

# The Cause of Cracking in Post-Tensioned Concrete Box Girder Bridges and Retrofit Procedures



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**O**n the premise that "Those who ignore history will be condemned to relive it," this paper attempts to document those types of problems that lead to cracking in post-tensioned concrete box girder bridges and have been encountered in both Europe and the United States.

These problems are not limited to those of prestressed box girder bridges, but may occur in other types of structures with similar conditions and details. Problems are generally attributed to poor quality control, inadequate design details, inferior workmanship, or lack of understanding as to how the structure will behave, either through ignorance or because a particular phenomenon is unknown to the current state-of-the-art, or a combination of all

these factors.

Cracking in post-tensioned concrete box girders generally results, in a broad sense, for the following reasons: inadequate flexural and shear capacity, non-consideration of thermal stresses, insufficient attention to stresses developed by curvature of tendons, improper or inappropriate construction techniques, lack of quality workmanship to meet the tolerances necessary for problem free structures, and understrength materials.

It should also be noted that cracks are not totally avoidable in post-tensioned concrete box girder bridges, since not all portions of the structure are precompressed in three directions and local concrete shrinkage is not always controllable. Certain cracks may not be structurally serious, while others are. It

is, therefore, important for the structural engineer to determine the structural significance of a crack and its effect on the serviceability of the structure.

In general, in today's technology it is very rare that the triggering mechanism can be traced to a single source. Each of the sources cited above, when taken individually, usually produce stresses or overstresses that are minor and within allowable limits. However, if one or a number of the sources is ignored or inadvertently overlooked, a superposition of stress can occur such that the capacity of the concrete is exceeded and the only relief the structure has from this over-stress is to crack.

Therefore, it can be seen that it is very difficult to diagnose a specific triggering mechanism. It generally requires an extensive investigative effort, usually by a process of elimination, to determine a single or small group of probable causative factors.

When serious and/or extensive cracking distress occurs in a project, there may be a concern for the integrity of the structure. This usually results in delay of the project, investigation, retrofit measures, determination of liability and legal involvements, which becomes costly in terms of both time and money. It is hoped that the data presented in this paper will help to dissipate the occurrence of these problems to the benefit of both the engineering and construction professions.

## FLEXURAL CRACKING

Flexural cracking is associated with tensile stresses that exceed the tensile capacity of concrete. They are generally found, in continuous girders, at the bottom of the girder in positive moment areas and at the top of the girder in negative moment areas (Fig. 1). In box girders, positive moment flexure cracks will traverse the bottom flange (soffit) width and if severe enough will propa-

## Synopsis

The appearance of distress in the form of cracking in any concrete structure is one of concern and consternation to all parties involved. It is evidence of an unfavorable stress condition and if serious enough could affect the structure's integrity. Cracking may occur for a number of reasons but usually is the result of the interaction of several causative factors. These factors taken individually may be minor but when superimposed can result in distress.

This paper presents documentation of problems that have led to cracking in post-tensioned concrete box girder bridges in Europe as well as in the United States. It discusses cracking resulting from inadequate flexural and shear capacity, thermal stress cracking, and cracking resulting from tendon curvature and tendon misalignment. Also discussed are possible retrofit measures.

The intent of this paper is to bring these problems to the attention of the engineering and construction professions with the hope of precluding their recurrence.

gate in the webs. Flexure cracks in the top of the section, in the negative moment area, occur less often since there is generally a moment redistribution that occurs which reduces the tensile stresses in this area.

In segmental box girders, flexure cracks will generally occur in or near the segment joints (Fig. 2). The crack width at the bottom can be on the order of 0.004 to 0.008 in. (0.1 to 0.2 mm). As the cracks propagate up to the top flange, they will become fine microcracks.

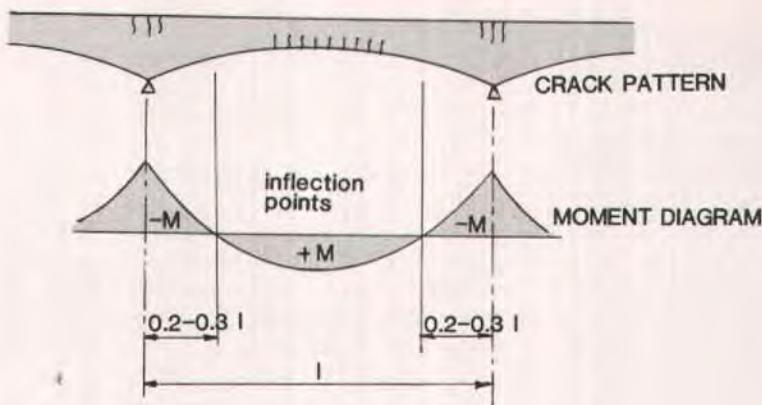


Fig. 1. Flexural cracking zones.

Flexural cracks can be small (microcracks) and the structure may appear to be intact. However, the width of these flexural cracks can vary and become quite large,  $\frac{1}{8}$  to  $\frac{3}{8}$  in. (3 to 10 mm), under the effects of external loading, such as dynamic traffic load or thermal gradient.

In areas near the support the effect of shear will be superimposed upon the flexural stresses producing, after initial vertical cracking, cracks that are inclined and become more and more diagonal in the webs, which may indicate insufficient shear strength (Fig. 3).

This type of cracking will be accompanied by deformation in the structure, visible and measurable deflection. Cracks of this type pose a risk to the reinforcement, prestress or conventional, that traverse the crack from corrosion or from fatigue damage under large and frequent stress variation.

Considerable care must be exercised in diagnosing the cause of cracks in this pattern. Inadequate flexural capacity may or may not necessarily be the causative factor. Excess tension and subsequent cracking behind blister anchorages can produce similar cracking (this effect will be discussed in a subsequent section). Therefore, cracking of this type may have its origin in inade-

quate flexural capacity, cracking behind anchorage blisters or a combination of both.

Causes of inadequate flexural capacity are varied in origin and it is seldom that a single cause will produce significant cracking. More generally, there are a number of causes which by themselves are insignificant, but when acting in concert with other causes produce subsequent cracking.

The more prevalent causes are as follows:

### (a) During Design

1. Inadequate consideration of, or underestimation of, stresses resulting from moment redistribution.
2. Underestimation of thermal stresses (expansion or contraction).
3. Lack of consideration of, or underestimation of, thermal gradient.
4. Overestimation of initial prestress resulting from optimistic coefficients of friction and wobble.
5. Underestimating relaxation loss in prestressing.
6. Selection of a tendon profile whereby a large variation of prestress eccentricity occurs (Fig. 4), and not checking the stresses at critical locations.<sup>2</sup>

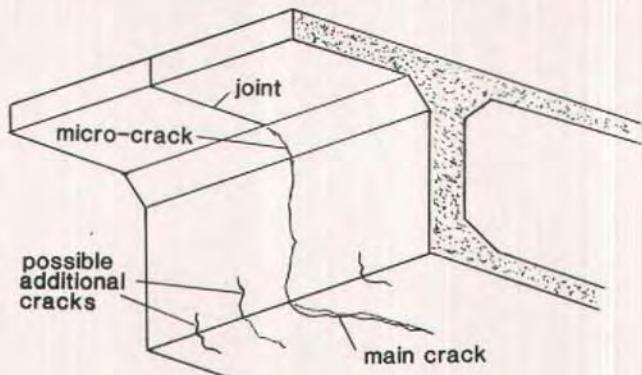


Fig. 2. Flexural cracks.

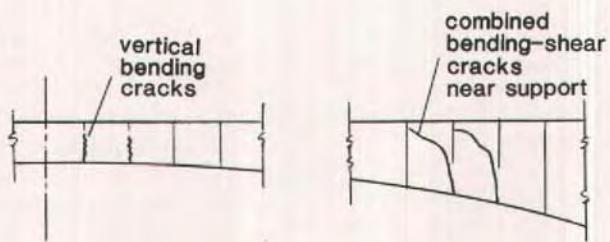


Fig. 3. Flexural cracks.

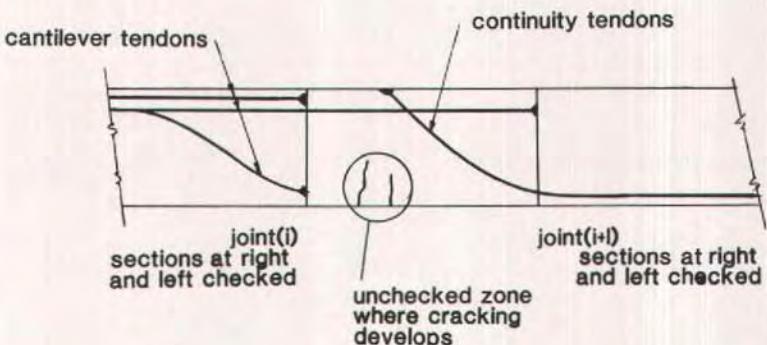


Fig. 4. Unverified zone of cracking.

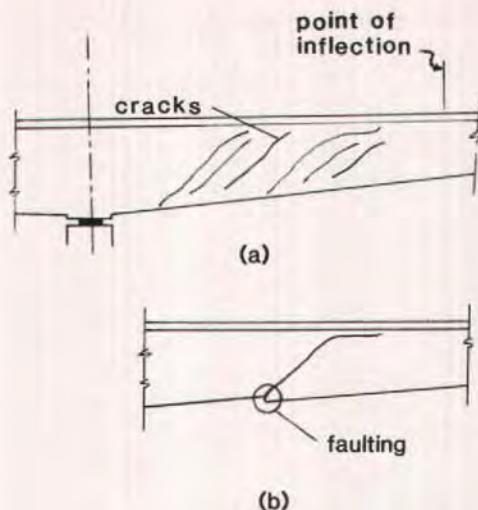


Fig. 5. Shear cracking.

### (b) During Construction

1. Losses as a result of excessive friction resulting from crushed tendon ducts.
2. Displacement of poorly secured tendon ducts which then alters the eccentricity and also produces additional friction.
3. Lack of isolation from the concrete of tendon couplers. If the coupler is positioned such that it cannot freely move as the tendon elongates under stressing, the prestress force distribution is altered.
4. Failure of strands or wires in a tendon.

### (c) Excess Permanent Loads

1. Underestimating the weight of construction equipment and permanent nonstructural loads (roadway thickness, overlay, utility lines not originally planned, etc.).
2. Accidental displacement of forms before or during concrete placement resulting in a greater thickness and heavier deck. The opposite can also

occur whereby the thickness is reduced resulting in decreased concrete area to accommodate stress.

3. Underestimation of the voluminal mass of concrete, especially where dense aggregates such as those of basaltic origin are used.

### (d) Secondary Stresses and Overloads

These are stresses experienced during construction and service life for which it may not have been designed.

1. Differential settlement of supports or movements of pier foundations which modify stresses and/or deformations.
2. Cracks caused in the concrete during or after setting that result from moving construction loads or shifting of formwork.
3. Faulty concreting sequence that can cause cracking as a result of differential shrinkage.
4. Overload from heavy truck traffic and accompanying dynamic effects.
5. Failure to recognize local effects such as, for example, curvature of tendons or stresses behind anchorages.

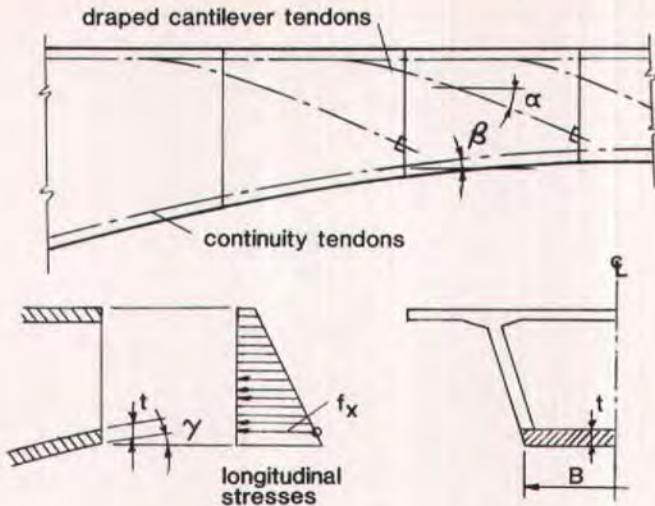


Fig. 6. Shear forces.

## SHEAR CRACKING

Shear cracks occur in the webs and are perceptibly inclined at approximately a 45 deg angle. They normally manifest themselves in a zone between the support and an inflection point (Fig. 5a). The web reinforcement traversing these cracks is subject to stress variations which can cause bond failure and fatigue damage. In addition, at the limit, the reinforcement can yield. This may result in the faulting of the intrados which is usually not visible to the eye (Fig. 5b), but can be instrumentally measured.

Most of the causes of flexural cracks are also responsible for shear cracks since a change in bending moment automatically changes shear stress.<sup>1</sup>

1. Insufficient prestressing
2. Excess permanent loads
3. Secondary stresses
4. Thermal effects

Shear cracks can also be initiated by bending cracks or high tensile stresses behind tendon anchorages. The following design oversights may also contribute to the formation of shear cracks:

1. Lack of recognition of shear distribution in the webs of multicell girders. The transverse design of the section does not take into account the true flexibility of the cross section. (This effect will be discussed in a subsequent section.)

2. Lack of double checking to the right and left of a section where a tendon terminates.

3. Failure to recognize the effect of inclination of continuity tendons in variable depth girders. Although the vertical component of a single tendon may be small, if there are a large number of tendons they can create a significant shear force.

The total net shear force,  $V_o$ , is the sum of the following terms (Fig. 6):<sup>3</sup>

1. Shear force due to applied loads equals  $V$ .
2. Reduction due to vertical component of draped tendons, where used equals  $-\sum P \sin \alpha$ .
3. Increase due to inclination of continuity tendons in the bottom flange for variable depth girders equals  $+\sum P \sin \beta$ .
4. Reduction due to the inclined prin-

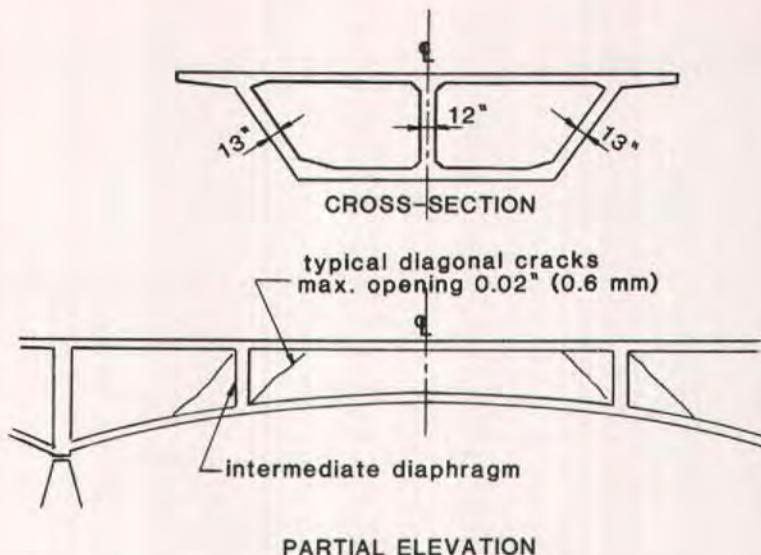


Fig. 7. Web cracking of center web.

cipal compressive stresses in the bottom flange (usually called the Resal effect after the engineer who first studied members of variable depth) equals  $-f_x t B \tan \gamma$ .

Thus:

$$V_o = V - \Sigma P \sin \alpha + \Sigma P \sin \beta - f_x t B \tan \gamma$$

### Cracking Produced by Incorrect Distribution of Forces

This example<sup>3,4</sup> is one of a two-cell, variable depth, box girder bridge constructed by the balanced cantilever method and consisting of five spans of 210, 370, 370, 390, and 230 ft (64, 113, 113, 119, and 70 m). External webs are 13 in. (340 mm) thick and the center web is 12 in. (300 mm) thick. Because the bridge was subsequently intended to carry monorail pylons, two intermediate diaphragms were provided at the third points of each span, as indicated in Fig. 7.

Longitudinal prestress was applied by straight tendons in the top flange and

vertically curved tendons in the bottom flange. Vertical prestress was applied in the webs to control shear stresses. Diagonal cracking was observed in the center web, only near the intermediate diaphragms, with a maximum crack opening of 0.02 in. (0.6 mm). In addition there was cracking in the bottom flange.

A complete investigation of the problems encountered revealed that cracking was the result of the superposition of several adverse effects, any of which might have been harmless if considered separately:

1. The computation of shear stresses failed to take into account the adverse effect (usually neglected) of the vertical component of continuity prestress in the bottom flange of a girder with variable depth. The curvature of continuity tendons distributed in the bottom flange is such that a downward reaction is produced at the webs that is additive to the shear in the web (see above).

2. The distribution of shear stresses was made under the conventional structural analysis assumption that the

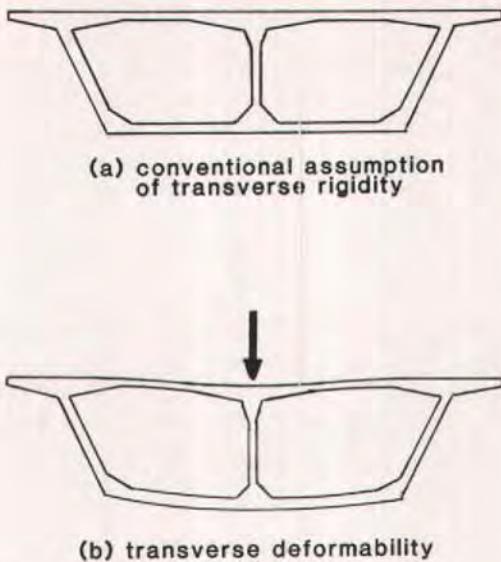


Fig. 8. Transverse stiffness of a two-cell box.

cross section was rigid and transversely undeformable (Fig. 8a). However, depending upon the stiffness of component parts of the cross section this assumption may be erroneous (Fig. 8b). Conventional analysis, assuming a rigid undeformable cross section, assumes that a centered loading is evenly distributed among the webs. In fact, a center loading induces a transverse deformation in the top and bottom flanges with a displacement of the center web larger than that of the external webs. Fig. 9 indicates influence lines for shear in the center web and the external web. The transverse behavior appears very different from that of a transversely undeformable section and may produce cracking in the bottom flange as indicated in Fig. 10. For any particular cross section, a more rigorous analysis is indicated than that usually conducted.

3. The vertical web prestress was partially lost into the intermediate diaphragms, and the actual vertical compressive stress was lower than assumed.

4. Vertical prestress is usually applied

with short bar tendons and even if equipped with a fine thread they are not completely reliable unless special precautions are taken under close supervision. Even a small anchor set significantly reduces the prestress force, and it is not unlikely that the actual prestress force is only three-fourths or even two-thirds of the theoretical prestress.

### **Cracking as a Result of Indirect Support**

The ideal position of bearings at a pier is directly under the webs of a box girder. The shear from the web is transferred directly from the web to the bearings, and there is need only for a simple inside diaphragm designed to transfer the shear stress, due to possible torsion moments, to the substructure. However, to avoid spalling the bottom outside corner of the soffit, the edge of the bearing should be a minimum of 2 in. (50 mm) clear of the corner. If a chamfer exists at the corner, the clear distance should be measured from the

edge of the chamfer (Fig. 11a).

A more complicated situation arises when the bearings are offset with respect to the webs (Fig. 11b). Mild steel reinforcement and possibly prestressing steel must be provided in the cross sec-

tion immediately above the pier to accommodate the following:<sup>3</sup>

1. Suspend all shear stresses carried by the web under Point A, where a 45 deg line starting at the bearing edge intersects the web centerline (hatched area in the shear diagram).

2. Balance the moment ( $V \times d$ ) induced by the bearing effect.

Obviously the most severe situation is that where a single bearing is used, or a pair of bearings is sufficiently offset, and that the 45 deg line from the bearing does not intersect the web centerline (Figs. 11c and 11d).

An approach to the determination of the forces involved is by means of a truss analogy,<sup>5,6,7</sup> using trusses with compression struts formed in the plane of the webs and diaphragms. Fig. 12 shows the truss analogy for the case of a single bearing at the centerline of the cross section. Where the web trusses and the diaphragm truss intersect, the web is stressed in the same manner as if a load were suspended from the bottom at the position of the diaphragm.

Reinforcement requirements should be based upon an ultimate load condition. Suspension reinforcement is required to transfer the ultimate load in the web to the diaphragm. The resisting suspension reinforcement in the web should be in addition to the normal shear reinforcement.

This suspension reinforcement should, in addition, be distributed in the

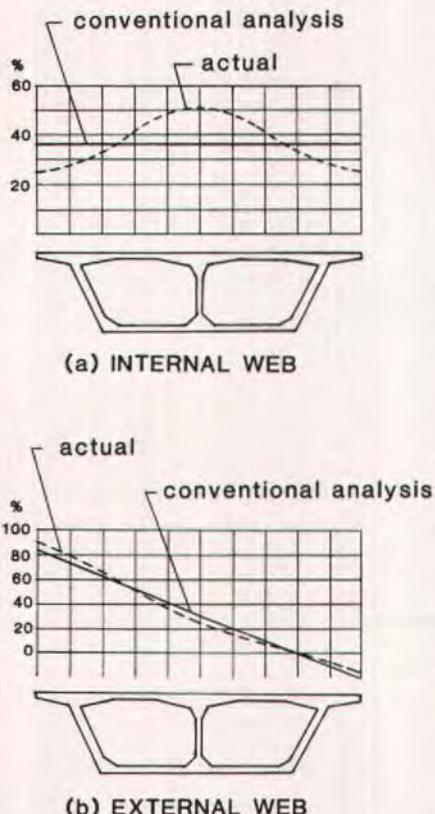


Fig. 9. Shear influence lines.

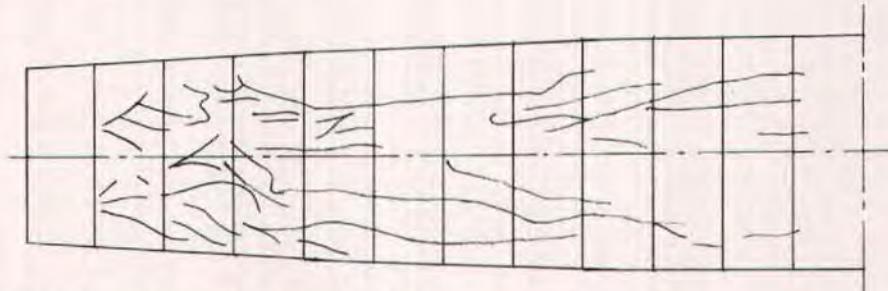


Fig. 10. Cracking pattern in bottom flange.

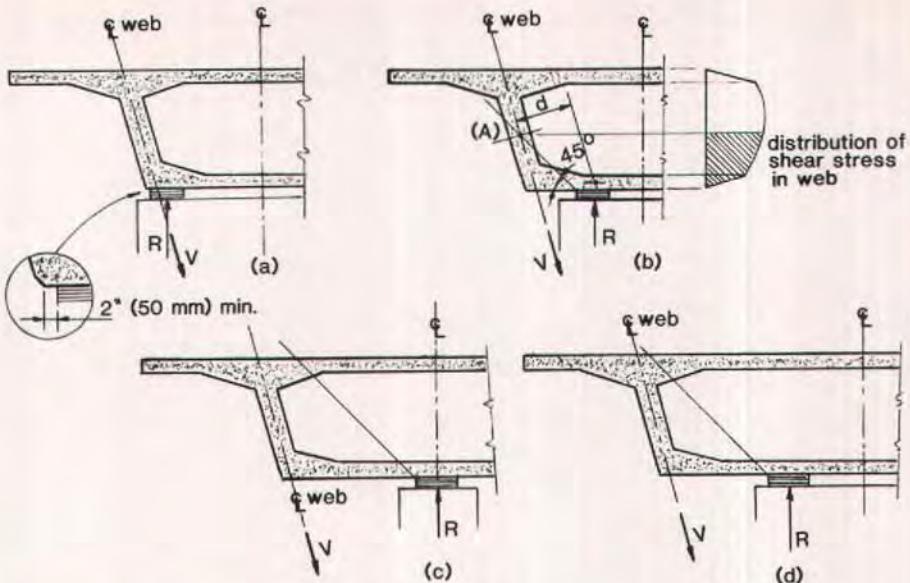


Fig. 11. Position of bearings.

hatched area of the intersection zone (Fig. 12d), such that 70 percent of it is located in the web.<sup>7</sup> It should be the full depth of the girder and adequately anchored.

In many instances, provisions are made (or should be made) for future jacking up of the superstructure in the event the bearings require replacement. The design drawings should indicate the intended position of the jacks and the consequences of this temporary re-positioning of reactions should be accommodated in the design.

## THERMAL STRESS CRACKING

The current AASHTO Specifications<sup>8</sup> recognize thermal expansion and contraction in the design of bridge structures. The rise and fall of temperature should be determined for the site of the structure under consideration. In general, AASHTO requirements for tem-

perature rise and fall of a concrete structure about an assumed temperature at time of construction is as follows:

Type of climate	Temperature	
	Rise	Fall
Moderate climate	30°F (16.7°C)	40°F (22.2°C)
Cold climate	35°F (19.4°C)	45°F (25.0°C)

The thermal coefficient for normal weight concrete is taken as  $6 \times 10^{-6}$  per deg F ( $10.8 \times 10^{-6}$  per deg C). Thus, by using the equation:

$$e = \alpha l \Delta t \quad (1)$$

where

$e$  = total change in length

$\alpha$  = thermal coefficient of expansion (or contraction)

$l$  = length of a member from a reference point

$\Delta t$  = change in temperature of an

unrestrained member

The change in length of an unrestrained member can be determined. Where a member is restrained (fixity of supports or friction in bearings) the thermal expansion or contraction is prevented by reactions at the supports which then cause stresses in the member. This stress is calculated by assuming that the thermal expansion or contraction first occurs and then forces (reactions) are applied at the restraints to bring the member back to its original length.

From Eq. (1) the strain in the member is determined as:

$$\epsilon = e/l = \alpha \Delta t \quad (2)$$

The change in length of the member per unit of length accompanying the unit stress "s" caused by the restraint is:

$$\epsilon = s/E \quad (3)$$

where  $E$  is the compressive or tensile modulus of elasticity. Thus:

$$\epsilon = s/E = \alpha \Delta t \text{ or } s = E \alpha \Delta t \quad (4)$$

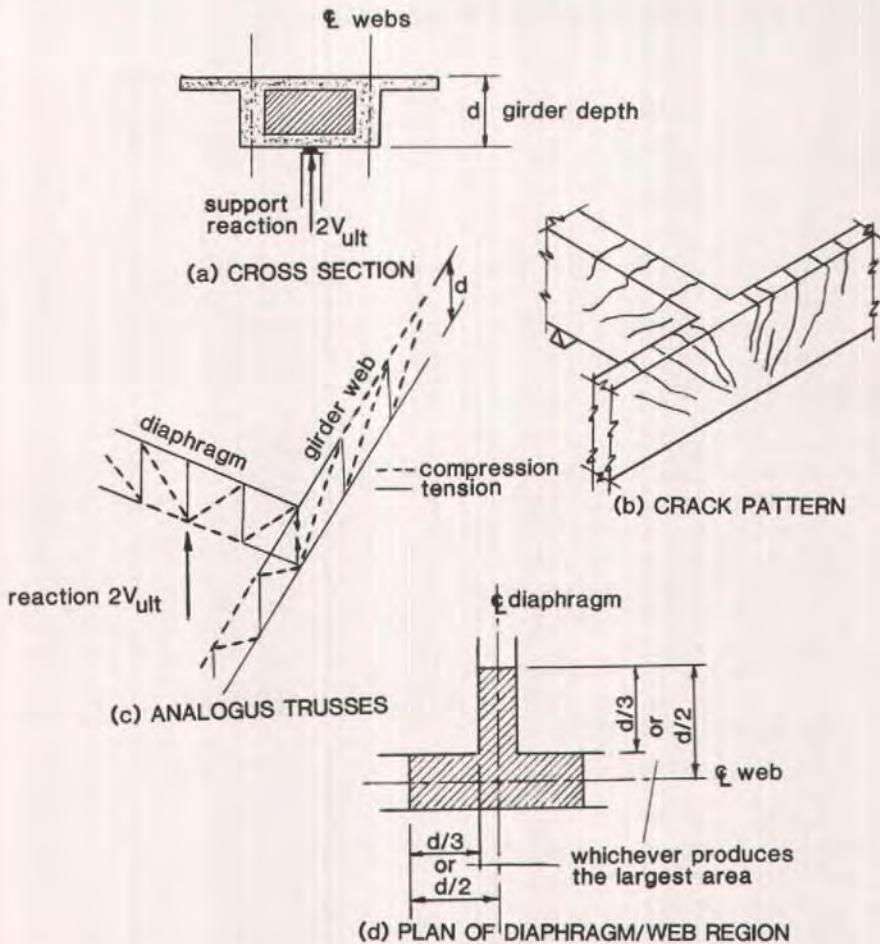


Fig. 12. Indirect bearing support.

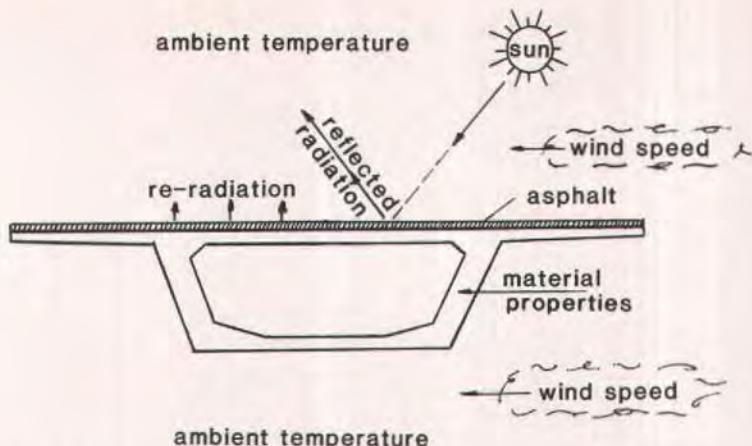


Fig. 13. Thermal response parameters.

From the above, expected movements or stresses can be determined for support conditions and expansion joints and bearings as appropriate can be provided. However, occasionally distress is still evident from displaced bearings and/or damaged expansion joints. Presumably, this distress occurs from unanticipated temperature rise or fall.

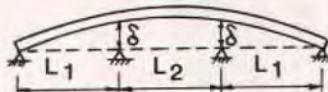
Although the above consideration has been a traditional approach to thermal stresses in bridge structures, bridge engineers have relatively recently become aware of another thermal phenomenon, namely, thermal gradient. As a result of the poor thermal conductivity of concrete, a temperature gradient will occur through the depth of a concrete member. The gradient is a function of a number of variables (Fig. 13) such as solar radiation, ambient temperature, wind speed fluctuations, material properties, surface characteristics, and section shape.<sup>9,10,11</sup>

This type of thermal gradient will cause external restraint forces in statically indeterminate structures such as continuous girders. If in a continuous bridge the interior supports (points of fixity) are released, the structure will attempt to camber upward, assuming an

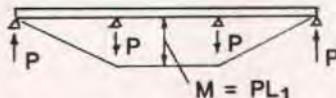
increase in temperature of the upper surface with respect to the lower surface. Since the structure is restrained from cambering freely, restraint moments and shears are produced (Fig. 14).<sup>1,12</sup>

In prestressed concrete structures there may be zones, under dead load plus prestress, where there is a low compressive stress reserve and the stresses due to restraint forces can easily reach values that exceed the tensile strength of the concrete. This usually leads to vertical cracks very close to intermediate supports. Cracking as a result of thermal gradients have been reported<sup>13</sup> and is considered to be one of four prime causes of cracking in prestressed concrete bridges.<sup>5</sup> In some cases the thermal gradient stresses can be larger than the live load stresses. These cracks can be further enhanced or aggravated by stresses resulting from bearing pressure at the supports or radial tendon stresses from curved tendons in the support area.<sup>12</sup>

The magnitude of the restraining forces is a function of the thermal gradient assumed and can be linear or nonlinear. Several countries recognize the existence of thermal gradient and make



FREE DEFORMATION



### RESTRAINT FORCES AND MOMENTS

Fig. 14. Moment caused by thermal gradient.

recommendations for their consideration in design. Most of these recommendations require different shaped gradients and intensities to be evaluated (Fig. 15). However, it should be noted that they are influenced by local meteorology and construction practices.

The New Zealand gradient<sup>14</sup> (Fig. 15a) is a fifth power parabola for a depth of 1200 mm (48 in.) with the temperature ( $T$ ) at the concrete surface related to an asphalt thickness ( $d$ ) in mm. This curve is used for webs and that portion of the deck not above an enclosed cell, i.e., deck flange cantilevers.

For decks above an enclosed cell a linear curve is used of  $5 - 0.05d^{\circ}\text{C}$  per 100 mm (4 in.) thickness with the temperature ( $T$ ) at the top surface. A linear temperature increase from  $0^{\circ}\text{C}$  to  $1.5^{\circ}\text{C}$  is used over the bottom 200 mm (8 in.) thickness of the section.

For structures shallower than 1400 mm (56 in.) the two parts of the solid curve are to be superimposed. For box girders the transverse effects are obtained by using the dotted curve. On a bridge which is to be surfaced, the temporary unsurfaced condition is to be checked by using a value for ( $T$ ) of  $27^{\circ}\text{C}$ .

Thermal gradient curves used in England<sup>15</sup> are indicated in Fig. 15b for deck warming and deck cooling. These

curves assume a thickness of surfacing of 40 mm (1.6 in.) for Groups 1 and 2 loading and 100 mm (4 in.) for Groups 3 and 4. For other thicknesses of surfacing, the values of ( $T$ ) are adjusted according to prescribed tables that are presented in the British Code of Practice.

In Australia<sup>7</sup> the thermal gradient curve takes the shape indicated in Fig. 15c. In France<sup>3</sup> (Fig. 15d) a linear variation is used with a  $10^{\circ}\text{C}$  gradient used in combination with dead load, shrinkage, and creep. A  $5^{\circ}\text{C}$  gradient is used with all loads (including live load and impact).

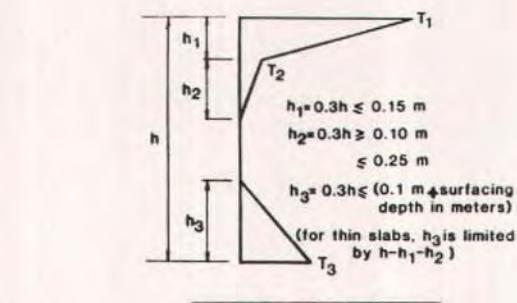
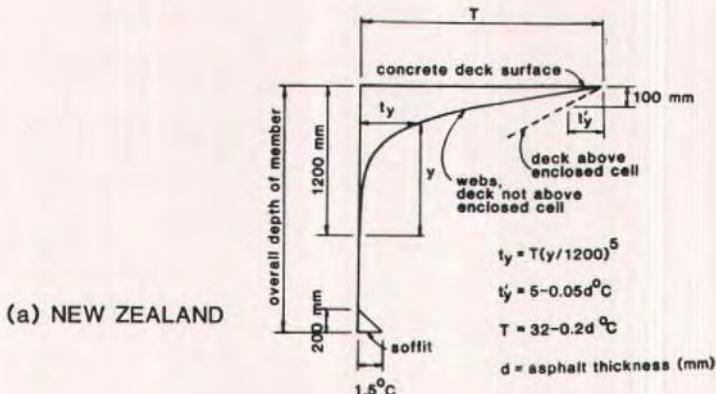
Currently in the United States there are no general criteria for thermal gradient, although a recommendation is presented in Ref. 16. However, the National Cooperative Highway Research Program (NCHRP) has a current Project 12-22 titled "Thermal Effects in Concrete Bridge Superstructures" that will hopefully result in recommendations to be used for United States practice.

The effects of thermal gradient are usually computed by assuming the gradient to be constant throughout the bridge superstructure length, which is not necessarily the case. Fig. 16 indicates the results for the case of a typical span fixed at both ends (the case of a long structure with many identical spans). The stress at the bottom fiber depends upon the following two factors:

1. Variation of height between span center and support (ratio  $h_1/h_0$ ).
2. Position of the center of gravity within the section (ratio  $c_2/h_0$ ).

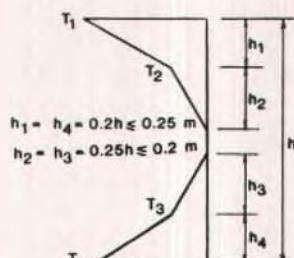
The stress increases rapidly when the variation in depth is more pronounced. For normal proportions the effect of gradient is increased by 50 percent in variable depth girders compared to constant depth girders.<sup>3</sup>

Another source of thermal stress (strain) related cracking is that associated with heat of hydration, subsequent cooling and shrinkage. Heat of hydration is a chemical process that is a func-



$h$	$T_1$	$T_2$	$T_3$
m	°C		
≤ 0.2	8.5	3.5	0.5
0.4	12.0	3.0	1.5
0.6	13.0	3.0	2.0
> 0.8	13.5	3.0	2.5

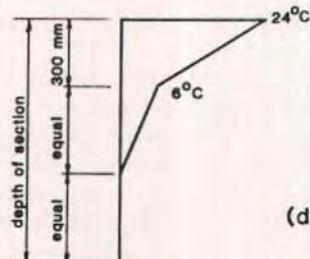
(deck warming)



$h$	$T_1$	$T_2$	$T_3$	$T_4$
m	°C			
≤ 0.2	2.0	0.5	0.5	1.5
0.4	4.5	1.4	1.0	3.5
0.6	6.5	1.8	1.5	5.0
0.8	7.6	1.7	1.5	6.0
1.0	8.0	1.5	1.5	6.3
> 1.5	8.4	0.5	1.0	6.6

(deck cooling)

(c) AUSTRALIA



(d) FRANCE

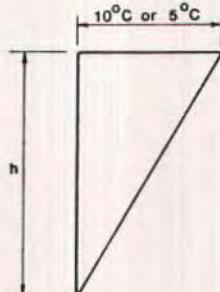


Fig. 15. Thermal gradient diagrams of various countries.

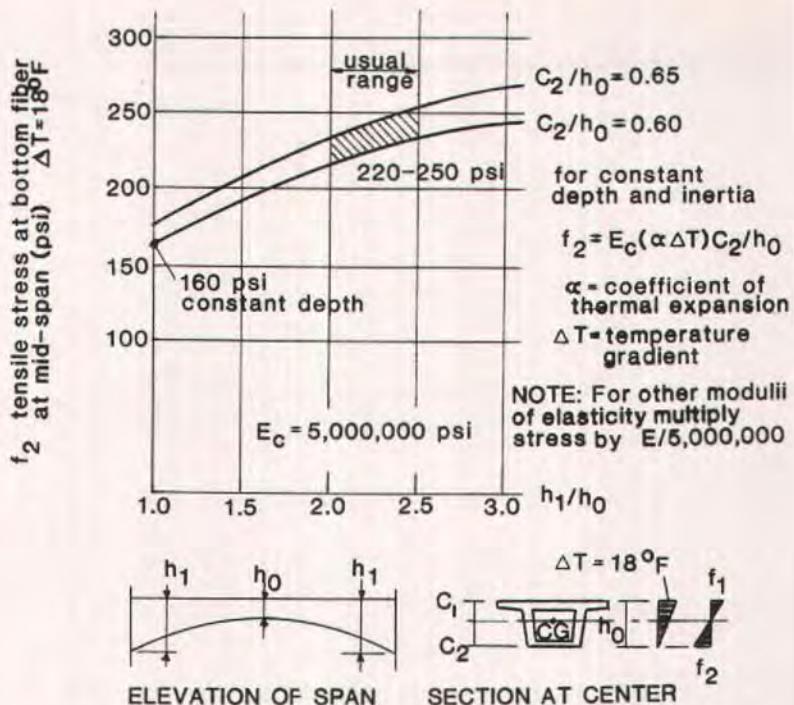


Fig. 16. Effect of thermal gradient on box girder decks.

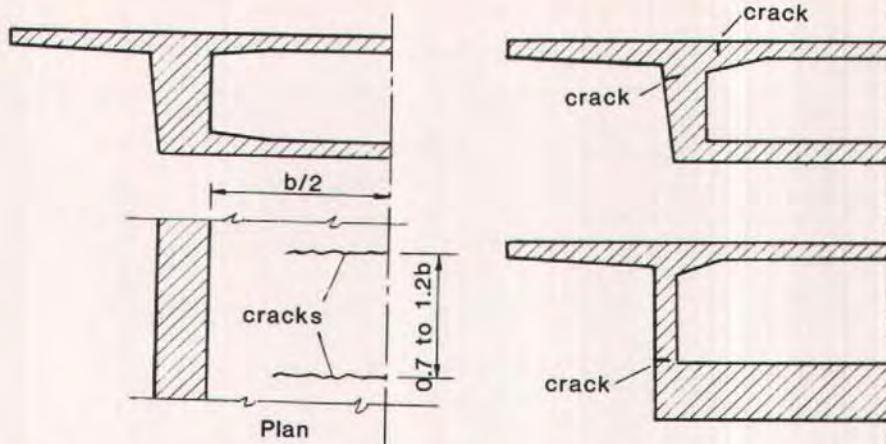
tion of the type and amount of cement, thickness of concrete members and temperature of aggregate, mixing water and surrounding atmosphere. As a result of heat of hydration and shrinkage, especially between thick and thin portions of a cross section, large residual stresses can be produced which exceed the tensile strength of "green" concrete.

Thin parts of the cross section will cool faster than adjoining thick parts producing residual tensile stress and cracking. There is always a potential of cracking when a thin member is attached to a relatively larger member. The thinner member will always be subjected to a faster rate of temperature change (cooling), larger creep and shrinkage than the larger member and thus the thinner member will crack.<sup>1,12</sup>

In box girders with thick webs relative to the thickness of the bottom flange,

the bottom thin flange will cool faster and attempt to shrink at a faster rate than the webs and is therefore being restrained. To relieve tensile strains, produced by restraint of the webs, the thin bottom flange has no other alternative but to crack (Fig. 17a). In a transverse section, large differences in thickness can produce restraining moments which can produce horizontal cracks in the webs (Fig. 17b).

A similar thermal stress (strain) induced cracking can occur in staged construction whereby the cross section is constructed in two or more pours requiring construction joints. When fresh young concrete is placed against hardened older concrete there is a danger of cracking. Initially, the fresh concrete will have an increase in temperature resulting from heat of hydration, then when cooling occurs the new concrete is



(a) CRACKS IN THIN BOTTOM FLANGE

(b) HORIZONTAL CRACKS IN WEBS

Fig. 17. Thermal shrinkage cracking.

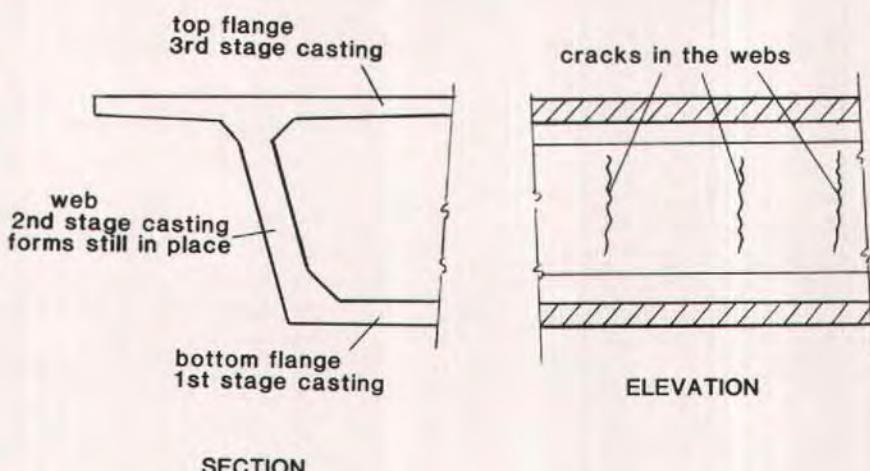


Fig. 18. Web shrinkage cracks.

restrained from shrinking by the older concrete.

Usually, because of insufficient longitudinal reinforcement and/or lack of sufficient bond development between the reinforcement and the young concrete, there is insufficient capacity for the younger concrete to resist the re-

straint to shrinkage produced by the older concrete and cracking of the younger concrete will occur. If the forms are left in place too long, they will present a large surface area of friction that will also hamper the shrinkage of the new concrete and thus cause cracking (Fig. 18).<sup>1</sup>

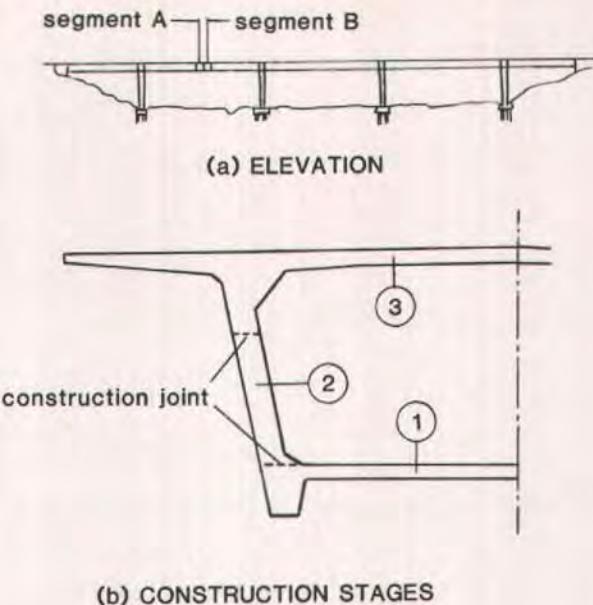


Fig. 19. Klement Gottwald Bridge.

As an example of temperature variation in a box girder bridge, data extracted from Ref. 17 is presented here to illustrate the point. The research effort involved the instrumentation and temperature data collection in two adjacent segments near the midspan of an interior span (Fig. 19a) during construction of the Klement Gottwald Bridge over the Nusle Valley in Prague, Czechoslovakia. Segment A had been previously cast and Segment B is the segment being constructed. Segment length is 11.5 ft (3.5 m) and is cast in three stages as indicated in Fig. 19b.

Concrete temperature data obtained in December 1969 was influenced by the following factors:

1. Extremely cold ambient temperature. The temperature was below  $0^{\circ}\text{C}$  and in the final phases of data collection the temperature was often below  $-20^{\circ}\text{C}$ .

2. Preheating of the concrete mix, because of the low ambient temperature. Hot water was used in preparing the mix such that the average mix temperature at

placing was approximately  $35^{\circ}\text{C}$ . Blown hot air was used as a protection against freezing; however, the distribution was uneven.

These conditions are far from normal and the results have to be considered as extreme, nevertheless, the results are indicative of thermal distribution in this type of cross section.

Temperature distribution during construction of Segment B is indicated in Fig. 20. Fig. 20a shows the temperature distribution 10 hours after casting Stage 1. Note that the bottom flange has a thickness of 11.8 in. (300 mm). Fig. 20b shows the temperature distribution 20 hours after casting Stage 2 (44 hours after casting Stage 1). The web thickness is 23.6 in. (600 mm).

At this point in time the temperature at Point "a" was  $38.6^{\circ}\text{C}$ . Fig. 20c indicates the temperature distribution 24 hours after casting Stage 3 (7 days after casting Stage 2). The temperature at Point "b" was  $51.1^{\circ}\text{C}$  and the temperature immediately above this point, Point

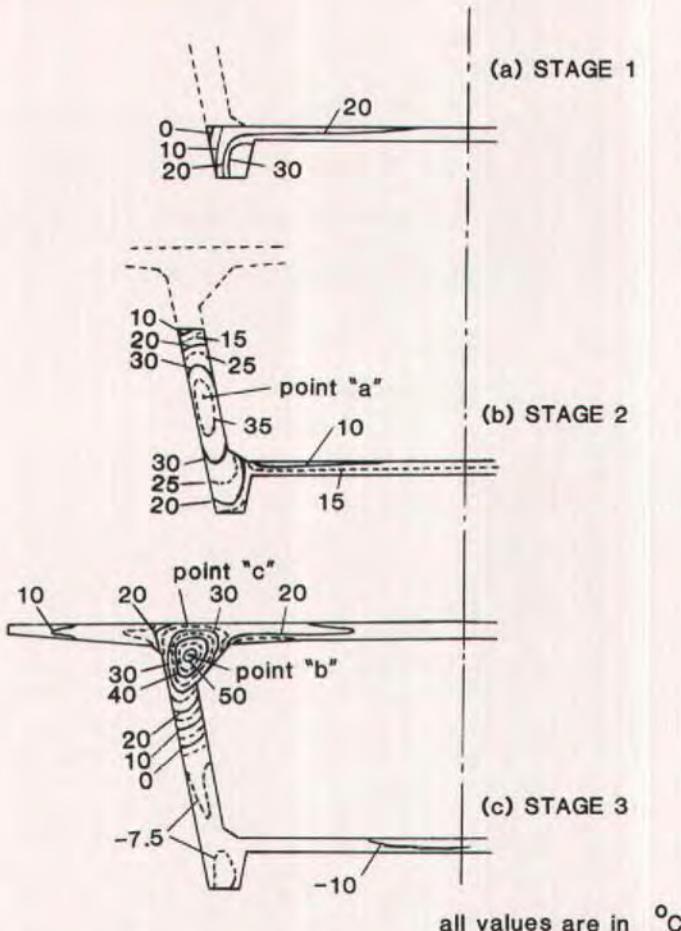


Fig. 20. Temperature distribution.

"c", on the surface of the deck, is approximately  $20^{\circ}\text{C}$  and the temperature in the bottom portion of the web and the bottom flange is below  $0^{\circ}\text{C}$ .

The temperature distribution with time for Points "b" and "c" with respect to placing Stage 3 is shown in Fig. 21. These curves indicate, as would be expected, a heat dissipation at the surface (Point "c") as compared with the heat dissipation at an internal position (Point "b"). A comparison of the two curves indicates a maximum gradient between Points "b" and "c" at 15 hours after

casting Stage 3 as  $32.2^{\circ}\text{C}$  for a vertical distance of 28.7 in. (730 mm) between the two points.

Time-temperature distribution along the centerline of the web in the longitudinal direction is shown in Fig. 22. The heavy dashed line indicates the joint between the previously cast Segments A and B. The curves show, between the points of 0.5 to 1.2 m (1.6 to 4 ft) from the end of the segment, the influence of the locally effective hot air heating of the segment and also its poor heat distribution.

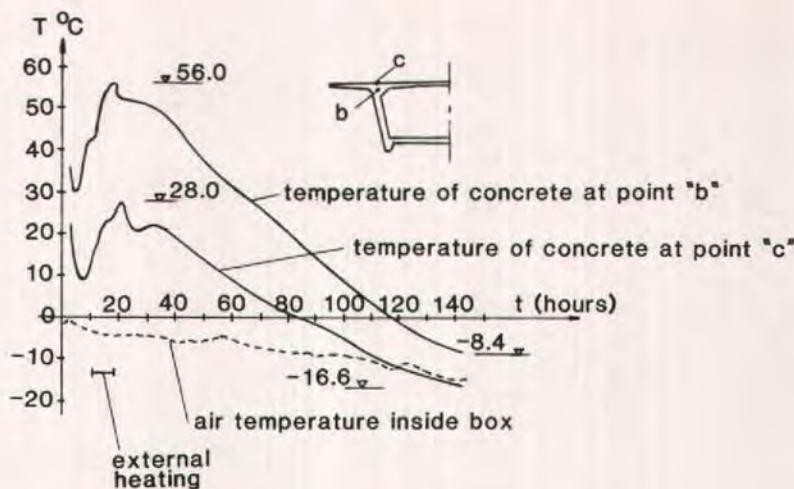


Fig. 21. Temperature distribution with time (Points b and c).

These curves are also noticeably influenced by the low temperature of the previously cast Segment A and the cool ambient temperature. The maximum temperature gradient decreases gradually after the first few hours of hydration heat development. This gradual decrease in gradient is due initially to the heating up of the adjacent segment and later on to decrease in heat of hydration and cooling of the web of the segment.

Thin members cool at a faster rate than thicker members which, therefore, create considerable tensile stress. When fresh concrete is cast against old concrete there is a potential for cracking in the younger concrete which, when subsequent cooling occurs, is hampered from shortening by the older concrete. This usually occurs during the first few cool evenings when the strength of concrete is low and low tensile stresses can cause cracking. At this point, because bond is not sufficiently developed, reinforcement cannot prevent opening of the cracks. Restraint forces due to temperature differences must be minimized to the extent possible by avoiding large differences in thickness

of contiguous members and proper curing procedures.<sup>12</sup>

There are basically two methods of heat curing concrete: (1) preheating the fresh concrete before placement in the forms and (2) heat curing the concrete after consolidation in the forms.<sup>3</sup>

In the first case the concrete is preheated to about 85 to 90°F (30 to 35°C). This operation is achieved in several ways:

1. Steam heating the aggregates — a simple solution that presents the disadvantage of changing the aggregate water content.
2. Heating the water — a solution that has limited efficiency, because of the small proportion of water in comparison with the other components (water at 140°F raises the concrete temperature by approximately 20°F).

3. Direct heating of the concrete mix by injecting steam into the mixer itself — the best solution and the one most easily controlled.

To avoid heat loss, the forms are generally insulated and some source of radiant heat is installed inside the segment.

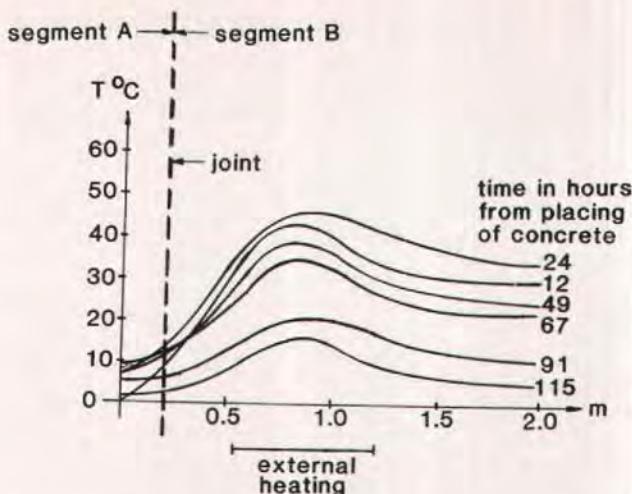


Fig. 22. Longitudinal temperature distribution in web (Stage 2).

In the second case, the concrete is heated in its form inside a thermal container in which low pressure steam is circulated. The heat curing cycle should include (Fig. 23):<sup>3</sup>

1. An initial curing period of 2 or 3 hours, during which the concrete is kept at the ambient temperature.
2. An increase in temperature at a low rate of less than 36°F (20°C) per hour.
3. A period, depending on the concrete strength to be achieved, during which the temperature is held constant and below 150°F (65°C).
4. A period during which the concrete is cooled at a rate similar to that used for the temperature increase.

A similar form of cracking occurs at a closure pour either at midspan in balanced cantilever construction or longitudinal closure pours between the tips of adjacent box girder flanges (Fig. 24).<sup>1</sup> As the closure pour concrete hardens, the units being connected may undergo deformations (solar heat) as discussed previously or by transient moving construction loads and/or as a result of thermal shrinking as discussed previously. The prestressing may not close

these cracks, which can cause a deviation of the isostatic lines of compression toward the uncracked zones, altering the prestressing distribution in the section under consideration resulting in insufficient bearing strength.

To avoid this situation the members being joined must be rigidly held in position. Normally, in cast-in-place balanced cantilever construction a form traveler is used to clamp the two cantilever tips rigidly in place. For precast segments or for longitudinal closure pours between flange tips an external rigid clamping device must be employed (strongbacks). In addition, proper curing procedures must be employed to control shrinkage stresses.

Another difficulty during curing is that resulting from large temperature differences between the curing environment of the segment being cast and the outside environment of the cast-against segment (Fig. 25). If a temperature gradient exists across the cast-against segment, its shape will change. The segment will return to its original shape when the temperature gradient falls back to zero.

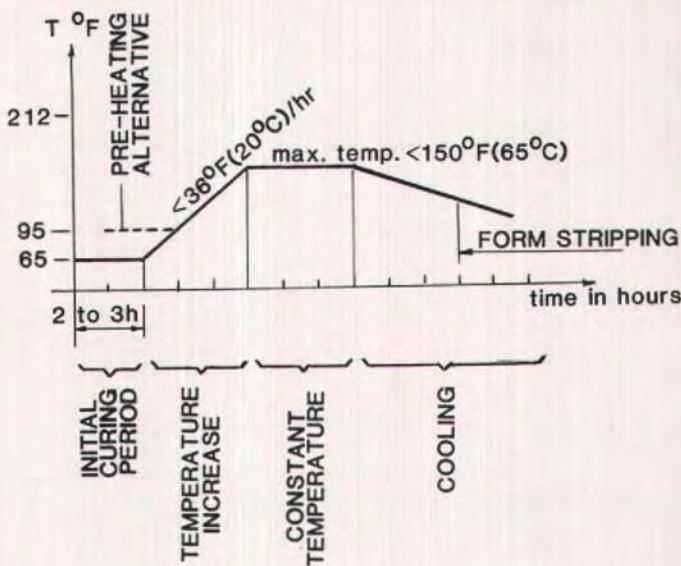


Fig. 23. Heat treatment cycle.

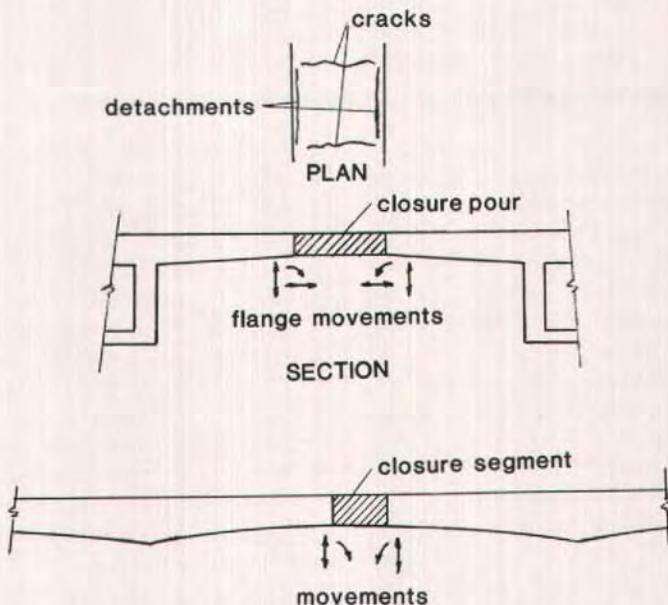


Fig. 24. Closure pour cracks.

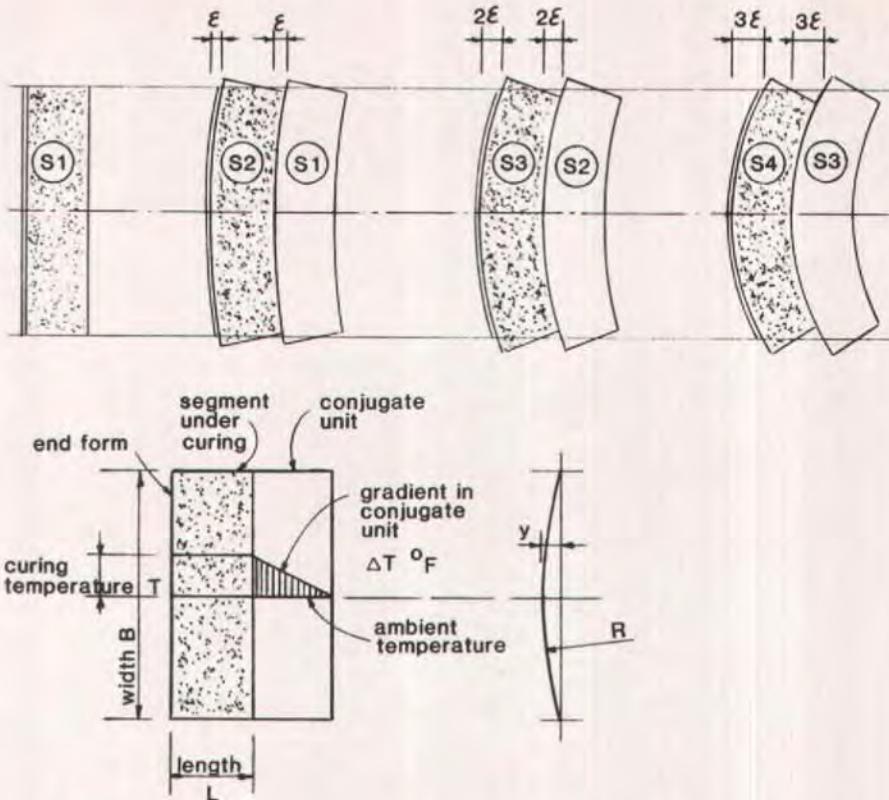


Fig. 25. Effect of improper curing of segments in short-line casting.

This will result in match cast surfaces that are not necessarily parallel full length. This condition is evidenced in the top slab joints; the joints will be tight at the curbs and a small gap may be noticed at the center. The problem becomes cumulative if this condition exists for subsequent castings. With the tremendous prestressing force, the segments develop longitudinal cracks as the segments are seated against each other.

This also results in an unaccounted variable when evaluating the bridge camber. For match-cast segments, it is important to enclose both the cast against segment and the segment being cast in an isothermal enclosure. This will prevent longitudinal thermal gra-

dients being set up which could lead to horizontal curvature of segments and consequent lack of matching in the structure. This effect is particularly significant for segments with high width to length ratios (in excess of 6).<sup>2,3</sup>

## CRACKING AT OR NEAR ANCHORAGES

Certain anchorage positions, such as an anchorage blister on a thin flange as shown in Fig. 26, should be avoided. Wherever possible, anchorage blisters for continuity tendons should be placed in a fillet between the web and flange where the transverse section has the largest rigidity. If this type of detail can-

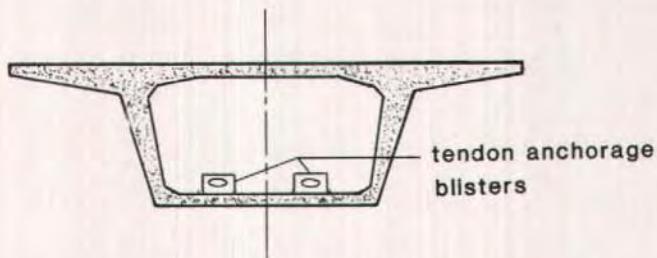


Fig. 26. Anchorage blister position to be avoided.

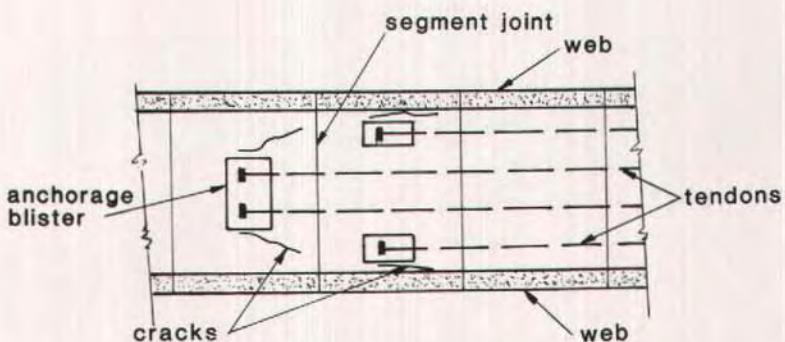


Fig. 27. Cracking at anchorages.

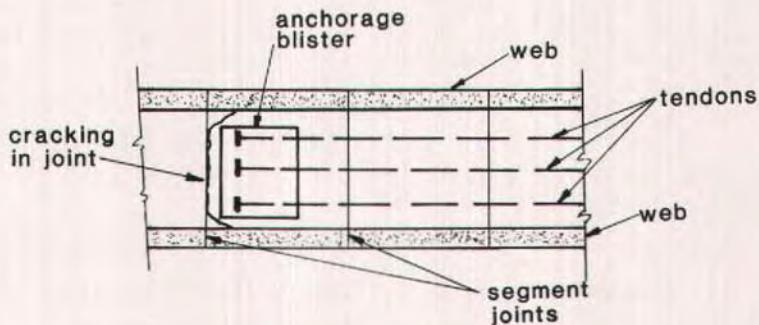


Fig. 28. Cracking in joint behind anchorage.

not be avoided, then particular attention must be paid in design and construction to the zone concerned.<sup>3,18</sup>

Cracking will originate in the bottom flange behind continuity anchorage blisters and propagate forward toward the webs, along a line forming a 30 to 45 deg angle with the longitudinal axis of the girder (Fig. 27). If the rear face of the blister is located in close proximity to a segment joint, cracking may develop in the joint (Fig. 28). In some situations the diagonal or herringbone cracks in the bottom flange continue to propagate to the webs forming a 30 to 45 deg angle with a horizontal axis (Fig. 29).<sup>1,2</sup>

If the crack reaches a segment joint in the web it may cross the joint, but usually travels vertically up the joint to a point below the top flange where it may continue vertically in the joint, resume a diagonal path in the web proper, or cease to propagate. Obviously, the shear

capacity of the webs will be reduced and failure of the structure may be imminent.<sup>2</sup> It should be noted that this type of cracking is not symptomatic with anchorage blisters; the same cracking pattern has been observed with anchorages embedded in the flange, i.e., no blisters.

It is possible that this type of cracking from continuity tendon anchorages in the bottom flange may join with similar cracks originating from cantilever tendon anchorages in the top flange (Fig. 30). In this situation, the center portion of the span may become an inverse "key-stone" (Fig. 31) and immediate measures are required to prevent collapse of the structure.<sup>1</sup>

This situation occurred in a box girder constructed by incremental launching<sup>3</sup> (Fig. 32). Permanent prestress was achieved by straight tendons placed in the top and bottom flanges, as required

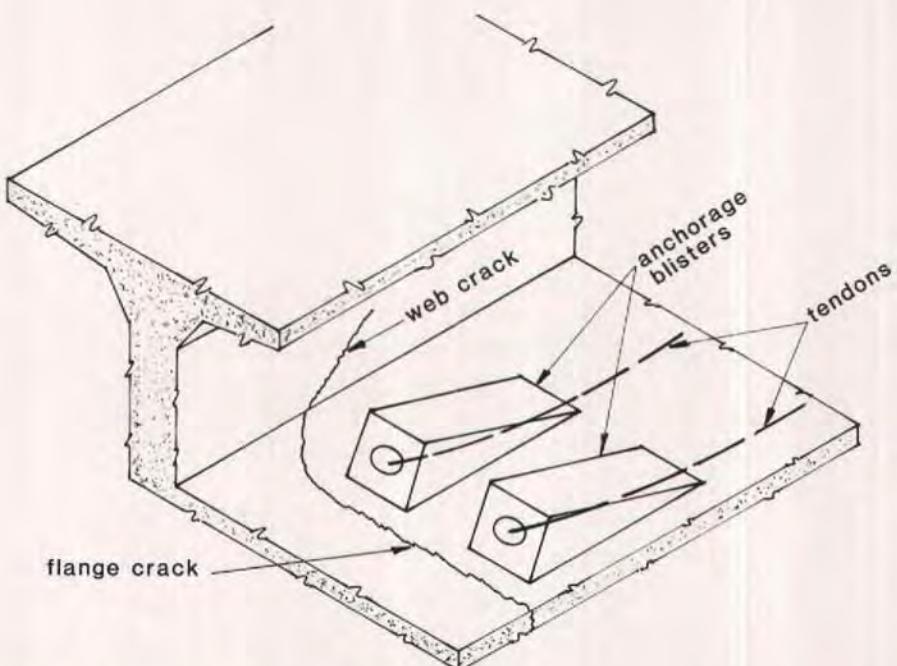


Fig. 29. Anchorage cracking propagating to web.

by the distribution of moments. During launching an additional uniform pre-stress was applied to the constant depth single box section, which produced an average compressive stress of 520 psi ( $3.6 \text{ MN/m}^2$ ). Near each pier there was a vertical pre-stress designed to reduce web diagonal stresses to allowable values.

During launching a diagonal crack appeared through both webs between the blisters provided in the box for anchorage of top and bottom tendons. The corresponding shear stress was 380 psi ( $2.67 \text{ MN/m}^2$ ), and there was no vertical pre-stress in that zone.

The principal tensile stress at the centroid of the section was 200 psi ( $1.4 \text{ MN/m}^2$ ), which is far below the cracking strength of plain concrete. In fact, the

webs of the box section were subjected to additional tensile stresses due to the distribution of the large concentrated forces of the top and bottom pre-stress. The truss analogy shown in Fig. 32 indicates clearly that such tensile stresses are superimposed on the normal shear and diagonal stress due to the applied dead load and may therefore produce cracking. This could have been prevented by extending the vertical pre-stress in the webs further out toward midspan.

The cause of this type of cracking is the application of a tendon force to a small area of slab, in its plane, compressing the slab forward of the force in a zone of radial compression stresses that cause tensile stresses behind the point of application of the tendon force (Fig.

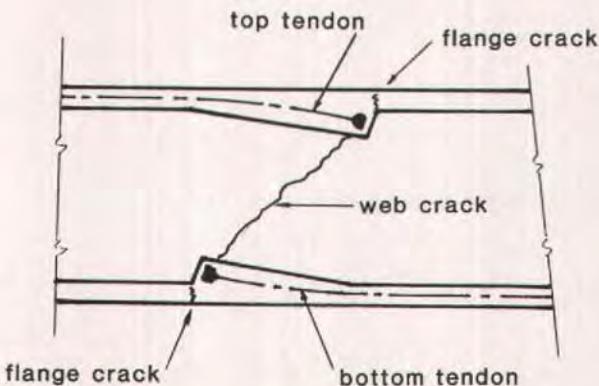


Fig. 30. Insufficient lap between anchorages.

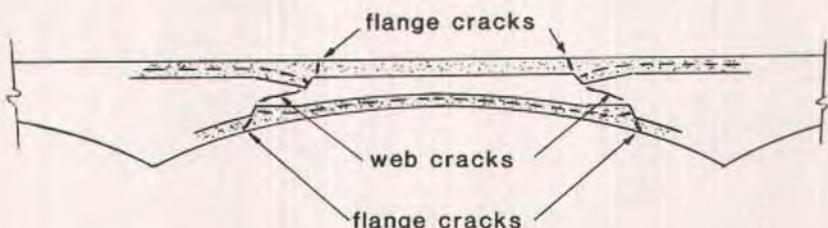


Fig. 31. Symmetrical cracking causing inverted "key-stone."

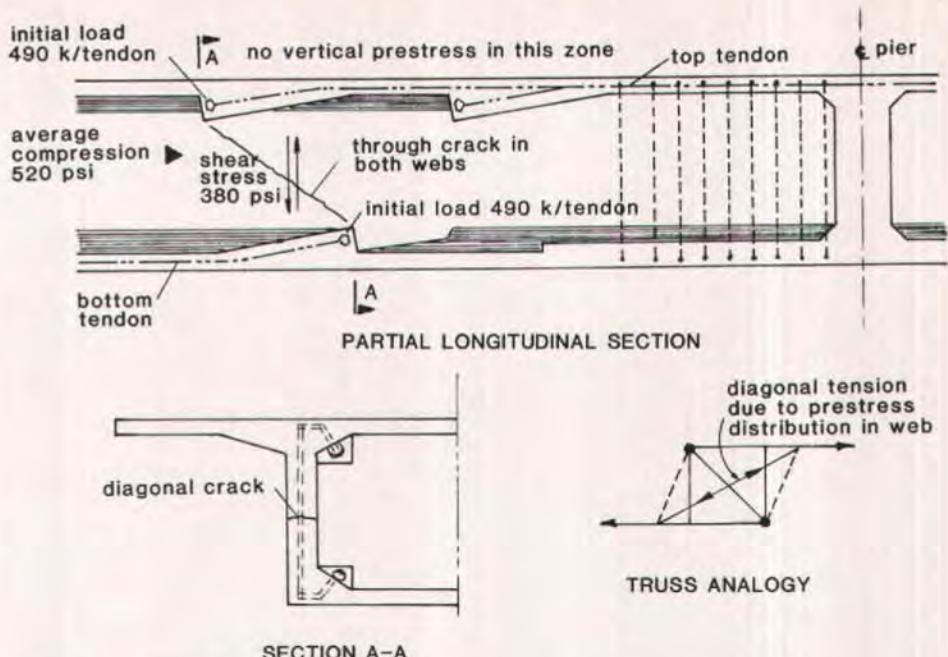


Fig. 32. Web cracking under high prestress force.

33).<sup>1,2</sup> Among the parameters that can produce this type of cracking are the following.<sup>1</sup>

1. Termination of numerous continuity tendons in a single section, particularly when the joint behind the anchorages is not compressed by other tendons and when passive longitudinal reinforcement does not traverse the joints between segments or is inadequate.

2. Insufficient horizontal offset between tendon anchorages in the top and bottom flanges (Fig. 30).

If this type of cracking and/or failure is to be inhibited, the design, detailing, and placement of reinforcement in these highly stressed zones must be carefully considered. Location of blisters must be considered and there must be sufficient longitudinal reinforcement behind the blister or anchorage and sufficient transverse reinforcement abreast of the anchorage to accommodate the initial

tendon forces. In a recent failure of this type, after epoxy injection of the cracks, additional external post-tensioning was provided to close-up and induce compression in open segment joints and cracks.

## CRACKS RESULTING FROM VERTICAL CURVATURE OF SOFFIT TENDONS

This type of distress is associated with variable depth girders and occurs in the positive moment area. It is manifested by longitudinal cracks occurring in the bottom flange, at the juncture of the bottom flange and web, and possibly in the web (Fig. 34).<sup>1</sup>

Tendons for continuity or positive moment prestress may not, or even should not, always be located in the fillet between web and bottom flange. They may be located in the bottom

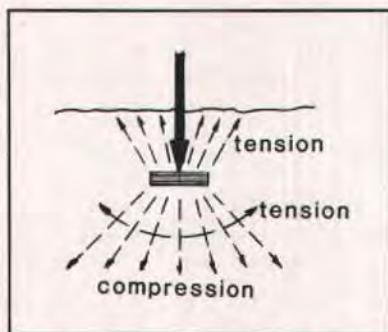


Fig. 33. Stress pattern at anchorage.

flange proper. When a variable depth girder is used, the bottom flange has a curvature in the vertical plane, which must be followed by the prestressing tendons (Fig. 35).

Unless careful consideration is given to that fact at the concept and detailed design stages, difficulties are likely to develop. This is indicated in Fig. 35, which shows the free-body diagrams of stresses in the bottom flange due to the curvature. Curvature of a tendon induces a downward radial force, which must be resisted by transverse bending of the bottom flange between the webs.<sup>3</sup>

Longitudinal compressive stresses in the bottom flange similarly induce an upward radial reaction in the flange, counteracting at least in part the effect of the tendons. Unfortunately, when the full live load and variable effects, such as thermal gradients, are applied to the superstructure, the longitudinal stresses vanish and consequently the partial negation of the effect of tendon curvature is lost.

Therefore, the effect of tendon curvature adds fully to the dead load stresses of the concrete flange. The corresponding flexural stresses may be four to five times greater than the effect due to dead load only, and if sufficient reinforcement is not provided for this effect, heavy cracking is to be expected and

possibly failure. Practically, this situation may be aggravated by tendon cusps associated with tendon profile deviations which will be subsequently discussed (see Fig. 57).

As a numerical example (Fig. 36) of secondary stresses due to vertically curved prestressing tendons, assume the following:<sup>3</sup>

Longitudinal radius of curvature

$$= 1000 \text{ ft (304.8 m)}$$

Weight of bottom slab

$$= 100 \text{ psf (4.8 kN/m}^2)$$

Compressive stress:

Unloaded bridge = 2000 psi

$$(13.8 \text{ kN/m}^2)$$

Loaded bridge = 0

Tendon capacity = 280 kips at 10 in.

$$\text{spacing (1.25 MN at 0.254 m)}$$

The radial load resulting from pre-stress compression is then determined as:

$$\begin{aligned} N/R &= f_c t/R = (2000)(8)(12)/1000 \\ &\approx 200 \text{ psf (9.6 kN/m}^2) \end{aligned}$$

The tendon radial load is:

$$\begin{aligned} F/R &= 280,000/(10/12)(1000) \\ &\approx 340 \text{ psf (16.3 kN/m}^2) \end{aligned}$$

The total loads on the bottom flange are:

$$\begin{aligned} (1) \text{ During construction (dead load)} \\ &= 100 \text{ psf (4.8 kN/m}^2) \end{aligned}$$

$$\begin{aligned} (2) \text{ Unloaded bridge} \\ &= 100 - 200 + 340 \\ &= 240 \text{ psf (11.5 kN/m}^2) \end{aligned}$$

$$\begin{aligned} (3) \text{ Loaded bridge} \\ &= 100 + 340 \\ &= 440 \text{ psf (21 kN/m}^2) \end{aligned}$$

Bending moment

$$\begin{aligned} &= w l^2/12 \\ &= 9 \text{ ft-kips/ft (40,000 N-m/m)} \end{aligned}$$

Stress in the bottom flange

(uncracked section):

$$\begin{aligned} f &= M/S \\ &= (9000)(12)/(12 \times 8^2/6) \\ &= 840 \text{ psi (5.8 MN/m}^2) \end{aligned}$$

At this stress level the tensile strength of the concrete would be exceeded, producing the longitudinal cracks depicted in Fig. 34.

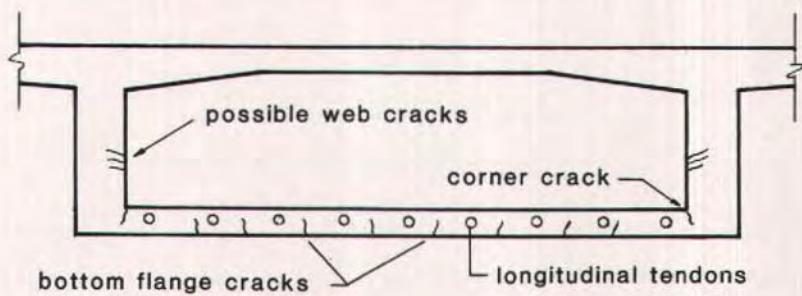


Fig. 34. Cracking from curved soffit tendons.

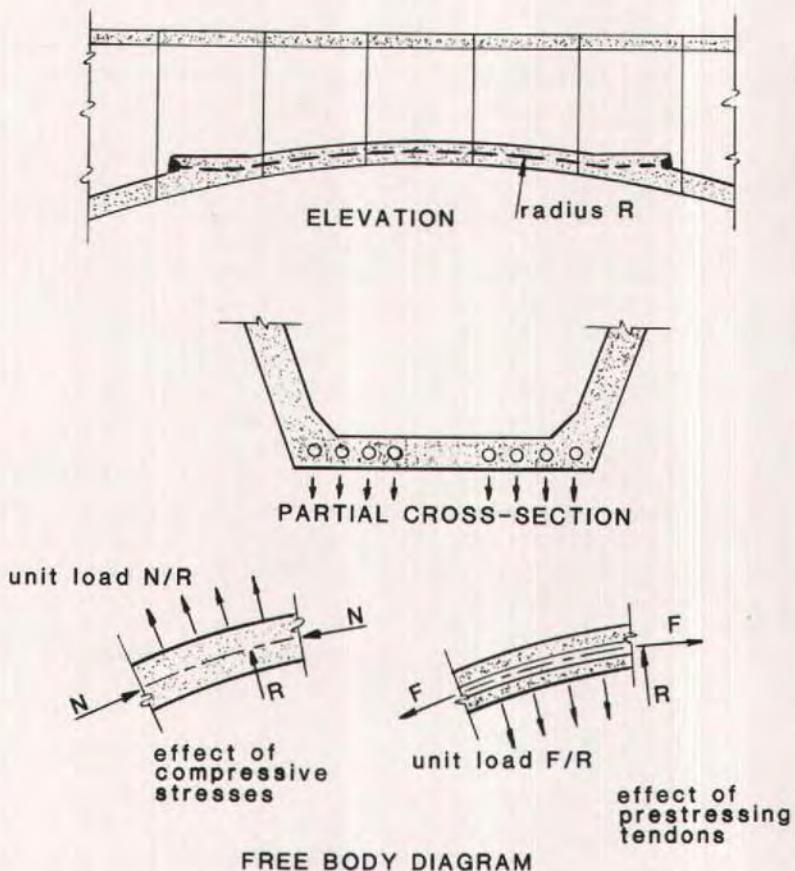


Fig. 35. Vertically curved tendon.

Assumed Longitudinal Radius 1,000 ft.

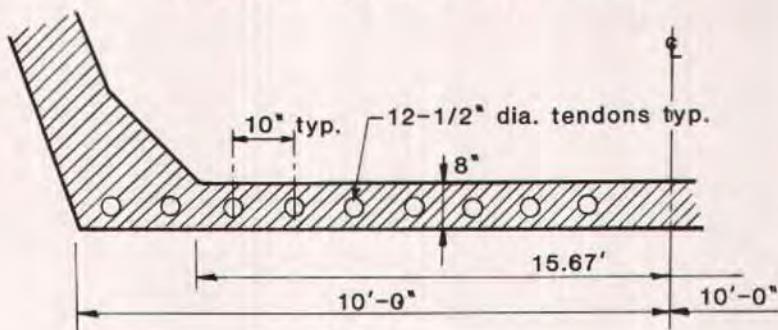


Fig. 36. Numerical example.

## PULLOUT OF HORIZONTALLY CURVED TENDONS

A problem has occurred relatively recently with small radius horizontally curved, post-tensioned box girder bridges. The problem has occurred at two known sites during construction. It concerns the lateral force produced in the webs by the tensioning of horizontally curved longitudinal tendons.

The first example\* is a continuous, cast-in-place, post-tensioned concrete box girder bridge, with three spans of 176, 234, and 176 ft (53.6, 71.3, and 53.6 m) with slightly over half of the eastern 176 ft (53.6 m) span aligned on a horizontal curve with a 250 ft (76 m) radius (Fig. 37). This 176 ft (53.6 m) curved span has three 12 in. (0.305 m) thick intermediate diaphragms, located at each quarter point of the span.

In cross section (Fig. 38) the superstructure is a two-cell box with top flange width of 34 ft (10.4 m), a bottom flange width of 23 ft (7 m), and a depth of

9 ft (2.74 m). The top flange varies in thickness transversely from 8 $\frac{1}{2}$  to 12 in. (0.22 to 0.305 m). The bottom flange varies in thickness longitudinally from 7 $\frac{1}{2}$  in. (0.19 m) at midspan to 14 in. (0.356 m) at the piers.

The bridge was post-tensioned with 12 draped tendons, which are continuous throughout the entire length of the structure. Four bundled tendons were placed in each web and all tendons were jacked from both ends at the abutments.

The first failure (Fig. 39) occurred during prestressing operations when the twelfth and last tendon, located in the north web along the inside of the curve, was near its full prestress. A loud noise, described as a "bang," was heard and all four tendons in that web broke away from the web for almost the entire length of the curved east span, tearing the curved reinforced web along the profile of the tendons. Two days later, the east quarter-span length of tendons in the south web, along the outside of the curve, suddenly broke out of the concrete web, again making a loud noise.

An immediate inspection of the structure indicated that the horizontally curved tendons, exerting a radial horizontal pressure, had overloaded the reinforced concrete webs. This overload

\* "Las Lomas Bridge — Causes of Structural Failure," Report to Department of the Army, San Francisco District Corps of Engineers, T. Y. Lin International, August 1979 (Unpublished).

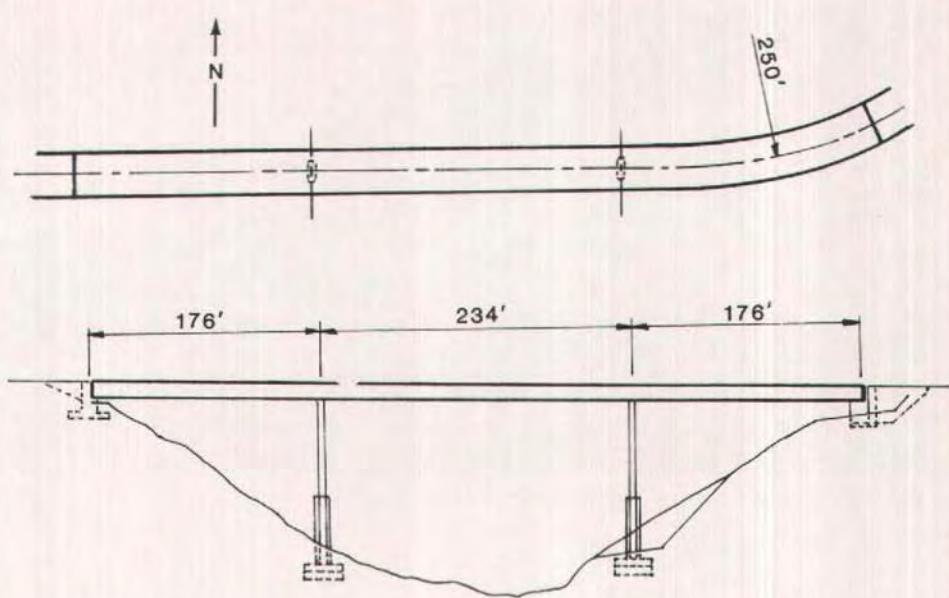


Fig. 37. Bridge elevation and plan.

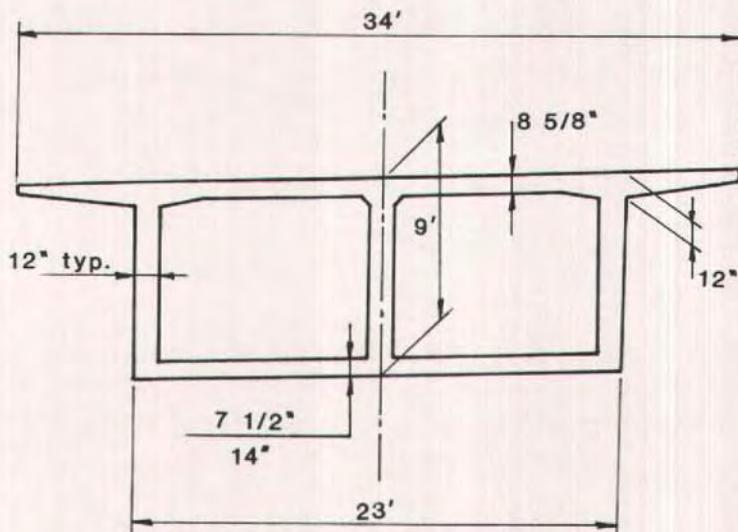


Fig. 38. Typical cross section.

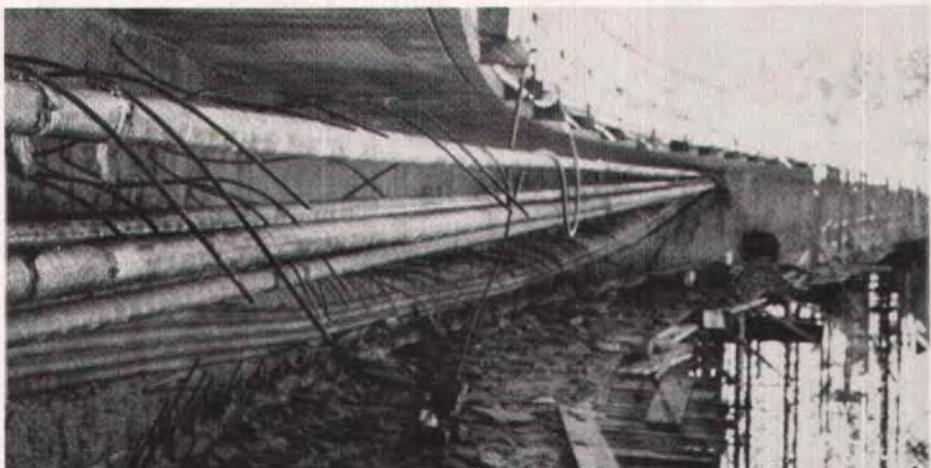


Fig. 39. Horizontal pullout of curved tendons (courtesy of T. Y. Lin).

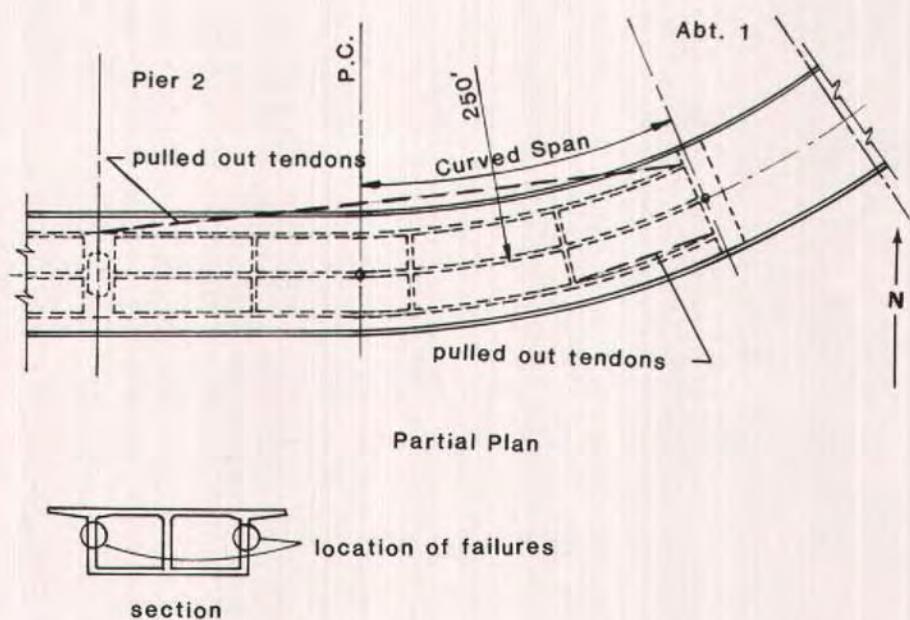


Fig. 40. Zones of web failure.

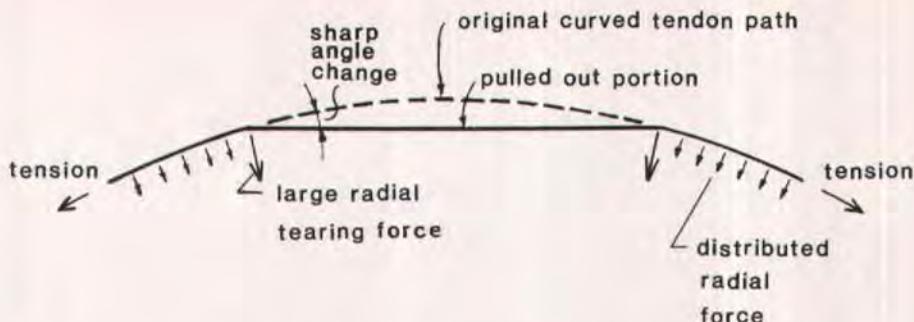


Fig. 41. Progressive failure along tendon.

had caused the concrete to fail, allowing the tendons to straighten out and pull away from the webs (Fig. 40). The failure was particularly evident in the inner (north) web, where the tendons had pulled well clear of the concrete. In the outer (south) web the failure (inside the box) had not progressed past the first quarter-span diaphragm. The center web was undamaged. Additional observations indicated that the tendons were continuing to pull away from the webs, and that failure had progressed several more feet towards the abutment and the pier (Fig. 41).

The second example\* occurred on an interchange ramp (Fig. 42). This structure is a post-tensioned, cast-in-place, continuous, two-cell concrete box girder, constructed essentially on a curve with a radius of 540 ft (164.6 m). The structure, between expansion hinges, experienced concrete spalling and cracking during construction on the south web of the box along the curve as indicated in Figs. 43 and 44.

During the first post-tensioning operations, four tendons out of six in each web were fully post-tensioned, then noise was heard and cracking and spalling were observed along the south web.

After that, most of the tendons were released except two tendons along the north web and one tendon along the center web which were not released.

It should be pointed out that a review of the construction documents and the construction method for both structures revealed no serious discrepancies between the construction method as compared to the construction documents and specifications. Further, the AASHTO Standard Specifications for Highway Bridges<sup>8</sup> does not provide guidance for the design of curved post-tensioned box girders.

In both these structures, there was a combination of relatively sharp curvature, thin concrete cover over the tendons and the bundling of a number of large sized tendons close together. These failures are somewhat unique in that the problem would not have surfaced in the case of a flatter curvature, thicker concrete cover over the tendons, or adequate spreading of the tendons into individual ducts as compared to bundled ducts.

The analysis of the failure of these structures can be divided into three separate actions (Fig. 45) which need to be considered in design:

1. The global or overall girder action of the bridge together with its supporting piers and abutments.
2. Regional beam action of each web

\*"Kapiolani Interchange On-Ramp Project No. 1-H1-(157):24," Design Review: Interim Report to Department of Transportation, State of Hawaii, T. Y. Lin International, April 1982 (Unpublished).

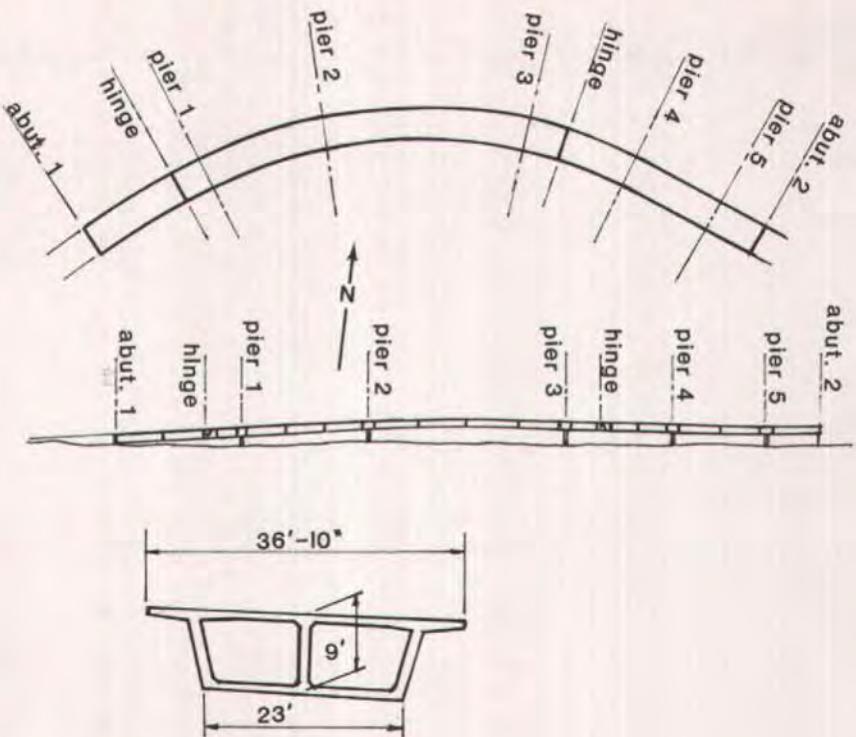


Fig. 42. Plan, elevation and section.

supported at the top and bottom flanges as a beam.

3. Local slab action of the concrete cover over the tendons.

It is important from both the analysis and the design viewpoint to look into all three of these actions and their effect on both the concrete and the reinforcement in the webs of the boxes.

One point concerning global action of the girder is the restraining effect of the piers and the bridge bearings. A curved bridge tends to shrink along certain directions, depending on the flexibility of the piers, while the movement of the bearings may be restrained to take place along a different direction. Thus, secondary moments in the horizontal direction are produced in the girder of these two structures.

It has been determined that such

stresses are not a major factor. Thus, it is believed that the global action for both of these structures had only minor effects on the failures experienced, and were not quantitatively considered.

Regional action considers each web as a beam supported at the top and bottom flanges. This beam (Fig. 46) is acted upon by the radial force from the tendons. The radial tendon force produces shear and bending in the web, as it spans vertically. The bending moment and shear in the web are also influenced by the fact that the web concrete is under longitudinal compression from tendon prestress.

This precompression has an arching effect which partly counteracts the radial force from the tendons, thus reducing the bending moment produced by the tendon force. In other words, the

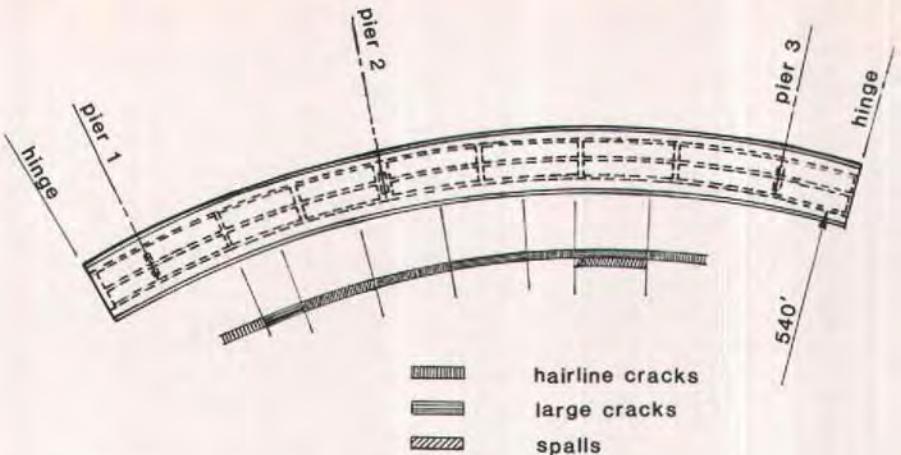


Fig. 43. Failure of south web.

tendency to push radially inward by the curved tendons is counteracted to some degree by the tendency to push radially outward by the concrete. This is shown

schematically in Fig. 47.

This condition of equilibrium is often taken for granted so that the local and regional effects of these forces are not normally calculated, in spite of the fact that they may be far from being balanced, depending on the degree of curvature and amount of prestress tendon pressure occurring locally or regionally. The inward pressure from the tendons is often much higher than the outward radial arch pressure from the concrete. Such a force imbalance has been resisted without failure in thousands of structures constructed, although some of them may be on the verge of failure and not detected.

The stirrups in the beams are designed, as usual, to resist the vertical web shear at the ultimate range. Generally, the webs of the box do not crack under normal loadings and the stirrups are seldom called into action. On the other hand, at ultimate load, where the beam webs do crack, then the stirrups are called into full action. At that time the stirrups may or may not have excess capacity to serve as the beam flexure reinforcement for the web in the vertical direction.

It appears that the stirrups should be



Fig. 44. Web spalling (courtesy of Clarence R. Yamamoto).

provided to take ultimate web shear first and to resist the web-beam action in addition. In the second failure example, where #5 stirrups at 12 in. (0.305 m) centers were provided, they were overstressed in beam action alone, even without their service as shear stirrups.

Local slab action is probably the most important item that actually caused the failure in both of these structures. The local slab action can be studied in different ways. First, consider the cover as a slab 2 in. (50 mm) thick (reinforced with #5 bars along the inside face only) acting as a two-span continuous beam with a central support furnished by the concrete between the two sets of bundled tendons, as shown on the left of Fig. 48.

This can be analyzed as a plain concrete beam acting in shear and in flexure. However, there is a tendency for concrete to shrink against the more rigid ducts. As a result, the tensile strength of the concrete can be entirely lost, both in the support area between bundled tendons and maybe in the 2 in. (50 mm) slab itself. Since it is not considered good

practice to design plain concrete to take tension, this analysis has very little meaning.

The second analysis, shown to the right in Fig. 48, assumes that the concrete has cracked and is unable to supply tension for the support area between the bundled tendons. If this is the case, the slab will act as a single span fixed at the ends; however, the reinforcement is in one plane on the inside face only. This can offer very little resistance to the radial force from the tendons, both in bending and in shear. The stresses are exceedingly high.

An extraordinarily high percentage of reinforcing steel would be required to resist such moments. Unfortunately, the tendons were tied to the stirrups on the inside of the curvature; thus the stirrup reinforcement is in position next to the tendon ducts such that it does not function to resist positive moment produced by the radial forces. Therefore, the concrete cover essentially is a plain concrete slab and its resistance is nil.

The order of magnitude of the stress produced by the three causes discussed,

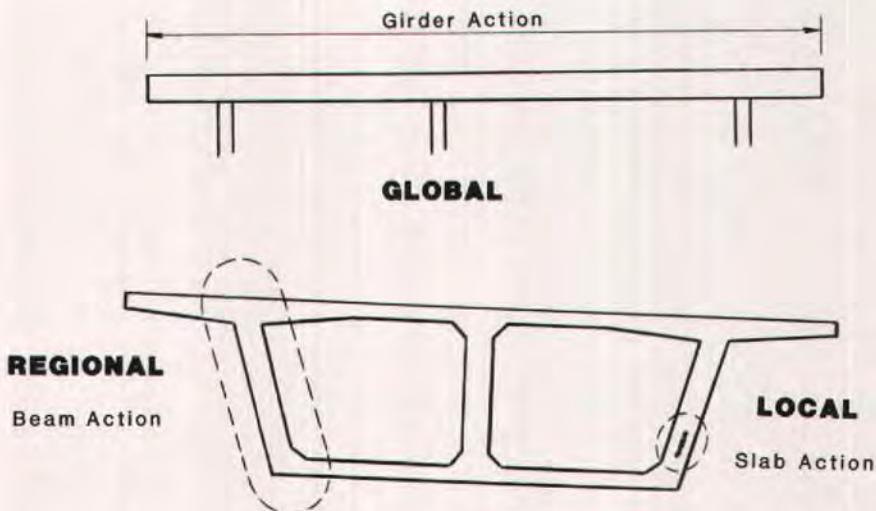


Fig. 45. Box girder actions due to horizontally curved tendons.

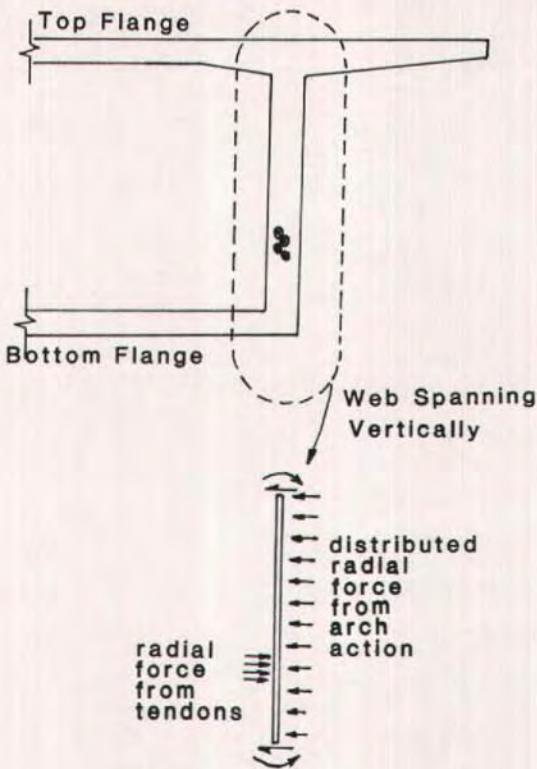


Fig. 46. Vertical span of web.

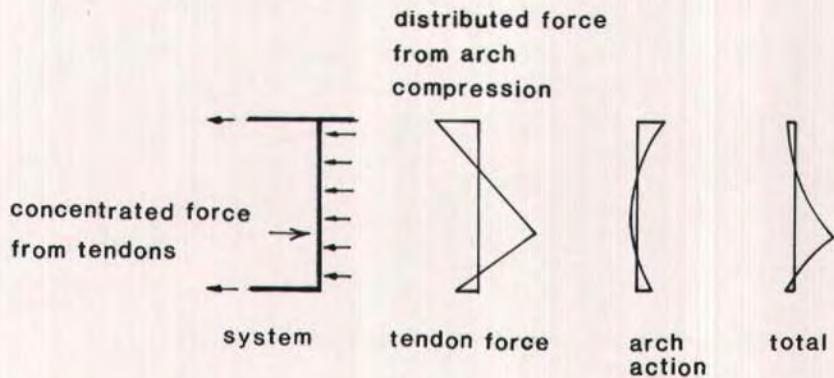


Fig. 47. Moments in vertical span of web.

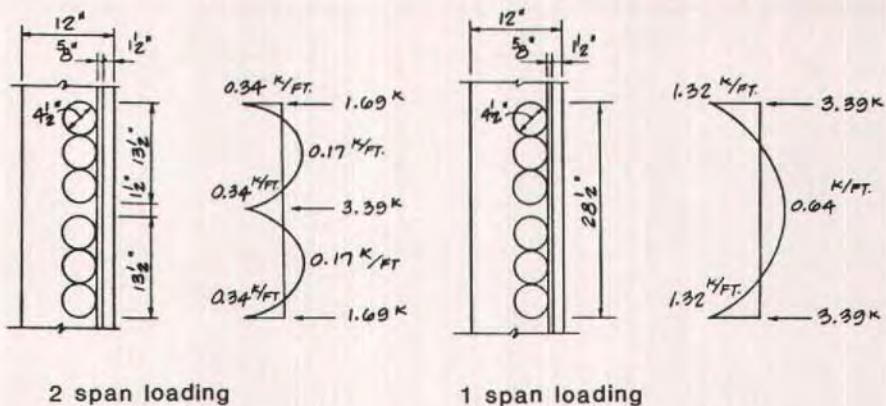


Fig. 48. Slab action.

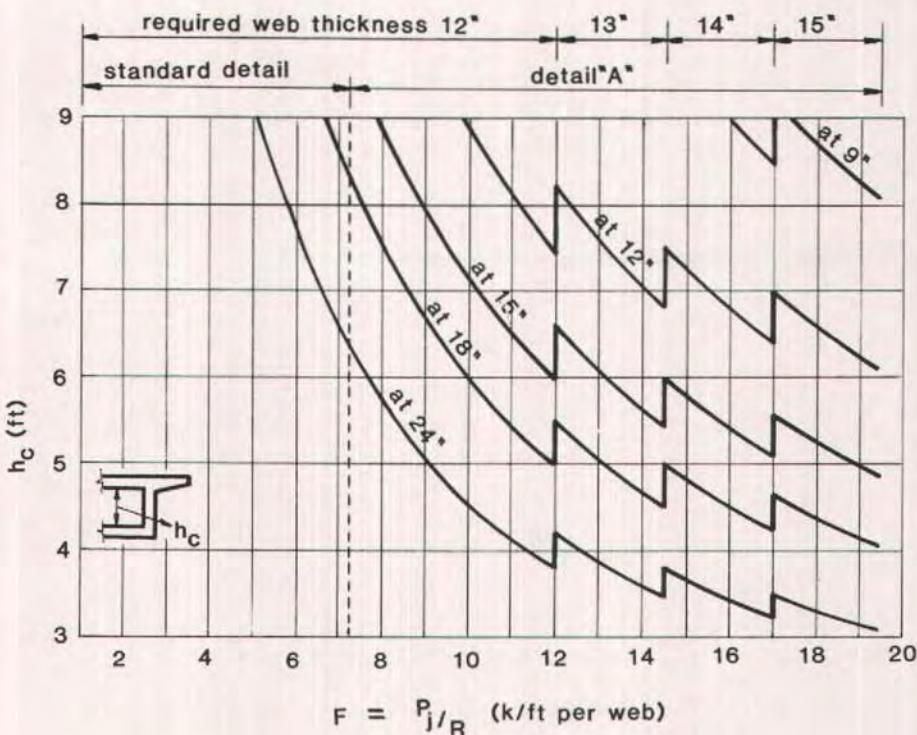


Fig. 49. Chart for determining web reinforcement (#5 stirrups).

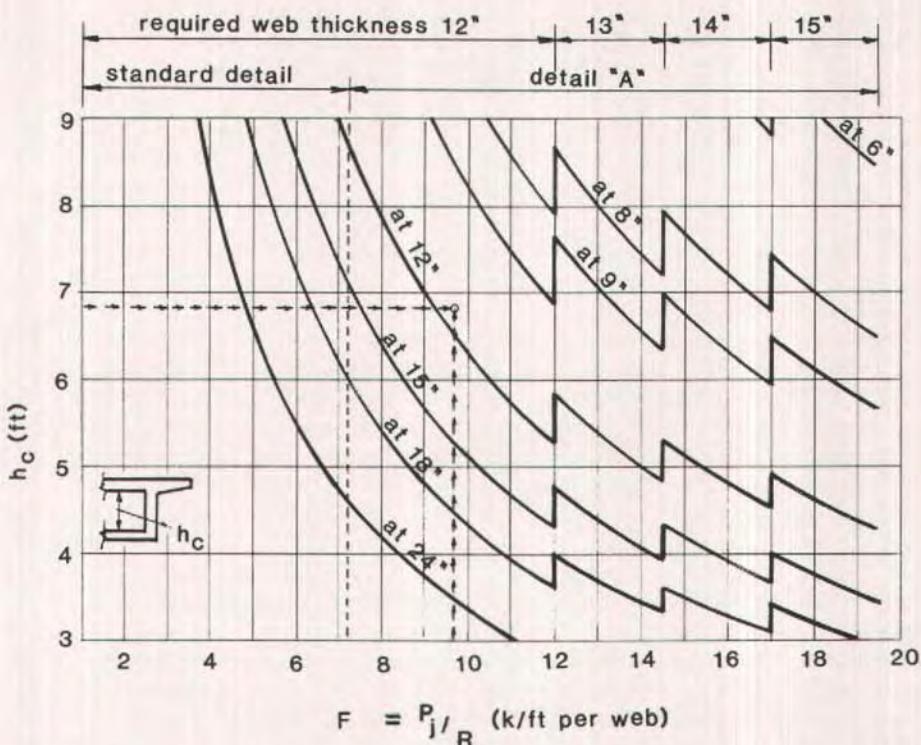


Fig. 50. Chart for determining web reinforcement (#6 stirrups).

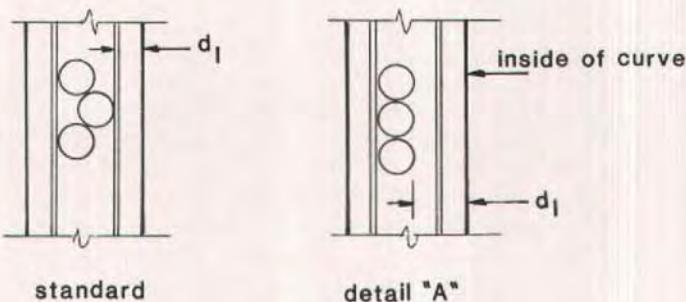


Fig. 51. Tendon placement details.

when compared to the allowable stresses in concrete or its reinforcement is approximately as follows:<sup>\*</sup>

1. For global secondary horizontal stress ..... 1 to 10 percent
2. For regional bending stress ..... 5 to 50 percent
3. For local slab bending stresses ..... 10 to 300 percent

Of course, every case is different and could fall outside of these ranges. But the percentages cited present some reasonable order of magnitude.

Another important factor in the above is the bundling of the large tendons. It appears that bundled tendons should not be used for horizontally curved bridges with a radius under 700 ft (213 m), unless proper design considerations are implemented.

It therefore appears that, for both these structures, the local slab action was the primary cause of failure, but the regional beam action could have been a contributory cause, and could by itself have overstressed some of the stirrups, even if not to the point of failure. The global action had a relatively small effect upon these failures.

In the design of curved post-tensioned concrete box girders the designer must consider the lateral prestress force. In recognition of this problem the California Department of Transportation (CALTRANS) has prepared and implemented design guidelines.<sup>†</sup> They have prepared charts and details to be used as a check of girder webs for containment of tendons and adequate stirrup reinforcement to resist flexural bending.

As an example, assume that the design of the girder requires a prestress jacking force ( $P_j$ ) of 2900 kips (12.9 MN) per web, a radius of curvature ( $R$ ) of 300 ft (91.4 m), and a vertical inside height of web ( $h_c$ ) of 6.83 ft (2.08 m). By dividing

the jacking force ( $P_j$ ) by the radius ( $R$ ), a lateral prestress force ( $F$ ) of 9.67 kips per ft (14 kN/m) is obtained.

The first step is to enter the chart (Fig. 49) with this value ( $F$ ) on the horizontal axis of the graph and travel vertically upward until the ordinate ( $h_c$ ) of web height is reached. The chart then indicates that a web thickness of 12 in. (0.305 m), #5 stirrups at 9 in. (228 mm) spacing, and tendon placement detail "A" as opposed to a "standard detail" is required.

A similar set of curves (Fig. 50) has been prepared for #6 stirrups. In cases that specify final prestress force ( $P_f$ ) rather than jacking prestress force ( $P_j$ ), it may be assumed that  $P_j$  is equal to 1.25  $P_f$ . Up to a lateral prestress force pressure ( $F$ ) of 7.2 kips per ft (10.5 kN/m), shown by the vertical dashed line, the chart indicates a "standard" tendon placement. Above this value, Detail "A" is required.

The "standard detail" indicated at the left of Fig. 51, as the name implies is a standard detail for bundled tendons. Detail "A", at the right of Fig. 51, indicates that the tendons are stacked vertically and placed against the outside stirrup with respect to curvature. This is done to provide a greater thickness for local slab action and to put the inside stirrup in a position to participate as active reinforcement in the slab action.

Further, Tendon Detail "A" requires #4 ties between the stirrups and adjacent to the bundled tendons, and a hoop around the bundled tendon hooked to the outside stirrup (Fig. 52). This then mobilizes the outboard stirrup and the central concrete area behind the inboard stirrup in participating to resist the pull-out of the tendons.

The application of the California guidelines is based upon the following assumptions:

1. The girder web is assumed to be a beam with a length equal to the clear distance between top and bottom flanges.

\*Private correspondence with Dr. T. Y. Lin.

†Memo to Designers, "Curved Post-Tensioned Bridges," 11-31, California Department of Transportation, November 1982.

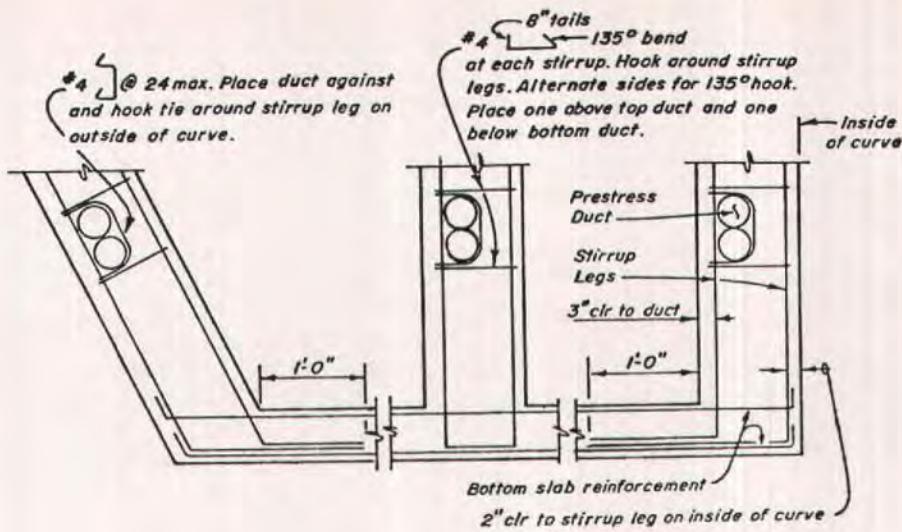


Fig. 52. Tendon placement Detail "A".

2. The lateral prestress force is assumed acting at midheight of the web.

3. The moment is calculated by simple beam formula reduced 20 percent for continuity between web and flanges with a 1.0 load factor applied.

4. The shear and bending stresses in the web stirrups are additive. However, for the purpose of these design aids the stirrups are considered capable of handling these stresses independently for the following reasons:

- The ultimate moment is calculated for the maximum condition of the lateral prestress force ( $F$ ) acting at midheight of the web span. This occurs at only two points in a span due to tendon drape.
- The jacking force,  $P_j$ , is used in the calculations of ultimate moment, and at the time  $P_j$  is applied, the structure is supported on falsework. When the falsework is removed and vertical shear forces act, the pre-stressing force has been reduced by losses.

It appears that these guidelines should be incorporated in the design of horizontally curved post-tensioned girders which have a radius of approximately 700 ft (213 m) or less.

## OTHER CRACKING MODES ASSOCIATED WITH CURVED TENDONS

Cracks have occurred in thin webs following the profile of curved tendons at some distance from the anchorage. This type of cracking appears in construction, during tendon stressing, and later stabilizes. Nevertheless, these cracks can lead to corrosion of the ducts and eventually the tendon. If moisture penetrates these cracks and subsequently freezes, spalling can occur. Although this type of cracking occurs in thin webs, it can occur in thin flanges where there is horizontal curvature of the tendons.

This type of crack usually occurs where there is significant tendon curvature and multi-strand tendons. Where

there is tendon curvature, a uniform lateral pressure is set-up that is normal to the tendon axis and in the plane of curvature. This laterally distributed pressure may cause cracking along the tendon profile at a prestress force lower than that required to initiate cracking at the anchorage zone.

The strands in a multi-strand tendon, as it is being tensioned, tend to flatten out toward the inside of the tendon curvature. This action then creates large lateral forces on the duct and side cover resulting in cracking and/or spalling. The failure mechanism is illustrated in Fig. 53.<sup>21,23</sup>

To preclude this type of cracking or failure, confinement reinforcing is required in the area of tendon curvature. A very excellent research program was conducted at the University of Texas at Austin concerning this phenomenon, along with cracking at the anchorage zone proper, and is presented in Refs. 19 through 23. An empirical design method to control cracking along the tendon profile is given in Ref. 23.

Another type of cracking is associated with the anchorage of external tendons in a diaphragm. The general arrangement of external tendons in a segmental box girder bridge is illustrated in Fig.

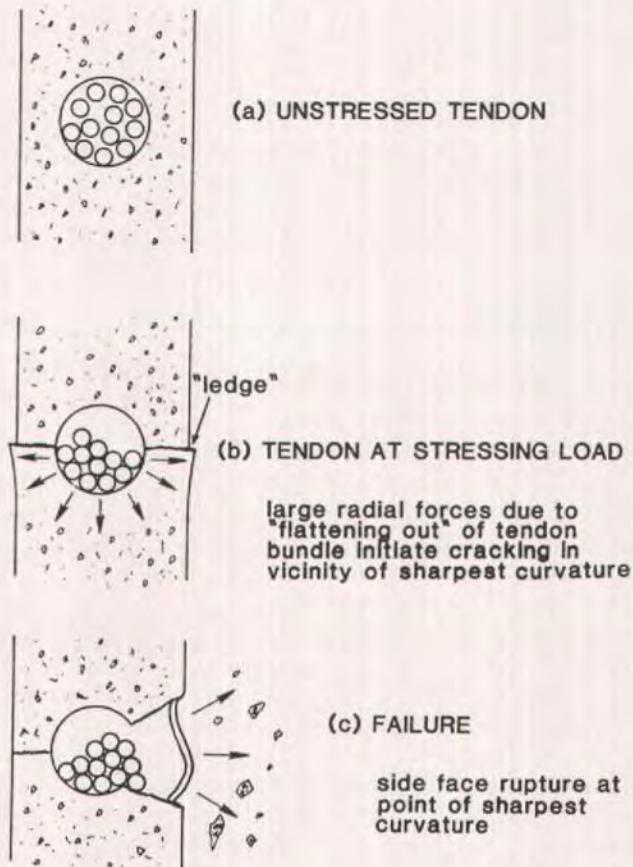


Fig. 53. Multistrand failure in a curved tendon.

post-tensioned tendons  
in polyethylene ducts

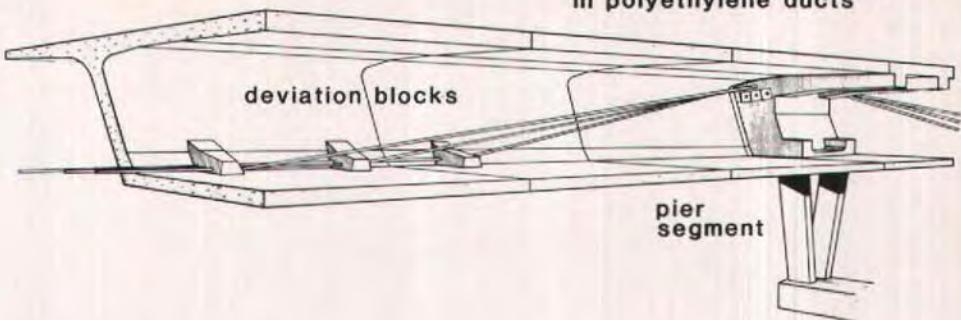


Fig. 54. External tendon configuration.

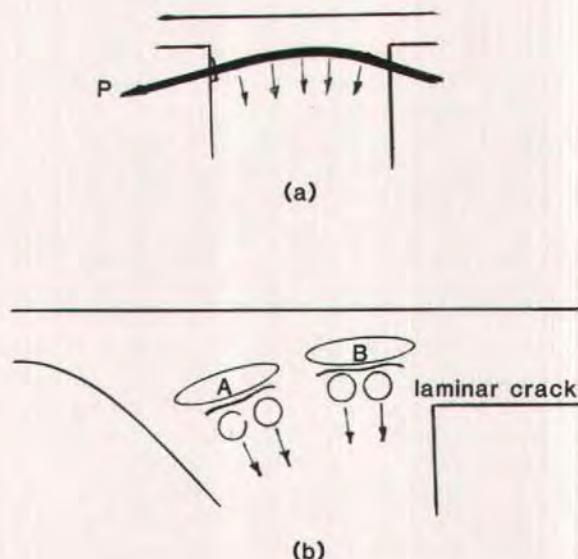


Fig. 55. Diaphragm cracking from tendon curvature.

54. The external tendons pass through the diaphragm along a vertically curved profile and are anchored in the diaphragm face which produces a radial pressure as indicated in Fig. 55a. As a result, a tensile stress field is set-up above the tendons (Fig. 55b) similar to the tension stress field behind an anchorage blister. This tension field may then cause a laminar crack above the tendons.

In addition, a splitting crack may develop as a result of multi-strand compaction in the zone of curvature as discussed above. It appears that the laminar crack either by itself or in conjunction with the splitting crack, over a period of time will propagate to the surface and produce spalling. Spalled areas of the deck have been as large as 3 x 3 ft (1 x 1 m). The spalled surface in two projects were located generally over the

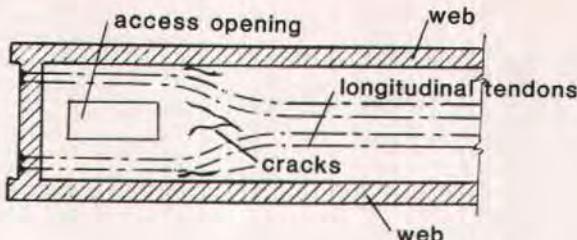


Fig. 56. Cracking at an access opening.

tensile zone B in Fig. 55 b.

This is probably associated with the fact that there is less cover over the tendons in Zone B and that Zone B represents the jacking end of the tendons and thus a larger prestress force. Zone A is associated with the dead-end anchorages. It appears that tie reinforcement should be provided to not only confine the splitting crack but also to provide reinforcement to resist the tension generated above the tendons.

Diaphragms can be elements of a bridge that have a highly complex stress field. This is particularly true if, in addition to vertically curved longitudinal tendons, there exist transverse tendons in the top flange and straight or curved tendons in its plane to accommodate torsion. A finite element method of analysis should be utilized to evaluate the state of stress in these members.

Where longitudinal tendons are flared or deflected around access openings in a bottom flange, there is a potential for severe cracking to occur as illustrated in Fig. 56. The flange in front of the opening is literally being torn or ripped apart as a result of the tension created by the tendon curvature.<sup>1</sup>

## CRACKING AND SPALLING FROM TENDON MISALIGNMENT

Longitudinal ducts at the segment joints are usually placed at their proper

position, being held in position by the bulkhead form or by the position of ducts in the segment cast against. However, if flexible tubing is used with an insufficient number of supporting chairs or ties, or if they are deflected downward by the weight of the wet concrete being placed, or by workmen walking in the fresh concrete, the duct profile will have an angle break or cusp at each joint. In addition to the increased friction losses, there is a potential danger of local spalling and bursting of the intrados of the bottom flange (Fig. 57a).<sup>1,2,3,24</sup>

Cracking may also appear in the top of the flange midway between the segment joints. Depending on the spacing between adjacent tendons and therefore the area of concrete between tendons to resist the tension developed, a laminar crack may develop (Fig. 57b).

Laminar cracking may then propagate to a spalling failure during further tensioning of the tendon (Fig. 57c) which will produce a deformation in the reinforcing steel and displacement of the tendon. In some instances, spalling has occurred during water pressurization tests or during grouting, especially when all tendons in a group are tested or grouted simultaneously (Fig. 57d).

Occasionally, because of the wrong height chairs or mislocation, the profile will be inadvertently placed in an inverted profile as shown in Fig. 57e. Obviously, the cracking and/or spalling pattern will be inverted from that shown in Fig. 57a.

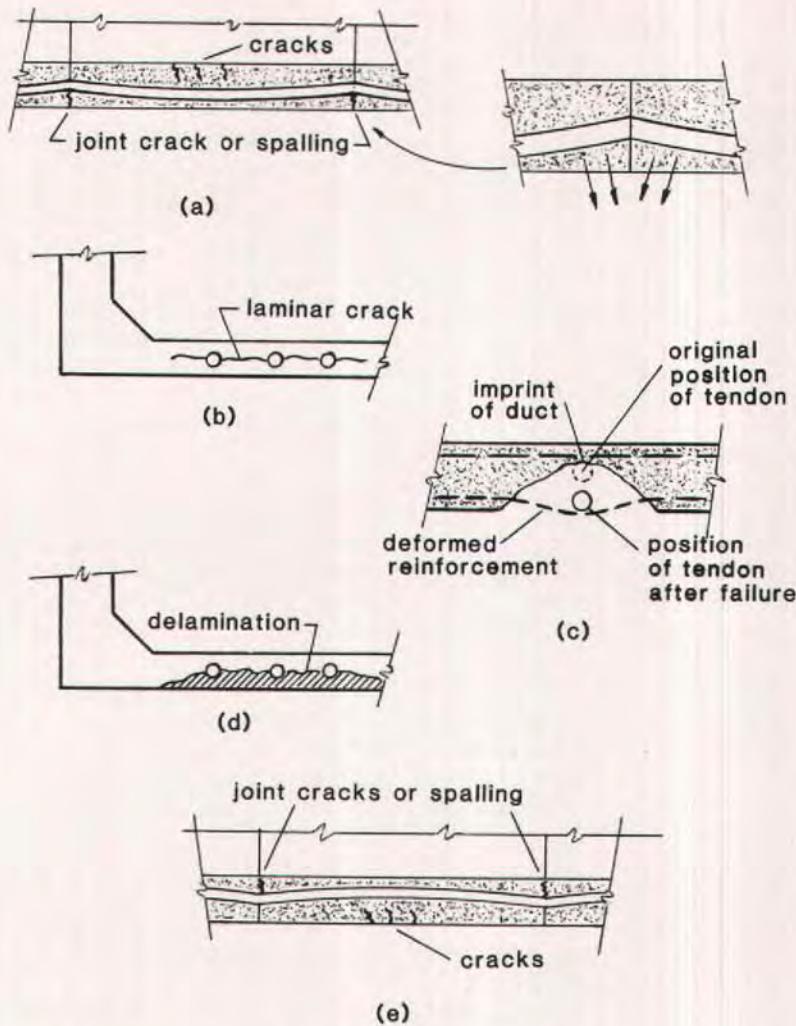


Fig. 57. Effects of misalignment of tendon ducts.

This problem can be avoided by the use of rigid ducts properly positioned and securely tied to the reinforcing cage, use of a rigid mandrel placed inside of the duct during casting, and providing work bridges for workmen so that they do not walk in the wet concrete.

In a recently reported example<sup>24,25</sup> of this type, a 16 x 32 ft (5 x 10 m) area of the upper portion of a deck slab laminated and spalled. Investigation of the

problem indicated that during construction the longitudinal tendons had been deflected downward because of an interference with the deflected profile of the transverse tendons.

The result of the upward components of the two curved tendon systems probably created a laminar crack upon tensioning of the tendons. Air pressure used to blow out the remaining unstressed tendon ducts probably propa-

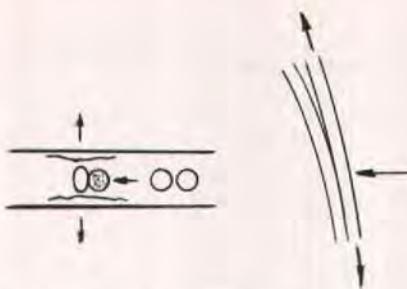


Fig. 58. Lateral duct crushing.

gated the laminar crack initiated during stressing and precipitated the spalling. A factor closely associated with this failure was the numerous closely spaced longitudinal ducts.

It may have been possible to substitute 24-0.6 in. (15.4 mm) strand tendons for the 19-0.5 in. (12.7 mm) strand tendons specified thus reducing the number of tendons by 40 percent and increasing the horizontal (in-plane) area of concrete at mid-depth of the flange by 70 percent.<sup>24</sup> In effect, numerous very closely spaced ducts produce a "built-in" lamination.

Subsequent to the failure described above, #3 C-shaped vertical ties at 24 in. (610 mm) centers were installed between tightly spaced tendons to tie together the top and bottom portions of the flange. These ties cannot prevent cracking but do function as crack propagation arresters.<sup>25</sup>

There are several other interacting causes that may have contributed to other laminar spalling. Although the design had the ducts spaced apart, in reality in many instances they touched, especially where there was a horizontal curvature of the ducts. In some instances where there was a horizontal curvature and there were closely adjoining ducts, stressing of the tendon on the outer side of the curve caused the duct to crush into the inner duct, distorting it vertically and causing spalling (Fig. 58).<sup>24</sup>

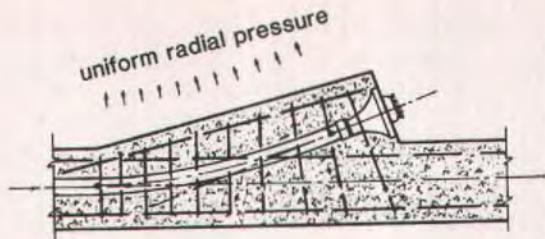
To avoid the problem of duct crushing, a number of alternatives are available to restrain the duct being tensioned from displacing into and crushing an adjacent ungrouted duct, namely, maintaining a clearance of one duct diameter between ducts, use of prebent steel tubes of adequate thickness in the zone of curvature, or providing adequate tie reinforcement.

Close spacing of longitudinal ducts combined with transverse ducts and longitudinal and transverse reinforcing steel in the top and bottom of the flange makes it very difficult to effectively place concrete. Concrete does not always work around and under the ducts and reinforcement. Workmen often stand or walk in the fresh concrete. This pushes the ducts and reinforcement down and when the men have moved, it springs back up, which may cause a laminar separation in the concrete.

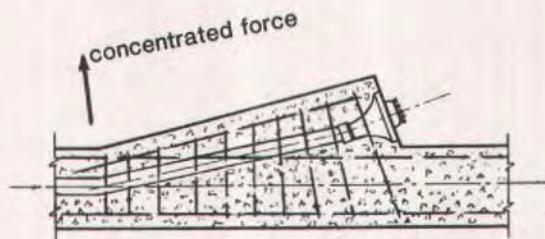
Apparently, vibration does not always extend deep enough to force concrete up between closely spaced ducts and knit the top and bottom layers together. Bleed water may collect under flat ducts and in the "V" formed by two adjoining longitudinal ducts. Much of the problem can be alleviated by selection of tendon size and duct spacing to maximize the concrete area at mid-depth of the flange, as discussed above.

Another form of cracking, spalling and failure is associated with misplacement of a curved tendon in a blister anchorage. The design assumes that the tendon profile as it leaves the flange (or web) and enters the anchorage blister is a smooth curve (Fig. 59a) with a uniform pressure along the curve. In reality, this is seldom accomplished in construction. The duct usually has an angular "kink" instead of a smooth curve transition, which produces a concentrated force at the toe of the blister (Fig. 59b).

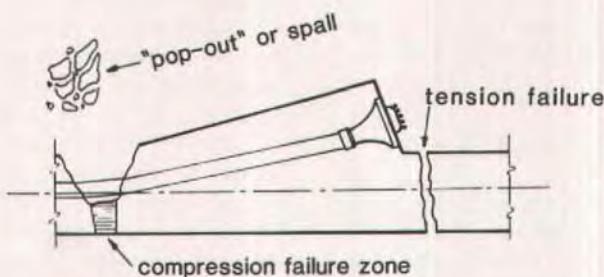
This then leads to a spalling or "pop-out" failure (Fig. 59c). The concrete area in the slab forward of the blister is then reduced, leading to a compression fail-



(a) DESIGN ASSUMPTION



(b) ACTUAL CONDITION



(c) FAILURE ZONES

Fig. 59. Anchorage blister failure.

ure at that location, tension failure behind the blister and shearing along the longitudinal sides of the blister.

To avoid the "kink" in the duct, a heavier gauge duct or prebent tube should be used, tie confining reinforcement around the tendon should be extended well forward of the blister, adequate reinforcement extending from the blister into the flange (or web) behind it should be employed to cater to the ten-

sile forces behind the blister, and adequate transverse reinforcement should be used to cater to shear forces alongside of the blister.

The design of blisters requires, in addition to reinforcement for anchorage bursting stresses, adequate reinforcement for shear and bending between the blister and the flange or web interface. Consideration should be given to Sections 11.7 and 11.9 of the ACI Building

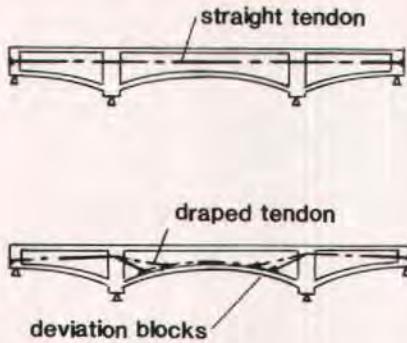


Fig. 60. Tendon profiles.

Code requirements for shear friction and corbels.<sup>26</sup> A check should be made of the localized bending induced into the flange or web away from the anchorage and reinforcement provided accordingly. A minimum radius of tendons of 15 ft (4.5 m) is recommended with an absolute minimum of 10 ft (3 m). A minimum clear tangent length of 1 ft (0.3 m) is recommended from the anchorage cone or trumpet.

## RETROFIT MEASURES

The purpose of retrofitting is to correct a deficiency in a structure the result of which has been manifested by formation of cracks. Thus, in order to propose a corrective measure, the source(s) of the problem must be determined. As previously pointed out, cracking rarely results from a single cause but more likely from a combination and interaction from several sources. The analysis and investigation of the problem are frequently difficult. Some of the phenomena exhibited are not well understood either in qualitative or quantitative terms. Therefore, the repair method can rarely be presented in simple terms.

To further compound the problem, a proposed repair must be analyzed with respect to the structure as a whole to insure that a detrimental effect or adverse stress condition is not produced at some

other location in the structure. Without the benefit of previous experience and behavioral data over time (perhaps as much as 10 years) no judgment can be made relative to a particular retrofit measure. Therefore, the retrofit measures discussed below must not be considered as absolute or automatically applicable to typical problem solutions, nor should they be attempted without rigorous analysis and application of engineering judgment.<sup>1</sup>

## Epoxy or Mortar Injection

Epoxy and mortar injection of cracks is well known and the mechanics of implementation will not be expounded upon in this paper. Rather, two precautions are presented which if not considered may inhibit the repair.

Effective repair by injection requires a stable crack. If the crack width continues to widen while polymerization of epoxy or curing of grout is taking place, the effectiveness of the repair is jeopardized. The crack may be "working" as a result of traffic loads or as a result of thermal gradient. On a particular project, it was possible to measure significant crack opening from the cool of the morning through late afternoon heating and subsequent closing when the structure cooled down during the evening. Obviously the structure should be closed to traffic. To minimize or preclude the effects of thermal gradient the deck surface should be kept cool by frequent water spraying.

The second precaution is one concerning pressure injection of laminar cracks. The problem here is one of fracture mechanics, i.e., driving the crack by pressure injection, further complicated by the fact that there usually is no cross reinforcement to arrest crack propagation.<sup>24</sup> Either a low pressure injection procedure or some external means of applying a pressure transverse to the crack, clamping the two portions of the member together, will be required.

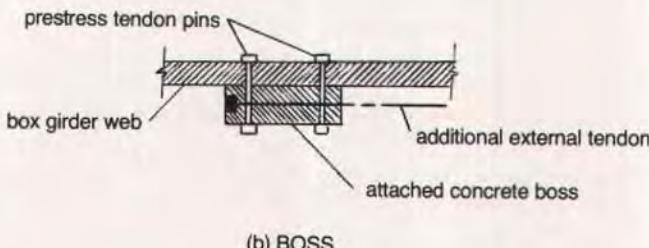
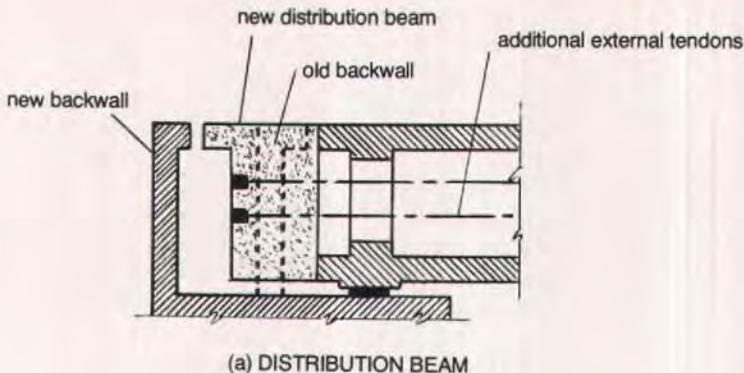


Fig. 61. Tendon anchorage.

## Flexural Cracks

In this situation the intent of the retrofit is to restore the integrity of the girder by recompressing it where it has decompressed and subsequently cracked. There are two retrofit measures that can be considered: support level adjustment or additional prestressing.<sup>1</sup>

Support level adjustment is simply a method of prestressing whereby supports are jacked in such a manner as to produce a moment that induces compressive stresses at the point where the member has been decompressed. This method is attractive because of its relative ease of implementation and low cost. However, because the method is one of forced displacement its effectiveness is short term as a result of subsequent moment redistribution.

As an example,<sup>1</sup> this method was used on a 15 year old structure in France

where it was observed that the moment developed was decreased by 20 percent after 16 months and that less than half would remain after 3 years. It is a suitable measure to be used as a temporary basis until a permanent retrofit can be devised or as a means of creating compression after grouting of the cracks and before permanent prestress is applied.

Additional permanent prestressing can be obtained by tensioning added longitudinal external tendons. To the extent possible, this method should be preceded by grouting of the cracks so that the compression being applied can be transferred.

The external tendon profile can be either straight or polygonal (Fig. 60) depending upon the problem. The straight tendon profile presents few problems with regard to installation, however, it is not very efficient and has only a minor effect in improving shear strength. The

polygonal profile is a more satisfactory solution but requires construction of deviation blocks or saddles which increase friction loss. In both methods the tendon should be rigidly attached to the superstructure at sufficiently close intervals to preclude elastic instability (buckling) of the girder and to prevent resonance in the tendons if the period of the structure is near that of the tendons.<sup>1,27</sup>

The installation of external tendons generally requires three elements for transfer of prestress: a transverse distribution member or diaphragm at the abutments, deviation blocks anchored to the web or flange or both, and a support at existing diaphragms.

The transverse distribution member at the abutments is simply a massive reinforced or prestressed beam constructed across the end of the girder for the purpose of transferring the external prestress force to the girder. In this

manner concentrated local stresses are not imparted to the original girder or diaphragm, i.e., stresses behind the tendon anchorage bearing plate are distributed to the new transverse member.

In all probability the existing diaphragm is not properly reinforced to accept these stresses. Therefore, high capacity tendons can be utilized. The disadvantage is that this construction requires removal of the existing abutment backwall, reconstruction of a new backwall, and the construction of the transverse distribution member, all of which require that the bridge be shut-down to traffic (Fig. 61a).<sup>1,27</sup>

An alternative is to anchor the longitudinal prestress tendons in a boss, of either concrete or steel, attached by prestressing to the existing structure (Fig. 61b). This method is effective in transmitting prestress to the cross section but has the disadvantage of inducing high

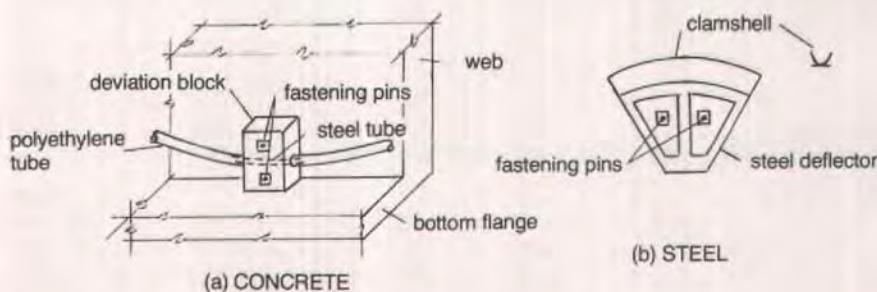


Fig. 62. Deviation saddle.

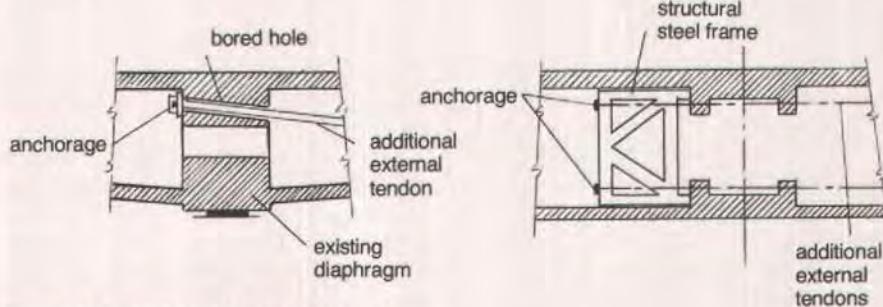


Fig. 63. Anchorage at diaphragms.

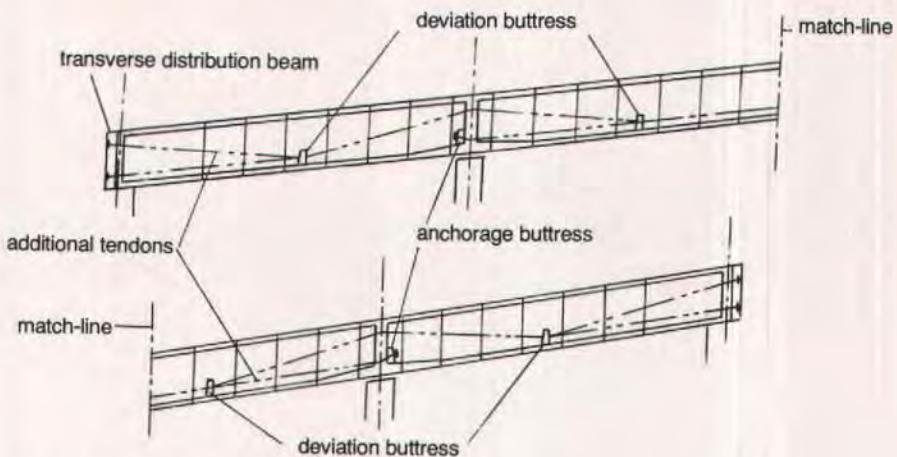


Fig. 64. Additional prestressing retrofit.

local stresses (bending, shear, and tension in the web behind the boss) which the existing structure may be incapable of accepting. Another disadvantage is the very short tendons used to pin the boss to the web. These short tendons are very sensitive to losses and could result in the boss sliding which would, of course, lead to unanticipated losses in the longitudinal tendons. Therefore, particular attention must be given to the:<sup>1</sup>

1. Methodology to be followed to reduce losses through the improper fastening of anchorages during tensioning operations.

2. Distribution of the anchoring force along the entire length of the boss (transfer of prestressing from the boss to the structure).

3. Treatment of the surface of the joint between the anchoring boss and the structure, and the resulting coefficient of friction.

4. Value of the coefficient of friction between the fastening mortar and the boss, if it is made of steel, and

5. Ratio between the fastening force and force created by the anchored tendons.

Where a polygonal tendon profile is

used, deviation blocks or saddles have to be provided to achieve the profile. These devices can be either concrete or steel (Fig. 62). They are attached to the existing web or flange by prestressing pins.<sup>1,27</sup> They are subject to the same difficulties as bosses discussed above.

At existing diaphragms the tendons can pass through or be anchored (Fig. 63a) provided they have sufficient strength and are adequately attached to the girder cross section to transfer the resulting longitudinal shear. Depending upon the diaphragm detail it may be necessary to provide a structural steel frame to transfer the longitudinal pre-stress force (Fig. 63b). With existing diamond coring technology 3 in. (80 mm) diameter holes can be drilled to depths of approximately 6.5 to 10 ft (2 to 3 m), with a longitudinal tolerance of  $\frac{1}{16}$  in. to  $\frac{3}{8}$  in. (5 to 10 mm).

A recent application of this system in the United States (Fig. 64) used the transverse distribution member at the abutments and a concrete buttress for a deviation saddle that was keyed and doweled into the existing flange and web (Fig. 65). Where tendons were anchored to existing diaphragms, the diaphragm was locally reinforced by a sim-

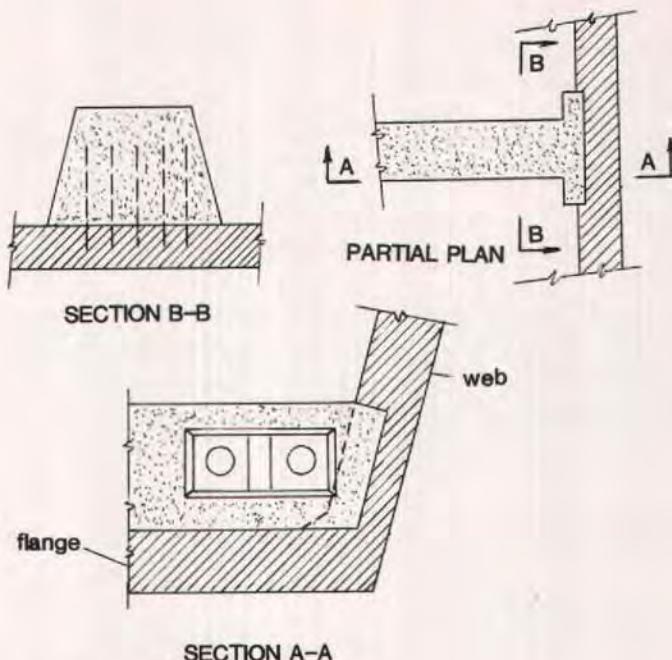


Fig. 65. Retrofit deviation buttress.

ilar buttress arrangement (Fig. 66).

A potential repair method for the type of cracking shown in Fig. 34 is illustrated in Fig. 67 and consists of supporting the bottom flange by a suspension system.<sup>1</sup> If the crack width is less than 0.008 in. (0.2 mm), the cracks are harmless with regard to environmental effects; if the crack width exceeds this dimension it should be epoxy injected. In some cases the crack width may reach  $\frac{3}{8}$  in. (1 mm). In this case consideration might be given to applying external forces to the section so as to return it to, or near to, an undeformed state.

This can be accomplished (in Fig. 67) by jacking the suspension rods at the deck level producing the external loads indicated in Fig. 68. However, this method or any other method (tried or untried previously) should not be attempted without careful evaluation and analysis to determine that adverse effects are not being introduced at the

section in question or at some other location in the structure.

Another possible repair method is that indicated in Fig. 69, whereby the flange thickness is increased or transverse ribs are added to assist in resisting the flexural stresses.<sup>1</sup> However, some means of providing external support to the original bottom flange to resist the added wet concrete weight, so as not to aggravate the already overstressed original flange, must be provided. In addition, or concurrently, a method of external loading is required to return as near as possible the original flange to its undeformed position, otherwise the stress condition in the bottom flange is not alleviated and the effectiveness of the procedure is questionable.

### Shear Cracks

Repair of shear cracks in the web of the structure illustrated in Fig. 7 was accomplished by adding vertical prestress

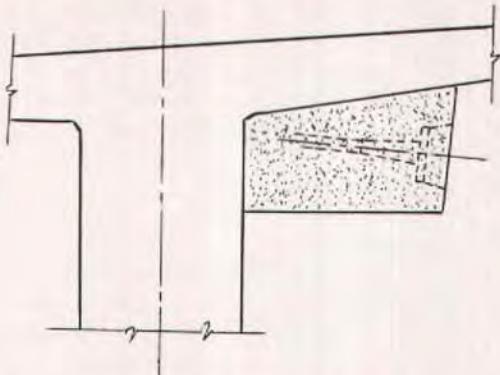
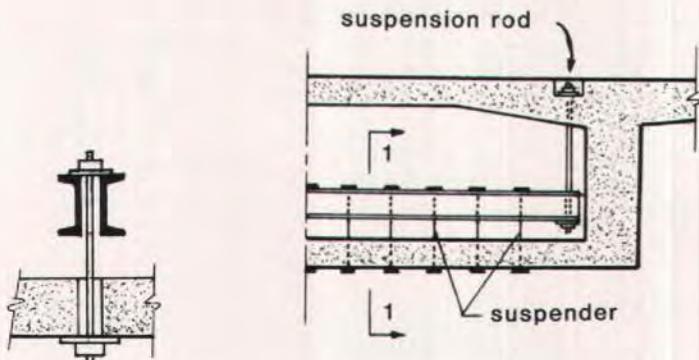


Fig. 66. Retrofit anchorage buttress at diaphragm.



Section 1-1

Fig. 67. Repair system.

in the center web after injecting the cracks. Two series of Dywidag vertical prestress tendons were utilized to accomplish the retrofit in the following operations:<sup>4</sup>

1. The overlay and membrane was removed in such a manner as to allow restoration.

2. The theoretical location of vertical tendons was marked with chalk on the concrete.

3. Anchorage pockets were chipped out with a lightweight pneumatic hammer; this had to be accomplished very carefully because in the process the

upper layer of reinforcement and longitudinal tendon ducts were exposed.

4. Since at some locations the longitudinal cantilever tendons in the top flange terminate, it was possible to bore a  $2\frac{1}{4}$  in. (56 mm) hole through the web to within 10 in. (250 mm) of the bottom of the box. At this point a gamma ray source was placed in the hole and at the same time a sensitive plate was placed underneath the box. If the exposure indicated that there was no interfering longitudinal bottom flange tendon, the drilling was continued and a  $1\frac{1}{4}$  in. (32 mm) diameter vertical Dywidag bar

tendon was placed (Fig. 70a). Where there was interference from either a longitudinal top or bottom tendon, the single tendon was replaced by two 1 in. (26.5 mm) diameter Dywidag bars on either side of the web (Fig. 70b).

Placement of retrofit shear tendons as indicated above is very satisfactory in theoretical terms but has the following disadvantages:<sup>1,27</sup>

In the case depicted by Fig. 70a:

1. An anchorage is located near the roadway surface and therefore is sus-

ceptible to deicing chemicals.

2. Because of anchorage slip and/or anchor set losses associated with short tendons, it is difficult to obtain a theoretical prestress force.

3. Boring is always a sensitive operation and often impossible, especially when boring the full depth of the web.

In the case depicted by Fig. 70b, the boring problem is significantly minimized; however, the problem of anchorage location with respect to the roadway surface and losses in short tendons are

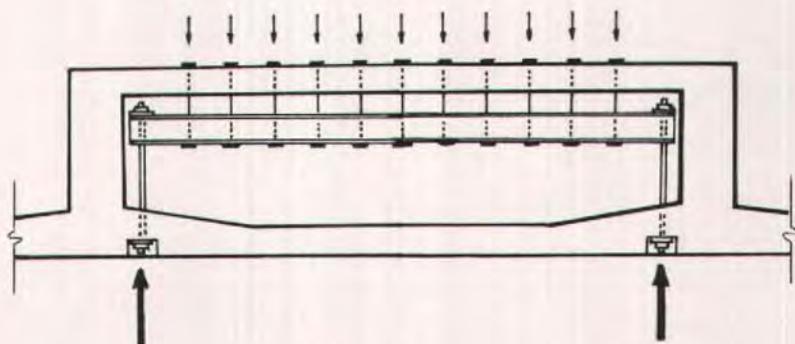


Fig. 68. Counteracting external loading.

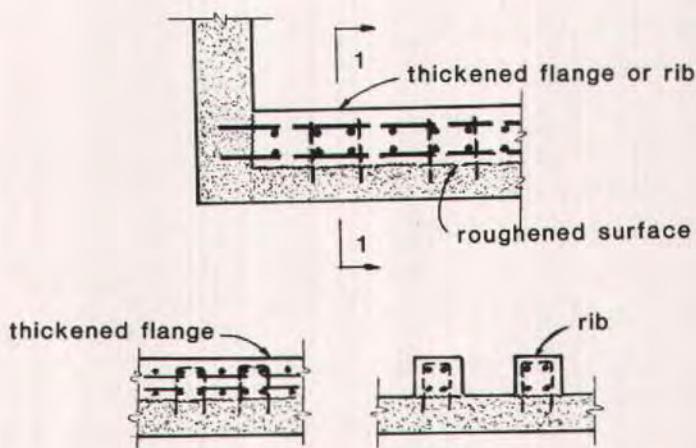


Fig. 69. Repair system.

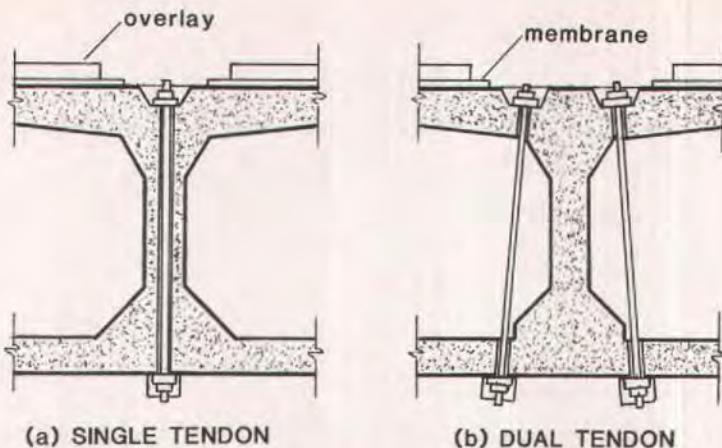


Fig. 70. Retrofit shear tendons.

retained and the following additional disadvantages are introduced:

1. Punching shear through the flanges.
2. Secondary bending stresses in the flanges and webs. The bending in the webs is particularly true of thin webs and where an external web is concerned, where vertical tendons would only be located at the inside of the box.

The retrofit measures used for the repairs of cracked webs for a structure in Canada<sup>28</sup> utilized a similar technique for the vertical internal webs as described above. However, the inclined exterior webs were more difficult to retrofit since no bar tendons could be used at the outside face of the web.

Boring through the external web was impossible because of the two side-by-side upper longitudinal tendons (Fig. 71a). Preliminary calculations indicated that placing tendons along the inside face of the web only would produce undesirable large tensile stresses in the outside surface of the web. Therefore, a decision was made to thicken the outside webs as shown in Fig. 71a.

To produce the required prestress in the thickened web, tendons had to be spaced at  $7\frac{1}{2}$  in. (190 mm). To preclude

weakening of the top and bottom flanges by the 2 in. (51 mm) diameter holes required, the tendons were staggered in a vertical and inclined orientation as shown in Fig. 71a. The shearing force produced at the interface of the existing web and the web thickening is largely resisted by bond. To prevent shearing at the interface, mechanical doweling was provided by  $\frac{5}{8}$  in. (16 mm) diameter bolts drilled into the web which supported a grid of conventional reinforcement (Fig. 71b).

## DESIGN/CONSTRUCTION RECOMMENDATIONS

In the previous sections the author has attempted to delineate the causes of cracking in post-tensioned concrete bridges. To minimize the occurrence of cracking during construction, or at some future time in the life of the structure, the designer should re-evaluate the following major considerations upon conclusion of the design:

1. Has the effect of tendon curvature been accommodated?
  - (a) Horizontal pullout.
  - (b) Vertical curvature of soffit ten-

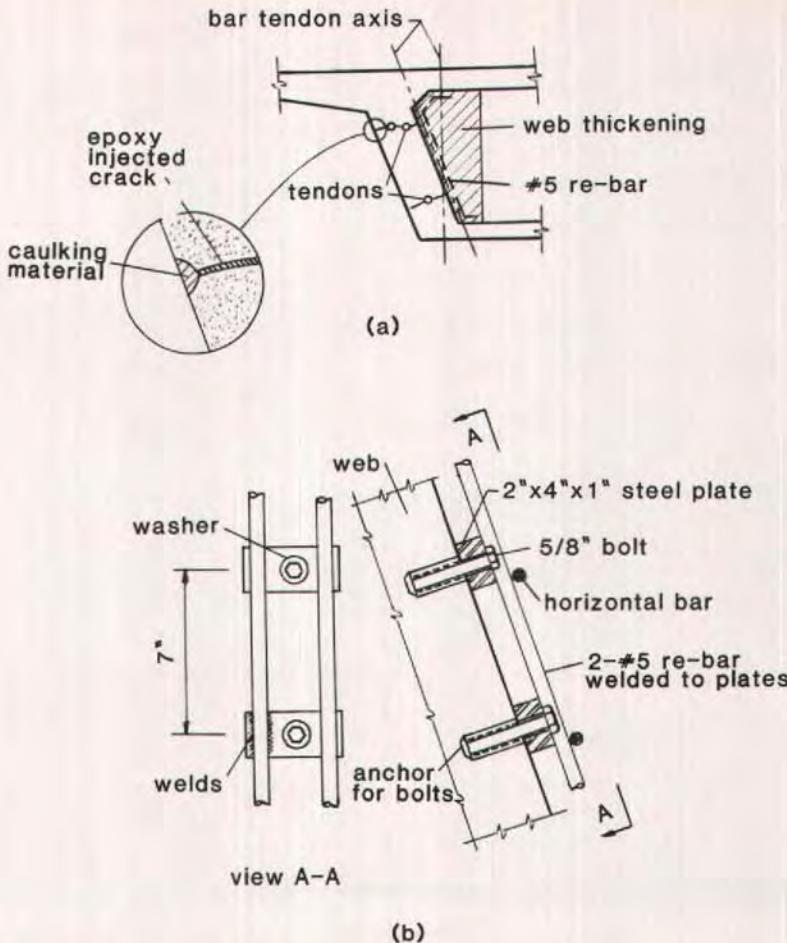


Fig. 71. Retrofit of inclined external web.

- dons.
  - (c) Curvature near anchorage zones in thin webs.
  - (d) Diaphragm cracking and deck lamination over diaphragms.
  - (e) Adequate spacing between horizontally curved ducts to prevent crushing.
2. Have blisters or embedded dead end anchorages been designed to adequately accommodate stresses at these locations?
  3. Have stresses at bearings been adequately addressed and have provi-

sions been made in design for future jacking in the event of bearing replacement?

4. Have thermal stresses and thermal gradients been addressed?

5. Are there locations in the structure where large variations of prestress eccentricity occur and have critical sections to either side of these locations been checked?

6. Has the transverse design of the section taken into account the true flexibility of the cross section (shear distribution in multicell girders)?

It is recommended that provisions for future prestressing (say 10 percent of the design longitudinal prestress force) be provided for in the initial design. Providing anchorage bearing plates and trumpets in the diaphragms and intermediate deviation saddles for future placement of external tendons is a very minor cost during initial construction. To provide this capability after initial construction can be a very expensive and difficult task.

During construction a number of simple precautions can preclude costly time delays and repairs.

1. Use of stiff pipe mandrels and work bridges to preclude displacement or offsetting of tendons from segment to segment during fabrication or casting.

2. Verify that ducts are not being "kinked" at tendon curvature locations. Use prebent heavy gauge ducts or steel tubes when curving of tendons is necessary.

3. Verify that adequate spacing is provided between ducts at points of horizontal curvature to prevent crushing of ducts, i.e., spread the ducts as much as possible.<sup>25</sup>

4. Be aware of and check construction loads to preclude overstressing and cracking i.e., stock piling of material on the deck.

5. Immediately bring to the attention of the Engineer any interferences between ducts or ducts and reinforcement and carefully evaluate the consequences of duct relocation or cutting or reinforcement.

6. Careful adherence to and monitoring of concrete curing methods. Avoid temperature gradients between segments during match-casting operations. Avoid accelerated curing.<sup>25</sup>

7. Minimize or eliminate any warping or distortion of segments during storage when the concrete is still at an early age.

## CONCLUDING REMARKS

An attempt has been made in this paper to document the current state-of-knowledge with regard to reasons for cracking in post-tensioned concrete box girder bridges. However, the reasons for cracking are so varied, complex and interactive that it is impossible to present a document that is all inclusive and authoritative. As with any type of construction with any material, problems arise that require the attention of not only the designer, but contractors and subcontractors as well. No matter how good the design, if the structure is not properly constructed, there will be problems. Conversely, no matter how diligent the contractor, if the design details are poor, problems will result. Obviously, if both the design and the construction are poor, problems are compounded.

Technological progress is only of value if it is measured against both pos-

itive and negative experiences. However, there exists in our society a cautious attitude toward dealing with unfavorable developments, errors, and damages which inhibits technological progress. Unfortunately, when a problem occurs the initial reaction is to seek out the guilty. This then leads to the situation whereby, under direction from the legal profession, all parties concerned are advised not to discuss the problem because of potential liability and litigation.

This is to some extent understandable since none of the parties involved desire to jeopardize their rights or to incriminate themselves. The disadvantage is that it may take years before the case is resolved in the courts and in the meantime the public safety at large may be jeopardized because the same errors or problems are perpetuated in possibly hundreds of additional projects. This impasse requires resolution and cooper-

ation from both the legal and engineering professions.

The only way our technology can advance is if problems are openly and frankly discussed, resolved and information disseminated to the profession.

Education by dissemination of information is the only viable methodology available to minimize the recurrence of the problem. If this is not done, we run the risk of losing credibility with decision makers and society in general.

## ACKNOWLEDGMENTS

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**NOTE:** Discussion of this paper is invited. Please submit your comments to PCI Headquarters by November 1, 1985.