceleration at bedrock at the site expressed as a fraction of gravity (Section 5.5.3)

A_b = area of an individual bar, sq. in.

A_c = area of core of spirally reinforced compression measured to the outside diameter of the spiral, sq. in.

A_g = gross area of section, sq. in.

A_h = area of shear reinforcement parallel to main tension reinforcement, sq. in.

A_{ps} = area of prestressed reinforcement in tension zone, sq. in.

A_{pst} = total area of prestressing steel (Chapter 11)

A_s = area of tension reinforcement, sq. in.

A'_s = area of compression reinforcement, sq. in.

A_{sa} = area of bonded reinforcement in tension zone, sq. in.

A_{se} = area of stirrups transverse to potential bursting crack and within a distance s, in.

A_{sf} = area of reinforcement to develop compressive strength of over-hanging flanges of I- and T-sections, sq. in.

A_{st} = total area of longitudinal reinforcement (in compression members), sq. in.

A_t = area of one leg of a closed stirrup resisting torsion within a distance s, sq. in.

A_l = total area of longitudinal reinforcement to resist torsion sq. in.

A_v = area of shear reinforcement within a distance, s, or area of shear reinforcement perpendicular to main reinforcement within a distance, s, for deep beam, sq. in.

A_{vf} = area of shear-friction reinforcement, sq. in.

A_{vh} = area of shear reinforcement parallel to the main reinforcement within a distance s, sq. in.

A_w = area of an individual wire, sq. in.

A_1 = loaded area, sq. in (Sections 7.3.15 and 9.11.3.1)

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, sq. in., (Sections 7.3.15 and 9.11.3.1)

b = width of compressive face of member, in.

b_a = width of bearing plate measured in the same direction as d_a

b_e = width of concrete section in plane of potential bursting crack, in.

b_0 = periphery of critical section for slabs and footings, in.

b_v = the width of the cross-section being investigated for horizontal shear, in.

b_w = web width, or diameter of circular section, in.

B = buoyancy (Chapter 5)

B = band width (Section 11.5.4)

c = distance from extreme compressive fibre to neutral axis, in.

c_w = shape factor used to change basic wind pressure

c_1 = size of rectangular or equivalent rectangular column, capital or bracket measured in the direction in which moments are being determined.

c_2 = size of rectangular or equivalent rectangular column, capital or bracket measured transverse to the direction in which moments are being determined.

C = construction, handling and erection loads (Section 5)

C = a coefficient equal to A.R.S/Z (Section 5.5.3)

C = a stiffness parameter (Section 5.7.5.3)

C_m = a factor relating the actual moment diagram to an equivalent uniform moment diagram

CF = centrifugal force (Chapter 5)

d = distance from extreme compressive fibre to centroid of tension reinforcement, in.

d = size of pile (Chapter 11)

d' = distance from extreme compressive fibre to centroid of compression reinforcement, in.

d'' = distance from centroid of gross section, neglecting

the reinforcement, to centroid of tensile reinforcement, in.

d_a = distance from edge of member to centre of anchor plate, in.

d_b = nominal diameter of bar, wire, or prestressing strand, in.

d_c = thickness of concrete cover measured from the extreme tensile fibre to the centre of the bar located closest thereto, in.

D = dead load

D = distribution factor (Table 5.7.5.3)

DS = displacement of supports

e = eccentricity of design load parallel to axis measured from the centroid of the section. It may be calculated by conventional methods of frame analysis, in.

e = base of Napierian logarithms

e_b = M_b/P_b = eccentricity of the balanced condition load-moment relationship, in.

E = effective width of concrete slab resisting wheel or other concentrated load, ft.

E = earth pressure (Chapter 5)

E = efficiency or the decimal fraction of the single pile value to be used for each pile in the group (Chapter 11)

E_c = modulus of elasticity of concrete, psi (Section 7.2.4, Section 8.1.1 (d), and Section 9.3)

E_{ci} = modulus of elasticity of concrete at transfer of stress, psi

E_s = modulus of elasticity of steel, psi

EI = flexural stiffness of compression members

EQ = earthquake force

f = sag of suspension cables

f_a = axial stress, psi (Chapter 11)

f_b = bending stress, psi (Chapter 11)

f_b = average bearing stress in concrete on loaded area

f_c = extreme fibre compressive stress in concrete at service loads

f'_c = specified compressive strength of concrete, psi

f'_{ci} = compressive strength of concrete at time of initial prestress, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

f_{cds} = change in concrete stress at centre of gravity of prestressing steel due to all dead loads except the dead load acting at the time the prestressing force is applied, psi

f_{cp} = concrete bearing stress under anchor plate of post-tensioning tendon, psi

f_{cir} = concrete stress immediately after transfer at centre of gravity of prestressing steel, psi

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi

f_d = stress due to dead loads at the extreme fibre of a section at which tensile stresses are caused by applied load, psi

f_f = stress range ksi

f_h = tensile stress developed by a standard hook, psi (Section 13.2.8.1)

f_l = loss in effective prestress, psi

f_{lc} = loss in prestressing steel stress due to creep, psi

f_{le} = loss in prestressing steel stress due to elastic shortening, psi

f_{lf} = loss in prestressing steel stress due to friction, psi

f_{lp} = total loss in prestressing steel stress, psi

f_{lr} = loss in prestressing steel stress due to relaxation, psi

f_{ls} = loss in prestressing steel stress due to shrinkage, psi

$f_{\min}$ = minimum stress level where tension is positive and compression is negative, psi

f_0 = fundamental natural frequency of the guideway, Hz.

f_{pc} = maximum allowable compressive stress exclusive of effective prestress, psi (Chapter 11)

f_{pc} = compressive stress in the concrete, after all prestress losses have occurred, at the centroid of the cross-section resisting the applied loads or at the junction of the web and flange when the centroid lies in the flange, psi. (In a composite member, f_{pc} will be the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to bending moments resisted by the precast member acting alone.)

f_{pe} = compressive stress in concrete due to prestress only after all losses, at the extreme fibre of a section at which tensile stresses are caused by applied loads, psi

f_{po} = steel stress at jacking end of post-tensioning tendon, psi

f_{ps} = calculated stress in prestressing steel at design load, psi

f_{pu} = ultimate strength of prestressing steel, psi

f_r = modulus of rupture of concrete, psi

f_s = tensile stress in reinforcement at service loads, psi

f_{se} = effective stress in prestressing steel, after losses, psi

f'_s = stress in compressive reinforcement, psi

f'_{sb} = stress in compressive reinforcement at balanced conditions

f_t = extreme fibre tensile stress in concrete at service loads

x_1 = shorter centre-to-centre dimension of a closed rectangular stirrup

y = longer overall dimension of a rectangular part of a cross-section

y_1 = longer centre-to-centre dimension of a closed rectangular stirrup

y_t = distance from the centroidal axis of cross-section, neglecting the reinforcement, to the extreme fibre in tension

z = a quantity limiting distribution of flexural reinforcement

Z = reduction for ductility and risk assessment (Section 5.5.3)

α = total angular change of prestressing steel profile in radians from jacking end to any point x from that end

α = angle between inclined shear reinforcement and longitudinal axis of member

α_v = ratio of stiffness of shearhead arm to surrounding composite slab section

β_b = ratio of area of bars cut off to total area of bars at the section

β_c = ratio of the short side to the long side of the concentrated load or reaction area in Section 7.3.13.2

β_d = the ratio of maximum design load moment to maximum design total load moment, always positive

β_1 = a factor defined in Section 7.3.3

σ = moment magnification factor for compression members

μ = coefficient of friction (Section 7.3.9.4)

μ = curvature friction coefficient (Table 9.4.3)

ρ = constant for standard hook

ρ = ratio of nonprestressed reinforcement = A_s/bd

ρ = density of air

ρ' = compression reinforcement ratio = A'_s/bd

ρ_b = reinforcement ratio producing balanced conditions (Section 7.3.4.3)

ρ_f = $A_{sf}/b_w d$

$\rho_{\min}$ = minimum tension reinforcement ratio = A_s/bd in Section 7.3.4.1

ρ_p = ratio of prestressed reinforcement, A_{ps}/bd

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of a spirally reinforced compression member

ρ_v = $A_s + A_h/bd$

ρ_w = reinforcement ratio $A_s/b_w d$

φ = capacity reduction factor

ϕ = $\tan^{-1}(d/s)$ expressed in degrees (Chapter 11)

χ_1 = distance from load to point of support, ft. (Section 10.4.5)

χ_2 = distance from centre of rail post section under investigation (Section 10.4.5)

ψ = curvature at section due to shrinkage

ω = $\rho f_y/f'_c$

ω' = $\rho' f_t/f'_c = A'_s f_y/bdf'_c$

ω_p = $\rho_p f_{ps}/f'_c = A_{ps} f_{ps}/bdf'_c$

$\omega_w, \omega_{pw}, \omega_w$ = reinforcement indices for flanged sections computed as for ω, ω_p, and ω' except that b is the web width and the steel area is that required to develop the compressive strength of the web only.

APPENDIX 7

Some of the U.K. (DTp.) Department of Transport Bridge Engineering Technical Memoranda (now DTp., previously DoE. — Dept. of Environment):

NUMBER	TITLE	DATE
H2/74	Suite of road design and analysis programs Program HECB/R/12 (FREEWAY)	11/3/74
H5/74	Suite of road design and analysis programs Program system (HOPS)	15/5/74
H6/74	Design flows for motorways and rural all-purpose roads (Corrigendum: 1/4/75)	12/8/74
H9/74	System of drainage and analysis programs Program HECB/R/7/ (DAPHNE) and HECB/R/11 (SAFRON)	3/9/74
H6/75	Design of rural motorway-to-motorway interchanges General guidelines	25/4/75
H9/75	Grass cutting and hedgerow treatment on trunk roads and motorways	11/7/75
H10/75	Clearance to bridges on dual two-lane roads	25/7/75
H13/75	Highways optimisation program system (HOPS) Hops user guide	11/9/75
H15/75	Suite of road design and analysis programs Program HECB R/18 (SETTLE)	30/9/75
H17/75	Design of rural motorway-to-motorway interchanges Single lane links	1/12/75
H18/75	Design of rural motorway-to-motorway interchanges Merging and diverging lanes	1/12/75

NUMBER	TITLE	DATE
H3/76	Model contract document for site investigation	Feb 1976
H7/76	Standardisation in format and preparation of drawings	4/6/76
H15/76	Introduction to specification for road and bridge work 1976 and notes for guidance on specification	17/12/76
H3/77	Suite of highway maintenance analysis programs Program suite HECB/R16 (CHART)	15/6/77
H4/77	Standard terms of appointment for testing firms employed on roadworks	21/6/77
H5/77	Method of measurement for road and bridge works 1977 and notes for guidance and library of standard item descriptions	29/6/77
H6/77	Suite of slope stability programs Program HECB/R/22 (CIRCA 1.0)	9/12/77
H2/78	Highway cost model system program HECB/R/20 (COSMOS)	11/1/78
H3/78	Notes for guidance and library of standard item descriptions for use with the method of measurement for road and bridge works, 1977 (Corrigendum, 1/3/78)	12/1/78
H5/78	Model contract document for topographical surveys	Feb 1978

NUMBER	TITLE	DATE
IM 4	Pulverised fuel ash Backfilling to structures	19/12/69
IM 5	Formation of continuity joints in bridge decks	21/1/70
IM 7	Use of stainless steel in bridge works	27/7/70
IM 9	Line and level on log-span structures	14/8/70
IM 11	PTFE in bridge bearings	23/10/70
IM 12	New proprietary products	30/4/71
BE 5	The design of highway bridge parapets (Third revision 16/11/73)	30/9/70
BE 11	Lightweight aggregate concrete for use in highway structures	20/4/69
BE 13	Fatigue risk in Bailey bridges	8/4/68
BE 14	Headroom standards (Addendum 1; 29/1/71)	11/12/68
BE 16	Provisional fatigue requirements for steel bridges	Jan. 69
BE 18	Suite of bridge design and analysis programs	10/7/69
BE 23	Shear key decks	27/11/70
BE 27	Waterproofing and surfacing of bridge decks	17/6/70
BE 28	Unauthorised access to motorway bridges	11/6/71
BE 29	MOT/C & CA standard bridge beams	2/7/71
BE1/72	Department of the Environment suite of bridge design and analysis programs	28/3/72
BE3/72	Expansion joints for use in highway bridge decks	31/10/72
BE7/72	Appointment of testing firms employed on bridgeworks	15/8/72
BE1/73	Reinforced concrete for highway structures (1st Revision; 9/8/73)	30/1/73
BE2/73	Prestressed concrete for highway structures	30/1/73
BE3/73	The assessment of highway bridges for construction and use vehicles	28/2/73
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BE7/73	Rules for the design and construction of preflexed beams in highway bridges	9/10/73
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