ROAD AND BRIDGE WORKS

THE REPUBLIC OF UGANDA

MINISTRY OF WORKS AND TRANSPORT

ROAD DESIGN MANUAL

Volume 4: Bridge Design

January 2010

PREAMBLE

This Road Design Manual **Volume IV: Bridge Design** is one of a series of Engineering Specifications, Standards, Manuals and Guidelines issued by and with the authority of the Engineer-in-Chief, Ministry of Works and Transport. The Manual is part of the revised Road Design Manual, November 1994.

The four Volumes of the Road Design Manual include:

a) Road Design Manual: Vol. I Geometric Design;
b) Road Design Manual: Vol. II Drainage Design;
c) Road Design Manual: Vol. III Pavement Design; and
d) Road Design Manual: Vol. IV Bridge Design.

The Manual gives guidance and recommendations to the Engineers responsible for the design of roads in Uganda. It complements the Ministry's efforts in providing guidance to the construction industry by setting uniform standards to be used in the construction of infrastructure facilities that meet the needs of the users.

The Manual serves as nationally recognized document, the application of which is deemed to serve as a standard reference and ready source of good practice for the design of bridges, and will assist in a cost effective operation and an environmentally sustainable development of bridges in the country's road network.

The major benefits to be gained in applying this Manual are the harmonization of professional practice and the ensuring of appropriate levels of safety, health and economy with due consideration of the objective conditions and need of the country.

Further, this Manual is a technical document, which, by its very nature, requires periodic updating from time to time arising from the dynamic technological developments and changes. The Ministry, therefore, welcomes proposals on areas for further development and revision stemming from the actual field experience and practice. It is hoped that the comments will contribute to future revisions of the Manual expected to lead to better and more economical designs.

Ministry of Works and Transport
P.O. Box 10
Entebbe

January 2010

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1. SCOPE

Documents Comprising this British Standard

This specification for loads should be read in conjunction with the other Part of BS 5400 which deals with the design, material and workmanship of steel, concrete and composite bridges.

Loads and Factors Specified In This Part of BS 5400

This part of BS 5400 specifies nominal loads and their application, together with the partial factors γ_{fL} to be used in deriving design loads. The loads and load combinations specified are for highway, railway and foot/cycle track bridges in the United Kingdom. Where different loading regulations apply, modifications may be necessary.

Wind and Temperature

Wind and temperature effects relate to conditions prevailing in the United Kingdom and Eire. If the requirements of this Part of BS 5400 are applied outside this area, relevant local data should be adopted.

2. REFERENCES

The titles of the standards publications referred to in this Part of BS 5400 are listed on the inside back cover.

3. PRINCIPLES, DEFINITIONS AND SYMBOLS

Principles

* Part 1 of this standard sets out the principles relating to loads, limit states, load factors, etc.

Definitions

For the purposes of this Part of BS 5400 the following definitions apply.

Loads

External forces applied to the structure and imposed deformations such as those caused by restrain of movement due to changes in temperature.

Load Effects

The stress resultants in the structure arising from its response to loads (as defined in **3.2.1)**

Dead Load

The weight of the materials and parts of the structures that are structural elements, but excluding superimposed materials such as road surfacing, rail track ballast, parapets, mains, ducts, miscellaneous furniture, etc.

Superimposed Dead Load

The weight of all material forming loads on the structures that are not structural elements.

Live Loads

Loads due to vehicle or pedestrian traffic.

Primary Live Loads

Vertical live loads, considered as static loads, due directly to the mass of traffic.

Secondary Live Loads

Live loads due to changes in speed or direction of the vehicle traffic, e.g. lurching, noising, centrifugal, longitudinal, skidding and collision loads.

Adverse and Relieving Areas and Effects

Where an element or structure has an influence line consisting of both positive and negative parts, in the consideration of loading effects which are positive, the positive areas of the influence line are referred to as adverse areas and their effects as adverse effects and the negative areas of the influence line are referred to as relieving areas and their effects as relieving effects. Conversely, in the consideration of loading effects which are negative, the negative areas of the influences line are referred to as adverse areas and their effects as adverse effects and the positive areas of the influence line are referred to as relieving areas and their effects as relieving effects.

Total Effects

The algebraic sum of the adverse and relieving effects.

NOTE. Where elements in a positive area influence line are being considered the total effects may be negative, in which case the equivalent positive value will be the least negative effect, and where in negative effects are being considered the total effects may be positive, in which case the equivalent negative value will be the least positive effect. In either case the maximum negative or positive total effect should also be considered.

Dispersal

The spread of load through surfacing fill, etc.

Distribution

The sharing of load between directly loaded members and other members not directly loaded as a consequence of the stiffness of intervening connecting members, as e.g. diaphragms between beams, or the effects of distribution of a wheel load across the width of a plate or slab.

Highway Carriageway and Lanes

(Figure 1 gives a diagrammatic description of the carriageway and traffic lanes).

Carriageway

That part of the running surface which includes all traffic lanes, hard shoulders, hard strips and marker strips. The carriageway width is the width between raised kerbs. In the absence of raised kerbs it is the width between safety fences, less the amount of set-back required for these fences, being not less than 0.6m or more than 1.0m from the traffic face of each fence.

Traffic Lanes

The lanes that are marked on the running surface of the bridge and are normally used by traffic.

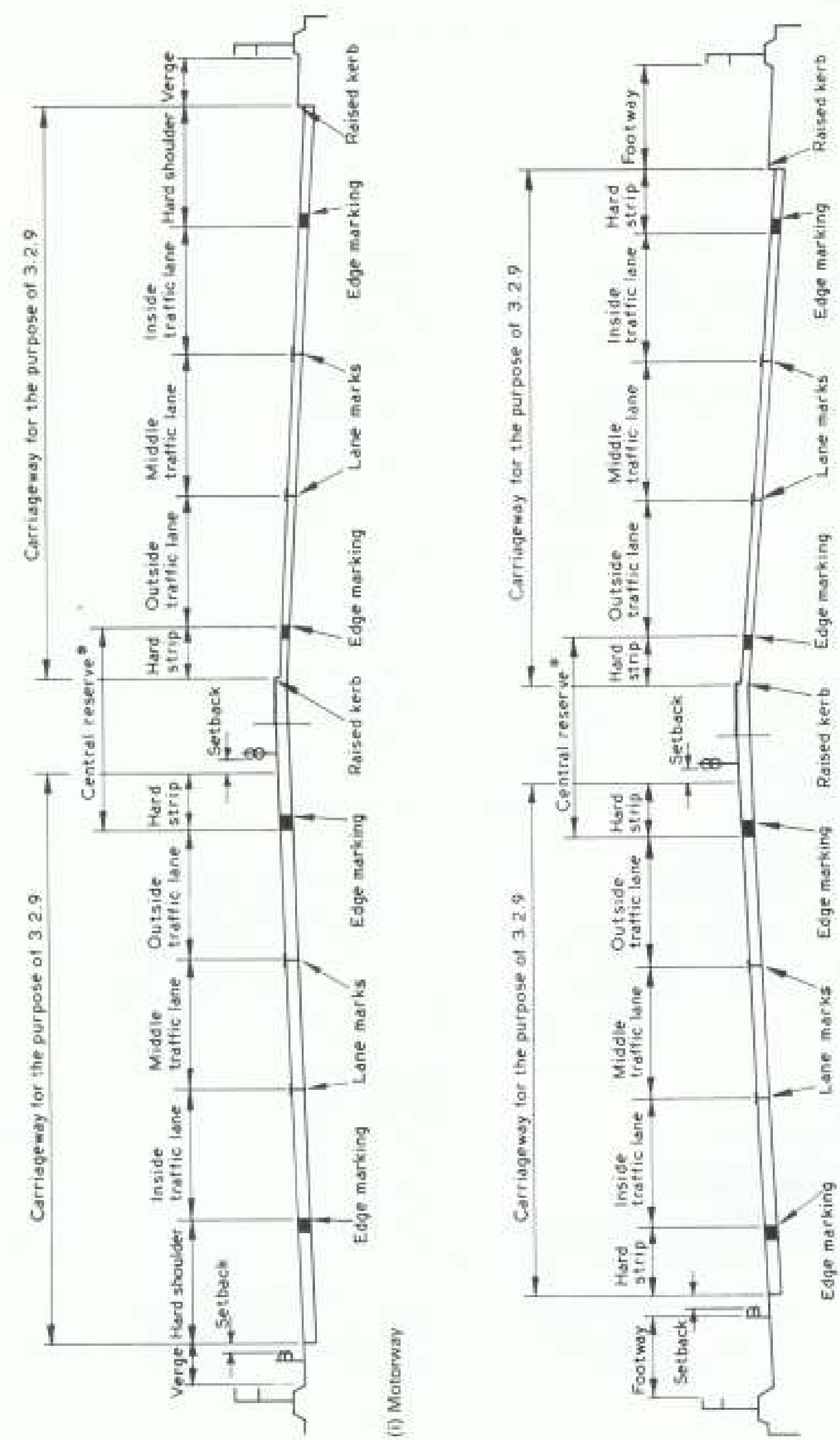
Notional Lanes

The notional parts of the carriageway used solely for the purpose of applying the specified live loads.

Carriageway Widths of 4.6m or More

Notional lanes shall be taken to be not less than 2.3m or more than 3.8m wide. The carriage way shall be divided into the least possible integral number of notional lanes having equal widths as follows:

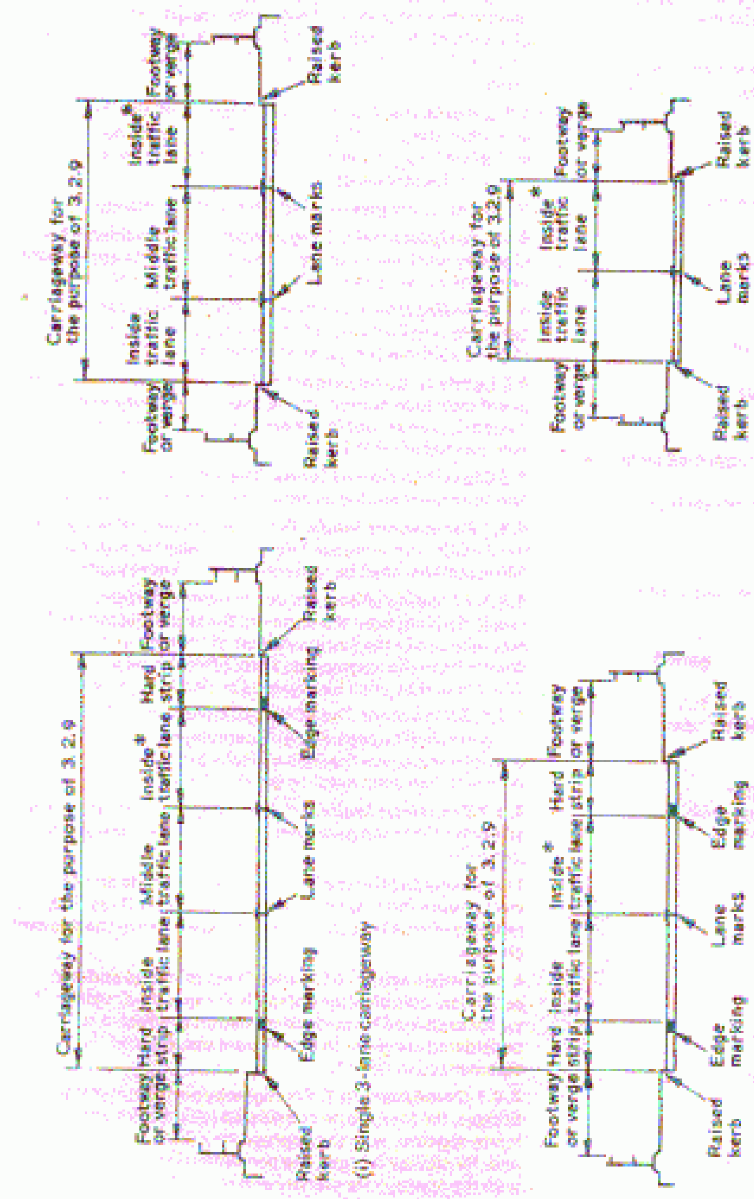
Carriageway width m	No. of notional lanes
4.6 up to and including 7.6	2
Above 7.6 up to and including 11.4	3
Above 11.4 up to and including 15.2	4
Above 15.2 up to and including 19.0	5
Above 19.0 up to and including 22.8	6



(a) Superstructures : dual carriageway

*Central reserve will be split on separate superstructures

Figure 1. Highway carriageway and traffic lanes



(i) Single 3-lane carriageway

(ii) Single 2-lane carriageway

(b) Bridge carrying a single carriageway

*Where the carriageway carries unidirectional traffic only, this lane becomes the outside traffic lane.

NOTE 1. The same definitions of inside, middle and outside have been used for individual lanes.

NOTE 2. Where a safety fence replaces a raised kerb the limits of the footway or verge and the hard strip shall be as shown in figure 1(a).

Figure 1 (concluded)

Carriageway Widths of Less Than 4.6m.

Carriageway shall be taken to have a number of notional lanes:

$$= \frac{\text{Width of carriageway (in meters)}}{3.0}$$

Where the number of lanes is not an integer, the loading on the fractional part of a lane shall be taken pro rata the loading for one lane.

Dual Carriageway Structures

Where dual carriageway are carried on one superstructure, the number of notional lanes on the bridge shall be taken as the sum of the number of notional lanes in each of the single carriage ways as specified in **3.2.9.3.1.**

Bridge Components

Superstructure

In a bridge, that part of the structure which is supported by the piers and abutments.

Substructure

In a bridge, the wing walls and the piers, towers and abutments that support the superstructure.

Foundation

That part of the substructure in direct contact with, and transmitting load to, the ground.

Symbols

The following symbols are used in this Part of BS 5400.

A	Maximum vertical acceleration
A_1	Solid area in normal projected elevation
A_2	Solid area in projected elevation normal to the longitudinal wind direction
A_3	Area in plan used to derive vertical wind load
B	Width used in deriving wind load
C	Spacing of plate girders used in deriving drag factor
C_D	Drag coefficient
C_L	Lift coefficient
D	Depth used in deriving wind load
D_1	Depth of deck
D_2	Depth of deck plus soil parapet
D_3	Depth of deck plus live load
d_L	Depth live load
F	A factor used in deriving centrifugal load on railway tracks
f_o	Fundamental natural frequency of vibration
F	Pulsating point load
Fc	Centrifugal load
H	Depth (see figure 9)

K	A constant used to derive primary live load on foot/cycle track bridges
K	Configuration factor
K_1	A wind coefficient related to return period
K_2	Hourly wind speed factor
I	Main span
I_1	Length of the outer spans of a three-span superstructure
L	Loaded length
N	Number of beams or box girders
P	Equivalent uniformly distributed load
P_L	Nominal longitudinal wind load
P_t	Nominal transverse wind load
P_V	Nominal vertical wind load
Q	Dynamic pressure head
R	Radius of curvature
S_1	Funneling factor
S_2	Gust factor
T	Thickness of pier
V	Mean hourly wind speed
v_c	Maximum wind gust speed
V'_c	Minimum wind gust speed
V_t	Speed of highway or rail traffic
W	Load per meter of lane
Ys	Static deflection
γ_{f1}	Factor, which takes account of the possibility of unfavourable deviation of the loads from their nominal values
γ_{f2}	Factor, which takes account of the reduced probability that various loadings acting together will all attain their nominal values simultaneously.
γ_{f3}	See section **4.1.3** of this part
γ_{fL}	Partial load factor (Y_{f1} x Y_{f2})
δ	Logarithmic decrement of decay of vibration
η	Shielding factor
Ψ	Dynamic response factor
N	Number of axles (see appendix D)
T	Time in seconds (see **C.3**)
	Temperature different (see figure 9 and appendix E)

4. LOADS: GENERAL

Loads and Factors Specified

Nominal Loads

Where adequate statistical distributions are available, nominal loads are those appropriate to a return period of 120 years. In the absence of such statistical data, nominal load values that are considered to approximate to a 120-year return period are given.

Design Loads

Nominal loads shall be multiplied by the appropriate value of γ_{fL} to derive the design load to be used in the calculation of moments, shears, total loads and other effects for each of the limit states under consideration period. Values of γ_{fL} are given in each relevant clause and also in table 1.

Additional Factor γ_{f3}

γ_{f3}. Moments, shears, total loads and other effects of the design loads are also to be multiplied by γ_{f3} in certain circumstances (see **4.3.2** of Part 1 of this standard).

Values of γ_{f3} are given in Part 3, 4 and 5 of this standard.

Fatigue Loads

Fatigue loads to be considered for highway and railway bridges, together with the appropriate values of γ_{fL} , are given in part 10 of this standard.

Deflection and Camber

For the purposes of calculating deflection and camber the nominal loads shall be adopted. (i.e. γ_{fL} shall be taken as unity).

Loads to be considered

The loads to be considered in different load combinations, together with the specified values γ_{fL}, are set out in the appropriate clauses and summarized in table 1.

Classification of Loads

The loads applied to a structure are regarded as either permanent or transient.

Permanent Loads

For the purposes of this standard, dead loads, superimposed dead loads and loads due to filling material shall be regarded as permanent loads.

Loading Effects not due to External Action

Loads deriving from the nature of the structural material, its manufacture or the circumstances of its fabrication are dealt with in the appropriate Parts of this standard. Where they occur they shall be regarded as permanent loads.

Settlement

The effect of differential settlement of supports shall be regarded as a permanent load where there is reason to believe that this will take place, and no special provision has been made to remedy the effect.

Transient Loads

For the purposes of this standard all loads other than permanent ones shall be considered transient.

The maximum effects of certain transient loads do not coexist with the maximum effects of certain others. The reduced effects that can coexist are specified in the relevant clauses.

Combinations of Loads

Three principal and two secondary combinations of loads are specified; values of γ_{fL} for each load for each combination in which it is considered are given in the relevant clauses and also summarized in table 1.

Combination 1

For highway and foot/cycle track bridges, the loads to be considered are the permanent loads, together with the appropriate primary live loads, and, for railway bridges, the permanent loads, together with the appropriate primary and secondary live loads.

Table 1 . Loads to be Taken in Each Combination with Appropriate

ULS: ultimate limit state
SLS: Serviceability limit state

Clause number	Load		Limit state	γ_{fL} to be considered in combination 1	2	3	4	5
5.1	**Dead:** steel		ULS*	1.05	1.05	1.05	1.05	1.05
			SLS	1.00	1.00	1.00	1.00	1.00
5.2		concrete	ULS*	1.15	1.15	1.15	1.15	1.15
			SLS	1.00	1.00	1.00	1.00	1.00
	Superimposed dead		ULS†	1.75	1.75	1.75	1.75	1.75
			SLS†	1.20	1.20	1.20	1.20	1.20
5.1.2.2&5.2.2.2	Reduced load factor for dead and superimposed dead load where this has a more severe total effect		ULS	1.00	1.00	1.00	1.00	1.00
5.3	**Wind:**	during erection	ULS		1.10			
			SLS		1.00			
		with dead plus superimposed dead load only, and for members primarily resisting wind loads	ULS		1.40			
			SLS		1.00			
		with dead plus superimposed dead plus other appropriate combination 2 loads	ULS		1.10			
			SLS		1.00			
		relieving effect of wind	ULS		1.00			
			SLS		1.00			
5.4	**Temperature:**	restraint to movement, except frictional	ULS			1.30		
			SLS			1.00		
		frictional restraint	ULS					1.30
			SLS					1.00
		effect of temperature difference	ULS			1.00		
			SLS			0.80		
5.6	**Differential settlement**		ULS	to be assessed and agreed between the engineer and the appropriate authority				
5.7	**Exceptional loads**		SLS					
5.8	**Earth pressure:**	retained fill and/or live load surcharge	ULS	1.50	1.50	1.50	1.50	1.50
			SLS	1.00	1.00	1.00	1.00	1.00
		relieving effect	ULS	1.00	1.00	1.00	1.00	1.00
5.9	**Erection:** temporary loads		ULS		1.15	1.15		
6.2	**Highway bridges live loading:**	HA alone	ULS	1.50	1.25	1.25		
			SLS	1.20	1.00	1.00		
6.3		HA with HB or HB alone	ULS	1.30	1.10	1.10		
			SLS	1.10	1.00	1.00		
6.5	**Centrifugal load and associated primary live load**		ULS (each secondary live load shall be considered separately together with the other combination 4 loads as appropriate)				1.50	
			SLS				1.00	
6.6	**Longitudinal load:**	HA and associated primary live load	ULS				1.25	
			SLS				1.00	
		HB and associated primary live load	ULS				1.10	
			SLS				1.00	
6.7	**Accidental skidding load and associated primary live load**		ULS				1.25	
			SLS				1.00	
6.8	**Vehicle collision load with bridge parapets and associated primary live load**		ULS				1.25	
			SLS				1.00	
6.9	**Vehicle collision load with bridge supports‡**		ULS				1.25	
			SLS				1.00	
7	**Foot/cycle track bridges:**	live load and parapet load	ULS	1.50	1.25	1.25	1.25	
			SLS	1.00	1.00	1.00	1.00	
8	**Railway bridges:** Type RU and RL primary and secondary live loading		ULS	1.40	1.20	1.20		
			SLS	1.40	1.00	1.00		

γ_{fL} shall be increased to at least 1.10 and 1.20 for steel and concrete respectively to compensate for inaccuracies when dead loads are not accurately assessed.
† γ_{fL} may be reduced to 1.2 and 1.0 for the ULS and SLS respectively subject to approval of the appropriate authority (see **5.2.2.1)**
‡This is the only secondary live load to be considered for foot/cycle track bridges.
NOTE. For loads arising from creep and shrinkage, or from welding and lack of fit, see Parts 3,4 and 5 of this standard, as appropriate.

Combination 2

For all bridges, the loads to be considered are the loads in combination 1, together with those due to wind and where erection is being considered temporary erection loads.

Combination 3

For all bridge, the loads to be considered are the loads in combination 1, together with those arising from restraint due to the effects of temperature range and difference, and where erection is being considered, temporary erection loads.

Combination 4

Combination 4 does not apply to railway bridges except for vehicle collision loading on bridge supports. For highway bridges, the loads to be considered are the permanent loads and the secondary live loads, together with them. Secondary live load shall be considered separately and are not required to be combined. Each shall be taken with its appropriate associated primary live load.

For foot/cycle track bridges, the only secondary live load to be considered is the vehicle collision load with bridge supports (see **6.9**).

Combination 5

For all bridges, the loads to be considered are the permanent loads, together with the load due to friction at **bearings***.

Application of Loads

Each element and structure shall be examined under the effects of loads that can coexist in each combination.

Selection to Cause Most Adverse Effect

Design loads shall be selected and applied in such a way that the most adverse total effect is caused in the element or structure under consideration.

Removal of Superimposed Dead Load

Consideration shall be given to the possibility that the removal [1]of superimposed dead load from part of the structure may diminish its relieving effect. In so doing the adverse effects of live load on the elements of the structure being examined may be modified to the extent that the removal of the superimposed dead load justified this.

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*Where a member is required to resist the loads due to temperature restraint within the structure and to frictional restraint of temperature-induced movement at bearings, the sum of these effects shall be considered. An example is the abutment anchorage of a continuous structure where temperature movement is accommodated by flexure of piers in some spans and by roller bearings in others.

† It is expected that experience in the use of this standard will enable users to identify those load cases and combinations (as in the case of BS 153) which govern design provisions, and it is only those load cases and combinations which need to be established for use in practice.

‡ In course of revision.

Live Load

Live load shall not be considered to act on relieving areas except in the case of wind on live load when the presence of light traffic is necessary to generate the wind loads.

Wind on Relieving Areas

Design loads due to wind on relieving areas shall be modified in accordance with **Section 5.3.**

Overturning

The stability of the structure and its parts against overturning shall be considered for the ultimate limit state.

Restoring Moment

The least restoring moment due to the unfactored nominal loads shall be greater than the greater overturning moment due to the design loads (i.e γ_{fL} for the ultimate limit state x the effects of the nominal loads).

Removal of Loads

The requirements specified in **4.5.2** relating to the possible removal of superimposed dead load shall also be taken into account in considering overturning.

Foundation Pressures, Sliding on Foundations, Loads on Piles, etc.

In the design of foundations, the dead load (see 5.1), the superimposed dead load (see **5.2**) and load due to filling material (see **5.8.1**) shall be regarded as permanent loads and all live loads, temperature effects and wind loads shall be regarded as transient loads, except in certain circumstances such as a main line railway bridge outside a busy terminal where it may be necessary to assess a proportion of live load as being permanent.

The design of foundations shall be based on the principles set out in CP 2004.

Design Loads to be Considered with CP 2004

CP 2004 has not been drafted on the basis of limit state design; it will therefore be appropriate to adopt the nominal loads specified in all relevant clauses of this standard as design loads (taking γ_{fL} = 1.0 and γ_{f3} = 1.0) for the purpose of foundation design in accordance with CP 2004.

5. LOADS APPLICABLE TO ALL BRIDGES

Dead Load

Nominal Dead Load

The nominal dead load initially assumed shall be accurately checked with the actual weights to be used in construction and, where necessary adjustments shall be made to reconcile any discrepancies.

Design Load

The factor, γ_{fL} to be applied to all parts of the dead load, irrespective of whether these parts have an adverse or relieving effect, shall be taken for all five load combinations as follows.

	For the ultimate limit state	**For the serviceability limit state**
Steel	1.05	1.0
Concrete	1.15	1.0

Except as specified in **5.1.2.1** and **5.1.2.2.**

These values of γ_{fL} assume that the nominal dead load has been accurately assessed, that the weld metal and bolts etc.. in steel work and the reinforcement, etc, in concrete have been properly quantified and taken into account and that the densities of materials have been confirmed.

Approximations in Assessment of Load

Any deviation from accurate assessment of nominal dead load for preliminary design or for other purposes should be accompanied by an appropriate and adequate increment in the value of $\gamma_{fL.}$ Values of 1.2 for steel and 1.2 for concrete for the ultimate limit state will usually suffice to allow for the minor approximations normally made. It is not possible to specify the allowances required to be set against various assumptions and approximations, and its is the responsibility of the engineer to ensure that the absolute values specified in **5.1.2** are met in the completed structure.

Alternative Load Factor

Where the structure or element under consideration is such that the application of γ_{fL} as specified in **5.1.2** for the ultimate limit state causes a less severe total effect (see **3.2.6**) than would be the case if γ_{fL}, applied to all parts of the dead load, had been taken as 1.0, value of 1.0 shall be adopted.

Superimposed Dead Load

Nominal Superimposed Dead Load

The nominal superimposed dead load initially assumed shall in all cases be accurately checked with the actual weights to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies.

Where the superimposed dead load comprised filling e.g. on spandrel filled arches, consideration shall be given to the fill becoming saturated.

Design Load

The factor, $\gamma_{fL,}$ to be applied to all parts of the superimposed dead load, irrespective of whether these parts have an adverse or relieving effect, shall be taken for all five load combinations as follows:

For the ultimate limit state	For the serviceability limit state
1.75	1.20

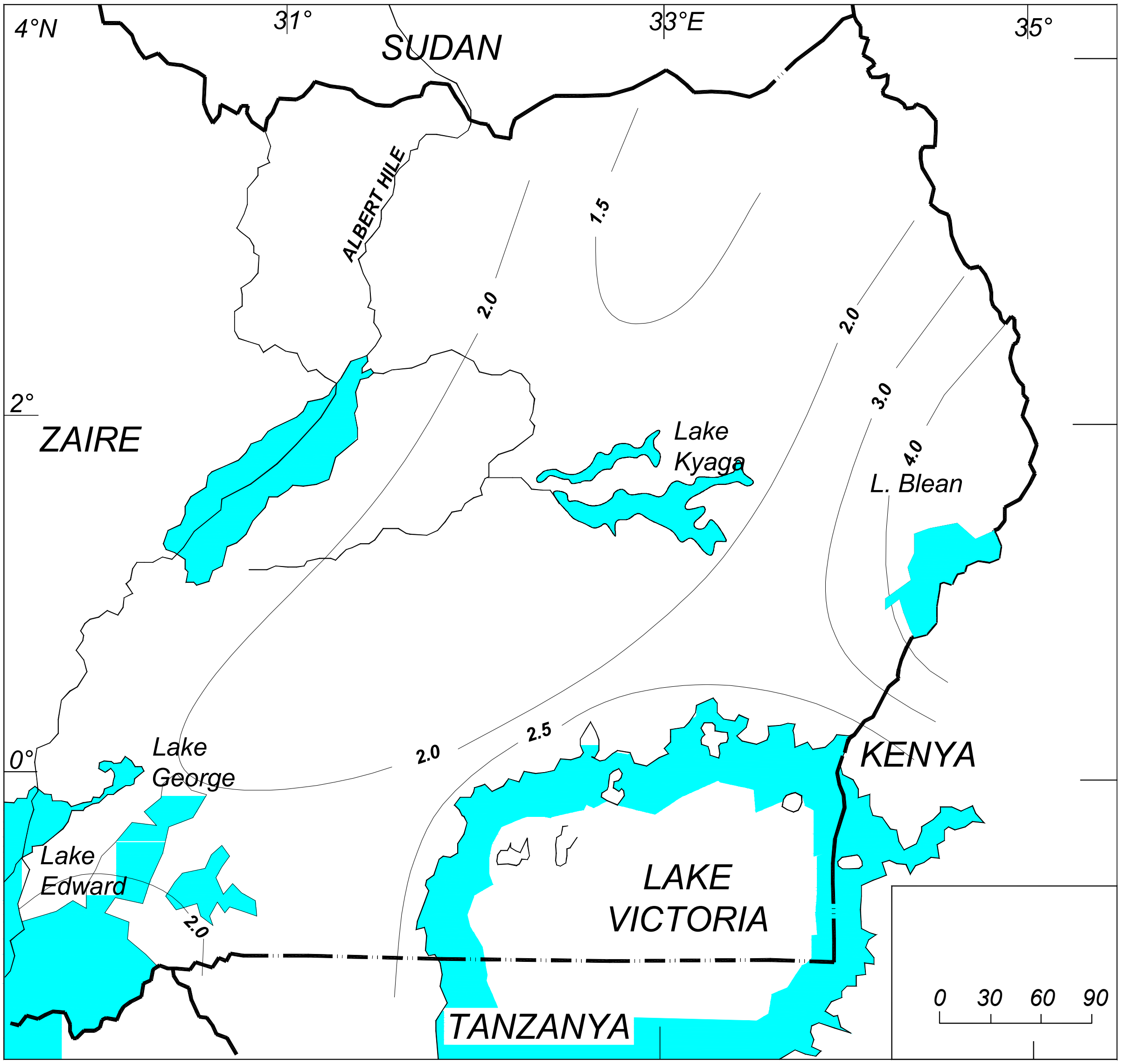
Except as specified in **5.2.1** and **5.2.2** (Note also the requirements of **4.5.2**).

Reduction of Load Factor

The value of γ_{fL} to be used in conjunction with the superimposed dead load may be reduced to an amount not less than 1.2 for the ultimate limit state and 1.0 for the serviceability limit state, subject to the approval of the appropriate authority which shall be responsible for ensuring that the nominal superimposed dead load is not exceeded during the life of the bridge.

Alternative Load Factor

Where the structure or element under consideration is such that the application of γ_{fL} as specified in **5.2.2** for the ultimate limit state causes a less severe total effect (see **3.2.6**) that would be the case if γ_{fL}, applied to all parts of the superimposed dead load, had been taken as 1.0, values of 1.0 shall be adopted.


4°N
31°
33°E
35°
SUDAN
ALBERT HILE
1.5
2.0
2.0
3.0
4.0
2°
ZAIRE
Lake
Kyaga
L. Blean
0°
Lake
George
2.0
2.5
KENYA
Lake
Edward
2.0
LAKE
VICTORIA
0 30 60 90
TANZANYA

Wind Load

Figure 2 Wind Intensity Map of Uganda

The terrain categories shall be classified as:

Category 1 - Exposed open terrains with few or no obstructions (open sea coasts, flats, treeless plains)

Category 2 - Open terrains with scattered obstructions (airfields, open parklands, sparsely built up suburbs)

Category 3 - Terrains with numerous closely-spaced obstructions having the size of domestic houses (well-wooded suburbs, towns and industrial areas fully or partially developed)

Category 4 - Terrains with numerous, large high, closely-spaced obstructions (large city centers)

The variations of velocity pressures with the various categories can be seen in Table 10.

Table 2: Terrain Coefficients for Various Terrain Categories

Height to top of structure (m)	**Velocity pressure kN/m²**	
	Open terrain (Category 1,2)	**Built-up areas (Category 3,4)**
5	0.81	0.37
10	0.92	0.47
15	1.00	0.57
20	1.06	0.66
25	1.10	0.72
30	1.12	0.76

Horizontal Wind Pressure

General

Pressures specified herein shall be assumed to be caused by a base design wind velocity, V_e, of 160 km/h. ***Pressures listed below shall be adjusted for the Ugandan base Design wind velocity shown in Table 2. For instance, if the design base wind speed in Uganda is 100km/h, then all the pressures shall be multiplied by a factor of 100/160=0.625.*** Wind load shall be assumed to be uniformly distributed on the area exposed to the wind. The exposed area shall be the sum of areas of all components, including floor system and railing, as seen in elevation taken perpendicular to the assumed wind direction. This direction shall be varied to determine the extreme force effect in the structure or in its components. Areas that do not contribute to the extreme force effect under consideration may be neglected in the analysis.

For bridges or parts of bridges more than 10 000 mm above low ground or water level, the design wind velocity, V_{DZ}, should be adjusted according to:

$$V_{DZ} = 2.5V_o\left(\frac{V_{10}}{V_B}\right)\ln\left(\frac{Z}{Z_o}\right)$$

Where:

V_{DZ} = design wind velocity at design elevation, z (km/h)

V_{10} = wind velocity at 10000mm above low ground or above design water level (km/h)

V_B = base wind velocity of 160km/h at 10 000mm height.

Z = height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 10 000mm

V_o = friction velocity, a meteorological wind characteristic taken, as specified in Table 1, for various upwind surface characteristic (km/h)

Z_o = friction length of upstream fetch, a meteorological wind characteristic taken as specified in Table 3(mm)

Table 3 - Values of V_o and Z_o for Various Upstream Surface Conditions

Condition	Open Country	Suburban	City
V_0 (km/h)	13.2	17.6	19.3
Z_0 (mm)	700	1000	2500

V_{10} may be established from:

- Site-specific wind surveys, and
- In the absence of better criterion, the assumption that $V_{10}=V_B$ = 160 km/h.

COMMENTARY

C5.3.1.1

Base design wind velocity varies significantly due to local conditions. For small and/or low structures, wind usually does not govern. For large and/or tall bridges, however, the local conditions should be investigated.

Pressures on windward and leeward sides are to be taken simultaneously in the assumed direction of wind.

Typically, a bridge structure should be examined separately under wind pressures from two or more different directions in order to ascertain those windward, leeward, and side pressures producing the most critical loads on the structure.

Equation 1 is based on boundary layer theory combined with empirical observations and represents the most recent approach to defining wind speeds for various conditions as used in meteorology. In the past, an exponential equation was sometimes used to relate wind speed to heights above 10 000 mm. This formulation was based solely on empirical observations and had no theoretical basis.

$$V_{DZ} = CV_{10}\left(\frac{Z}{10000}\right)^{a}$$

The purpose of the term C and exponent "a" was to adjust the equation for various upstream surface conditions, similar to the use of Table 1. Further information can be found in Liu (1991) and Simiu (1973, 1976).

The following descriptions for the terms "open country", "suburban", and "city" in Table 2 are:

- Open Country - Open terrain with scattered obstructions having heights generally less than 10 000 mm. This category includes flat open country and grasslands.
- Suburban - Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family or larger dwellings. Use of this category shall be limited to those areas for which representative terrain prevails in the upwind direction at least 500 000 mm.
- City - Large city centers with at least 50 percent of the buildings having a height in excess of 2100 mm. Use of this category shall be limited to those areas for which representative terrain prevails in the upwind direction at least 800 000 mm. Possible channeling effects of increased velocity pressures due to the bridge or structure's location in the wake of adjacent structures shall be taken into account.

Wind Pressure on Structures: WS

General

If justified by local conditions, a different base design wind velocity may be selected for load combinations not involving wind on live load. The direction of the design wind shall be assumed to be horizontal, unless otherwise specified in Article 10.1.3. In the absence of more precise data, design wind pressure, in MPa, may be determined as

$$P_D = P_B\left(\frac{V_{DZ}}{V_B}\right)^2 = P_B\frac{V_{DZ}{}^2}{25600}$$

P_B = base wind pressure specified in Table 4 (MPa)

Table 4 – Base Pressures, P_B Corresponding to V_B = 160km/h

STRUCTURAL COMPONENT	WINDWARD LOAD, MPa	LEEWARD LOAD, MPa
Trusses, Columns and Arches	0.0024	0.0012
Beams	0.0024	NA
Large Flat Surfaces	0.0019	NA

The wind loading shall not be taken less than 4.4N/mm in the plane of a windward chord and 2.2 N/mm in the plane of a leeward chord on truss and arch components, and not less than 4.4 N/mm on beam or girder components.

Loads from Superstructures

Where the wind is not taken as normal to the structure, the base wind pressures, P_B, for various angles of wind direction may be taken as specified in Table 1 and shall be applied to a single place of exposed area. The skew angle shall be taken as measured from a perpendicular to the longitudinal axis. The wind direction for design shall be that which produces the extreme force effect on the component under investigation. The transverse and longitudinal pressures shall be applied simultaneously.

Table 5 - Base Wind Pressures, P_B, for Various Angles of Attack and V_B = 160 km/hr

	Columns and Arches		Girders	
Skew Angle of Wind	**Lateral Load**	**Longitudinal Load**	**Lateral Load**	**Longitudinal Load**
Degrees	**MPa**	**MPa**	**MPa**	**MPa**
0	0.0036	0	0.0024	0
15	0.0034	0.0006	0.0021	0.0003
30	0.0031	0.0013	0.0020	0.0006
45	0.0023	0.0020	0.0016	0.0008
60	0.0011	0.0024	0.0008	0.0009

Forces Applied Directly to the Substructure

The transverse and longitudinal forces to be applied directly to the substructure shall be calculated from an assumed base wind pressure of 0.0019 MPa. For wind directions taken skewed to the substructure, this force shall be resolved into components perpendicular to the end and front elevations of the substructure. The component perpendicular to the end elevation shall act on the exposed substructure area as seen in end elevation, and the component perpendicular to the front elevation shall act on the exposed areas and shall-be applied simultaneously with the wind loads from the superstructure.

COMMENTARY

C5.3.2.3

The stagnation pressure associated with a wind velocity of 160 km/h is $1.23x10^{-3}$ MPa, which is significantly less than the values specified in Table 4. The difference reflects the effect of gusting combined with some tradition of long-time usage.

The pressures specified in N/mm^2 or MPa (= N/mm^2) should be chosen to produce the greater net wind load on the structure.

Wind tunnel tests may be used to provide more precise estimates of wind pressures. Such testing should be considered where wind is a major design load.

For trusses, columns, and arches, the base wind pressures specified in Table 5 are the sum of the pressures applied to both the windward and leeward areas.

Wind Pressure ON Vehicles: W_L

When vehicles are present, the design wind pressure shall be applied to both structure and vehicles. Wind pressure on vehicles shall be represented by an interruptible, moving force of 1.46 N/mm acting normal and 1800 mm above, the roadway and shall be transmitted to the structure.

When wind on vehicles is not taken as normal to the structure, the components of normal and parallel force applied to the live load may be taken as specified in Table 6 with the skew angle taken as referenced normal to the surface.

Table 6 - Wind Components on Live Load

Skew Angle	Normal Component	Parallel Component
Degrees	N/mm	N/mm
0	1.46	0
15	1.28	0.18
30	1.20	0.35
45	0.96	0.47
60	0.50	0.55

COMMENTARY

C5.3.3.1

Based on practical experience, maximum live loads are not expected to be present on the bridge when the wind velocity exceeds 90 km/h. The load factor corresponding to the treatment of wind on structure only in Load Combination Strength III would be

$(90160)^2$ (1.4) = 0.44, which has been rounded to 0.40 in the Strength IV Load Combination. This load factor corresponds to 0.3 in Service I.

The 1.46 N/mm wind load is based on a long row of randomly sequenced passenger cars, commercial vans, and trucks exposed to the 90 km/h design wind. This horizontal live load, similar to the design lane load, should be applied only to the tributary areas producing a force effect of the same kind.

Vertical Wind Pressure

Unless otherwise determined in article 5.3.5, a vertical upward wind force of 9.6×10^{-4} MPa times the width of the deck, including parapets and sidewalks, shall be considered to be a longitudinal line load. This force shall be applied only for limit states that do not involve wind on live load, and only when the direction of wind is taken to be perpendicular to the longitudinal axis of the bridge. This lineal force shall be applied at the windward quarter-point of the deck width in conjunction with the horizontal wind loads specified in Article 5.3.1.

COMMENTARY

C5.3.4.

The intent of this article is to account for the effect resulting from interruption of the horizontal flow of air by the superstructure. This load is to be applied even to discontinuous bridge decks, such as grid decks. This load may govern where overturning of the bridge is investigated.

Aeroelastic Instability

General

Aeroelastic force effects shall be taken into account in the design of bridges and structural components apt to be wind-sensitive. For the purpose of this article, all bridges, and structural components thereof with a span length to width or depth ratio exceeding 30.0 shall be deemed to be wind-sensitive.

The vibration of cables due to the interaction of wind and rain shall also be considered.

COMMENTARY

C5.3.5.1

Because of the complexity of analyses often necessary for an in-depth evaluation of structural aeroelasticity, this article is intentionally kept to a simple statement. Many bridges, decks, or individual structural components have been shown to be aeroelastically insensitive if their length-to-width or length-to-depth ratios are under about 30.0, a somewhat arbitrary value helpful only in identifying likely wind-sensitive cases.

Flexible bridges, such as cable-supported or very long spans of any type, may require special studies based on wind tunnel information.

In general, appropriate wind tunnel tests involve simulation of the wind environment local to the bridge site. Details of this are part of the existing wind tunnel state of the art and are beyond the scope of this commentary.

Aeroelastic Phenomena

The aeroelastic phenomena of vortex excitation, galloping, flutter, and divergence shall be considered where applicable.

COMMENTARY

C5.3.6.

Excitation due to vortex shedding is the escape of wind-induced vortices behind the member, which tend to excite the component at its fundamental natural frequency in harmonic motion. It is important to keep stresses due to vortex-induced oscillations below the "infinite life" fatigue stress. Methods exist for estimating such stress amplitudes, but they are outside the scope of this commentary.

Tubular components can be protected against vortex-induced oscillation by adding bracing, strakes, or tuned mass dampers or by attaching horizontal flat plates parallel to the tube axis above and/or below the central third of their span. Such aerodynamic damper plates should lie about one-third tube diameter above or below the tube to allow free passage of wind. The width of the plates may be the diameter of the tube or wider.

Galloping is a high-amplitude oscillation associated with ice-laden cables or long, flexible members having aerodynamically unsymmetrical cross-sections. Cable stays, having circular sections, will not gallop unless their circumferences are deformed by ice, dropping water, or accumulated debris.

Flexible bridge decks, as in very long spans and some pedestrian bridges, may be prone to wind-induced flutter, a wind-excited oscillation of destructive amplitudes, or, on some occasions, divergence, an irreversible twist under high wind. Analysis methods, including wind tunnel studies leading to adjustments of the deck form, are available for prevention of both flutter and divergence.

Control of Dynamic Responses

Bridges and structural components thereof, including cables, shall be designed to be free of fatigue damage due to vortex-induced or galloping oscillations. Bridges shall be designed to be free of divergence and catastrophic flutter up to 1.2 times the design wind velocity applicable at bridge deck height.

COMMENTARY

C5.3.7

Cables in stayed-girder bridges have been successfully stabilized against excessive dynamic responses by attaching automotive dampers to the bridge at deck level or by cross-tying multiple cable -stays.

Wind Tunnel Tests

Representative wind tunnel tests may be used to satisfy the requirements of Articles 5.3.6 and 5.3.7.

COMMENTARY

C5.3.8.

Wind tunnel testing of bridges and other civil engineering structures is a highly developed technology, 'which may be used to study the wind response characteristics of a structural model or to verify the results of analysis (Simiu 1976).

Temperature

General

Daily and seasonal fluctuations in shade air temperature, solar radiation, re-radiation, etc… cause the following:

(a) changes in the overall temperature of the bridge, referred to as the effective bridge temperature. Over a prescribed period, there will be a minimum and a maximum, together with a range of effective bridge temperatures, resulting in loads and/or or load effects within the bridge due to:

(1) restraint of associated expansion or contraction by the form of construction (e.g. portal frame, arch, flexible pier, elastomeric bearings) referred to as temperature restraint, and
(2) friction at roller or sliding bearings where the form of the structure permits associated expansion and contraction, referred to as frictional bearing restraint;

(b) differences in temperature between the top surface and other levels through the depth of the superstructure, referred to as temperature difference and resulting in associated loads and /or load effects within the structure.

Effective bridge temperatures are derived from the isotherms of shade air temperatures shown in figure 7 and 8. These shade air temperatures are appropriate to mean sea level in open country and a 120-year return period.

Minimum and Maximum Shade Air Temperatures

For all bridges, extremes of shade air temperature for the location of the bridge shall be obtained from available maps of isotherms.

For foot/cycle track bridges, subject to the agreement of the appropriate authority, a return period of 50 years may be adopted, and the shade air temperatures may be reduced as specified in **5.4.2.1**.

During erection, a 50-year return period may be adopted for all bridges and the shade air temperatures may be reduced as specified in **5.4.2.1.** Alternatively, where a particularly erection will be completed within a period of one or tow days for which reliable shade air temperature and temperature range predictions can be made, these may be adopted.

Adjustment for a 50-year Return Period

The minimum shade air temperature, shall be adjusted by the addition of 2^0c.

The maximum shade air temperature, shall be adjusted by the subtraction of 2^0c.

Adjustment for Height above Mean Sea Level

The values of shade air temperature shall be adjusted for height above sea level by subtracting 0.5^0C per 100m height for minimum shade air temperatures and 1.0^0c per 100m height for maximum shade air temperatures.

Minimum and Maximum Effective Bridge Temperatures

The minimum and maximum effective bridge temperatures for different types of construction shall be derived from the minimum and maximum shade air temperatures by reference to table 10 and 11 respectively. The different types of construction are as shown in figure 9.

Table 10. Minimum effective bridge temperature

Minimum shade air temp.	Minimum effective bridge temp.		
	Type of superstructures		
	Group 1 & 2	Group 3	Group 4
^{0}C	^{0}C	^{0}C	^{0}C
-24	-28	-19	-14
-23	-27	-18	-13
-22	-26	-18	-13
-21	-25	-17	-12
-20	-23	-17	-11
-19	-22	-16	-11
-18	-21	-15	-10
-17	-20	-15	-10
-16	-19	-14	-9
-15	-18	-13	-9
-14	-17	-12	-8
-13	-16	-11	-8
-12	-15	-10	-7
-11	-14	-10	-6
-10	-12	-9	-6
-9	-11	-8	-5
-8	-10	-7	-4
-7	-9	-6	-3
-6	-8	-5	-3
-5	-7	-4	-2

Table 11. Maximum effective bridge temperature

Maximum shade air temp.	Maximum effective bridge temp.		
	Type of superstructure		
	Group 1 & 2	Group 3	Group 4
^{0}C	^{0}C	^{0}C	^{0}C
24	40	31	27
25	41	32	28
26	41	33	29
27	42	34	29
28	42	34	30
29	43	35	31
30	44	36	32
31	44	36	32
32	44	37	33
33	45	37	33
34	45	38	34
35	46	39	35
36	46	39	36
37	46	40	36
38	47	40	37

NOTE. See figure 9 for different types of superstructure.

Adjustment for Thickness of Surfacing

The effective bridge temperatures are dependent on the depth of surfacing on the bridge deck and the values given in table 10 and 11 assume depths of 40mm for groups 1 and 2 and 100mm for groups 3 and 4. Where the depth of surfacing differs from these values, the minimum and maximum effective bridge temperatures may be adjusted by the amounts given in table 12.

Table 12. Adjustment to effective bridge temperature for deck surfacing

Deck Surface	Addition to minimum effective bridge temp.			Addition to maximum effective bridge temp.		
	Groups 1 & 2	Group 3	Group 4	Group 1 & 2	Group 3	Group 3
	^{0}C	^{0}C	^{0}C	^{0}C	^{0}C	^{0}C
Unsurfaced	0	-3	-1	+4	0	0
waterproofed 40mm	0	-3	-1	+2	+4	+2
surfacing 100mm	0	-2	-1	0	+2	+1
surfacing* 200mm	-	0	0	-	0	0
surfacing*	-	+3	+1	-	-4	-2

*Surfacing depths include waterproofing.

Range of Effective Bridge Temperature

In determining load effects due to temperature restraint, the effective bridge temperature at the time the structure is effectively restrained shall be taken as datum in calculating expansion up to the maximum effective bridge temperature and contraction down to the minimum effective bridge temperature.

Temperature Difference

Effects of temperature differences within the superstructure shall be derived from the data given in figure 9.

Positive temperature differences occur when conditions are such that solar radiation and other effects cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences occur when conditions are such that heat is lost from the top surface of the bridge deck as a result of reradiation and other effects.

Adjustment for Thickness of Surfacing

Temperature differences are sensitive to the thickness of surfacing, and the date given in figure 9 assume depths of 40mm for groups 1 and 2 and 100mm for groups 3 and 4. For other depths of surfacing different values will apply. Values for other thicknesses of surfacing are given in appendix E.

Combination with Effective Bridge Temperatures

Maximum positive temperature differences shall be considered to coexist with effective bridge temperatures at above 25^{0}C (groups 1 and 2) and 15^{0}C (groups 3 and 4). Maximum reversed temperature differences shall be considered to coexist with effective bridge temperatures up to 8^{0}c below the maximum for group 1 and 2, up to 4^{0}C below the maximum for group 3, and up to 2^{0}C below the maximum for group 4.

Coefficient of Thermal Expansion

For the purpose of calculating temperature effects, the coefficient of thermal expansion for structural steel and for concrete may be taken as 12 x $10^{-6}/^0$C, except when limestone aggregates are used in concrete, when a value of 7 x $10^{-6}/^0$C shall be adopted for the concrete.

Nominal Values

Nominal Range of Movement

The effective bridge temperature at the time the structure is attached to those parts permitting movement shall be taken as datum and the nominal range of movement shall be calculated for expansion up to maximum effective bridge temperature and for contraction down to the minimum effective bridge temperature.

Nominal Load for Temperature Restraint

The load due to temperature restraint of expansion or contraction for the appropriate effective bridge temperature range (see **5.4.4**) shall be taken as the nominal load.

Where temperature restraint is accompanied by elastic deformations in flexible piers and elastomeric bearings, the nominal load shall be derived as specified in **5.4.7.2.1** to **5.4.7.2.2.**

5.4.7.2.1. Flexure of Piers

For flexible piers pinned at one end and fixed at the other, or fixed at both ends, the load required to displace the pier by the amount of expansion or contraction for the appropriate effective bridge temperature range (see **5.4.4**) shall be taken as the nominal load.

5.4.7.2.2. Elastomeric Bearings

For temperature restraint accommodated by shear in an elastometric, the load required to displace the elastometric by the amount of expansion or contraction for the appropriate effective bridge temperature range (see **5.4.4**) shall be taken as the nominal load.

The nominal load shall be determined in accordance with **5.14.2.6** of BS 5400: Section 9.1: 1983.

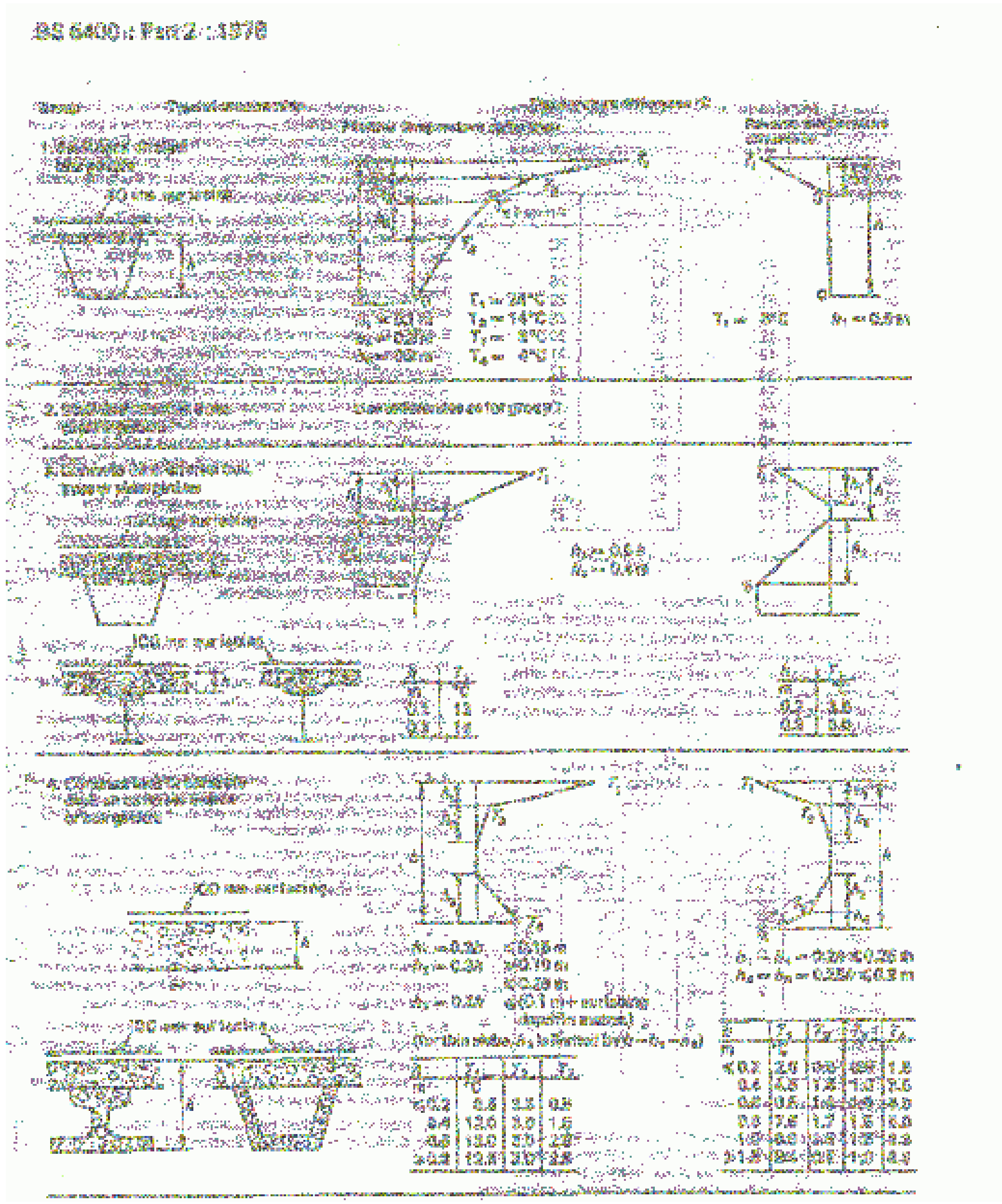


Figure 9 – Temperature difference for different types of construction

Nominal Load for Frictional Bearing Restraint

The nominal load due to the frictional bearing restraint shall be derived from the nominal dead load (see **5.1.1**), the nominal superimposed dead load (see **5.2.1**) and the snow load (see **5.7.1**), using the appropriate coefficient of friction given in table 2 and 3 of BS 5400: Section 9.1:1983.

Nominal Effects of Temperatures Difference

The effects of temperature difference shall be regarded as nominal values.

Design Values

Design Range of Movement

The design range of movement shall be taken as 1.3 times the appropriate nominal value for the ultimate limit state and 1.0 times the nominal value for the serviceability limit state.

For the purpose of this clause the ultimate limit state shall be regarded as a condition where expansion or contraction beyond the serviceability range up to the ultimate range would cause collapse or substantial damage to main structural members. Where expansion or contraction beyond the serviceability range will not have such consequences, only the serviceability range need be provided for.

Design Load for Temperature Restraint

For combination 3, γ_{fL} shall be taken as follows.

For the ultimate limit state	For the serviceability limit state
1.30	1.00

Design Load for Frictional Bearing Restraint

For combination 5, γ_{fL} shall be taken as follows.

For the ultimate limit state	For the serviceability limit state
1.30	1.00

5.4.8.3.1. Associated Vertical Design Load

The design dead load (see **5.1.2**) and design superimposed dead load (see **5.2.2**) shall be considered in conjunction with the design load due to frictional bearing restraint.

Design Effects of Temperature Difference

For combination 3, γ_{fL} shall be taken as follows.

For the ultimate limit state	For the serviceability limit state
1.00	0.80

Effects of Shrinkage and Creep, Residual Stresses etc.

Where it is necessary to take into account the effects of shrinkage or creep in concrete, stresses in steel due to rolling, welding or lack of fit, variations in the accuracy of bearing levels and similar sources of strain arising from the nature of the materials or its manufacture or from circumstances associated with fabrication and erection, requirements are specified in the appropriate parts of this standard.

Differential Settlement

Where differential settlement is likely to affect the structure in whole or in part, the effects of this shall be taken into account.

Assessment of Differential Settlement

In assessing the amount of differential movement to be provided for, the engineering shall take into account the extent to which its effect will be observed and remedied before damage ensues.

Load Factors

The value of γ_{fL} shall be chosen in accordance with the degree of reliability of assessment, taking account of the general basis of probability of occurrence set out in Part 1 of this standard and the provisions for ensuring remedial action.

Exceptional Loads

Where other loads not specified in this standard are likely to be continued, e.g. the effects of earthquakes, stream flows or ice packs, these shall be taken into account. The nominal loading to be adopted shall have a value in accordance with general basis of probability of occurrence set out in Part 1 of this standard.

Snow Load

Snow loading should be considered in accordance with local conditions; for those prevailing in Great Britain, this loading may generally be ignored in combinations 1 to 4 (see **4.4.1** to **4.4.4**) but there are circumstances, e.g. opening bridges or where dead load stability is critical, when consideration should be given to it, any snow load shall be included in combination 5. (see **4.4.5**) as superimposed dead load.

Design Loads

For exceptional design loads, γ_{fL} shall be assessed in accordance with the general basis of probability of occurrence set out in Part 1 of this standard.

Earth Pressure on Retaining Structures

Filling Material

Nominal Load

Where filling material is retained by abutments or other parts of the structure, the loads calculated by soil mechanics principles from the properties of the filling material shall be regarded as nominal loads.

The nominal loads initially assumed shall be accurately checked with the properties of the materials to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies.

Consideration shall be given to the possibility that the filling material may become saturated or may be removed in whole or in part from either side of the fill-retaining part of the structure.

Design Load

For all five deign load combinations, γ_{fL} shall be taken as follows.

For the ultimate limit state	For the serviceability limit state
1.5	1.00

Except as defined in **5.8.1.3.**

Alternative Load Factor

Where the structure or element under consideration is such that the application of γ_{fL} as given in **5.8.1.2** for the ultimate limit state causes a less severe total effect (see **3.2.6**) than would be the case if γ_{fL}, applied to all parts of the filling materials, has been taken as 1.0, values of 1.0 shall be adopted.

Live Load Surcharge

The effects of live load surcharge shall be taken into consideration.

Nominal Load

In the absence of more exact calculations the nominal load due to live load surcharge for suitable material properly consolidated may be assumed to be:

(a) for HA loading : 10kN/m^2;
(b) for HB loading :
45 units : 20kN/m^2 (intermediate values);
25 units : 10kN/m^2 by interpolation).
(c) for RU loading : 50kN/m^2 on areas occupied by tracks;
(d) for RL loading : 30kN/m^2 on areas occupied by tracks.

Design Load

For combinations 1 to 4, γ_{fL} shall be as specified in **5.8.1.2.**

Erection Loads

For the ultimate limit state, erection loads shall be considered in accordance with **5.9.1** to **5.9.5.**

For the serviceability limit state, nothing shall be done during erection that will cause damage to the permanent structure or will alter its response in service from that considered in design.

Temporary Loads

Nominal Loads

The total weight of all temporary materials, plant and equipment to be used during erection shall be taken into account. This shall be accurately assessed to ensure that the loading is not underestimated.

Design Loads

For the ultimate limit state for combinations 2 and 3, γ_{fL} shall be taken as 1.15, except as specified in **5.9.1.3.**

Relieving Effect

Where any temporary materials have a relieving effect, and have not been introduced specifically for this purpose, they shall be considered not to be acting. Where, however, they have been so introduced, precautions shall be taken to ensure that they are not inadvertently removed during the period for which they are required. The weight of these materials shall also be accurately assessed to ensure that the loading is not over estimated. This value shall be taken as the design load.

Permanent Loads

Nominal Loads

All dead and superimposed dead loads affecting the structure at each stage of erection shall be taken into account.

The effects of the method of erection of permanent materials shall be considered and due allowance shall be made for impact loading or shock loading.

Design Loads

The design loads due to permanent loads for the ultimate limit state for combinations 2 and 3 shall be as specified in **5.1.2** and **5.2.2** respectively.

Disposition of Permanent and Temporary Loads

The disposition of all permanent and temporary loads at all stages of erection shall be taken into consideration and due allowances shall be made for possible inaccuracies in their location. Precautions shall be taken to ensure that the assumed disposition is maintained during erection.

Wind and Temperature Effects

Wind and temperature effects shall be considered in accordance with **5.3** and **5.4** respectively.

Snow and Ice Loads

When climatic conditions are such that there is a possibility of snowfalls or of icing, an appropriate allowance shall be made. Generally, a distributed load of 500N/m^2 may be taken as adequate but may require to be increased for regions where there is a possibility of snowfalls and extremes of low temperature over a long period. The effects of wind in combination with snow loading may be ignored.

6. HIGHWAY BRIDGE LIVE LOADS

General

Standard highway loading consists of HA and HB loading.

HA loading a formula loading representing normal traffic in Great Britain. HB loading is an abnormal vehicle unit loading. Both loading is an abnormal vehicle unit loading. Both loadings include impact. (See appendix A for the basis of HA and HB loading).

Loads to be Considered

The structure and its elements shall be designed to resist the more severe effects of either:

design HA loading (see **6.4.1**) or
design HA loading combined with design HB loading (see **6.4.2**).

Notional Lanes, Hard Shoulders, etc.

The width and number of notional lanes, and the presence of hard shoulders, hard strips, verges and central reserves are integral to the disposition of HA and HB loading. Requirements for deriving the width and number of notional lanes for design purpose are specified in **3.2.9.3.**

Distribution Analysis of Structure

The effects of the design standard loading shall, where appropriate be distributed in accordance with a rigorous distribution analysis or from data derived from suitable tests.

Type HA Loading

Type HA loading consists of a uniformly distributed load (see **6.2.1**) and knife edge load (see **6.2.2**) combined, or of a single wheel load (see **6.2.5**).

Nominal Uniformly Distributed Load (UDL)

The UDL shall be taken as 30kN per linear meter of notional lane for loaded lengths up to 30m, and for loaded lengths in excess of 30m it shall be derived from the equation.

$W = 151\left(\frac{1}{L}\right)^{0.475}$ but not less than 9

Where L is the loaded length (in m) and W is the load per meter of lane (in kN).

Values of this load per linear meter of notional lane are given in table 13 and the loading curve is illustrated in figure 10.

Table 13. Type HA uniformly distributed load

Loaded length	Load	Loaded length	Load	Loaded length	Load
M	KN/m	m	KN/m	m	KN/m
Up to 30	30.0	73	19.7	160	13.6
32	29.1	76	19.3	170	13.2
34	28.3	79	18.9	180	12.8
36	27.5	82	18.6	190	12.5
38	26.8	85	18.3	200	12.2
40	26.2	90	17.8	210	11.9
42	25.6	95	17.4	220	11.7
44	25.0	100	16.9	230	11.4
46	24.5	105	16.6	240	11.2
49	23.8	110	16.2	255	10.9
52	23.1	115	15.9	270	10.6
55	22.5	120	15.5	285	10.3
58	21.9	125	15.2	300	10.1
61	21.4	130	15.0	320	9.8
64	20.9	135	14.7	340	9.5
67	20.5	140	14.4	360	9.2
70	20.1	145	14.2	380 and	9.0
		150	14.0	above	

NOTE. The loaded length for the member under consideration shall be the base length of the adverse area (see **3.2.5**). Where there is more than one adverse area, as for continuous construction, the maximum effect should be determined by consideration of any adverse area or combination of adverse areas using the loading appropriate to the base length or the total combined base lengths.

Nominal Knife Edge Load (KEL)

The KEL per notional lane shall be taken as 120kN.

Distribution

The UDL and KEL shall be taken to occupy one notional lane, uniformly distributed over the full width of the lane and applied as specified in **6.4.1.**

Dispersal

No allowance for the dispersal of the UDL and KEL shall be made.

Single Nominal Wheel Load Alternative to UDL and KEL

One 100kN wheel, placed on the carriageway and uniformly distributed over a circular contact area assuming an effective pressure of 1.1N/mm^2 (i.e 340mm diameter) shall be considered.

Alternatively, a square contact area may be assumed, using the same effective pressure (i.e. 300mm side).

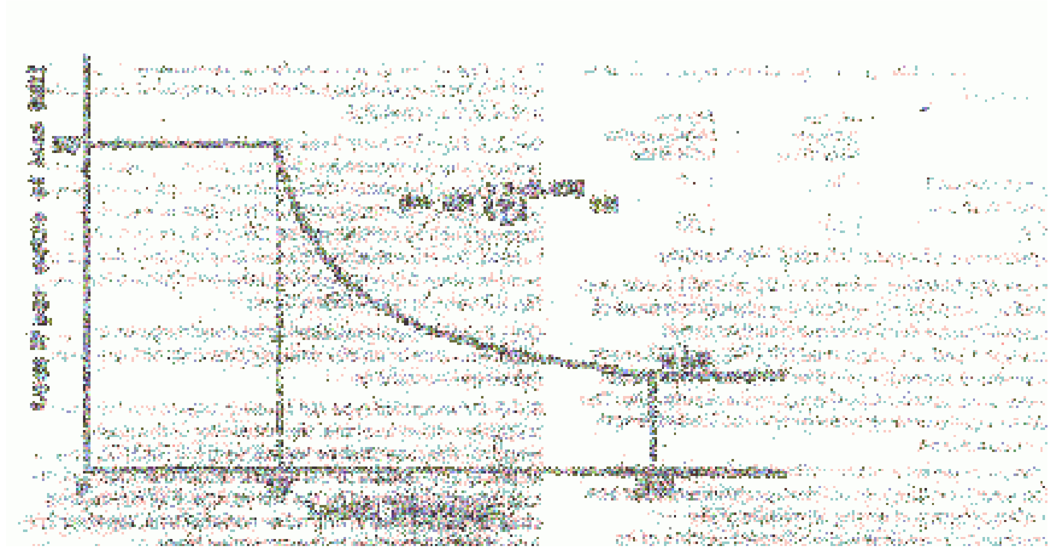


Figure 10. Loading curve for HA UDL

Dispersal

Dispersal of the single nominal wheel load at a spread-to-depth ratio of 1 horizontally to 2 vertically through asphalt and similar surfacing may be assumed, where it is considered that this may take place.

Dispersal through structural concrete slabs may be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the neutral axis.

Design HA Loading

For design HA load considered alone, γ_{fL} shall betaken as follows.

	For the ultimate limit state	**For the serviceability limit state**
For combination 1	1.50	1.20
For combinations 2 and 3	1.25	1.00

Where HA loading is coexistent with HB loading
(see *6.4.2)* γ_{fL}, as specified in *6.3.4,* shall be applied to HA loading.

Type HB Loading

For all public highway *bridges* in Great Britain, the minimum number of units of type H B loading that shall normally be considered is 25, but this number may be increased up to 45 if so directed by the appropriate authority.

Nominal HB Loading

Figure 11 shows the plan and axle arrangement for one unit of nominal HB loading. One unit shall be taken as equal to 10 kN per axle (i.e. 2.5 kN per wheel).

The overall length of the H B vehicle shall be taken as 10, 15, 20, 25 or 30 m for inner axle spacings of 6,11,16, 21 or 26 m respectively, and the effects of the most severe of these cases shall be adopted.

The overall width shall be taken as 3.5 m.

Contact Area

Nominal HB wheel loads shall be assumed to be uniformly distributed over a circular contact area, assuming an effective pressure of 1.1 N/mm^2.

Alternatively, a square contact area may be assumed, using the same effective pressure.

Dispersal

Dispersal of HB wheel loads at a spread-to-depth ratio of 1 horizontally to 2 vertically through asphalt and similar surfacing may be assumed, where it is considered that this may take place.

Dispersal through structural concrete slabs may be taken at a spread-to-depth ratio of 1 horizontally to 1 vertically down to the neutral axis.

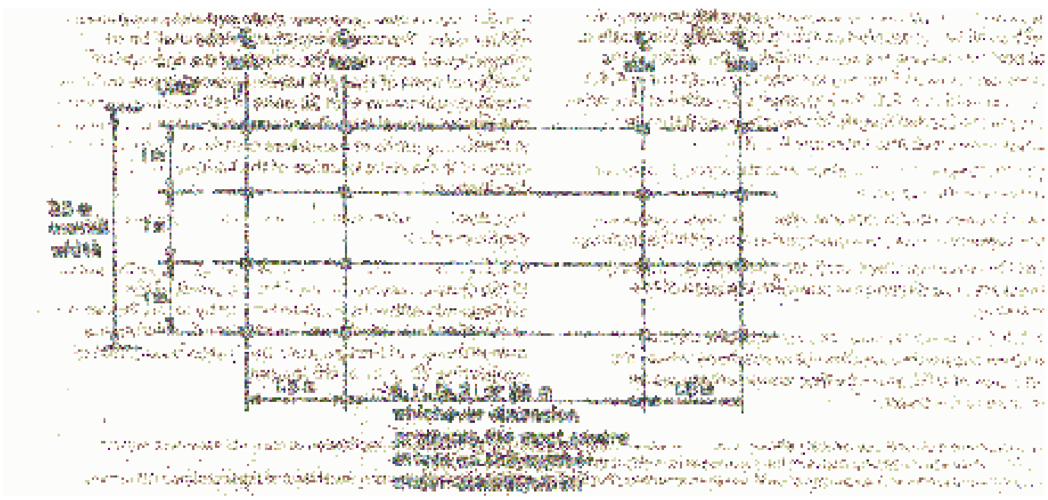


Figure 11. Dimensions of HB vehicles

Design HB Loading

For design H B load, γ_{fL} shall be taken as follows.

	For the ultimate limit state	For the serviceability limit state
For combination 1	1.30	1.10
For combinations 2and3	1.10	1.00

Application of Types HA and HB Loading

Type HA Loading

Type HA UDL and KEL loads shall be applied to two notional lanes in the appropriate parts of the influence line for the element or member under consideration* and one-third type HA UDL and KEL loads shall be similarly applied to all other notional lanes except where otherwise specified by the appropriate authority. The KEL shall be applied at one point only in the loaded length of each notional lane.

Where the most severe effect is caused by locating portions of loaded length on one side of the superstructure over one portion of its length and on the other side of the superstructure in a longitudinally adjacent portion of its length, this shall be taken into consideration, using the loading appropriate to the combined length of the loaded portions.

2

Multilevel Structures

Where multilevel superstructures are carried on common substructure members (as, e.g., columns of a multilevel interchange) each level shall be loaded and the loaded length shall be taken as the aggregate of the loaded lengths in each superstructure that has the most severe effect on the member under consideration.

Transverse Cantilever Slabs, Central Reserves and Outer Verges

HA UDL and KEL shall be replaced by the arrangement of HB loading given in *6.4.3.*

Combined Effects

Where elements of a structure can sustain the effects of live load in two ways, i.e. as elements in themselves and also as parts of the main structure (e.g. the top flange of a box girder functioning as a deck plate), the element shall be proportioned to resist the combined effects of the appropriate loading specified in *6.4.2.*

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* In consideration of local (not global) effects, where deviations from planarity may be critical, the application of the knife edge load without the UDL immediately adjacent to it may have a more severe effect than with the UDL present.

† In considering plates and slabs care should be taken to ensure that the free edge is adequate to resist the effects of the associated HB vehicle specified.

‡ This is the only exception to the rule that not more than one HB vehicle shall be considered to act on a structure. The 25 unit vehicle is to be regarded as a substitute for HA loading for these elements only.

Knife Edge Load (KEL)

The KEL shall be taken as acting as follows.

(a) On plates, right slabs and skew slabs spanning or cantilevering longitudinally or spanning transversely: in a direction parallel to the supporting members or at right angles to the unsupported edges, whichever has the most severe effect. Where the element spans transversely, the KEL shall be considered as acting in a single line made up of portions having the same length as the width of the nominal lanes and having the intensities set out in *6.4.1.* As specified in *6.4.1,* the KEL shall be applied at one point only in the loaded length. Where plates or slabs are supported on all four sides see *6.4.3.1.*
(b) On longitudinal members and stringers: in a direction parallel to the supports.
(c) On piers, abutments and other members supporting the superstructure: in a direction in line with the bearings.
(d) On cross members, including transverse cantilever brackets: in a direction in line with the span of the member.

Single Wheel Load

The HA wheel load is applied to members supporting small areas of roadway where the proportion of UDL and KEL that would otherwise be allocated to it is small.

Types HB and HA Loading Combined

Types HB and HA loading shall be combined and applied as specified in *6.4.2.1* and *6.4.2.2.*

Type HB Load

Type HB loading shall be taken to occupy any transverse position on the carriageway, and in so doing will lie either wholly within one notional lane or will straddle two notional lanes: No other primary live loading shall be considered for *25* m in front of, to *25* m behind, the HB vehicle in the one lane occupied by the HB vehicle when it is wholly in one lane or in the two lanes when the HB vehicle is straddling them.

Only one HB load is required to be considered on any one superstructure or on any substructure supporting two or more superstructures.

Associated Type HA Loading

Where the HB vehicle is wholly within one lane, the remainder of the loaded length of this lane shall be loaded with full HA UDL only, of intensity appropriate to the loaded length that includes the total length displaced by the H B vehicle. Full HA loading shall be considered in one other notional lane, together with one-third HA loading in the remaining lanes.

Where the HB vehicle straddles two lanes, the following alternatives for associated HA highway loading shall be considered:

Either

(a) the remainder of the loaded length of both straddled lanes shall be loaded with full HA U DL only, of intensity appropriate to the loaded length that includes the total length displaced by the H B vehicle; all other lanes shall be loaded with one-third HA loading;
or

(b) the remainder of the loaded length of one straddled lane shall be loaded with full HA U DL only and the remainder of the loaded length of the other straddled lane shall be loaded with one-third HA UDL only. The intensity of the full HA UDL and one-third HA UDL shall be that appropriate to a loaded length that includes the total length displaced by the HB vehicle. Full HA loading shall be considered in one other notional lane, together with one-third HA loading in the remaining lanes.

Figure *12* illustrates type HB loading in combination with type HA loading.

Highway Loading on Transverse Cantilever Slabs, Slabs Supported on All Four Sides, Central Reserves and Outer Verges

Type HB loading shall be applied to the elements specified in *6.4.3.1* to *6.4.3.3.*

Transverse Cantilever Slabs and Slabs Supported on all Four Sides

Transverse cantilever slabs shall be so proportioned as to resist the effects of the appropriate number of units of type HB loading placed in one notional lane in combination with *25* units of HB loading placed in one other notional lane. Proper consideration shall be given to transverse joints of transverse cantilever slabs and to the edges of these slabs because of the limitations of distribution.

This does not apply to members supporting transverse cantilever slabs.

Central Reserves

On dual carriageways the portion of the central reserve isolated from the rest of the carriageway either by a raised kerb or by safety fences is not required to be loaded with live load in considering the overall design of the structure, but it shall be capable of supporting *25* units of HB loading.

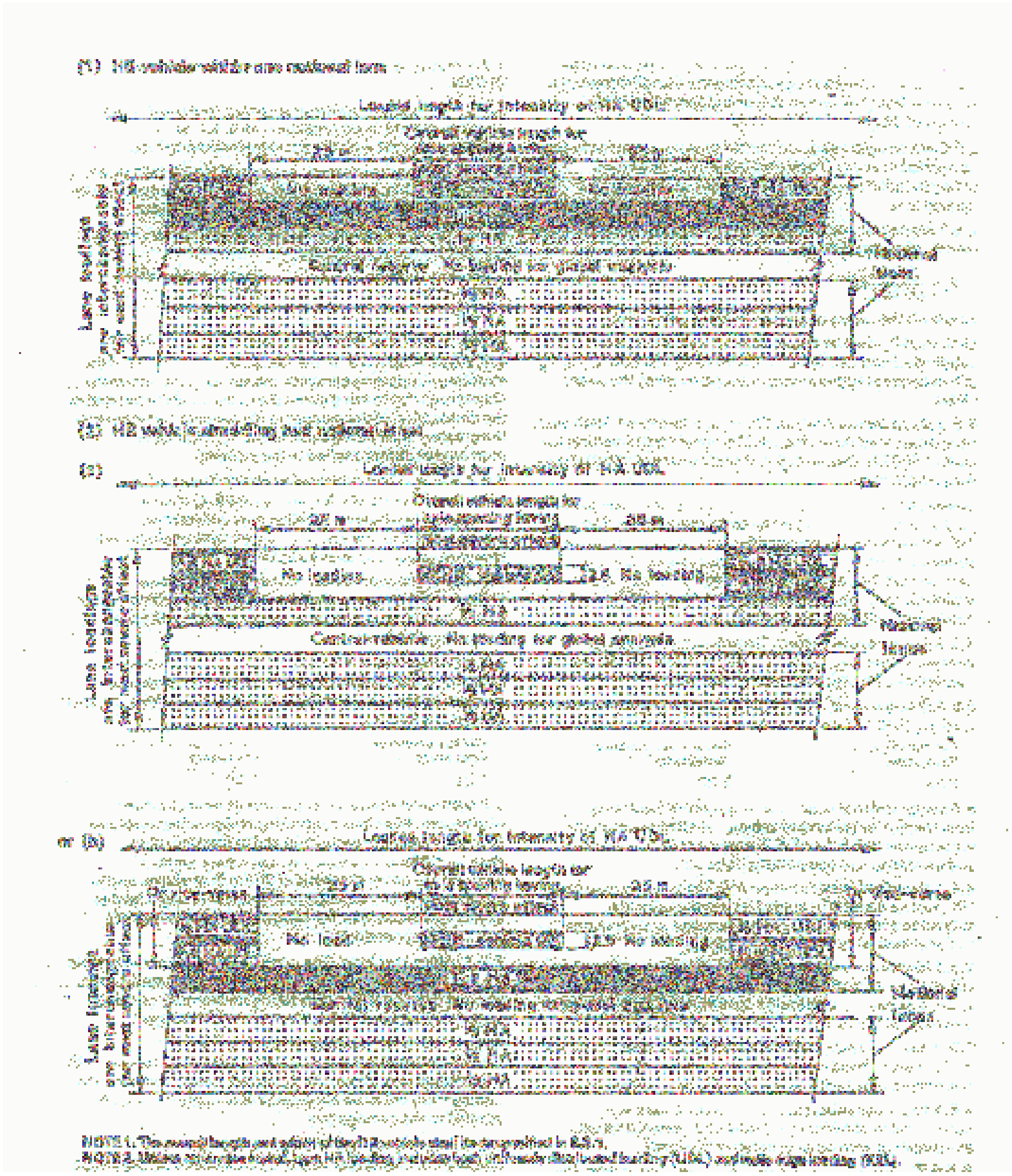

Figure 12. Type HA of HB highway loading in combination

Outer Verges

Outer verges are not required to be loaded with live load in considering the overall design of the structure, but shall be capable of supporting any four wheels of 25 units of HB loading.

Centrifugal Load

On curved bridges, point loads shall be applied in any two notional lanes at 50 m centres, acting in radial direction at the surface of the road and parallel to it.

Nominal Centrifugal Load

The nominal centrifugal load F_c shall be taken as

$$F_c = \frac{30000}{r + 150} kN$$

where r is the radius of curvature of the lane (in m).

Each load F_c, shall be either taken as a single load or subdivided into two parts of one-third F_c, and two-thirds F_c at 5m centres longitudinally, whichever gives the lesser effect.

Associated Nominal Primary Live Load

With each centrifugal load there shall also be considered a vertical live load of *300* kN, distributed uniformly over the notional lane for a length of 5 m.

Where the centrifugal load is subdivided, the vertical live load shall be subdivided in the same proportions. The 100 kN portion shall be considered to act as a point load coincident with the one-third F_c. load and the 200 kN portion shall be considered as a distributed load applied uniformly over the notional lane for a length of 1 m and coincident with the two-thirds F_c load.

Load Combination

Centrifugal load shall be considered in combination 4 only and need not be taken as coexistent with other secondary live loads.

Design Load

For the centrifugal and primary live load, γ_{fL} shall be taken as follows:

For the ultimate limit state	For the serviceability limit state
1.50	1.00

Longitudinal Load

The longitudinal load resulting from traction or braking of vehicles shall be taken as the more severe of *6.6.1* or *6.6.2,* applied at the road surface and parallel to it in one notional lane only.

Nominal Load for Type HA

The nominal load for HA shall be 8 kN/m of loaded length plus 200 kN, subject to a maximum of 700 kN, applied to an area one notional lane wide x the loaded length.

Nominal Load for Type HB

The nominal load for HB shall be 25 % of the total nominal HB load adopted, applied as equally distributed between the eight wheels of two axles of the vehicle, 1.8 m apart (see 6.3).

Associated Nominal Primary Live Load

Type HA or HB load, applied in accordance with *6.4,* shall be considered to act with longitudinal load as appropriate.

Load Combination

Longitudinal load shall be considered in combination 4 only and need not be taken as coexistent with other secondary live loads.

Design Load

For the longitudinal and primary live load, γ_{fL} shall be taken as follows.

	For the ultimate Limit state	**For serviceability Limit state**
For HA load	1.25	1.00
For HB load	1.10	1.00

Accidental Load Due to Skidding

On straight and curved bridges a single point load shall be considered in one notional lane only, acting in any direction parallel to the surface of the highway.

Nominal Load

The nominal load shall be taken as 250 kN.

Associated Nominal Primary Live Load

Type HA loading, applied in accordance with *6.4.1,* shall be considered to act with the accidental skidding load.

Load Combination

Accidental load due to skidding shall be considered in combination 4 only, and need not be taken as coexistent with other secondary live loads.

Design Load

For the skidding and primary live load, γ_{fL} shall be taken as follows.

For the ultimate limit state	For the serviceability limit state
1.25	1.00

Loads Due to Vehicle Collision with Parapets*

Nominal Load

In the design of the elements of the structure supporting parapets, the actual loads, moments or shears required to bring about the collapse of the parapet or the connection of the parapet to the element (whichever is the greater) shall be regarded as the nominal loads, moments or shears applied to the element.

Associated Nominal Primary Live Load

Any four wheels of 25 units of HB loading (see *6.3)* shall be considered in whatever position they will have the most adverse effect on the element; where their application has a relieving effect they shall be ignored.

Load Combination

Loads due to vehicular collision with parapets shall be considered in combination 4 only, and need not be taken as coexistent with other secondary live loads.

Design Load

For the load transmitted by the collapse of the parapet or its connection to the element supporting the parapet and for the primary live load, γ_{fL} shall be taken as follows.

For the ultimate limit state	For the serviceability limit state
1.25	1.00

Collision Loads on Supports at Bridges Over Highways

Members supporting bridge superstructures, both in the central reservation and at the road edge, shall be protected by safety fences where traffic is permitted to travel at speeds of 80 km/h or above, and in other cases where damage to the supports might lead to severe consequences. For foot/cycle track bridges see *7.1.4* (and appendix B).

Nominal Load

The nominal loads are given in table 14, together with their direction and height of application, and are to be considered as acting horizontally on bridge supports (but see *7.1.4* for foot and cycle bridges).

Supports shall be capable of resisting the load transmitted from the guard rail applied simultaneously with the residual load above the guard rail. Loads normal to the carriageway are to be considered separately from loads parallel to the carriageway.

Associated Nominal Primary Live Load

No primary live load is required to be considered on the bridge.

Load Combination

Collision load shall be considered in combination 4 only, and need not be taken as coexistent with other secondary live loads.

Design Load

For the vehicle collision load on supports, γ_{fL} shall be as follows:

For the ultimate limit state	For the serviceability limit state
1.25	1.00

Table 14. Collision loads on supports of bridges over highways

	Load normal to the carriageway below	Load parallel to the carriageway below	Point of application on bridge support
Load transmitted from guard rail	kN 150	kN 50	Anyone bracket attachment point or, for free standing fences, 0.75m above carriageway level
Residual load above guard rail	100	100	Atthemost severe point between 1 m and 3 m above carriageway level

Bridges over Railways, Canals or Navigable Water

Collision loading on supports of bridges over railways, canals or navigable water shall be as agreed with the appropriate authority.

Loading for Fatigue Investigations

For loading for fatigue investigations, see Part 10 of this standard.

Dynamic Loading on Highway Bridges

The effects of vibration due to traffic are not required to be considered.

7. FOOTWAY AND CYCLE TRACK LIVE LOAD

Bridges Supporting Footways or Cycle Tracks only

Nominal Live Load

The nominal live load on elements supporting footways and cycle tracks only shall be taken as follows:

(a) for loaded lengths of 30 m and under, a uniformly distributed live load of 5.0 kN/m^2;

(b) for loaded lengths in excess of 30 m, k x 5.0 kN/m^2, where k is the

$$\frac{\text{nominal HA UDL for appropriate loaded length (in kN/m)}}{\text{30 kN/m}}$$

Special consideration shall be given to the intensity of the live load to be adopted on loaded lengths in excess of 30 m where exceptional crowds may be expected (as, for example, where a footbridge serves a sports stadium).

Nominal Load on Pedestrian Parapets

The nominal load shall be taken as 1.4 kN/m length, applied at a height of 1 m above the footway or cycle track and acting horizontally.

Design Load

For the live load on footways and cycle tracks and for the load on pedestrian parapets, γ_{fL} shall be taken as follows:

	For the ultimate limit state	**For the serviceability limit state**
For combination 1	1.50	1.00
For combinations 2 and 3	1.25	1.00

Collision Load on Supports of Foot/Cycle Track Bridges

The load on the supports of foot/cycle track bridges shall be a single load of 50 kN applied horizontally in any direction up to a height of 3 m above the adjacent carriageway. This is the only secondary live load to be considered for foot/cycle track bridges.

Associated Nominal Primary Live Load

No primary live load is required to be considered on the bridge.

Design Load

For combination 4, γ_{fL} shall be taken as follows:

For the ultimate limit state	**For the serviceability limit state**
1.25	1.00

Vibration Serviceability

Consideration shall be given to vibration that can be induced in foot/cycle track bridges by resonance with the movement of users. The structure shall be deemed to be satisfactory where its response, as calculated in accordance with appendix C, complies with the limitations specified therein.

Elements Supporting Footways or Cycle Tracks and a Highway or Railway

Nominal Live Load

On footways and cycle tracks carried by elements that also support highway or railway loading, the nominal live load shall be taken as 0.8 of the value specified in 7.1.1. (a) or (b), as appropriate, except where crowd loading is expected, in which case special consideration shall be given to the intensity of live loading to be adopted.

Where the footway (or footway and cycle track together) is wider than 2 m these intensities may be further reduced by 15 % on the first metre in excess of 2 m and by 30 % on the second metre in excess of 2 m. No further reduction for widths exceeding 4 m shall be made. These intensities may be averaged and applied as a uniform intensity over the full width of footway.

Where a main structural member supports two or more highway traffic lanes or railway tracks, the footpath and cycle track loading to be carried by the main member may be reduced to the following.

On footways : 0.5 of the value given in 7.1.1 (a) and (b), as appropriate.

On cycle tracks: 0.2 of the value given in 7.1.1 (a) and (b), as appropriate.

Special consideration shall, however, be given to structures where there is a possibility of crowds using cycle tracks, which could coincide with exceptionally heavy highway loading.

Nominal Wheel Load

Where the footway or cycle track is not protected from highway traffic by an effective barrier, any four wheels of 25 units of HB loading (see 6.3) acting alone in any position shall be considered.

Associated Nominal Primary Live Load

Associated nominal primary live load on the carriageway or rail track shall be derived and applied in accordance with 6.4 or clause 8, as appropriate.

Load due to Vehicle Collision with Parapets

Where the footway or cycle track is not protected from the highway traffic by an effective barrier, the nominal loads on elements of the structure supporting parapets shall be as specified in 6.8.

Design Load

γ_{fL}, to be applied to the nominal loads shall be as follows:

(a) for live loading on footways and cycle tracks, as specified in 7.1.4;
(b) for highway live loading, as specified in 6.2.7 and 6.3.4;
(c) for railway live loading, as specified in 8.4;
(d) for loading derived from vehicle collision with parapets, as specified in 6.8.

8. RAILWAY BRIDGE LIVE LOAD

General

Standard railway loading consists of two types, RU and RL.

RU loading allows for all combinations of vehicles currently running or projected to run on railways in the Continent of Europe, including the United Kingdom, and is to be adopted for the design of bridges carrying main line railways of 1.4 m gauge and above.

RL loading is a reduced loading for use only on passenger rapid transit railway systems on lines where main line locomotives and rolling stock do not operate.

The derivation of standard railway loadings is given in appendix D.

Nominal primary and associated secondary live loads are as given in 8.2.

Nominal Loads

Type RU Loading

Nominal type RU loading consists of four *250* kN concentrated loads preceded, and followed, by a uniformly distributed load of 80 kN/m. The arrangement of this loading is as shown in figure 13.

Type RL Loading

Nominal type RL loading consists of a single 200 kN concentrated load coupled with a uniformly distributed load of *50* kN/m for loaded lengths up to 100 m. For loaded lengths in excess of 100 m the distributed nominal load shall be 50 kN/m for the first 100 m and shall be reduced to 25 kN/m for lengths in excess of 100 m, as shown in figure 14.

Alternatively, two concentrated nominal loads, one of 300 kN and the other of 150 kN, spaced at 2.4 m intervals along the track, shall be used on deck elements where this gives a more severe condition. These two concentrated loads shall be deemed to include dynamic effects.

Dynamic Effects

The standard railway loadings specified in 8.2.1 and 8.2.2 (except the 300 kN and 150 kN concentrated alternative RL loading) are equivalent static loadings and shall be multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by track and wheel irregularities. The dynamic factors given in 8.2.3.1 and 8.2.3.2 shall be adopted, provided that maintenance of track and rolling stock is kept to a reasonable standard.

Type RU Loading

The dynamic factor for RU loading applies to all types of track and shall be as given in table 15.

Table 15. Dynamic factor for type RU loading

Dimension L	Dynamic factor for evaluating	
	Bending moment	Shear
m		
up to 3.6	2.00	1.67
from 3.6 to 67	$0.73+\frac{2.16}{\sqrt{L-0.2}}$	$0.82+\frac{1.44}{\sqrt{L-0.2}}$
over 67	1.00	1.00

In deriving the dynamic factor, L is taken as the length (in m) of the influence line for deflection of the element under consideration. For unsymmetrical influence lines, L is twice the distance between the point at which the greatest ordinate occurs and the nearest end point of the influence line. In the case of floor members, 3 m should be added to the length of the influence line as an allowance for load distribution through track.

The values given in table 16 may be used, where appropriate.

Type RL Loading

The dynamic factor for RL loading, when evaluating moments and shears, shall be taken as 1.20, except for unballasted tracks where, for rail bearers and single-track cross girders, the dynamic factor shall be increased to 1.40.

The dynamic factor applied to temporary works may be reduced to unity when rail traffic speeds are limited to not more than 25 km/h.

Dispersal of Concentrated Loads

Concentrated loads applied to the rail will be distributed both longitudinally by the continuous rail to more than one sleeper, and transversely over a certain area of deck by the sleeper and ballast.

It may be assumed that only two-thirds of a concentrated load applied to one sleeper will be transmitted to the bridge deck by that sleeper, and that the remaining one-third will be transmitted by the two sleepers either side.

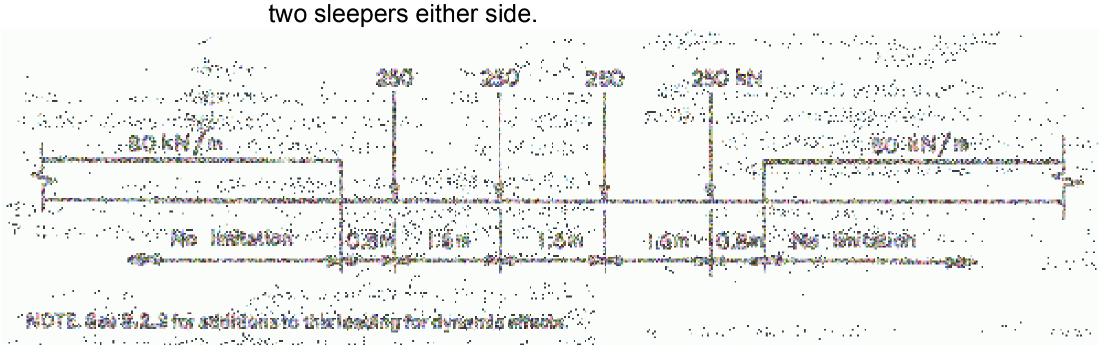

Figure 13. Type RU Loading

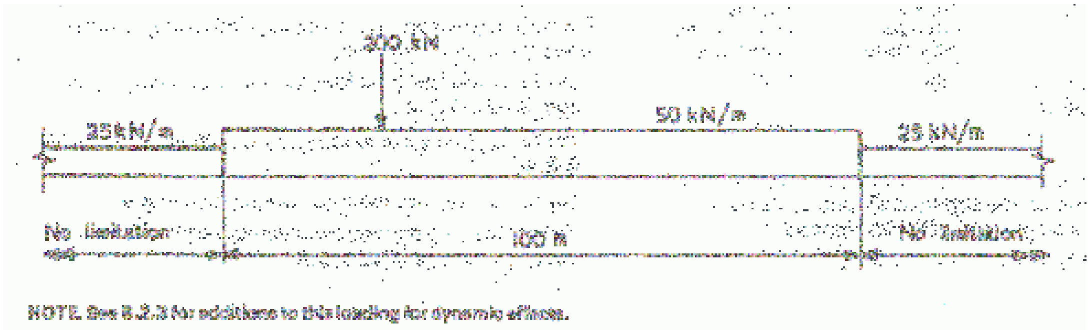

Figure 14. Type RL loading

Table 16. <u>Dimension L used in calculating the dynamic factor for RU loading</u>

	Dimension L
Main girders:	
simply supported	Span
Continuous	For 2,3,4,5 and more spans 1.2, 1.3, 1.4, 1.5 x mean span, but at least the greatest span
Portal frames and arches	1/2 span
Floor members:	
Simply supported rail bearers	Cross girder spacing plus 3m
Cross girders loaded by simply supported rail bearers	Twice the spacing of cross girders plus 3m
End cross girder or trimmers	4m
Cross girders loaded by continuous deck elements and any element in a continuous deck system	The lesser of the span of the main girders and twice the main spacing

The load acting on the sleeper under each rail may be assumed to be distributed uniformly over the ballast at the level on the underside of the sleeper for a distance of 800 mm symmetrically about the centre-line of the rail or to twice the distance from the centre line of the rail to the nearer end of the sleeper, whichever is the lesser. Dispersal of this load through the ballast onto the supporting structure shall be taken at 5° to the vertical.

The distribution of concentrated loads applied to a track not supported on ballast shall be calculated on the basis of the relative stiffnesses of the rail, its support on the bridge deck and the bridge deck itself.

In designing the supporting structure for the loads transmitted from the sleepers, distributed as set out above, any further distribution arising from the type of construction of the deck may be taken into account.

Deck Plates and Similar Local Elements

Irrespective of the calculated distribution of axle loads, all deck plates and similar local elements shall be designed to support a nominal load of 250 kN for RU loading and 168 kN for RL loading at any point of support of a rail. These loads shall be deemed to include all allowances for dynamic effects and lurching.

Application of Standard Loadings

Type RU or RL loading shall be applied to each and every track as specified in 4.4. Any number of lengths of the distributed load may be applied, but for RL loading the total length of 50 kN/m intensity shall not exceed 100 m on any track. The concentrated loads shall only be applied once per track for any point under consideration.

Lurching

Lurching results from the temporary transfer of part of the live loading from one rail to another, the total track load remaining unaltered.

The dynamic factor applied to RU loading will take into account the effects of lurching, and the load to be considered acting on each rail shall be half the track load.

The dynamic factor applied to RL loading will not adequately take account of all lurching effects. To allow for this, 0.56 of the track load shall be considered acting on one rail concurrently with 0.44 of the track load on the other rail. This redistribution of load need only be taken into account on one track where members-support two tracks. Lurching may be ignored in the case of elements that support load from more than two tracks.

Nosing

An allowance shall be made for lateral loads applied by trains to the track. This shall be taken as a single nominal load of 100 kN, acting horizontally in either direction at right angles to the track at rail level and at such a point in the span as to produce the maximum effect in the element under consideration.

The vertical effects of this load on secondary elements such as rail bearers shall be considered.

For elements supporting more than one track a single load, as specified, shall be deemed sufficient.

Centrifugal Load

Where the track on a bridge is curved, allowance for centrifugal action of moving loads shall be made in designing the elements, all tracks on the structure being considered occupied. The nominal centrifugal load F, in kN, per track acting radially at a height of 1.8 m above rail level shall be calculated from the following formula:

$$F_c = \frac{P(v_t+10)^2}{127r} \times f$$

where

P is the static equivalent uniformly distributed load for bending moment when designing for RU loading; for RL loading, a distributed load of 40 kN/m multiplied by *L* is deemed sufficient

r is the radius of curvature (in m)

V_t is the greatest speed envisaged on the curve in question (in km/h)

$$f = 1 - \left[\frac{Vt - 120}{1000}\right] x \left[\frac{814}{v_t} + 1.75\right] \text{x} \left[1 - \sqrt{\frac{2.88}{L}}\right]$$

for *L* greater than 2.88 m and v_t over 120 km/h
= unity for *L* less than 2.88 m or v_t less than 120 km/h

L is the loaded length of the element being considered.

Longitudinal Loads

Provision shall be made for the nominal loads due to traction and application of brakes as given in table 17. These loads shall be considered as acting at rail level in a direction parallel to the tracks. No addition for dynamic effects shall be made to the longitudinal loads calculated as specified in this subclause.

For bridges supporting ballasted track, up to one-third of the longitudinal loads may be assumed to be transmitted by the track to resistances outside the bridge structure, provided that no expansion switches or similar rail discontinuities are located on, or within, 18 m of either end of the bridge. Structures and elements carrying single tracks shall be designed to carry the larger of the two loads produced by traction and braking in either direction parallel to the track.

Where a structure or element carries two tracks, both tracks shall be considered as being occupied simultaneously. Where the tracks carry traffic in opposite directions, the load due to braking shall be applied to one track and the load due to traction to the other. Structures and elements carrying two tracks in the same direction shall be subjected to braking or traction on both tracks, whichever gives the greater effect. Consideration, however, may have to be given to braking and traction, acting in opposite directions, producing rotational effects.

Where elements carry more than two tracks, longitudinal loads shall be considered as applied simultaneously to two tracks only.

Load Combinations

All loads that derive from rail traffic. Including dynamic effects, lurching, nosing, centrifugal load and longitudinal loads, shall be considered in combinations 1, 2 and 3.

Table 17. Nominal longitudinal loads

Standard loading type	Load arising from	Loaded length	Longitudinal load
RU	Traction (30% of load on driving wheels)	m up to 3	kN 150
		from 3 to 5	225
		from 5 to 7	300
		from 7 to **25**	**24(L-7)**+300
		over 25	750
	Braking (25 % of load on braked wheels)	upto3	125
		from 3 to 5	187
		from 5 to 7	250
		over **7**	**20(L-7)+ 250**
RL	Banking (25% of load on braked wheels)	up to 8	80
		from 8to30	10kN/m
		from 30 to 60	300
		from 60 to 100	5 kN/m
		over 100	500
	Braking (25% of load on braked wheels)	up to 8	64
		from 8 to 100	8 kN/m
		over 100	800

Design Loads

For primary and secondary railway live loads, γ_{fL} shall be taken as follows:

	For the ultimate limit state	For the serviceability limit state
Combination 1	1.40	1.10
Combination 2 and 3	1.20	1.00

Derailment Loads

Railway bridges shall be so designed that they do not suffer excessive damage or become unstable in the event of a derailment. The following conditions shall be taken into consideration:

(a) For the serviceability limit state, derailed coaches or light wagons remaining close to the track shall cause no permanent damage.
(b) For the ultimate limit state, derailed locomotives or heavy wagons remaining close to the track shall not cause collapse of any major element, but local damage may be accepted.
(c) For overturning or instability, a locomotive and one following wagon balanced on the parapet shall not cause the structure as a whole to overturn, but other damage may be accepted.

Conditions (a), (b) and (c) are to be considered separately and their effects are not additive. Design loads applied in accordance with 8.5.1 and 8.5.2 for types RU and RL loading, respectively, may be deemed to comply with these requirements.

Design Load for RU Loading

The following equivalent static loads, with no addition for dynamic effects, shall be applied:

(a) For the serviceability limit state, either
  (1) a pair of parallel vertical line loads of 20 kN/m each, 1.4 m apart, parallel to the track and applied anywhere within 2 m either side of the track centre line; or
  (2) an individual concentrated vertical load of 100 kN anywhere within the width limits specified in (1).
(b) For the ultimate limit state, eight individual concentrated vertical loads each of 180 kN, arranged on two lines 1.4 m apart, with each of the four loads 1.6 m apart on line, applied anywhere on the deck.
(c) For overturning or instability, a single line vertical load of 80 kN/m applied along the parapet or outermost edge of the bridge, limited to a length of 20 m anywhere along the span.

Loads specified in (a) and (b) shall be applied at the top surface of the ballast or other deck covering and may be assumed to disperse at 30 ° to the vertical onto the supporting structure.

Design Load for RL Loading

The following equivalent static loads, with no addition for dynamic effects, shall be applied:

(a) For the serviceability limit state, either
  (1) a pair of parallel vertical line loads of 15 kN/m each, 1.4 m apart, parallel to the track and applied anywhere within 2 m either side of the track centre line (or within 1.4 m either side of the track centre line where the track includes a substantial centre rail for electric traction or other purposes) ; or
  (2) an individual concentrated vertical load of 75 kN anywhere within the width limits specified in (1).
(b) For the ultimate limit state, four individual concentrated loads each of 120 kN, arranged at the corners of a rectangle of length 2.0 m and width 1.4 m, applied anywhere on the deck.

(c) For overturning and instability, a single vertical line load of 30 kN/m, applied along the parapet or outermost edge of the bridge, limited to a length of 20 m anywhere on the span.

Loads specified in (a) and (b) shall be applied at the top surface of the ballast or other deck covering and may be assumed to disperse at 30° to the vertical onto the supporting structure.

Collision Load on Supports of Bridges Over Railways

* The collision load on supports of bridges over railways shall be as agreed with the appropriate authorities.

Loading for Fatigue Investigations

All elements of bridges subject to railway loading shall be checked against the effects of fatigue caused by repeated cycles of live loading. The number of load cycles shall be based on a life expectancy of 120 years for bridges intended as permanent structures. The load factor to be used in all cases when considering fatigue is 1.0.

For RU and RL loading the 120-year load spectrum, which has been calculated from traffic forecasts for the types of line indicated, shall be in accordance with Part 10 of this standard.
3

* Requirements for the supports of bridges over highways and waterways are specified in 6.9.

9. EARTHQUAKE LOADS

Introduction

Purpose

As there was no Seismic Loading section in the British Standards, Seismic loading from American Association of State Highway & Transport Officials (AASHTO) was customized for Uganda with appropriate changes.

These Standards establish design and construction provisions for bridges to minimize their susceptibility to damage from earthquakes.

The design earthquake motions and forces specified in these provisions are based on a low probability of their being exceeded during the normal life expectancy of a bridge. Bridges and their components that are designed to resist these forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

The principles used for the development of the provisions are:

1. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.
2. Realistic seismic ground motion intensities and forces are used in the design procedures.
3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

A basic premise in developing these seismic design standards was that they be applicable to all parts of Uganda. The seismic risk varies from very small to high across the country. Therefore, for purposes of design, three Zones (Seismic Performance Categories, SPC) are defined on the basis of an Acceleration Coefficient (A) for the site, determined from the map provided, and the Importance Classification (IC). Different degrees of complexity and sophistication of seismic analysis and design are specified for each of the three Seismic Performance Categories.

An essential bridge must be designed to function during and after an earthquake. A bridge is designated essential on the basis of Social/Survival and Security/Defense classifications presented in the Commentary.

Basic Concepts

Development of the Standards has been predicated on the following basic concepts:

- Hazard to lift be minimized.
- Bridges may suffer damage but have low probability of collapse due to earthquake motions.
- Function of essential bridges be maintained.
- Design ground motions have low probability of being exceeded during normal lifetime of bridge.
- Provisions be applicable to all of the United States.
- Ingenuity of design not be restricted.

Quality Assurance Requirements

Structural failures which have occurred during earthquakes and are directly traceable to poor quality control during construction are innumerable. The literature is replete with reports pointing out that collapse may have been prevented had proper inspection been exercised. To provide adequate seismic quality assurance requirement the engineer specified the quality assurance requirements, the Contractor exercises the control to achieve the desired quality and the owner monitors the construction process through special inspection. It is essential that each party recognizes its responsibilities, understands the procedures and has the capability to carry them out. Because the Contractor does the work and exercises quality control it is essential that the inspection be performed by someone approved by the owner and not the contractors direct employee.

Flow Charts for Use of Standards

Flow charts outlining the steps in the seismic design procedures are given in Figures 15 and 16. The Commentary provides background information to assist the user in understanding the intent of the Standards;

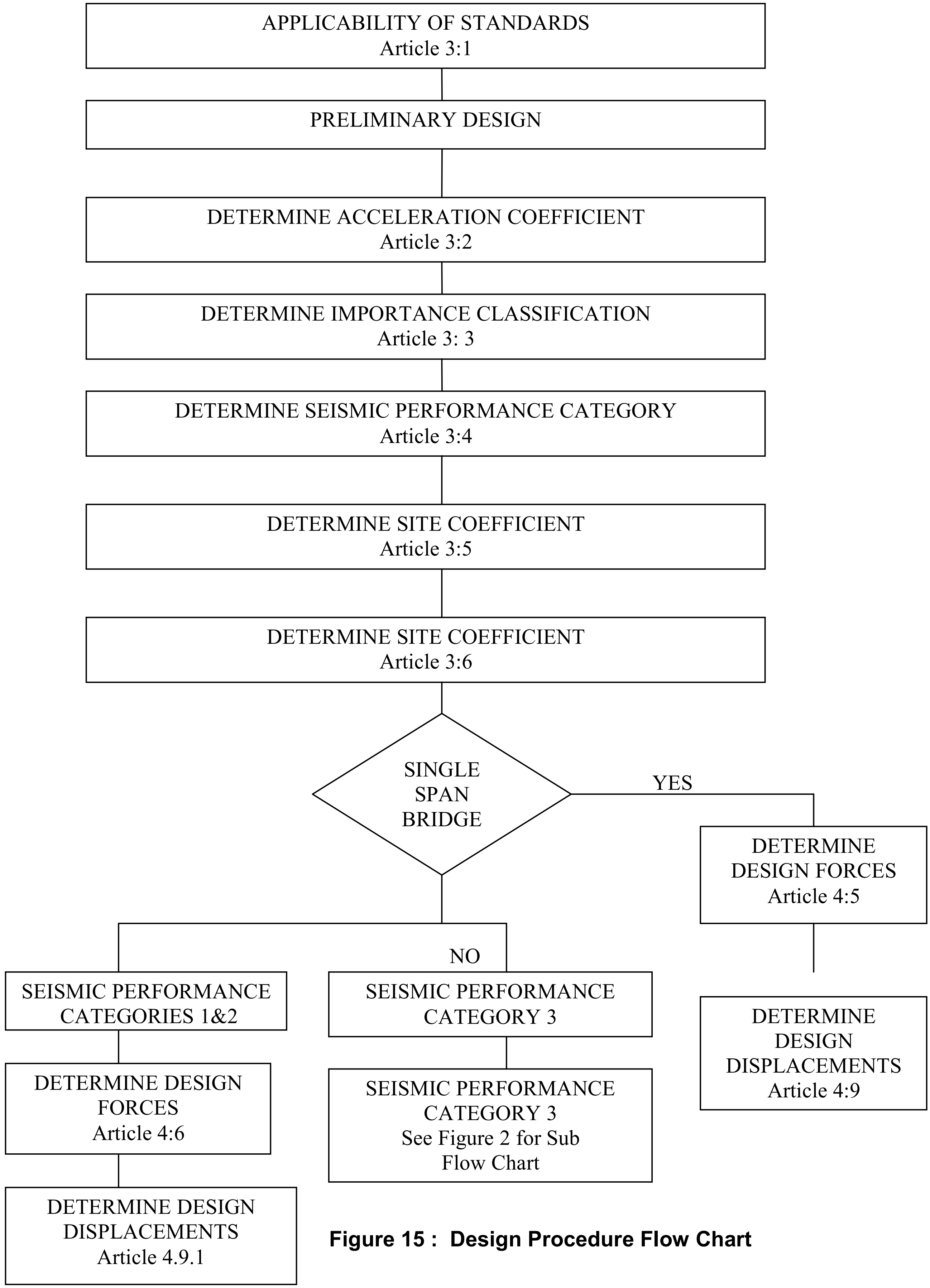

Figure 15 : Design Procedure Flow Chart

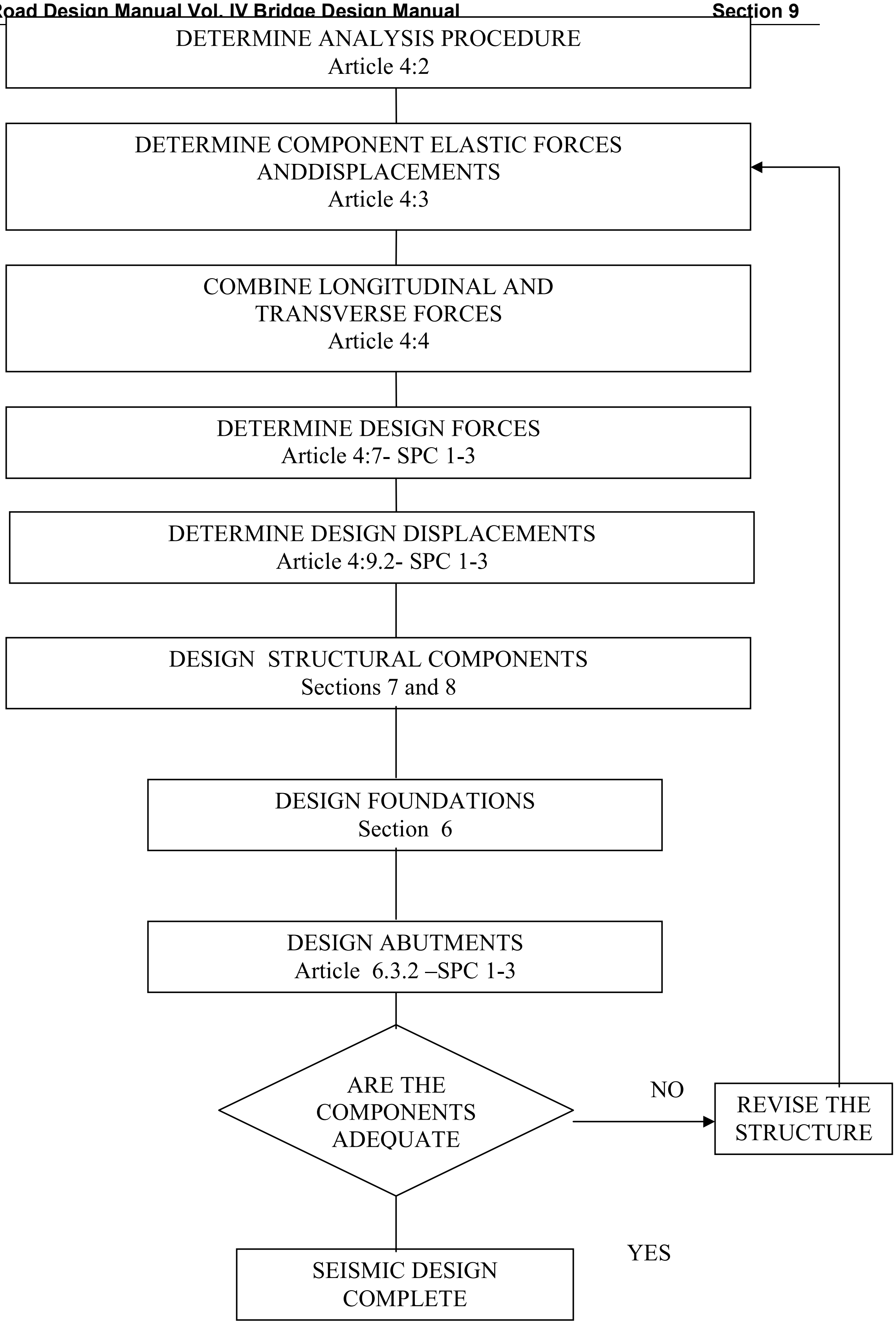

Figure 16. Sub Flow Chart for Seismic Performance Category 3

Symbols and Definitions

Notations

The following symbols and definitions apply to these Standards:

a	=	Vertical spacing of transverse reinforcement (hoops or stirrups) in rectangular reinforced concrete columns (mm)
A	=	Acceleration coefficient determined in Article 3.2 (dimensionless)
A_c	=	Area of reinforced concrete column core (mm^2)
A_g	=	Gross area of reinforced concrete column (mm^2)
A_s	=	Area of longitudinal reinforcement in concrete pile (mm^2)
A_{sh}	=	Total cross-sectional area of transverse reinforcement (hoops or stirrups) used in rectangular reinforced concrete columns (mm^2)
A_{vf}	=	Total amount of reinforcement normal to a construction joint (mm^2)
B	=	Loads resulting from buoyancy forces and used in the group load combinations of Eqs. 4-1 and 4-2
C_m	=	Coefficient used in steel design to account for boundary conditions (dimensionless)
C_s	=	Elastic seismic response coefficient defined in Article 5.2.1 (dimensionless)
C_{sm}	=	Elastic seismic response spectrum defined in Article 5.2.2 (dimensionless)
d	=	Diameter of a reinforced concrete column (mm^2)
D	=	Loads resulting from dead load and used in the group load combinations of Eqs. 4-1 and 4-2
E	=	Loads resulting from earth pressure and used in the group load combinations of Eqs. 4-1 and 4-2.
EQF	=	Modified seismic forces used in the group load combination of Eq. 4-2 and defined in Article 4.7.2
EQM	=	Modified seismic forces used in the group load combination of Eq. 4-1 and defined in Article 4.7.1
f_c	=	Specified compressive strength of reinforced concrete (psi or MPa)
f_y	=	Yield strength of reinforcement in reinforced concrete members (psi or MPa)
f_{yh}	=	Yield strength of transverse reinforcement (psi or MPa)
F_a	=	Axial stress in steel design that would be permitted if axial force alone existed (psi or MPa)
F	=	Buckling stress for load factor steel design (psi or MPa)
F_e	=	Euler buckling stress in the plane of bending (psi or MPa)
F_e	=	Euler buckling stress for service load steel design (psi or MPa)
F_y	=	Yield strength of structural steel design (psi or MPa)
g	=	Acceleration of gravity (in/sec^2 or cm/sec^2)
h_e	=	Core dimension of a rectangular reinforced concrete column (in or mm)

H	=	Height of a column or pier defined in Article 4.9 (ft or m)
I_C	=	Importance classification given in Article 3.3 (dimensionless)
K	=	Effective length factor used in steel design and given in Article 7.3 (dimensionless)
K_h	=	Seismic coefficient used to calculate lateral earth pressures and defined in Article 6.3.2 (dimensionless)
L	=	Length of bridge deck defined in Article 4.9 (ft. mm)
N	=	Minimum support length for girder specified in Article 4.9 (in or mm)
P_e (x)	=	Intensity of the equivalent static seismic loading applied to represent the primary mode of vibration in Article 5.3 (force/unit length)
P_n	=	Minimum axial load specified in Article 4.8.3 for columns and 4.8.4 for piers (lb or N)
P_o	=	Assumed uniform loading used to calculate the period in Article 5.3 (force/unit length)
P_u	=	Maximum strength of concentrically loaded steel columns (lb or N)
R	=	Response modification factor specified in Article 3.6 (dimensionless)
s	=	Spacing of spiral reinforcement in reinforced concrete columns (in or mm)
S	=	Site coefficient specified in Article 3.5.1 (dimensionless)
SF	=	Load resulting from stream flow forces and used in the group load combinations of Eqs. 4-1 and 4-2.
SPC	=	Seismic Performance Category specified in Article 3.4 (dimensionless)
T	=	Fundamental period of the bridge determined in Article 5.3 (sec.)
T_m	=	Period of the mth mode of vibration of a bridge (sec.)
V_j	=	Limiting shear force across a construction joint (lb or N)
V_u	=	Shear stress (psi or MPa)
V_u	=	Shear force (lb or N)
Vs(x),	=	Static displacement profiles resulting from applied loads Po and Pe, respectively, and used in Article 5.3
Ve(x)	=	(in or mm)
W(x)	=	Dead weight of the bridge superstructure and tributary substructure per unit length (force/unit length)
Ph	=	The ratio of horizontal shear reinforcement area to gross concrete area of vertical section – Article 8.4.2 (dimensionless)
Pn	=	The ratio of vertical shear reinforcement area to the gross concrete area of a horizontal section – Article 8.4.2 (dimensionless)
Ps	=	Volumetric ration of spiral reinforcement for a circular column (dimensionless)
		ϕ = Strength reduction factor (dimensionless)
α	=	Coefficient used to calculate the period of the bridge in Article 5.3 (length^2)
β	=	Coefficient used to calculate the period of the bridge in Article 5.3 (force – length)
γ	=	Coefficient used to calculate the period of the bridge in Article 5.3 (force – lenght^2)

General Requirements

Applicability of Standards

These Standards are for the design and construction of new bridges to resist the effect of earthquake motions. The provisions apply to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 500 ft (152.4m). Suspension bridges, cable-stayed bridges, arch type and movable bridges are not covered by these Standards but general considerations for designing such bridges are presented in the Commentary. Seismic design is usually not required for buried type (culvert) bridges.

The provisions specified in these Standards are minimum requirements.

No detailed seismic analysis is required for any single span bridge or for any bridge in Seismic Performance Category 1 & 2. For both single span bridges (Article 4.5) and bridges classified as SPC 1 & 2 (Article 4.6) the connections must be designed for specified forces and must also meet minimum support length requirements (Article 4.9).

Acceleration Coefficient

The coefficient A to be used in the application of these provisions shall be determined from the contour maps of Figure 17.

Local maxima (and minima) are given inside the highest (and lowest) contour for a particular region. Linear interpolation shall be used for sites located between contour lines between a contour line and local maximum (or minimum). It is recommended that a qualified professional be consulted to determine a value for the coefficient A for sites located close to active fault.

Table 9.1: Bedrock Acceleration ratio, A

Zone (SPC)	1	2	3
α_o	0.15	0.07	0.05

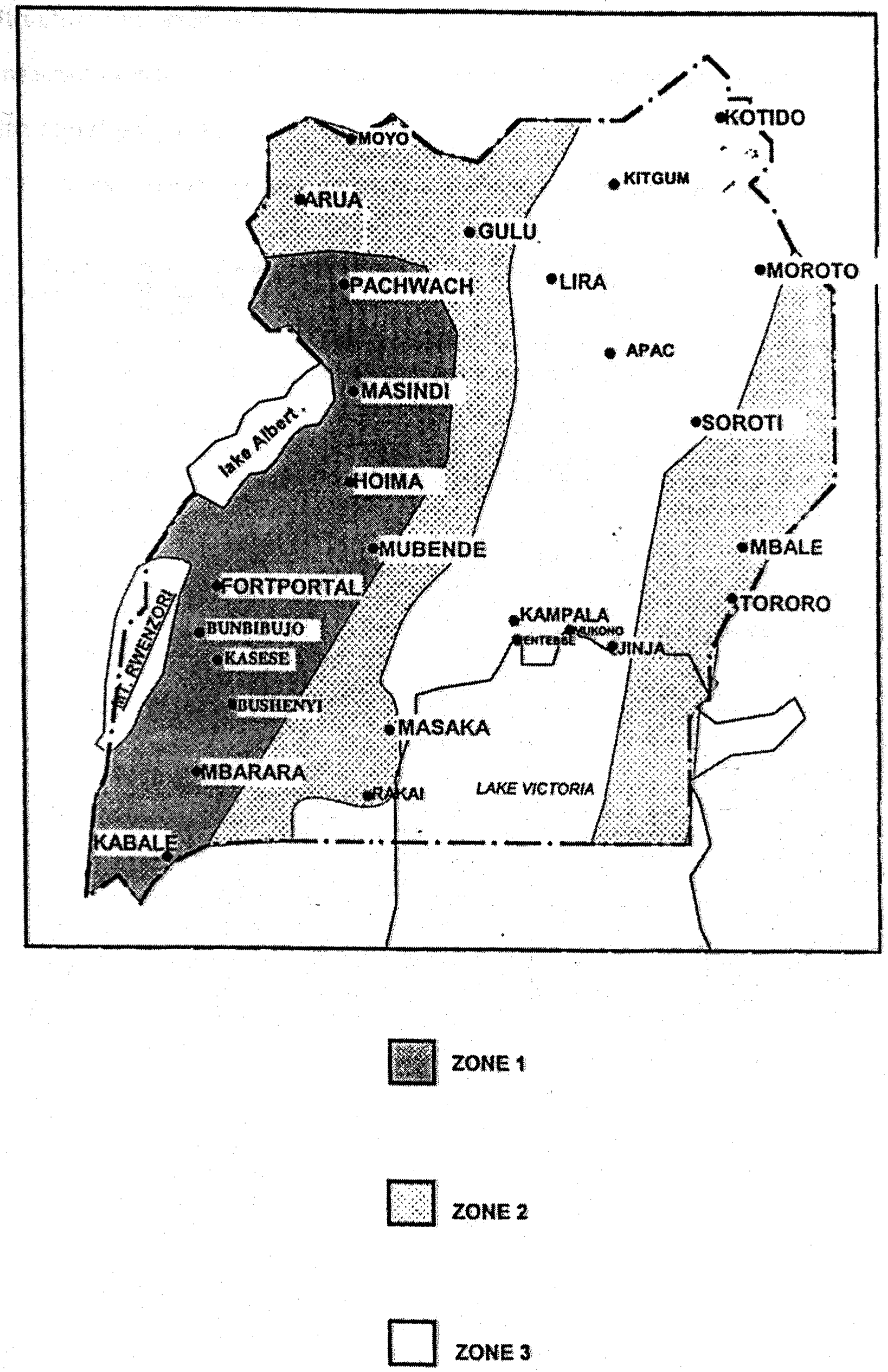

Figure 17. Seismic map of Uganda

Importance Classification

An Importance Classification (IC) shall be assigned for all bridges with an Acceleration Coefficient greater than 0.29 for the purpose of determining the Seismic Performance Category (SPC) in Article 3.4 as follows:

1. Essential bridges – IC = I
2. Other bridges – IC = II

Bridges shall be classified on the basis of Social/Survival and Security/Defense requirements, standards for which are given in the Commentary.

Seismic Performance Categories

Each bridge shall be assigned to one of three seismic Performance Categories (SPC), 1 through 3, based on the Acceleration Coefficient (A) and the Importance Classification (IC), as shown in Table 9.2. Minimum analysis and design requirements are governed by the SPC.

TABLE 9.2 : Seismic Performance Category (SPC)

Acceleration Coefficient	Importance Classification (IC)	
A	**I**	**II**
A $\leq$ 0.05	1	1
0.05 < A $\leq$ 0.07	2	2
0.07 < A $\leq$ 0.15	3	3

Site Effects

The effects of site conditions on bridge response shall be determined from a site coefficient (S) based on soil profile types defined as follows:

SOIL PROFILE TYPE I is a profile with either

1. Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/sec (762m/sec), or by other appropriate means of classification); or
2. Stiff soil conditions where the soil depth is less than 200 ft (61m) and the soil types overlying rock are stable deposits of sands, gravels, or still clays.

SOIL PROFILE TYPE II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 200 ft (61m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE III is a profile with soft to medium-stiff clays and sands, characterized by 30ft (9m) or more of soft to medium – stiff clays with or without intervening layers of sand or other cohesionless soils.

In locations where the soil properties are not known in sufficient detail to determined the soil profile type or where the profile does not fit any of the three types, the site coefficient of Soil Profile Type II shall be used. The soil profile coefficients apply to all foundation types including pile supported and spread footings.

Site Coefficient

The site coefficient (S) approximates the effects of the site conditions on t the elastic response coefficient or spectrum of Article 5.2 and is given in Table 9.3.

TABLE 9.3. Site Coefficient (S)

	Soil Profile Type		
	I	II	III
S	1.0	1.2	1.5

Response Modification Factors

Seismic design forces for individual members and connections of bridges classified as SPC 3 are determined by dividing the elastic forces by the appropriate Response Modification Factor (R) as specified in Article 4.7 and 4.8. The Response Modification Factors for the various components are given in Table 9.4.

TABLE 9.4. Response Modification Factor (R)

Substructure[1]	R	Connections	R
Wall – Type Pier[2]	2	Superstructure to Abutment	0.8
Reinforced Concrete Pile Bents		Expansion Joints within a span of the superstructure c	0.8
a. Vertical piles only	3		
b. One or more Batter piles	2	Columns, Piers or pile bents to Cap Beam of Superstructure[3]	1.0
Single columns	3	Columns or piers to Foundation [3]	1.0
Steel or composite steel and concrete pile bents			
a. Vertical piles only	5		
b. One or more batter piles	3		
Multiple column bent			

[1]The R-Factor is to be used for both orthogonal axes of the substructure.
[2]A wall-type pier may be designed as a column in the weak direction of the pier provided all the provision for columns in Chapter 8 are followed. The R-Factor for a single column can then be used.

Analysis and Design Requirements

General

The provisions of this chapter shall control the selection of the analysis procedure and seismic design forces and displacements. The elastic seismic forces shall be determined in accordance with the procedures for section 5. Material and foundation design requirements are given in section 6, 7 and 8.

EXCEPTION:

Seismic design requirements for single span bridges are given in Articles 4.5 and 4.9 and design requirements for bridges classified as SPC 1 & 2 are given in Article 4.6 and 4.9.1.

Analysis Procedure

Two minimum analysis procedures are defined and the applicable procedure for a given type of bridge, which depends on the number of spans, the geometrical complexity and the Seismic Performance Category, is given in Table 9.5. A more rigorous, generally accepted procedure may be used in lieu of the recommended minimum.

TABLE 9.5 : Analysis Procedures

Seismic Performance Category	Regular[1] Bridges with 2 or More spans	Irregular[2] Bridges with 2 or More Spans
1 & 2	-	-
3	1	1

[1] A “regular” bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded). For example a bridge may be considered regular if it is straight or described a sector of an arch not exceeding 90^0 and has adjacent columns, or piers that do not differ in stiffness by more than 25% (percentage differences is to be based on the lesser of tow adjacent quantities as the reference).

[2] An “irregular” bridge is any bridge that does not satisfy the definition of a regular bridge.

The two analysis procedures to be used are as follows:

PROCEDURE 1: Single –Mode Spectral Method
PROCEDURE 2 : Multimode Spectral Method

Details of these procedures are given in Section 5.

EXCEPTION:

Detailed seismic analysis is not required for a single span bridge or for bridges classified as SPC 1 & 2.

Determination of Elastic Forces and Displacements

For bridges classified as SPC 3 the elastic forces and displacements shall be determined independently along two perpendicular axes by use of the analysis procedure specified in Article 4.2. the resulting forces shall then be combined as specified in Article 4.4. Typically the perpendicular axes are the longitudinal and transverse axes of the bridge but the choice is open to the designer. The longitudinal axis of a curved bridge may be a chord connecting the two abutments.

Combination of Orthogonal Seismic Forces

A combination of orthogonal seismic forces is used to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquakes forces in tow perpendicular horizontal directions. The elastic seismic forces and moments resulting from analysis in the two perpendicular directions. The elastic seismic forces and moments resulting from analyses in the tow perpendicular directions of Article 4.3 shall be combined to form tow load cases as follows.

LOAD CASE I : Seismic forces and moments on each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in one of the perpendicular (longitudinal) directions to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse). (NOTE: The absolute values are used because a seismic force can be positive or negative).

LOAD CASE 2 : Seismic forces and moments on each of the principal axes of a member shall be obtained by adding 100% of the absolute value of the member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction (transverse) to 30% of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the first perpendicular direction (longitudinal).

Design Requirements for Single Span Bridges

A detailed seismic analysis is not required for single span bridges. However, the connections between the bridge span and the abutments shall be designed both longitudinally and transversely to resist the gravity reaction force at the abutment multiplied by the Acceleration Coefficient of the site. The minimum support lengths shall be as specified in Article 4.9.

Design Forces for Seismic Performance Category 1 & 2

The connection of the superstructure to the substructure shall be designed to resist a horizontal seismic force equal to 0.20 times the dead load reaction force in the restrained directions.

Design Forces for Seismic Performance Category 3

Design Forces for Structural Members and Connections

Seismic design forces specified in this subjection shall apply to:

(a) The superstructure, its expansion joints and the connections between the superstructure and the supporting substructure.
(b) The supporting substructure down to the base of the columns and piers but not including the footing, pile cap or piles.
(c) Components connecting the superstructure to the abutment.

Seismic design forces for the above components shall be determined by dividing the elastic seismic forces obtained from Load Case I and Load Case 2 of Article 4.4 by the appropriate Response Modification Factor of Article 3.6. The modified seismic forces resulting from the two load cases shall then be combined independently with forces from other loads as specified in the following group loading combination for the components. Note that the seismic forces are reversible (positive and negative) and the maximum loading for each component shall be calculated as follows:

Group Load = 1.0 (D+B+SF+E+EQM) (4-1)

where

D = dead load
B = buoyancy
SF = stream-flow pressure
E = earth pressure

EQM = elastic seismic force for either Load Case I or Load Case 2 of Article 4.4 modified by dividing by the appropriate R-Factor.

Each component of the structure shall be designed to withstand the forces resulting from each load combination and the additional requirements for Section 6, 7 and 8 of these Standards. For Service Load Design a 50% increase is permitted in the allowable stresses for structural steel and a 33.3% increase for reinforced concrete.

Design Forces for Foundations

Seismic design forces for foundations, including footing, pile caps, and piles shall be the elastic seismic forces obtained from Load Case I and Load Case 2 of Article 4.4 divided by the Response Modification Factor (R) specified below. These modified seismic forces shall then be combined independently with forces from other loads as specified in the following group loading combinations to determine two alternate load combinations for the foundations:

Group Load = 1.0 (D+B+SF+E+EQF) (4-2)

Where D, B, E and SF are defined in Article 4.7.1 and

EQF = the elastic seismic force for either Load Case 1 or Load Case 2 of Article 4.4 divided by half the R-Factor for the substructure (column or pier) to which it is attached.

EXCEPTION:

For pile bents the R-Factor shall not be divided by 2.

Each component of the foundation shall be designed to resist the forces resulting from each load combination according to the requirements of Section 6.

Abutments and Retaining Walls

The components (bearing, shear keys) connecting the superstructure to an abutment shall be designed to resist the forces specified in Article 4.7.1.

Design requirements for abutments are given in Article 6.3.2.

Design Displacements

Minimum bearing support lengths as determined in this section shall be provided for the expansion ends of all girders.

Seismic Performance Category 1 & 2

Bridges classified as SPC 1 & 2 shall meet the following requirements: Bearing seats supporting the expansion ends of girders, as shown in Figure 5, shall be designed to provide a minimum support length N (in or mm) measured normal to the face of an abutment or pier, not less than that specified below:

$$N = 8 + 0.02L + 0.08H \text{ (in)} \qquad (4\text{-}3A)$$

or

$$N = 203 + 1.67L + 6.66H \text{ (mm)} \qquad (4\text{-}3B)$$

where

L = length, in feet for Eq. 4-3A or meters for Eq. 4-3B, of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L shall be the sum of L1 and L2, the distances to either side of the hinge. For single span bridges L equals the length of the bridge deck. These lengths are shown in Fig. 5.

For abutments:

H = average height, in feet for Eq. 4-3A or meters for Eq. 4-3B, of columns supporting the bridge deck to the next expansion joint For columns and/or piers

H = column or pier height in feet for Eq. 4-3A or meters for Eq. 4-3B

For hinges within a span:

H = average height of the adjacent two columns or piers in feet for Eq. 4-3A or meters for Eq.4-3B.

Seismic Performance Category 3

The seismic design displacements shall be the maximum of these determined in accordance with Article 4.3 or those specified in Article 4.9.1.

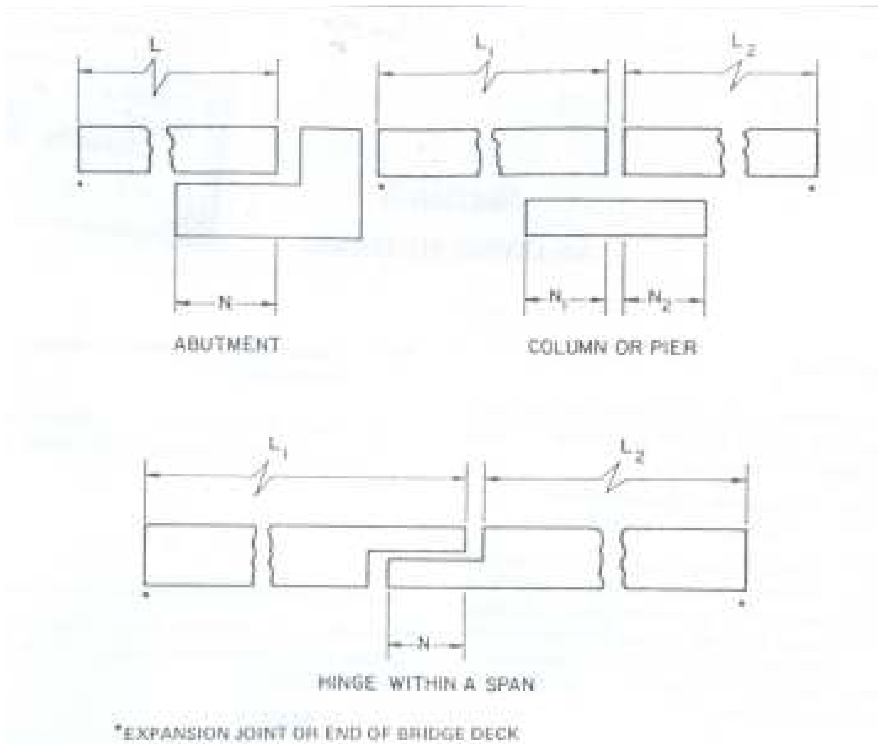
L
L1
L2
N
N1
N2
ABUTMENT
COLUMN OR PIER
HINGE WITHIN A SPAN
*EXPANSION JOINT OR END OF BRIDGE DECK

Figure 5 : Dimensions for Minimum Support Length Requirements

Analysis Methods

General

The requirements of this chapter shall control the seismic analysis of bridges prescribed in Article 4.2. Two analysis procedures are presented.

Procedure 1. Single-Mode Spectral Method
Procedure 2. Multimode Spectral Method

In both methods, all fixed column, pier or abutment supports are assumed to have the same ground motion at the same instant in time. At movable supports, displacements determined from the analysis prescribed in this section, which exceed the minimum requirements as specified in Articles 4.9.2 and 4.9.3, shall be used in design without reduction.

Elastic Seismic Response Coefficient and Spectrum

Elastic Seismic Response Coefficient:

Procedure 1

The elastic seismic response coefficient C_s used to determine the design forces is given by the dimensionless formula:

$$C_s = \frac{1.2AS}{T^{2/3}} \qquad (5\text{-}1)$$

Where:

A = the Acceleration Coefficient from Article 3.2,
S = the dimensionless coefficient for the soil profile characteristics of the site as given in Article 3.5,
T= the period of the bridge as determined in Article 5.3 or by other acceptable methods.

The value of C_s need not exceed 2.5A.

Elastic Seismic Response Spectrum

Procedure 2

The elastic seismic response coefficient for mode "m", C_{sm}, shall be determined in accordance with the following formula:

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \qquad (5\text{-}2)$$

where T_m = the period of the m^{th} mode of vibration.

The value of C_{sm} need not exceed 2.5A.

EXCEPTIONS:

1. For Soil Profile Type III soils, C_{sm} for modes other than the fundamental mode, which have periods less than 0.3 sec. may be determined in accordance with the following formula:

$$C_{sm} = A\,(0.8 + 4.0T_m) \qquad (5\text{-}3)$$

2. For structures in which any T_m exceeds 4.0 sec., the value of C_{sm} for that mode may be determined in accordance with the following formula:

$$Csm = \frac{3AS}{T_m^{4/3}} \qquad (5\text{-}4)$$

Single Mode Spectral Analysis Method-Procedure 1

The single mode spectral analysis method described in the following steps may be used for both transverse and longitudinal earthquake motions. Examples illustration their applications are given in the Commentary.

Step 1. Calculate the static displacements V_s (X) due to an assumed uniform loading P_o as shown in Figure 6. Abutment stiffness, if desired, can be incorporated by the procedure outlined in Article C5.3 of the Commentary.

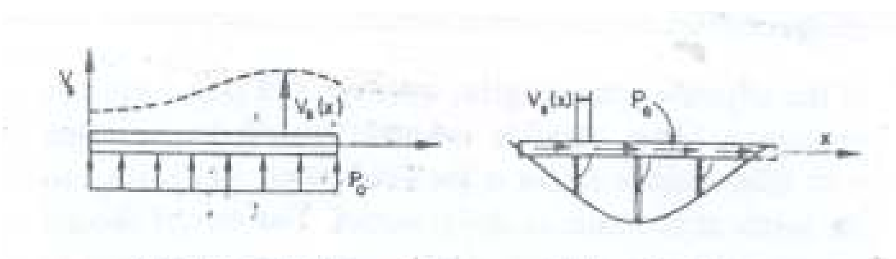

Figure 6: Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading

The uniform loading P_o is applied over the length of the bridge; it has units of force/unit length and is arbitrarily set equal to 1. The static displacement V_s(X) has units of length.

Step 2. Calculate factors α, β, and γ from the expressions

α = Vs(x)dx (5-5)

β = w(x)Vs(x)dx (5-6)

γ = w(x)vs(x)^{2}dx (5-7)

where w(x) is the weight of the dead load of the bridge superstructure and tributary substructure (see Supplement A) (force/unit length). The computed factors, α, β, and γ have units of (length^2), (force length), and (force length^2), (force length), and (force length^2), respectively.

The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included. (Generally, the inertial effects of live loads are not included in the analysis; however, the design of bridges having high live to traffic congestion is likely to occur should consider the probability of large live load being on the bridge during an earthquake).

Step 3. Calculate the period of the bridge using the expression:

$$T = 2\prod\sqrt{\frac{\gamma}{P_o g \alpha}} \qquad (5\text{-}8)$$

Where: g = acceleration of gravity (length/time^2).

Step 4. Calculate the equivalent static earthquake loading p_e (x) from the expression:

$$p\hat{e}(\mathrm{x}) = \frac{\beta C_s}{\gamma} w(x) V_s(x) \qquad (5\text{-}9)$$

Where:

C_s = the dimensionless elastic seismic response coefficient given by Eq. (5-1)

$P_e(x)$ = the intensity of the equivalent static seismic loading applied to represent the primary mode of vibration (force/unit length)

Step 5. Apply loading pe(x) to the structure as shown in Fig. 7 and determine the resulting member forces and displacements for design.

Multimode Spectral Analysis Method – Procedure 2

The multimode response spectrum analysis should be performed with a suitable space frame linear dynamic analysis computer program. Currently available computer programs are included in the Commentary.

General

The multimode spectral analysis method applies to bridges with irregular geometry which includes coupling in the three coordinate directions within each mode of vibration. These coupling effects make it difficult to categorize the modes into simple longitudinal or transverse modes of vibration will in general contribute to the total response of the structure. A computer program with space frame dynamic analysis capabilities should be used to determine coupling effects and multimodal contributions to the final response. Motions applied at the supports in any one of the two horizontal directions will produce forces along both principal axes of the individual members because of the coupling effects. For curved structures, the longitudinal motion shall be directed along a chord connecting the abutments and the transverse motion shall be applied normal to the chord. Forces due to longitudinal and transverse motions shall be combined as specified in Article 4.4.

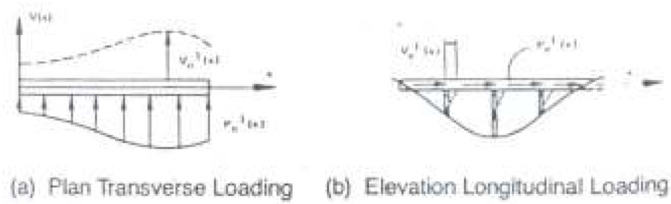

Figure 7: Bridge deck subjected to Equivalent Transverse and Longitudinal Seismic Loading

Mathematical Model

The bridge should be modeled as a three-dimensional space frame with joints and nodes selected to realistically model the stiffness and inertia effects of the structure. Each joint or node should have six degrees of freedom, three transnational and three rotational. The structural mass should be lumped with a minimum of three transnational inertial terms.

The mass should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included. (Generally, the inertial effects of live loads are not included in the analysis; however, the design of bridges having high live to dead load rations located in metropolitan areas where traffic congestion is likely to occur should consider to probability of a large live load being on the bridge during an earthquake.)

(A) Superstructure

The superstructure should, as a minimum, be modeled as a series of space frames members with nodes at such points as the span quarter points in addition to joints at the ends of each span. Discontinuities should be included in the superstructure at the expansion joints and abutments. Care should be taken to distribute properly the lumped mass inertia effects at these location. The effect of earthquake restrainers at expansion joints may be approximated by superimposing one or more linearly elastic members having the stiffness properties of the engaged restrainer units.

9.5.4.3 (B) Superstructure

The intermediate columns or piers should also be modeled as space frame members. Generally, for short, stiff columns having lengths less than one-third of either of the adjacent span lengths, intermediate nodes are not necessary. Long, flexible columns should be modeled with intermediate nodes at the third points in addition to the joints at the ends of the columns. The model should consider the eccentricity of the columns with respect to the superstructure. Foundation conditions at the base of the columns and at the abutments may be modeled using equivalent linear sprint coefficients.

Mode Shapes and Periods

The required periods and mode shapes of the bridge in the direction under consideration shall be calculated by established methods for the fixed base condition using the mass and elastic stiffness of the entire seismic resisting system.

Multimode Spectral Analysis

The response should, as a minimum, include the effects of a number of modes equivalent to three times the number of spans up to a maximum of 25 modes.

Member Forces and Displacements

The member forces and displacements can be estimated by combining the respective response quantities (e.g., force, displacement, or relative displacement) from the individual modes by the Square Root of the Sum of the Squares (SRSS) method. The member forces and displacements obtained using the SRSS method combining modes is generally adequate for most bridge systems because they have well-separated modes of vibration characterized by significant modes of vibration characterized by significant differences in the natural periods of each of the modes. For bridges with closely spaced modes (within 10%), other more appropriate methods of combining or weighting the individual contributions should be considered to obtain the total final response (see commentary).

Foundation and Abutment Design Requirements

General

This section includes only those foundation and abutment requirements that are specifically related to seismic resistant construction. It assumes compliance with all the basic requirements necessary to provide support for vertical loads and lateral loads other than those due to earthquake motions. These include, but are not limited to provisions for the extent of foundation investigation, fills, slope stability, bearing and lateral soil pressures, drainage, settlement control, and pile requirements and capacities.

Seismic Performance Category 1 & 2

There are no special seismic design requirements for this category.

Seismic Performance Category 3

Foundation and abutment seismic design requirements for SPC 3 are given in the following subsections.

Investigation

In addition to the normal site investigation report, the Engineer may require the submission of a report which shall include the results of an investigation to determine potential hazards and seismic design requirements related to (1) slope instability, (2) liquefaction, (3) fill settlement, and (4) increases in lateral earth pressure, all as a result of earthquake motions. Seismically – induced slope instability in approach fills or cuts may displace abutments and lead to significant differential settlement and structural damage. Fill settlement and abutment displacements due to lateral pressure increase may lead to bridge access problems and structural damage. Liquefaction of saturated cohesionless fills or foundation soils may contribute to slope and abutment instability, and lead to a loss of foundation bearing capacity and lateral pile support. Liquefaction failures of the above types have led to many bridge failures during past earthquakes (a discussion on liquefaction potential assessment is provided in the Commentary).

Foundation Design

For the load combinations specified in Article 4.7.2 the soil strength capable of being mobilized by the foundations shall be established in the site investigation report. Because of the dynamic cyclic nature of seismic loading, the ultimate capacity of the foundation supporting medium should be used in conjunction with these load combinations. Due consideration shall be given to the magnitude of the seismically-induced foundation settlement that the bridge can withstand.

Transient foundation uplift or rocking involving separation from the subsoil of up to one-half of the contact area of foundation pile group or up one-half of the contract area of foundation footings is permitted under seismic loading, provided that foundation soils are not susceptible to loss of strength under the imposed cyclic loading.

General comments on soil strength and stiffness mobilized during earthquakes, foundation uplift, lateral loading of piles, soil-structure interaction and foundation design in environments susceptible to liquefaction are provided in the Commentary.

Special Pile Requirements

The following special pile requirements are in addition to the requirements for piles in other applicable specifications.

Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities, required to resist seismic loads shall be determined by means of the design criteria established in the site investigation report, Note that the ultimate capacity of the piles should be used in designing for seismic loads.

All piles shall be adequately anchored to the pile footing or cap. Concrete piles shall be anchored by embedment of sufficient length of pile reinforcement (unless special anchorage is provided) to develop uplift forces but in no case shall this length be less than the development length required for the reinforcement. Each concrete-filled pipe pile shall be anchored by at least 4 reinforcing steel dowels with a minimum steel ratio of 0.01 embedded sufficiently as required for concrete piles. Timber and steel piles, including unfilled pipe piles, shall be provided with anchoring devices to develop all uplift forces adequately but in no case shall these forces be less than 10% of the allowable pile load.

All concrete piles shall be reinforced to resist the design moments, shears, and axial loads. Minimum reinforcement shall be not less than the following:

1. Cast-in-Place Concrete Piles. Longitudinal reinforcing steel shall be provided for cast-in-place concrete piles in the upper one-third (8 ft. or 2.4 m minimum) of the pile length with a minimum steel ratio of 0.005 provided by at least 4 bars. Spiral reinforcement or equivalent ties of 1/4 in. (6.3 mm) diameter or larger shall be provided at 9 in. (229 mm) maximum pitch, except for the top 2 ft (610 mm) below the pile cap reinforcement where the pitch shall be 3 in. (76 mm) in maximum.

2. Pre-cast Piles. Longitudinal reinforcing steel shall be provided for each pre-cast concrete pile with a minimum steel ratio of 0.01 provided by at least 4 bars. Spiral reinforcement or equivalent ties of No. 3 bars or larger shall be provided at 9 in. (229 mm) maximum pitch, except for the top 2 ft (610 mm) below the pile cap reinforcement where the pitch shall be 3 in. (76 mm) maximum.

3. Pre-cast – pre-stressed piles. Ties in pre-cast pre-stressed piles shall conform to the requirements of pre-cast piles.

Abutments

Free-Standing Abutments

For free-standing abutments or retaining walls which may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), the pseudo-static Mononobe-Okabe method of analysis (see Commentary) is recommended for computing lateral active soil pressures during seismic loading. A seismic coefficient equal to one-half the acceleration coefficient (k,, = A/2) is recommended. The effects of vertical acceleration may be omitted. Abutments should be pro portioned to slide rather than tilt, and provisions should be made to accommodate small horizontal seismically induced abutment displacements when minimal damage is desired at abutment supports. Abutment displacements of up to 10A. in. (254 mm) may be expected.

The seismic design of free-standing abutments should take into account forces arising from seismically-induced lateral earth pressures, additional forces arising from wall inertia effects and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely (e.g., elastomeric bearings).

For free-standing abutments which are restrained from horizontal displacement by anchors or batter piles, the magnitudes of seismically-induced lateral earth pressures are higher than those given by the Mononobe-Okabe method of analysis. As a first approximation, it is recommended that the maximum lateral earth pressure be computed by using a seismic coefficient k,, = 1.SA in conjunction with the Mononobe-Okabe analysis method.

Monolithic Abutments

For monolithic abutments where the abutment forms an integral part of the bridge superstructure, maximum earth pressures acting and the abutment may be assumed equal to the maximum longitudinal earthquake force transferred from the superstructure to the abutment. To minimize abutment damage, the abutment should be designed to resist the passive pressure capable of being mobilized by the abutment backfill, which should be greater than the maximum estimated longitudinal earthquake force transferred to the abutment. It may be assumed that the lateral active earth pressure during seismic loading is less than the superstructure earthquake load.

When longitudinal seismic forces are also resisted by piers or columns, it is necessary to estimate abutment stiffness in the longitudinal direction in order to compute the proportion of earthquake load transferred to the abutment (see Commentary Article C5.4.2).

Structural Steel

General

Design and construction of structural steel columns and connections shall conform to the requirements of other chapters and to the additional requirements of this Section. Either Service Load or Load Factor design may be used. If Service Load design is used the allowable stresses are permitted to increase by 50%.

Seismic Performance Category 1 & 2

No consideration of seismic forces is required for the design of structural components except for the design of the connection of the superstructure to the substructure as specified in Article 4.6.

Seismic Performance Categories 3

Where axial and flexural stresses are determined by considering secondary bending resulting from the design P-delta effects (moments induced by the eccentricity resulting from the seismic displacements and the column axial force), all axially loaded members may be proportioned in accordance with other chapters.

EXCEPTIONS:

1. The effective length factor, K, in the plane of bending may be assumed to be unity in the calculation of Fa, Fe, Fcr, or Fe.
2. The coefficient Cm is computed as for the cases where joint translation is prevented.

Reinforced Concrete

General

Design and construction of cast-in-place monolithic reinforced concrete columns, pier footings and connections shall conform to the requirements of other chapters of this Manual and to the additional requirements of this section. Either Service Load or Load Factor design may be used. If Service Load design is used the allowable stresses are permitted to increase by $33^{1}/_{3}$%.

Seismic Performance Category 1 & 2

No consideration of seismic forces is required for the design of structural components except for the design of the connection of the superstructure to the substructure as specified in Article 4.6.

Seismic Performance Category 3

For bridges classified as SPC 3 the minimum transverse reinforcement requirements at the top and bottom of a column shall be as required in Article 8.4.1(D). The spacing of the transverse reinforcement shall be as required in Article 8.4.1 (E) except that the maximum spacing is permitted to increase to 6 in. (152 mm).

COMMENTARY

SECTION 1 – INTRODUCTION

DESIGN PHILOSOPHY

Conceptually there are two seismic design approaches currently in use and both employ a "force design" concept. These are the current New Zealand and Caltrans criteria and are discussed in detail in references 1 and 2, respectively.

In the New Zealand Code, which accepts that it is uneconomical to design a bridge to resist a large earthquake elastically, bridges are designed to resist small-to-moderate earthquakes in the elastic range. For large earthquakes the design philosophy is that bridges be ductile where possible. Flexural plastic hinging in the columns is acceptable but significant damage to the foundations and other joints is not. Consequently, as a second step in the design process, forces resulting from plastic hinging in all columns are determined and the capacities of connections to columns are checked to determine if they are able to resist these forces. Hence, critical elements in the bridge are designed to resist the maximum forces to which they will be subjected in a large earthquake.

In the Caltrans approach the member forces are determined from an elastic design response spectrum for a maximum credible earthquake. The design forces for each component of the bridge are then obtained by dividing the elastic forces by a reduction factor(Z). The Z- Factor is 1.0 and 0.8, respectively, for hinge restrainers and shear keys. These components are therefore designed for expected and greater- than- expected (in the case of shear keys) elastic forces resulting from a maximum credible earthquake. Well-confined ductile columns are designed for lower-than-expected forces from an elastic analysis a Z varies from 4 to 8. This assumes that the columns can deform plastically when the seismic forces exceed these lower design forces. The end result is similar to the New Zealand approach although the procedures are quite different.

In assessing bridge failures of past earthquakes in Alaska, California and Japan, many loss-of –span type failures are attributed in part to relative displacement effects. Relative displacements arising from out –of-phase motion of different parts of a bridge, from lateral displacement and/or rotation of the foundations and differential displacement of abutments. Therefore in developing the Standards the design displacements and forces were considered equally important. Thus minimum support lengths at abutments, columns and hinge seats are specified, and for bridges in areas of high seismic risk ties between non continuous segments of a bridge are specified. Special attention to the problem of relative displacements is required for bridges with high columns or piers.

The methodology used in the Standards is, in part, a combination of the Caltrans and New Zealand " force design" approaches but also addresses the relative displacement problem. The complexity of the methodology increases as the seismic intensity of an area increases. Four additional concepts are included in the Standards that are not included in either the Caltrans or New Zealand approach. First, minimum requirements are specified for support lengths of girders at abutments, columns and hinge seats to account for some of the important relative displacement effects that cannot be calculated by current state-of –the art methods. A somewhat similar requirement is included in the latest Japanese bridge criteria. Second, member design forces are calculated to account for the directional uncertainty of earthquake motions and the simultaneous occurrence of earthquake motions and the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. Third, design requirements and forces for foundations are intended to minimize foundation damage which is not readily detectable. Fourth, a basic premise in developing the Standards was that they be applicable to all parts of the United States. In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, four seismic performance categories (SPC) were defined. The three categories permit variation in the design requirements and analysis methods in accordance with the seismic risk associated with a particular bridge location. Bridges classified as SPC 1 & 2 are designed for the lowest level of seismic performance.

For bridges classified as SPC 1 & 2, prevention of super-structure collapse is all that was deemed necessary for their level of seismic exposure. The requirements for these bridges are minimal and specify the support lengths for girders at abutments, columns and expansion joints, and that the design of the connections of the superstructure to the substructure be for 0.20 times the dead load reaction forces.

For bridges classified as SPC 3 the approach used is similar to that of Caltrans where elastic member forces are determined from a single- mode spectral method of analysis. Design forces for each components are obtained by dividing the elastic forces by a response modification factor (R). For connections at abutments, columns and expansion joints the R-Factor is either 1.0 or 0.8; therefore these components are designed for expected or greater-than-expected elastic forces. For columns and piers the R-Factor varies between 2 and 5 resulting in design forces lower than predicted by the elastic analysis. Therefore the columns are expected to yield when subjected to the forces of the design earthquake. This yielding in turn implies relative distortions of the structural system that must be considered in assessing the adequacy of the final bridge design. Design requirements to ensure reasonable ductility capacity of columns for bridges classified as SPC 3 are specified. Foundations are designed for twice the seismic design forces of a column or pier.

SEISMIC GROUND MOTION ACCELERATIONS

Although the probability is quite small, it is possible that in highly seismic areas near active faults to ground motions could exceed the design earthquake ground shaking. This possibility occurs because the seismic risk maps used herein were developed on a macro rather than micro scale and the influence of many smaller but active faults was not considered. Therefore, for locations near active faults it is recommended that a qualified professional be consulted to determine an appropriate value for the Acceleration Coefficient, A.

SOIL EFFECTS ON GROUND MOTION

It is generally recognized that the effects of local soil conditions on ground motion characteristics should be considered in structural design. Three fundamentally different approaches have been used:

- The first approach was based on the concept of potential resonance of a structure with the underlying soil. In the SEAOC building seismic requirements the seismic site-structure resonance coefficient varies from 1.0 to 1.5 depending on the ratio of the fundamental building period to the characteristic site period.

- In a second approach, the computer program SHAKE was used by Caltrans to develop soil amplification factors for its design criteria. The program analyzes a one-dimensional soil column. The Caltrans approach is limited because only vertically propagating one-dimensional soil effects are considered and several parameters which could have significant effects are not considered. These parameters include surface waves, oblique transmission of waves through the soil and the effects of reflection and refraction at the interfaces of different material layers.

- For the third approach representative ground motion spectral shapes were modified in ATC – 3 -06 [4] to determine corresponding values of effective peak ground acceleration and smoothed spectral shapes for there typical site conditions. These modifications were based on a study of ground motions recorded at locations with different site conditions and the exercise of experienced judgment in extrapolating beyond the data base. Coefficients were developed for each three typical soil conditions.

The ATC-3-06 approach for considering soil effects on ground motion is used in these standards and is discussed in more detail in Article C3.2.

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COMMENTARY

SECTION 3 – GENERAL REQUIREMENTS

C3.1 APPLICABILITY OF STANDARDS

The Standards present seismic design and construction requirements applicable to the majority of the high-way bridges to be constructed in Uganda. Bridges not covered by these provisions probably constitute 5 to 15% of the total number of bridges designed.

Special seismic design provisions would not be required for buried type structures. However, this decision may need reconsideration as more research data on the seismic performance of this type of structure becomes available.

The standards specify minimum requirements. More sophisticated design and /or analysis techniques may be utilized if deemed appropriate by the design engineer.

For bridge types not covered by these Standards the following factors should be considered.

1. The recommended elastic design force levels of the Standards should be applicable because force levels are largely independent of the type of bridge structure, although a project may warrant a site –specific study to determine appropriate design force levels. If the site is near an active fault zone it is also recommended that qualified professionals familiar with local conditions be consulted.

   It should be noted that the elastic design force levels of the standards are part of a design philosophy described in the introduction to this commentary. The appropriateness of both the design force levels and the design philosophy must be assessed before they are used for bridges that are not covered by these Standards.

2. The Multimode Spectral Procedure described in Article 5.4 should be considered, especially if the Acceleration coefficient for the bridge site is greater than 0.20. The designer should consider the pros and cons of using elastic and /or inelastic methods of time history analysis for larger and more complex types of bridges. If these methods are used, appropriate time histories must be determined as part of the site specific study. It is recommended that at least three ground motion time histories be used in this type of analysis.

3. Design displacements are as important as design forces and, where possible, the design methodology should consider displacements arising from the effects discussed in Article C4.9.

4. If a design methodology similar to that used in these standards is deemed desirable, the design requirements of Sections 6,7 and 8 should be used to ensure compliance with the design philosophy.

C3.2, C3.5 and C5.2 ACCELERATION COEFFICIENT, SITE EFFECTS AND ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRUM

The ground motion coefficient to be used with the Standards was originally developed as part of a similar but even more extensive study for buildings entitled " Tentative Provisions for the Development of Seismic regulations for Buildings" (ATC- 3-06). Since the ground motion coefficient and associated elastic response spectrum are independent of the structural system, the ATC-3-06 values are used in these Standards.

Two coefficients and two corresponding maps were developed in the ATC -3-06 provisions. The two coefficients are the Effective Peak Acceleration Coefficient, A_a, and the Effective Peak Velocity – Related Acceleration Coefficient $A_{v.}$

A major policy decision in the development of the bridge Standards was to use only the Effective Peak Velocity – Related Acceleration coefficient A_v would be used rather than the county – by county maps of ATC-3-06. The decision to use only one coefficient was made to simplify the bridge guidelines. The decision to use a contour rather than county-by-county map was made because it was felt that the local jurisdictional problems with buildings were not a major importance for bridges.

The following is pertinent test extracted from the commentary of ATC-3-06 provisions. For a more complete discussion see the ATc-3-06 Commentary.

A. INTRODUCTION

It must be emphasized at the outset that the specification of earthquake ground shaking cannot be achieved solely by following a set of scientific principles. First, the causes of earthquakes are still not well understood and experts do not fully agree as to how available knowledge should be interpreted to specify ground motion for use in design. Second, to achieve workable bridge design provisions it is necessary simplify the enormously complex matter of earthquake occurrence and ground motions. Finally, any specification of a design ground shaking involves balancing the risk of that motion occurring against the cost to society of requiring the structures be designed to withstand that motion. Hence judgment, engineering experience, and political wisdom are as necessary as scientific knowledge. In addition, it must be remembered that design ground shaking alone does not determine how a bridge will perform during a future earthquake; there must be a balance of the specified shaking with the rules used to assess structural resistance to that shaking.

The recommended reorganization maps and seismic design coefficients and spectra are the work of several committees and are based upon the best scientific knowledge available in 1976, adjusted and tempered by experience. The following sections explain the bases for the various recommendations, as a guide both to the user of the provisions and to those who will improve the provisions in the future. It is expected that the maps and coefficients will change with time, as the profession gains more knowledge about earthquakes and their resulting ground motions and as society gains greater insight into the process of establishing acceptable risk.

B. POLICY DECISIONS

The recommended ground shaking regionalization maps are based upon two major policy decisions. The first is a departure from past practice in the United States whereas the second is a currently accepted practice.

The first policy decision was that the probability of exceeding the design ground shaking should, as a goal, be assumed to b equal in all parts of the country. The desirability of this goal is accepted within the profession; however, there is some disagreement as to the accuracy of estimates of probability of ground motion as determined from current knowledge and procedures. Use of a contour map based on uniform probability of occurrence is a departure from the use of the present zone maps which are based on estimates of maximum ground shaking experienced during the recorded historical period without and consideration of how frequently such motions might occur. It is also recognized that the real concern is with the probability of structural failures and resultant casualties and that the geographical distribution of that the geographical distribution of that probability is not necessarily the same as the distribution of the probability of exceeding some ground motion. Thus the goal as stated is the most workable one for the present but not necessarily the ideal one for the future.

The second policy decision was that the regionalization maps should not attempt to microzone. In particular there was to be no attempt to locate actual faults on the regionalization maps, and variations of ground shaking over short distances – about 10 miles or less – were not to be considered. Any such microzoninig must be done by qualified professionals who are familiar with localized conditions. Many local jurisdictions may find if expedient to undertaken microzoning.

C. DESIGN EARTHQUAKE GROUND MOTION

The previous sections have discussed design ground shaking in general without being specific as to the meaning of the phrase. To state the concept rather than a precise definition, the design ground shaking for a location is the ground motion that the engineer should consider when designing a structure to provide a specified degree of protection for life safety and to prevent collapse.

At present, the best workable tool for describing design ground shaking is a smoothed elastic response spectrum for single degree-of-freedom systems. Such a spectrum provides a quantitative description of both the intensity and frequency content of a ground motion. Smoothed elastic response spectra for 5% damping were used as a basic tool for the development of the regionalization maps and for the inclusion of the effects of local ground conditions. In effect, the first policy decision was reinterpreted to mean the probability of exceeding the ordinates of the design elastic response spectrum for all structural periods for a given location would be roughly equal. Again, this concept should be looked upon as a gradual goal, and not one that can be strictly met on the basis of present knowledge.

This should not be interpreted to mean that a structure can necessarily be designed for the forces implied by an elastic response spectrum. The design philosophy associated with the elastic response spectrum is at least as important as the level of the response spectrum.

A smoothed elastic response spectrum is not necessarily the ideal means for describing the design ground shaking. A time history analysis would be better, but a single time history motion generally is not adequate. It would be better to use a set of three or more acceleration time histories with an average elastic response spectrum similar to the design spectrum. This approach may be desirable for structures of special importance but is not feasible for the vast majority of structures. This discussion is intended to emphasize that the design ground shaking is not a single motion, but rather a concept that encompasses a family of motions having the same over all intensity and frequency content but differing in some potentially important details of the time sequences of the motions.

A significant deficiency of the response spectrum is that is does not by itself include the duration of the shaking. The extent that duration affects elastic response is accounted for by the spectrum. However, the major effect of duration is upon possible loss of strength once a structure yields. Duration effects have not been explicitly considered in drawing up the recommended provisions, although in a general way it was envisioned that the design ground shaking might have a duration of 20 to 30 sec. The possibility that the design motion might be longer in highly seismic areas and shorter in less seismic areas was one of the considerations which influenced the design provisions for the various Seismic Performance Categories.

D. GROUND MOTION PARAMETERS

In developing the design provisions for buildings, two parameters were used to characterize the level of design ground shaking. These parameters are called the Effective peak Acceleration (EPA) and the Effective Peak Velocity-related Acceleration (EPV). These parameters do not at present have precise definitions in physical terms but their significance may be understood from the following paragraphs.

The meaning of EPA and EPV is better understood if they are considered as normalizing factors for construction of smoothed elastic response spectra fro ground motions of smoothed elastic response spectra for ground motions of normal duration (see Figure 8.) The EPA is proportional to spectral acceleration ordinates for periods in the range of 0.1 to 0.5 sec., while the EPV is proportional to spectral velocity ordinates at a period of about 1 sec. The constant of proportionality (for a 5% damping spectrum) is set at a standard value of 2.5 in both cases.

For purposes of computing the lateral force coefficient in Article 3.3 of the ATC-3-06 provisions, EPA and EPV are replaced by dimensionless acceleration coefficients A_a and A_v, respectively. The coefficient A_a is numerically equal to EPA when EPA is expressed as a decimal fraction of the acceleration of gravity; e.g., if EPA = 0.20 g the A_a = 0.20. The coefficient A_v is proportional to EPV as explained in Article 1.4(F) of the ATC-3-06 Commentary.

Compared with spectral shapes from studies conducted by Blume, Newmark, and Mohraz. It was considered appropriate to simplify the curves to a family of three by combining the spectra for rock and stiff soil conditions; the normalized spectral curves are shown in Figure 10. The curves in this figure thus apply to the following three soil conditions.

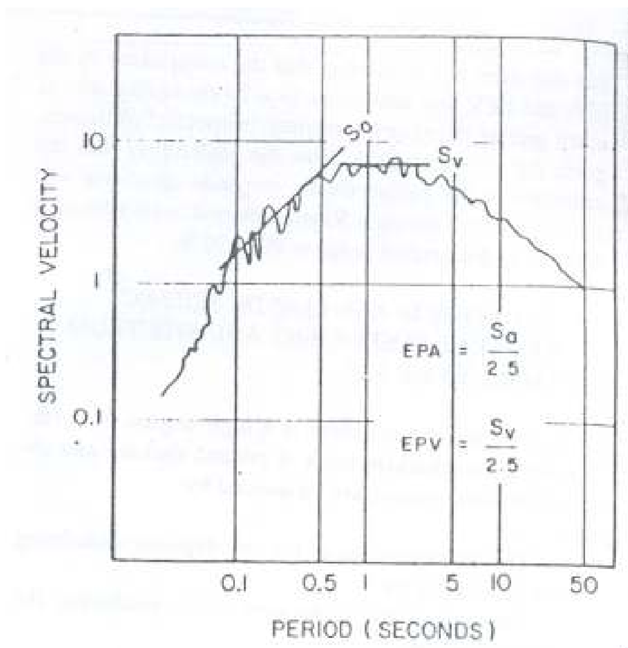

Figure 8. Schematic Representation Showing How Effective Peak Acceleration and Effective Peak Velocity Are Obtained From a Response Spectrum.

E. RISK ASSOCIATED WITH THE CONTOUR MAP

The probability that the recommended EPA and EPV at a given location will not be exceeded during a 50-year period is estimated to be about 90%. At present, this probability cannot be estimated precisely. Moreover, since the maps were adjusted and smoothed by the committee after consultation with seismologists, the risk may not be the same at all locations. It is believed that this probability of the design ground motion not being exceeded is in the range of 80% to 90%. The use of a 50-year interval to characterize the probability is a rather arbitrary convenience, and does not imply that all structures are thought to have a useful life of 50 years.

The probability that an ordinate of the design elastic response spectrum will not be exceeded, at any period, is approximately the same as the probability that the EPA and the EPV will not be exceeded. The veracity of this statement lies in the fact that the uncertainty in the EPA and EPV that will occur in a future earthquake is much greater than the uncertainty in spectral ordinates, given the EPA and EPV. Thus the probability that the ordinates of the design elastic response spectrum will not be exceeded during a 50-year interval is also roughly 90%, or in the general range of 80 to 95%.

F. SITE EFFECTS AND ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRUM (ARTICLE 3.5 AND 5.2)

At the present time there is a high degree of agreement that the characteristics of ground shaking and the corresponding spectra are influenced by:

1. The characteristics of the soil deposits underlying the proposed area.
2. The magnitude of the earthquake producing the ground motions.
3. The source mechanism of the earthquake producing the ground motions.
4. The distance of the earthquake from the proposed site and the geology of the travel path.

While it is conceptually desirable to include specific consideration of all four of the factors listed above it is not possible to do so at the present time because of lack of adequate data. Sufficient information is available to characterize in a general way the effects of specific soil conditions on effective peak acceleration and spectral shapes. The effects of the other factors are so little understood at this time that they are often not considered in spectral studies.

The present recommendations therefore only consider effects of site conditions and distance from the seismic source zone. At such times that potential effects of other significant parameters can be delineated and quantified, the current recommendations can be modified to reflect these effects.

Thus, the starting points in the development of the ground motion spectra are the seismic design regionalization maps that express, by contours, the EPA and the EPV that would be developed on firm ground.

SITE EFFECTS

The fact that the effects of local soil conditions on ground motion characteristics should be considered in structural design has long been recognized in many countries of the world. Most countries considering these effects have developed different design criteria for several different soil conditions. Typically these criteria use up to four different soil conditions. In the early part of the ATC-3-06, study consideration was given to four different conditions of local site geology.

On the basis of available data, the following four conditions were selected:

1. Rock-of any characteristic, whether it be shale like or crystalline in nature. In general, such material is characterized by a shear wave velocity greater than about 2,500 ft/sec (762 m/sec)
2. Stiff soil conditions or firm ground-including any site where soil conditions or firm ground-including any site where soil depth is less than 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
3. Deep cohesionless or stiff clay soil conditions – including sites where the soil depth exceeds about 2,500 ft (762 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
4. Soft to medium-stiff clays and sands- characterized by several tens of feet of soft to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

EFFECTIVE PEAK ACCELERATIONS FOR DIFFERENT SITE CONDITIONS

The values of EPA for rock conditions were first modified to determine corresponding values of EPA for the three other site conditions outlined above. This modification was based on a statistical study of peak accelerations developed at locations with different site conditions and the exercise of judgment to extrapolate beyond the database.

After evaluating these effects and rounding out the results obtained, the values of EPA were modified as follows. For the first three soil types (rock, shallow stiff soils and deep cohesionless or stiff clay soils) there is no reduction. For the fourth soil type (soft to medium-stiff clays) a reduction factor of 0.8 is used for all seismcity index areas. It should be pointed out that statistical data show that the reduction effect is not constant for all ground motion levels and that the value of the reduction factor is generally smaller than is recommended here.

SPECTRAL SHAPES

Spectral shapes representative of the different soil conditions discussed above were selected on the basis of a statistical study of spectral shapes developed on such soils close to the seismic source zone in past earthquakes. The mean spectral shapes determined directly from the study by Seed et al. based on 104 records, primarily from earthquakes in the Western United States, are shown in Figure 9. These spectral shapes were also compared with spectral shapes from studies conducted by Blume, Newmark, and Mohraz. Ti was considered appropriate to simplify the curves to a family of three by combining the spectra for rock and stiff soil conditions; the normalized spectral curves are shown in Figure 10. The curves in this figure thus apply to the following three soil conditions.

Soil Profile Type1: Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/sec (762 m/sec); or stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Soil Profile Type II: Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 ft (61m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Soil Profile Type III: Soft to medium-stiff clays and sands, characterized by 30 ft (9.1m) or more of soft to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

Ground motion spectra for 5% damping for the different map areas are thus obtained by multiplying the normalized spectral values shown in Figure 10 by the appropriate EPA and by the correction factor of 0.8 if Soil Profile III exists. The resulting ground motion spectra for an EPA of 0.4 are shown in Figure 11. The spectra from Figure 11 are shown in Figure 12 plotted in tripartite form. It can be readily seen in Figure 12 that for all soil conditions the response spectra for periods near 1 sec. are horizontal or equivalent to a constant spectral velocity. It should also be noted that these spectra are modified as discussed in the following section before they are used in the design provisions. On the basis of studies of spectral shapes conducted by Blume and Newmark, spectra for 2% damping may be obtained by multiplying the ordinates of Figure 10 by a factor of 1.25.

Spectra for vertical motions may be determined with sufficient accuracy by multiplying the ordinates of the spectra for horizontal motions by a factor of 0.67.

ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRA

The equivalent lateral force method of design requires that a horizontal force be accommodated in the structural design. The magnitude of this force is a function of several parameters including the Acceleration Coefficient, the type of site soil profile, and the fundamental period of the structure.

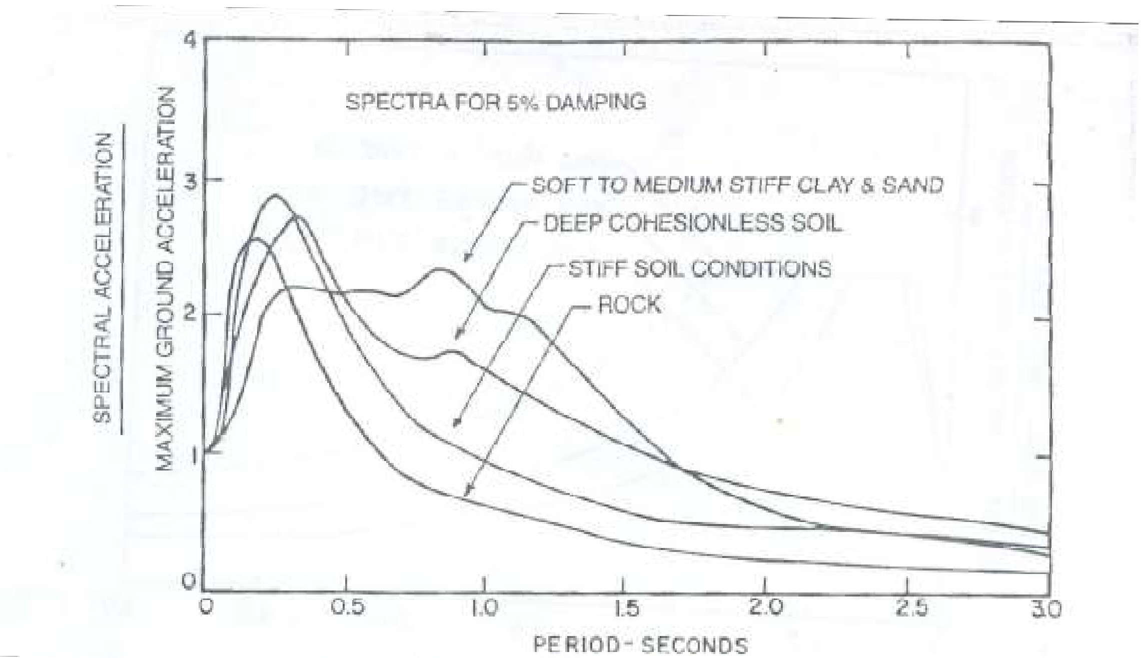

Figure 9 : Average Acceleration Spectra for Different Site Conditions (after Seed, et al., 1976)

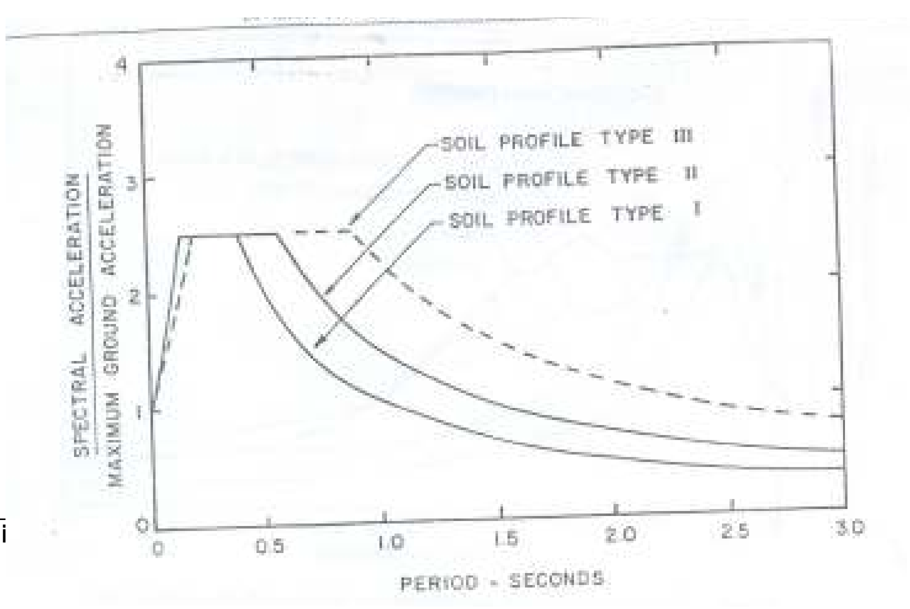

Figure 10 : Normalized Response Spectra

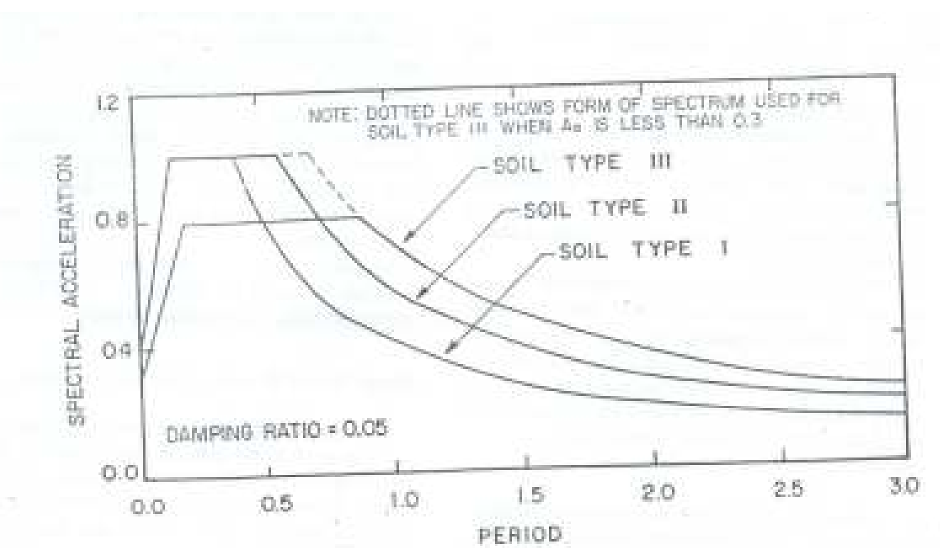

Figure 11 : Ground Motion Spectra for A = 0.4

For use in a design provision or code it is distinctly advantageous to express the later design force coefficient in as simple a manner as possible. The recommended procedure for determining the lateral design force coefficient Cs is given in Article 5.2 as follows:

$$C_s = \frac{1.25AS}{T^{2/3}} \qquad (5\text{-}1)$$

The value of Cs need not exceed 2.5A for Type I, II or III soils. The soil coefficient S is given in Table 2. the use of a simple soil factor in Eq. 5-1 directly approximates the effect of local site conditions on the design requirements. This direct method eliminates the need for the estimation of a predominant site period and the computation of a soil factor based on the site period and the fundamental period of the bridge.

This concludes the text abstracted from the Commentary of the ATC-3-06 provisions.

It is apparent from the discussion on spectral shapes in the foregoing paragraphs and Figures 10 that the recommended elastic acceleration response spectrum decreases approximately as 1/T for longer periods. However, because of the concerns associated with inelastic response of longer period bridges it was decided that the ordinates of the design coefficients and spectra should not decrease as rapidly as 1/T but should be proportional to $1/T^{2/3}$ in Eqs.5-1 and 5-2.

A comparison of the spectra resulting form Eqs.5-1 and 5-2 and those of the ATC-2-06 recommended elastic acceleration response spectra is given in Figure 13. The PEP decided that the elastic seismic response coefficient and spectra should be approximately 50% greater at a period of 2 sec. For the stiff soil condition than would be obtained by direct use of the recommended elastic acceleration response spectra. This increase should gradually decrease as the period of the bridge shortens. The two major reasons for introducing this conservatism in the design of long period bridges is:

1. The fundamental period of a bridge increases as the column height increases, the span length increases and the number of columns per bent decreases. Hence the longer the period the more likely those high ductility requirements will be concentrated in a few columns.

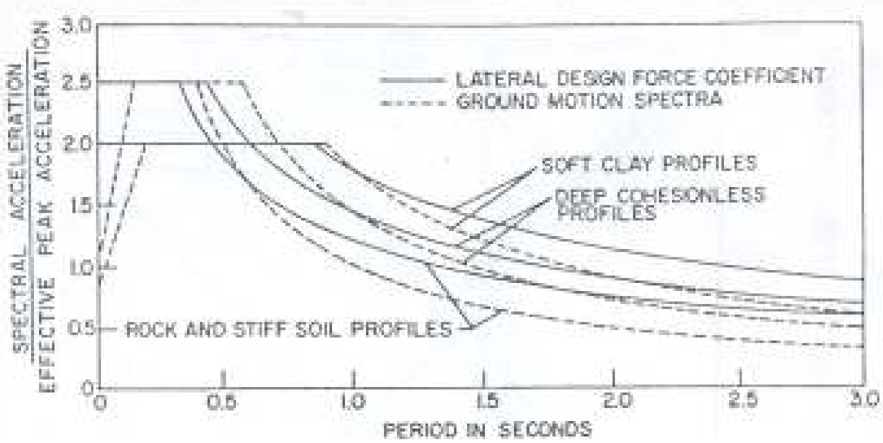

Figure 12 : Comparison of Free Filed Ground Motion Spectra and Lateral Design

Force Coefficients

2. Instability of a bridge is more of a problem as the period increases.

Other factors associated with modification of the elastic seismic response spectra are given in Article C5.2.2.

C3.3 IMPORTANCE CLASSIFICATION

The Importance Classification (IC) is used in conjunction with the Acceleration Coefficient (A) to determine the Seismic Performance Category (SPC) for bridges with an Acceleration Coefficient greater than 0.29. The SPC controls the degree of complexity and sophistication of the analysis and design requirements.

Two Importance Classifications are specified. An IC of I is assigned for essential bridges and II for all others. Essential bridges are those that must continue to function after and earthquake. The determination of the Importance Classification of a bridge is necessarily subjective. Consideration should be give to the following Social/Survival and Security/Defense requirements. An additional consideration would be average annual daily traffic.

The Social/Survival evaluation is largely concerned with the need for roadways during the period immediately following an earthquake. In order for civil defense, police, fire department or public health agencies to respond to a disaster situation a continuous route must be provided. Bridges on such routes should be classified as essential.

Survival and mitigation of the effects of the earthquake are of primary concern following a seismic event. Transportation routes to critical facilities such as hospitals, police and fire stations and communication centers must continue to function and bridges required for this purpose should be classified as essential. In addition a bridge that has the potential to impede traffic if it collapses no to an essential route should also be classified as essential.

The health and well-being of the community is another major concern. Victims with critical injuries or illnesses must be treated; food, water and shelter provided and utilities restored. Routes to such facilities as schools, arenas, etc., which could provide shelter or be converted to aid stations must suffer little or no damage and bridges on such routes should be classified as essential. Access must be available to power installations, water treatment plants, etc. and bridges required for these purposes should also be classified as essential.

The importance evaluation of a bridge for Social/Survival significance in a disaster situation depends on the range of options available and the possibility of a bridge being in parallel or series with other bridges in a roadway network. Discussion may be required with highway, civil defense and police officials.

A basis for the Security/ defense evaluation is the 1973 Federal-aid Highway Act which required that a plan for defense highways be developed by each state. This plan had to include, as a minimum, the Interstate and Federal-Aid Primary routes; however, some of these routes can be deleted when such action is considered appropriate by a state. The defense highway network provides connecting routes to important military installations, industries and resources not covered by the Federal-Aid Primary routes and includes:

1. Military bases and supply depots and National Guard installations.
2. Hospitals, medical supply centers and emergency depots.
3. Major airports.
4. Defense industries and those that could easily or logically be converted to such.
5. Refineries, fuel storage, and distribution centers.
6. Major railroad terminals, railheads, docks, and truck terminals.
7. Major power Plants including nuclear power facilities and hydroelectric centers at major dams.
8. Major communication centers.
9. Other facilities that the state considers important from a national defense viewpoint or during emergencies resulting form natural disasters or other unforeseen circumstances.

Bridges serve as important links in the Security/Defense roadway network and such bridges should be classified as essential.

C3.4 SEISMIC PERFORMANCE CATEGORIES

A basic premise in developing the Standards was that they be applicable to all parts of Uganda. The seismic risk varies from very small to high across the country and design requirements applicable to the higher risk areas are not always appropriate for the lower risk areas. In order to provide flexibility in specifying design provisions associated with areas of different seismic risk, three Seismic Performance Categories (SPC) were defined. The three categories permit variation in the requirements for methods of analysis, minimum support lengths, column design details, foundation and abutment design requirements in accordance with the seismic risk associated with a particular bridge location.

The seismic Performance Category is determined from the Importance Classification of Article 3.3 and the Acceleration Coefficient of Article 3.2. thus the importance of a bridge in a road network and the level of seismic exposure at a bridge site are used to determine the SPC. Different degrees of complexity in analysis and design requirements are specified for each SPC. Bridges classified as SPC 1, 2 & 3 are those designed for the lowest level of seismic performance.

C3.5 SITE EFFECTS

See Article C3.2 (F)

C3.4 RESPONSE MODIFICATION FACTORS

Response modification factors (R) shown in Table 3 are used to modify the component forces obtained from the elastic analysis. Inherent in the R values is the assumption that columns will yield when subjected to forces induced by the design ground motions and that connections and foundations are designed to accommodate the design ground motion forces with little, if any, damage.

The rationale used in the development of the R-Factors for columns, piers and pile bents was based on considerations of redundancy and ductility provided by the various supports, the wall type piper was judged to have minimal ductility capacity and redundancy in its strong direction and was assigned an R-Factor of 2. A multiple column bent with well-detailed columns, as specified in Section 8, was judged to have good ductility capacity and redundancy and was assigned the highest value of 5. The ductility capacity of single columns is similar to that of columns in a multiple column bent; however, there is no redundancy and therefore a lower R-Factor of 3 was assigned to single columns to provide a level of performance similar to that of multiple column bents. Unfortunately little information was available on the performance of pile bent substructures in actual earthquakes and the R-factors were based on the PEP's judgment of potential pile bent performance in comparison to the other three types of substructure. It was believed that there would be a reduction in the ductility capacity of pile bents with batter piles and therefore lower R-Factors were assigned to these systems.

The R-Factors of 1.0 and 0.8 assigned to connections means that the connections are designed for the design elastic forces and for greater than the design elastic forces in the case of abutments. This approach way adopted in part to accommodate the redistribution of forces that occurs when a bridge responds in elastically[8]. The other reason for adopting these values was to maintain the overall integrity of the bridge structure at these important joints. Increased protection can be obtained for a minimum increase in construction cost by designing connections for these larger force levels. Since these are the maximum forces that can be developed and are generally smaller than the elastic values the desired integrity will be obtained at lower cost.

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COMMENTARY

SECTION 4- ANALYSIS AND DESIGN REQUIREMENTS

C4.1 GENERAL

These Standards Provide for bridge design (sizing of individual members, commotions, and supports) based on internal forces derived by modifying the results from a linear elastic analysis. The provisions of the Standards assume that the columns may yield during an earthquake but that damage to connections and foundations will be minimized.

C4.2 ANALYSIS PROCEDURE

An elastic analysis procedure is used for seismic design of bridges to give the designer an indication of the force distribution to the structural members and to give him some indication of the relative deformations. It also provides the basis for the design of the components. The actual forces and displacements in a bridge subjected to the design ground motions may be quite different from those obtained form the elastic analysis because at these high levels of excitation the bridge may respond in elastically.

Two analytical procedures are specified. Procedure1, the single-mode spectral method of analysis, requires calculation of the fundamental period (T) of the bridge. A reasonable estimate of the elastic forces and displacements can then be made for regular bridges. Limits on the applicability of the method have not been exhaustively determined and the definition of a regular bridge give in Table 4 needs further study. The limits on the bridge does not meet the definition of a regular bridge. In these instances the multimode spectral method should be used.

Procedure2, the multimode spectral method of analysis, is the mote sophisticated of the two procedures and generally requires the use of a digital computer. It is very effective for analyzing the response of any linearly elastic structure to any prescribed dynamic excitation. Tow multimode methods of analysis are generally used; one is spectral analysis and the other is time history analysis. The spectral analysis does not directly account for the phase relationships between the modes of vibration where as the time history analysis does. A statistical approach (square root of the sum of the squares) is used to combine the contributions of different modes of vibration in the spectral analysis. This is a major limitation since the accuracy of this approach of bridges has not been thoroughly validated. The time history method accounts for the phase relationships between modes but requires the determination of appropriate ground motion time histories. Its major limitation is due to the uncertainties in ground motion studies. If a time history elastic or inelastic analysis is performed, a determination of appropriate time histories must be made as part of the site-specific study and it is recommended that a minimum of three different time histories be used.

The details of the two procedures are presented in the commentary of Section 5. For bridges classified as SPC C and D with three or more spans, the designer should seriously consider including the flexibility of the foundations and abutments in the analysis.

C4.3 DETERMINATION O ELASTIC FORCES AND DISPLACEMENT

Current knowledge of earthquake ground motions indicates that structures will be subjected to simultaneous ground motion in three orthogonal directions.[1] For many bridges the effect of the vertical component of motion may not be important and a detailed analysis in the vertical direction is not required.

To account for the two horizontal components of motion, an analysis is required in two orthogonal directions, generally the longitudinal and transverse directions of the bridge. Forces and moments resulting from these analyses are then combined as specified in Article 4.4 to account of the simultaneous occurrence of forces in two horizontal directions.

The forces and displacements obtained from an elastic analysis should be similar to those to which the bridge would be subjected if it responded elastically and the actual ground motion had similar characteristics to the design displacements.

C4.4 COMBINATION OF ORTHOGONAL SEISMIC FORCES

The method of combining forces for each of the load cases is given by means of an example. The two principal transverse axes of a column, abutments, pier, etc. may be designated as the z and y axes. The shear (V), moment (M), and axial (P) forces resulting from an analysis of the bridge subjected to loads in the transverse direction are designated as V^Tz, V^Ty, M^Tz, M^Ty, and PT, respectively. The corresponding forces resulting from an analysis of loads in the longitudinal direction are designated V^Lz,V^Ly, M^Lz, M^Ly, and P^L, respectively. The design shear (V^Dz, V^Dy), moment (M^Dz, M^Dy) and axial (P^D) forces for the Z and Y axes of the member for the two load cases are as follows:

LOAD CASE1

$V^Dz = 1.0\ |V^Lz| + 0.3\,|VTz|$
$V^Dy = 1.0\ |V^Ly| + 0.3\,|V^Ty|$
$M^Dz = 1.0\ |M^Lz| + 0.3\,|M^Tz|$
$M^Dy = 1.0\ |M^Ly| + 0.3\,|M^Ty|$
$P^D = 1.0\ |P^L| + 0.3\,|P^T|$

LOAD CASE 2

$V^Dz = 0.3\ |V^Lz| + 1.0\ |VTz|$
$V^Dy = 0.3\ |V^Ly| + 1.0\,|V^Ty|$
$M^Dz = 0.3\ |M^Lz| + 1.0\,|M^Tz|$
$M^Dy = 0.3\ |M^Ly| + 1.0\,|M^Ty|$
$P^D = 0.3\ |P^L| + 1.0\,|P^T|$

The symbol $|\ |$ denotes the absolute value or the magnitude of the force or moment without regard to its sign since a seismic force can act in either direction. It should be noted that, for a straight bridge with no skewed piers, columns or abutments, the above combinations simplify significantly because a transverse load will primarily produce moments and shear forces in the z direction of the structural member and the longitudinal load will primarily produce moments and shear forces in the y direction.

C4.5 DESIGN REQUIREMENTS FOR SINGLE SPAN BRIDGES

Single span bridges were separated from bridges with two or more spans for analysis and design requirements because of their response to seismic loads. This response was judged, on the basis of past performance, to be satisfactory provided there is sufficient support for the girders in both longitudinal and transverse directions. The design requirements for the connections are necessary to prevent damage and excessive deflections. The design forces are based on the premise that the bridge is very stiff. This assumption on the stiffness of the bridge also acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

C4.6 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY 1 & 2

Prior to the redesign phase of the project the PEP thought that the design of connections for wind forces would be satisfactory of anticipated seismic forces for bridges classified as SPC 1 & 2. However when the magnitude of the wind and seismic forces were compared for six bridges, it was found in almost all cases that, for an Acceleration Coefficient of 0.10, seismic forces were greater than wind forces. In some cases the difference was significant. Hence it was deemed necessary to include the requirement of this section for the design of the connections. The requirement is simple and some what conservative, especially for more flexible bridges, since the forces are based on the maximum elastic response coefficient. If the design forces are difficult to accommodate it is recommended that SPC 3 analysis and design procedures be used.

C4.7 DESIGN FORCES FOR SEISMIC PERFORMANCE CATEGORY 3

The seismic design forces specified for bridges classified as SPC 3 are intended to be relatively simple but consistent with the overall design concepts and methodology. Inherent in any simplification of a design procedure, however, is a degree of conservatism and for SPC 3 this occurs in the determination of the design forces for the foundations and connections to columns.

Article 4.7.1 specifies the design forces for the structural components of the bridge. In the first step the elastic forces of Load Cases 1 and 2 of Article 4.4 are divided by the appropriate R-Factors of Article 3.6. These forces are combined with those from other loads and the group loading combination is the same as that used in the current AASHTO Specifications with all γ and β factors equal to 1.0. Furthermore, in the Standards each component shall be designed to resist the two seismic group load combinations of Article 4.4, one including Load Case 1 and the other including Load Case 2. Each load case incorporates different proportions of bi-directional seismic loading. This may be important for some components (e.g., biaxial design of columns) and unimportant for others. In the design loads for each component from Article 4.4 can be taken as either positive or negative. The sign of the seismic force or moment that gives the maximum magnitude for the design force (either positive or negative) shall be used.

C4.9 DESIGN DISPLACEMENTS

In developing the Standards the PEP considered design displacements to be as important as design forces because many of the loss-of-span type failures in past earthquakes have been attributed in part to relative displacement effects.

The length of support provided at abutments, columns and hinge seats must accommodate displacements resulting from the overall inelastic response of the bridge structure, possible independent movement of different parts of the substructure, and out-of-phase rotation of abutments and columns resulting from traveling surface wave motions.

A reasonable estimate of the displacements resulting from the overall elastic dynamic response of the bridge structure can be obtained from the multi mode spectral method of analysis if the flexibility of the foundations is included. Better estimates can be obtained if an inelastic time history analysis is performed; however, this is not recommended in the Standards because of the complexities involved in performing this method of analysis. Either the elastic or inelastic time history analysis will give reasonable estimates of the out-of-phase movements of different parts of the substructure whereas the C4.9.1Seismic Performance Category 1 & 2 multimode method of spectral analysis will not. The recent work of Elms et al. 2.3 can be used to give the order of magnitude of abutment movement and the, recent work of Werner et al. 4.5 gives some indication or the effects of traveling waves on the responses of a limited number of bridges. However, much research remains ' to be done in both these areas.1

In summary, the current state of the art precludes a good estimate of the differential column and abutment displacements to be expected when a bridge is subjected to an earthquake. The PEP believes it necessary to specify minimum support lengths at abutments, piers and hinge seats to provide for the effects discussed above. If the displacements resulting from the elastic analysis of Article 4.3 exceed the minimum specified values, the values resulting from the elastic analysis must be used in the design. The minimum support lengths specified are dependent on the deck length between expansion joints and the column height since both dimensions influence one or more of the factors that cause the differential dis-placements. Although a considerable amount of judgment was exercised on the basis of current knowledge, the proposed criteria should be refined as the state of the art develops.

C4.9.1 Seismic Performance Category 1 & 2

Since an elastic analysis is not required for bridges classified as SPC 1 & 2 the minimum support lengths specified in Article 4.9.1 are the only design displacement requirements for these bridges.

C4.9.2 Seismic Performance Categories 3

For bridges classified as SPC 3 the design displacements are specified as the maximum of those determined from the elastic analysis of Article 4.3 or the minimum specified support lengths given by Eqs. 4-3 and 4-4. This either/or specification was introduced tq, account for larger displacements that may occur from the analysis of more flexible bridges. It was the opinion of the PEP that displacements obtained from the elastic analysis of bridges should provide a reasonable estimate of the displacements resulting from the inelastic response of the bridge. However, it must be recognized that displacements are very sensitive to the flexibility of the foundation and if the foundation is not included in the elastic analysis of Article 4.3, consideration should be given to increasing the specified displacements for bridges founded on very soft soils. This increase may be of the order of 50% or more but as with any generalization considerable judgment is required. A better method is to determine upper and lower bounds from an elastic analysis, which incorporates foundation flexibility. Special care in regard to foundation flexibility is required for bridges with high piers.

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COMMENTARY

SECTION 5 - ANALYSIS PROCEDURES

C5.1 GENERAL

This chapter of the Standards presents two analytical procedures to determine the distribution of forces for the prescribed seismic loadings. Both are based on linear elastic analysis techniques.

C5.2 ELASTIC SEISMIC RESPONSE COEFFICIENT AND SPECTRUM

C5.2.1 Elastic Seismic Response Coefficient

See Article C3.2(F)

C5.2.2 Elastic Seismic Response Spectrum

Equation 5-4 is to be used if a modal period exceeds 4 sec. It can be seen that Eqs. 5-4 and 5-2 coincide at T m = 4 see, so that the effect of using Eq. 5-4 is to provide a more rapid decrease in Csm as a function of T m than implied by Eq. 5-2. This modification is introduced in consideration of the known characteristics of earthquake response spectra at intermediate and long periods. At intermediate periods the average velocity spectrum of strong earthquake motions from large earthquakes (magnitude 6.5 and larger) is approximately horizontal; this implies that Csm should decrease as Iff m' In Eq. 5-2 Csm decreases as *Irr2m/3* for reasons discussed in Article C3.2(F), and this slower rate of decrease, if extended to very long period$, would result in an unbalanced degree of conservatism in the modal force for very flexible bridges. In addition, for very long periods, the average displacement spectrum of strong earthquake motions becomes horizontal; this implies that Csm, which is a form of acceleration spectrum, should decay as *Irr;..* The period at which the displacement response spectrum becomes horizontal depends on the size of the earth quake, being longer for large earthquakes, and a representative period of 4 sec. was chosen to make the transition.

A central feature of modal analysis is that the earthquake response is considered as a combination of the independent responses of the bridge vibrating in each of its important modes. As the bridge vibrates back and forth in a particular mode at the associated period, it experiences maximum values of member forces and displacements. The coefficient Csm is determined for each mode from Eq. 5-2 using the associated period of the mode, T m, in addition to the factors A and S, which are discussed elsewhere in this Commentary. An exception to this procedure occurs for higher modes of those bridges which have periods shorter than 0.3 sec. And which are founded on Type III soils. For such modes; Eq. 5-3 is used. Equation 5-3 gives values ranging from 0.8 A for very short periods to 2.0 A for T m = 0.3. Comparing these values with the limiting value of Cs of 2.0 A for Type III soils as specified following Eq. 5-2, it is seen that the use of Eg. 5-3, when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified by Eq. 5-2 and its limiting values. This shape is a conservative approximation to that of average spectra which are known to first ascend, then level off, and then decay as period increases-see Figures 9 and 10. Equation 5-2 and its limiting values conservatively replace the ascending portion for small periods by a level portion. For Type I or II soils, the ascending portion of the spectrum is completed by the time the period reaches a value near 0.1 or 0.2 see. On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Equation 5-3 is then a replacement for Type III soils and short (periods, which is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Eq. 5-2 for modal analysis in the case of Type III soils.

C5.3 SINGLE MODE SPECTRAL ANALYSIS METHOD

The single mode spectral analysis method is used to calculate the seismic design forces for bridges that respond predominantly in the first mode of vibration. The method, although completely rigorous from a structural dynamics point of view, reduces to a problem in static after the introduction of inertia forces. The method, as formulated, can be applied to many types of bridges which have both continuous and noncontinuous superstructures. Boundary conditions at the abutments and piers can also be modeled in include the effects of foundation flexibility.

Bridges are generally continuous systems consisting of many components which contribute the overall resistance capacity of the system. Consider a bridge subjected to a transverse earthquake ground motion. The bridges are composed of several spans restrained transversely at the end abutments and intermediate piers, as shown in Figure 14. Typically the bridge deck may have expansion joints at the piers or within the spans. These expansion joints do not have the capability to transmit transverse deck moments between adjacent deck sections. The equation of motion for a continuous system representing this system is conveniently formulated using energy principles. The principle of viral displacements may be used to formulate a generalized parameters may be used to formulate a generalized parameter" model may be formulated. To obtain an approximation to this mode shape, a uniform static loading, P_o, is applied to the superstructure and the resulting deflection, $v_s(x)$, is obtained. The dynamic deflection, v(x, t) of the structure under seismic excitation as shown in Figure 15 is then approximated by the shape function multiplied by a generalized amplitude function, (v(t) as shown by Eq. C5-1

$$V(x,t) = V_s(x)V(t) \qquad \text{(C5-1)}$$

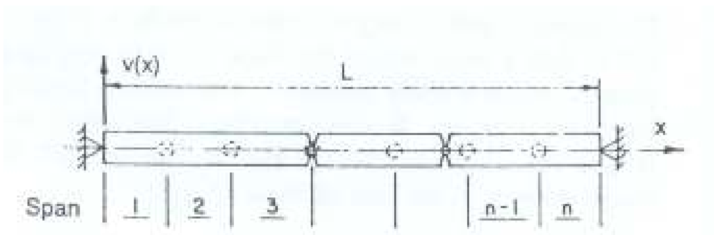

Figure 14 : Plan View of a Bridge Subjected to a Transverse Earthquake Motion

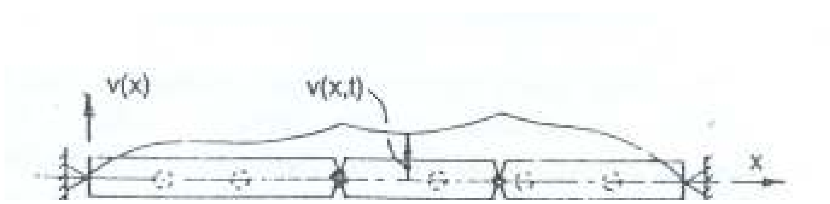

Figure 15 : Displacement Function Describing the Transverse Position of the Bridge Deck

The function will describe the deformed bridges structure in a manner which is consistent with the support conditions and intermediate expansion joint hinges in the deck. Note that it is an admissible function which satisfies the geometric boundary conditions of the systems.

Initially, to establish the deflected shape for the generalized parameter model, apply a uniform loading Po to the structure as shown in figure 16 assume that the loading is applied gradually so that the kinetic energy of the mass of the structure is zero. The external work, WE, done by the uniformly applied loading in deforming the structure is given by:

$$WE = \frac{P\hat{O}}{2}\int_{o}^{L} Vs(x)dx = \frac{P\hat{O}}{2}\alpha \qquad \text{(C5-2)}$$

This work will be stored internally in the elastic structure in the form of strain energy U; this

$$U = W_E \qquad \text{(C5-4)}$$

After $V_s(x)$ is determined using any standard static analysis approach, the integral in Eq. C5-3, appearing in Step 2 of the Standards, may be evaluated numerically.

If the uniform loading Po is suddenly removed, and the effects of damping are neglected, the structure will vibrate in the assumed mode shape shown in Figure 17 at a natural frequency determined by equating maximum kinetic energy to maximum strain energy (Rayleigh method); i.e.

$$T_{max} = U_{max} \qquad \text{(C5-5)}$$

The maximum kinetic energy of the system is given by:

$$T_{max} = \frac{\omega_2}{2g}\int_{o}^{L} w(x)Vs(x)^2\,dx = \frac{\omega^2 y}{2g} \qquad \text{(C5-6)}$$

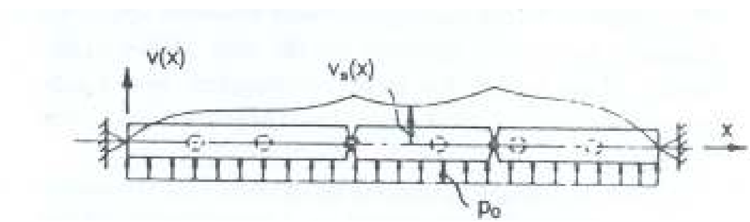

Figure 16 : Deflected Shape Due to Uniform Static Loading

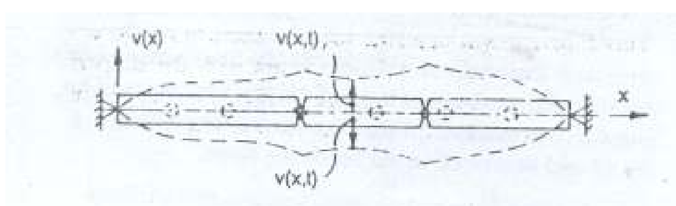

Figure 17 : Transverse Free Vibration of the Bridge in Assumed Mode Shape

Where:

$$\gamma = \int_{o}^{L} w(x)Vs(x)^2\,dx \qquad \text{(C5-7)}$$

and ω is the frequency of the vibrating system. The factor γ defined in Eq. C5-7 and appearing in Step 2 of the Standards, is evaluated numerically.

The maximum strain energy stored in the system is

$U_{max} = W_E$, (C5-8)

Using Eqs. C5-2, C5-6 and C5-8, Eq. C5-5 becomes

$$P_o\alpha = \frac{\omega^2\gamma}{g} \qquad \text{(C5-9)}$$

Introducing w = 21Trr into Eq. C5-9 and solving for the period T, yields

$$T = 2\prod\sqrt{\frac{\gamma}{p\hat{o}g\alpha}} \qquad \text{(C5-10)}$$

The generalized equation of motion for the single degree-of-freedom system subjected to a ground acceleration Vg(t) may be written as

$$V(t) + 2\xi v(t) + \omega^2 v(t) = \frac{-\beta v_g(t)}{\gamma} \qquad \text{(C5-11)}$$

where

$$\beta = \int_{o}^{L} w(t)V_s(x)dx \qquad \text{(C5-12)}$$

and ξ is the damping ratio to be prescribed. For most structures, a value of 0.05 is recommended. Using the standard acceleration response spectral value Cs in its dimensionless form,

$$Cs = \frac{Sa(\xi.T)}{g} \qquad \text{(C5-13)}$$

where Sa (ξ, T) is the pseudo acceleration spectral value.

The maximum response of the system is given by

V(x, t) max = v(t)max Vs(x) (C5-14)

Where

V(t)max =

$$v(t)_{\max} = \frac{\mathrm{Csg}\beta}{\omega^2\gamma} \qquad \text{(C5-15)}$$

Thus:

$$v(x,t)\max = \frac{\mathrm{Csg}\beta}{\omega^2\gamma} V_s(x) \tag{C5-16}$$

The static loading $P_e(x)$ which approximates the inertial effects associated with the displacement v(x, t) max is shown in Figure 18 and is given by:

$$Pe(x) = \frac{\beta Cs}{\gamma} w(x) Vs(x) \tag{C5-17}$$

C5.4 METHOD MULTIMODE SPECTRAL ANALYSIS

C5.4.1 General

The multi mode response spectrum analysis should be performed with a suitable linear dynamic analysis computer program. Programs generally available with these capabilities include: STRUDL, SAP2000, ANSYS, STARDYN, NASTRAN, EASE and MARC.

C5.4.2 Mathematical Model

The model type and degree of refinement depends on the complexity of the actual structure and the results desired in the analysis. Modeling a bridge for a dynamic analysis is currently more an art than a science. The overall objective is to produce a mathematical model that will represent the dynamic characteristics of the structure and produce realistic results consistent with the input parameters. This article is intended to provide some basic guidelines which will yield realistic results for most bridge structures. Although the terms "joint") and "node" are generally used interchangeably, for the purposes of these Standards the term "node" is used to indicate the use of a joint specifically for the purposes of mathematically modeling mass" or inertia characteristics. Condensation of mass terms should be done with care to prevent the loss of the inertia effects of the structure.

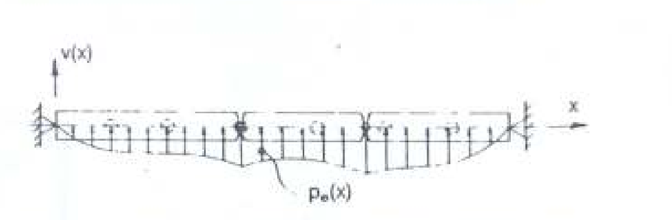

Figure 18 : Characteristic Static Loading Applied to the Bridge System

The force-displacement relationship at bridge abutments is a highly complex nonlinear problem and will be affected by the abutment design. In the absence of more accurate information, the following iterative technique may be used to determine an equivalent elastic transverse and longitudinal stiffness at the abutments to be used for the analysis of typical bridge structures. The procedure is outlined in the flowchart appearing in Figure 19 and described in the following steps:

1. Assume an initial abutment design and stiffness.
2. Analyze the bridge and determine the forces at the abutment. Perform the appropriate following steps:

   (a) If the force levels exceed the acceptable capacity of the abutment fill and/or piles, reduce the stiffness of the abutments until the analysis indicates force levels below the acceptable capacity.

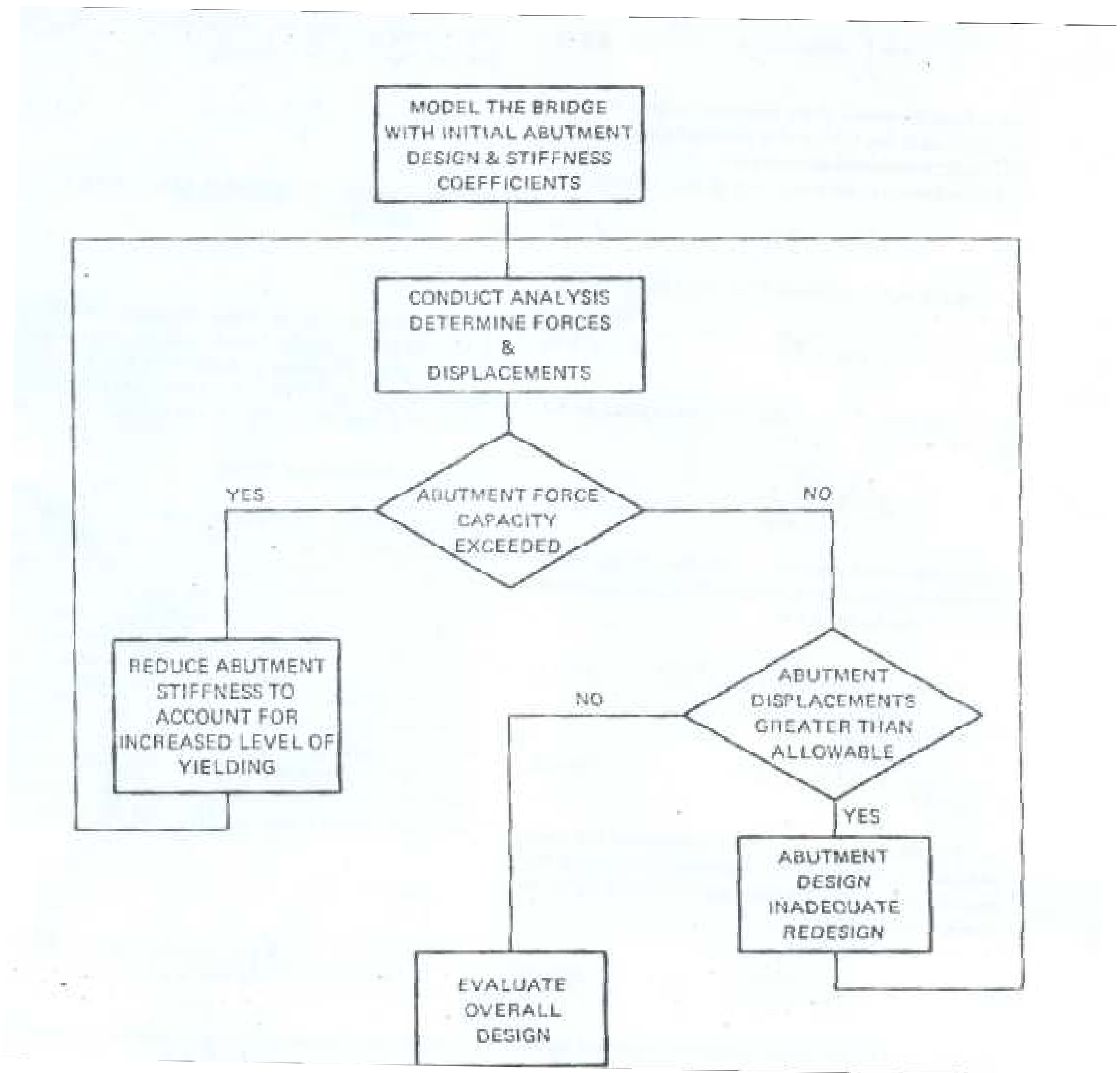

Figure 19 Iterative Procedure for Including Abutment Soil Effects in the Seismic Analysis of Bridges

(b) If the' force levels are below the acceptable capacity of the 'abutments, proceed to Step 3.

3. Observe the analyzed displacements at the abutment and take the appropriate following step:

   (a) If displacements exceed acceptable levels, the assumed abutment design is inadequate. Redesign the abutment and return to Step I.
   (b) If displacements are acceptable, the last assumed abutment stiffness is consistent with the assumed abutment design.

C5.4.3 Mode Shapes and Periods

The computer programs mentioned in Article C5.4.1 have the ability to calculate the mode shapes, frequencies and resulting member forces and displacements for a multi mode spectral analysis. The following equations summarize the equations used in such an analysis.

Mode shapes and frequencies should be obtained from the equation:

$$[\underline{k} - \omega 2m]\ \underline{v} = \underline{o} \qquad \text{(C5-18)}$$

using standard eigenvalue computer programs; where k and !Q are the known stiffness and mass matrices of the mathematical model, respectively, y is the displacement amplitude vector, and w is the frequency. This analysis will yield the dimensionless mode shapes ~ and their corresponding circular frequencies w.. W2, . . ., W_n. The mode periods can then be obtained using ϕ_1, $\phi_{2,....,}$ ϕ_n and their corresponding circular frequencies ω_1, $\omega_{2,......}$ $\omega_{n.}$ The mode periods can then be obtained using:

$$Ti = \frac{2\pi}{\omega_i}(i = 1,2.....,n) \qquad \text{(C5-19)}$$

C5.4.4 Multimode Spectral Analysis

The uncoupled normal mode equations of motion are of the form:

$$\acute{\mathrm{Y}}\mathrm{i(t)} + \omega_i^2 Y_2 i(t) = \frac{P_i(t)}{M_i} \qquad \text{(C5-20)}$$

where the subscript i refers to the mode number, Yi> Wi and ~i are the mode amplitude, frequency, and damping ratios, respectively, and the effective modal load Pi(t) and generalized mass Mi are given by:

$$P_i(t) = \phi_i^T mBV_{g(t)} \qquad \text{(C5-21)}$$

Mi = $\phi_i^T m\phi$

Where $\underline{B}$ is 'a vector containing ones and zeroes corresponding to those components in the direction of excitation v_g(t)and those components in the other orthogonal directions, respectively.

The maximum absolute value of Yi(t) during entire time-history of earthquake excitation is given by:

$$Yi(t)_{\max} = \frac{T_i^2 Sa(\xi i, Ti)\phi_i^T mb}{4\pi^2 \phi_i^T m\phi i} \qquad \text{(C5-22)}$$

where Sa (ξ_i, Ti) is the acceleration response spectral value for the prescribed earthquake excitation. In these, Standards Sa (~i, Ti) is obtained from the equation:

$$Sa(\xi_j, Ti) = gC_{sm} \tag{C5-23}$$

where C_{sm} is defined through the empirical relation given by Eqs. 5-2, 5-3 or 5-4.

To determine the maximum value of any particular response quantity Z(t) (e.g., a shear, moment, displacement or relative displacement), use is made of the fact that it is linearly related to the normal mode amplitude, i.e.,

$$Z(t) = \sum_{i=1}^{n} A_i Y_i(t) \qquad \text{(C5.24)}$$

where coefficients Ai are known. The maximum value of Z(t) during the duration of the earthquake can be estimated using the square root of the sum of the squares (SRSS) method for systems having well-separated modes, i.e., using

$$\left|Z(t)\right|_{\max} = \sqrt{\sum_{i=1}^{n} A_i^2 \left|Y_i(t)\right|^2{}_{\max}} \qquad \text{(C5-25)}$$

C5.4.5 Member Forces and Displacements

The member forces and displacements of an elastic structure are obtained by the superposition of the respective quantities of the individual modes of vibration. Generally, the maximum values for each mode do not occur simultaneously and thus the maximum value of each mode cannot be directly superimposed to obtain the maximum force or displacement of a member. The direct superposition (absolute sum) of the individual modal contributions thus provides an upper bound which is generally conservative and not recommended for design. A satisfactory estimate of a maximum value of a force or displacement can be obtained by taking the square root of the sum of the squares (SRSS) of the individual modal response as defined by Eq. C5-25.

The SRSS method is generally applicable to most bridges however there are some bridges, with unusual geometric features which cause some of the individual modes to have closely spaced periods to which this method may not be applicable. There are several methods currently available and new methods are emerging for combining these closely spaced modes. One of these methods, which may in some cases be conservative, suggests that the absolute values of closely spaced modes be added to the SRSS of the remaining modes. At present, however, there are not enough data available for bridges, comparing these response spectrum results with time history analyses to provide a verifiable basis on which to make recommendations.

COMMENTARY

SECTION 6-FOUNDATION AND ABUTMENT DESIGN REQUIREMENTS

C6.3 SEISMIC PERFORMANCE CATEGORIES 3

The Commentary on Section 6 is not broken down by Seismic Performance Category because most of the commentary on liquefaction, foundations, piles and abutments is applicable to all three categories.

C6.3.1 Foundations

C6.3.J(A) Investigation

Slope instability, liquefaction, fill settlement and increases in lateral earth pressure have often been major factors in contributing to bridge damage in past earthquakes. These earthquake hazards may be significant design factors for peak earthquake accelerations in excess of 0.1 g and should form part of a site specific investigation if the site conditions and the associated acceleration levels and design concepts suggest that such hazards may be of importance. Since liquefaction has contributed to many bridge failures, methods for evaluating site liquefaction potential are described in more detail below.

Liquefaction Potential!. Liquefaction of saturated granular foundation soils has been a major source of bridge failures during historic earthquakes. For example, during the 1964 Alaska earthquake, 9 bridges suffered complete collapse, and 26 suffered severe deformation or partial collapse. Investigations indicated that liquefaction of foundation soils contributed to much of the damage, with loss of foundation support leading to major displacements of abutments and piers. A study of seismically induced liquefaction and its influence on bridges has been compiled by Feritto and Forest in a report I to the Federal Highway Administration. A brief review of seismic design considerations for bridge foundations related to site liquefaction potential is given in reference 2. From the foundation failure documented in these reports and in the literature in general, it is clear that the design of bridge foundations in soils susceptible to liquefaction poses difficult problems. Where possible, the best design measure is to avoid deep, loose to medium-dense sand sites where liquefaction risks are high. Where dense or more competent soils are found at shallow depths, stabilization measures such as densification may be economical. The use of long ductile vertical steel piles to support bridge piers could also be considered. Calculations for lateral resistance would assume zero support from the upper zone of potential liquefaction, and the question of axial buckling would need to be addressed. Overall abutment stability would also require careful evaluation, and it may be preferable to use longer spans and to anchor abutments well back, from the end of approach fills.

A further design philosophy of bridges in liquefaction susceptible areas might be one of "calculated risk", at least for those bridges regarded as being less essential for communication purposes immediately after an earthquake. It may not be economically justifiable to design some bridges to survive a large earthquake in a liquefaction environment without significant damage. However, it may be possible to optimize a design so that the cost of repair of potential earthquake damage to those bridges does not exceed the cost of remedial measures and additional construction needed to avoid the damage. The approaches for determining the liquefaction potential at a site are outlined below.

A recent review of methodologies identifies two basic approaches for evaluating the cyclic liquefaction potential of a deposit of saturated sand subjected to earthquake shaking:

1. Empirical methods based on field observations of the performance of sand deposits in previous earthquakes and correlations between sites which have and have not liquefied and Relative Density of Standard Penetration Test (SPT) blow counts.
2. Analytical methods based on the laboratory determination of the liquefaction strength

characteristics of undisturbed samples and the use of dynamic site response analysis to determine the magnitude of earthquake-induced shearing stresses.

Both empirical and analytical methods require the level of ground acceleration at a site to be defined as a prerequisite for assessing liquefaction potential. This is often established from relationships between earthquake magnitude distance from the epicenter and peak acceleration.

For conventional evaluations using a "total stress" approach the two methods are similar, but differ only in the manner in which the field liquefaction strength is determined. In the "total stress" approach, liquefaction strengths are normally expressed as the ratio of an equivalent uniform or average cyclic shearing stress the acting on horizontal surfaces of the sand to the initial vertical effective stress σ'_0. As the first approximation, the cyclic stress ratio developed in the field because of earthquake ground shaking may be computed from an equation given by Seed and Idriss,[4] namely:

$$(Th)av/\sigma'_o = 0.65 r_d \, (a_{max}/g) \, (\sigma \hat{o}/\sigma'_o) \qquad \text{(C6-1)}$$

Where

a_{max} = maximum or effective peak ground acceleration at the ground surface

σ_o = total overburden pressure on sand layer under consideration

σ'_o = initial effective overburden pressure on sand layer under consideration

r_d = stress reduction factor varying from a value of I at the ground surface to 0.9 at a depth of about 30 ft (9.1 m).

Empirical Methods. Values of the cyclic stress ratio defined by Eq. C6- I have been correlated for sites which have and have not liquefied, with parameters such as relative density based on' SPT Data (Seed et al.,5 Castro[6]). The latest form of this type of correlation (after Seed3) is expressed in Figures 20 and 21. N I is the measured standard penetration resistance of the sand corrected to an effective overburden pressure of 1 ton/sq ft (95,800 *N/m²)* using the relationship

$$N_1 = NC_N \qquad \text{(C6-2)}$$

Thus, for a given site and a given maximum ground surface acceleration, the average stress ratio developed during the earthquake, $(T_h)av/\sigma'_0$, at which liquefaction may be expected to occur, is expressed by the empirical correlations shown by Figure 20. The correlations for different magnitudes reflect the influence of earthquake duration on liquefaction potential. The factor of safety against liquefaction can be determined by comparing the stress ratio required to cause liquefaction with that induced by the design earthquake.. It is suggested that a factor of safety of 1.5 is desirable to establish a reasonable measure of safety against liquefaction in the case of important bridge sites.

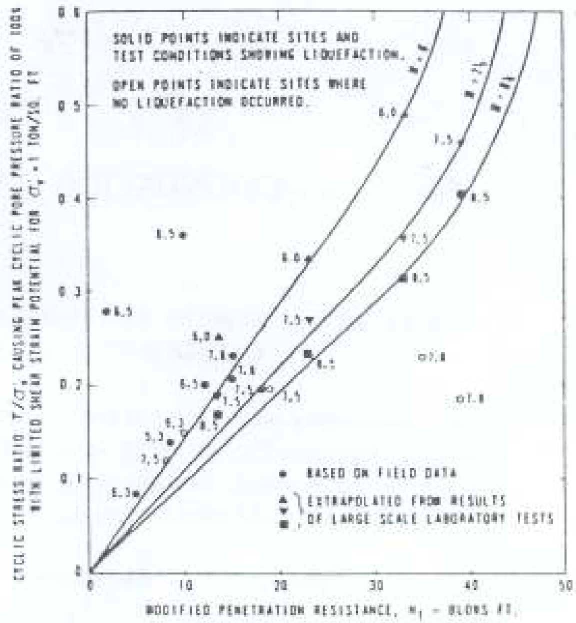

Figure 20 Correlation Between Field Liquefaction Behavior and Penetration Resistance

Figure 20 : Correlation Between Field Liquefaction Behavior and Penetration Resistance

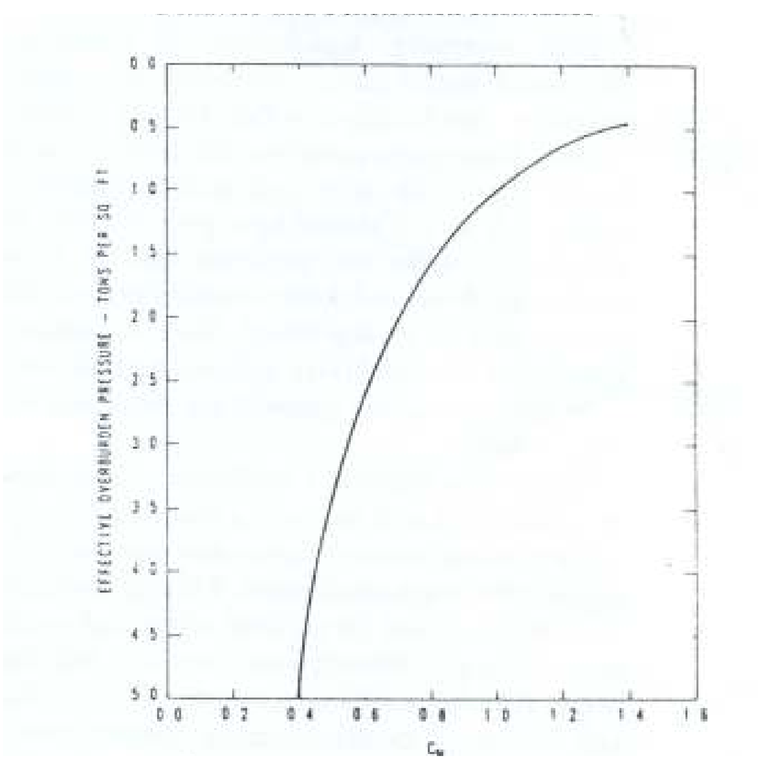

Figure 21 : Relationship Between C_N and Effective Overburden Pressure

A further extension of the empirical approach has recently been described by Dezfulian and

Prager,[7] where a correlation between cone penetrometer tests (CPT) and standard penetration tests (SPT) has enabled CPT measurements in sands (expressed as point resistance qc to be used as a measure of liquefaction potential. CPT have the advantage of being more economical than SPT, and since they can provide a continuous record of penetration.

Resistance with depth, potentially liquefiable thin seams of sand can be identified more readily. Whereas penetration tests have the clear advantage of being a field oriented liquefaction evaluation procedure, it must always be remembered that the empirical correlation has been established from a very limited database restricted to sites comprising primarily deposits of fine silty sand. The correlation may break down for sandy silts and gravely soils (where blow count data are difficult to interpret), and for coarser sands where partial drainage of excess pore pressures may occur during an earthquake. Furthermore, for situations where additional stresses are imposed by construction operations, care is needed in interpreting the correlation.

Analytical Methods. The analytical approach for evaluating liquefaction potential is based on a comparison between field liquefaction strengths established from cyclic laboratory tests on undisturbed samples, and earthquake-induced shearing stresses. In this approach it must be recognized that the development of a field liquefaction strength curve from laboratory test results, requires data adjustment to account for factors such as correct cyclic stress simulation, sample disturbance, aging effects, field cyclic stress history, and the magnitude of in situ lateral stresses. These adjustments require a considerable degree of engineering judgment. Also in many cases it is impossible to obtain undisturbed sand samples.

Once a liquefaction strength curve has been established, if a total stress analysis is used, liquefaction potential is evaluated from comparisons with estimated earthquake-induced shear stresses (as shown in Figure 22).

The earthquake-induced shear stress levels may be established from a simplified procedure, or more sophisticated assessments made using one dimensional “equivalent linear” dynamic response programs such as SHAKE. Average stress levels are established using the equivalent number of cycles concept (approximately 10 for M7 and 30 for M8.5 earthquakes). More recently, nonlinear programs have been introduced for response calculations.

An improved representation of the progressive development of liquefaction is provided by the use of an effective stress approach where pore water pressure increases are coupled to nonlinear dynamic response solutions, and the influence of potential pore water pressure dissipation during an earthquake is taken into account. This approach provides data on the time history of pore water pressure increases during and earthquake, as shown in Figure 23.

If is of interest to note that a rough indication of the potential for liquefaction may be obtained by making use of empirical correlations established between earthquake magnitude and the epicentral distance to the most distant field manifestations of liquefaction. Such a relationship has been described by youd and Perkins (Figure 24), and has been used as a basis for preparation of liquefaction-induced ground failure susceptibility maps.

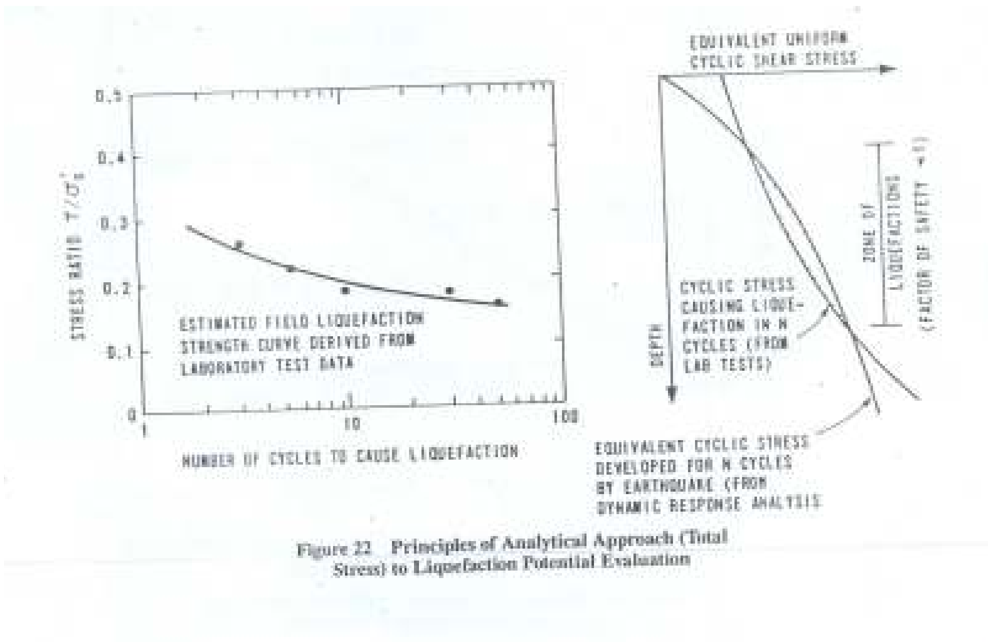

Figure 22 : Principles of Analytical Approach (Total Stress) to Liquefaction Potential Evaluation

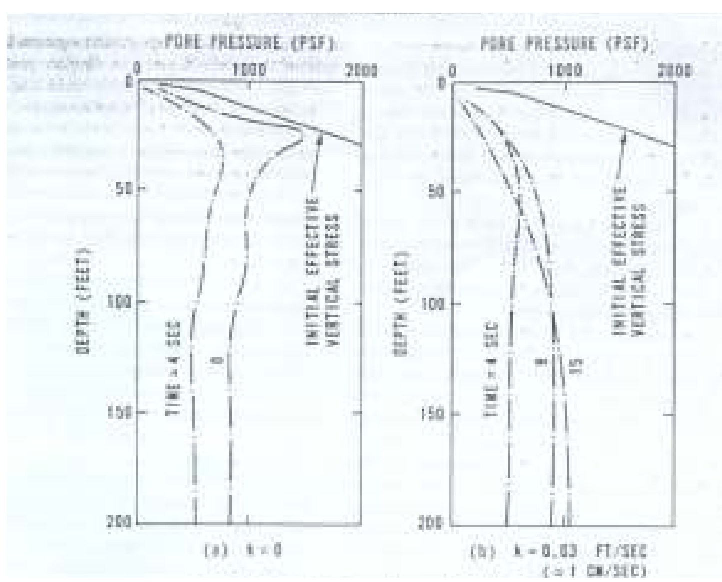

Figure 23 : Effective Stress Approach to Liquefaction Evaluation showing Effect of Permeability (After Finn et. Al., 1977)

C6.3.1(B) Foundation Design

The commonly accepted "practice for the seismic design of foundations" is to utilize pseudo-static approach, where earthquake-induced 'foundation loads are determined from the reaction forces and moments necessary for structural equilibrium. Whereas traditional bearing capacity design approaches are also applied, with appropriate capacity reduction factors if a measure of safety against "failure" is desired, a number of factors associated with the dynamic nature of earthquake loading should always be borne in mind.

Under cyclic loading at earthquake frequencies, the strength capable of being mobilized by many soils is greater than the static strength. For unsaturated cohesionless soils the increase may be about 10%, while for cohesivesoils, a 50% increase could occur. However, for softer saturated clays and saturated sands, the potential for strength and stiffness degradation under repeated cycles of loading must also be recognized. For bridges classified as SPC 3, the use of static soils strengths for evaluating ultimate foundation capacity provides a small implicit factor of safety and, in most cases, strength and stiffness degradation under repeated loading will not be a problem because of the smaller magnitudes of seismic events.

As earthquake loading is transient in nature, "failure" of soil for a short time during a cycle of loading may not be significant of perhaps greater concern is the magnitude of the cyclic. Foundation displacement *or* rotation associated with soil yield, as this could have a significant influence on structural displacements or bending moments and shear distributions in columns.

As foundation compliance influences the distribution of forces or moments in a structure and affects computation of the natural period, equivalent stiffness factors for' foundation systems are often required. In many cases, use is made of various analytical solutions which are available for footings or piles, where it is assumed that soil behaves as an elastic medium. In using these formulae, it should be recognized that equivalent elastic moduli for soils are a function of strain amplitude, and for high seismic loads modulus values could be significantly less than those appropriate for low levels of seismic loading. Variation of shear modulus with shearing strain amplitude in the case of sands is shown in Figure 25.

On the basis of field and experimental observations, it is becoming more widely recognized that transient foundation uplift or rocking during earthquake loading, resulting in separation of the foundation from the subsoil, is acceptable provided appropriate design precautions are taken (Taylor and Williamsl2). Experimental studies suggest that rotational yielding beneath rocking foundation can provide a useful form of energy dissipation. However, care must be taken to avoid significant induced vertical deformations accompanying possible soil yield during earthquake rocking, as well as excessive pier movement. These could lead to design difficulties with relative displacements.

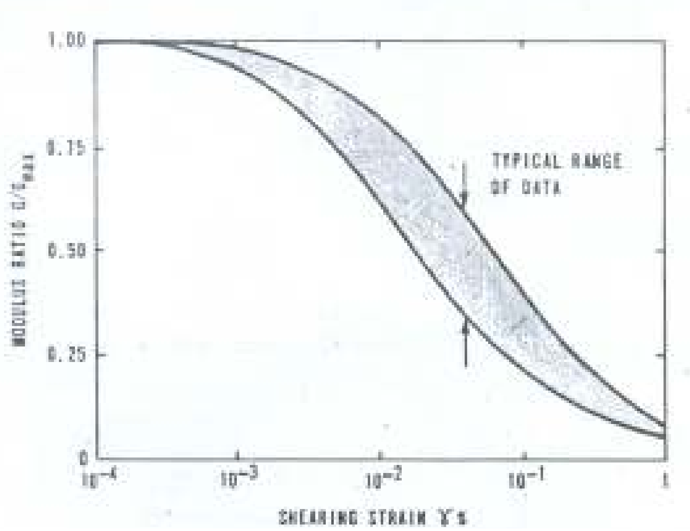

Figure 25 : Variation of Shear Modulus with Shearing Strain for Sands

Lateral Loading, of Piles~ Most of the well-known solutions for: computing, the lateral stiffness of vertical piles are based on the assumption of elastic behavior and utilize equivalent cantilever beam concepts[13]! the beam on elastic Winkler foundation method[14] or elastic continuum solutions[15]. However, the use of methods incorporating nonlinear subgrade reaction behavior that allows for soil failure may be important for high lateral loading of piles in soft clay and sand. Such a procedure is encompassed in the American Petroleum Institute (API) recommendations for offshore platform design.[16] The method utilizes nonlinear subgrade reaction or p-y curves for sands and clays which have been developed experimentally from field loading tests.

The general features of the API analysis in the case of sands are illustrated in Figure 26. Under large loads, a passive failure zone develops near the pile head. Test data indicate that the ultimate resistance, P_u for lateral loading is reached for pile deflections, Y_u. of about 3d/8O, where d is the pile diameter. Note that most of the lateral resistance is mobilized over a depth of about 5d. The API method also recognizes degradation in lateral resistance with cyclic loading, although in the case of saturated sands the degradation postulated does not reflect pore water pressure increases. The degradation in lateral resistance due to earthquake-induced free-filed pore-water pressure increases in saturated sands. Has been described by Finn and, Martin.[17] A numerical method which allows the use of API p-y curves to compute pile stiffness characteristics forms the basis of the computer program BMCOL 76 described by Bogard and Matlock.[18]

The influence of group action on pile stiffness is a somewhat controversial subject. Solutions based on elastic theory can be misleading where yield near the pile head occurs. Experimental evidence tends to suggest that group action is not significant for pile spacings greater than 4d to 6d.

For batter pile systems the computation of lateral pile stiffness is complicated by the stiffness of the piles in axial compression and tension. It is also important to recognize than bending deformations in batter pile groups may generate high reaction forces on the pile cap.

It should be noted that while batter piles are economically attractive for resisting horizontal loads. Such piles are very rigid in the lateral direction if arranged so that

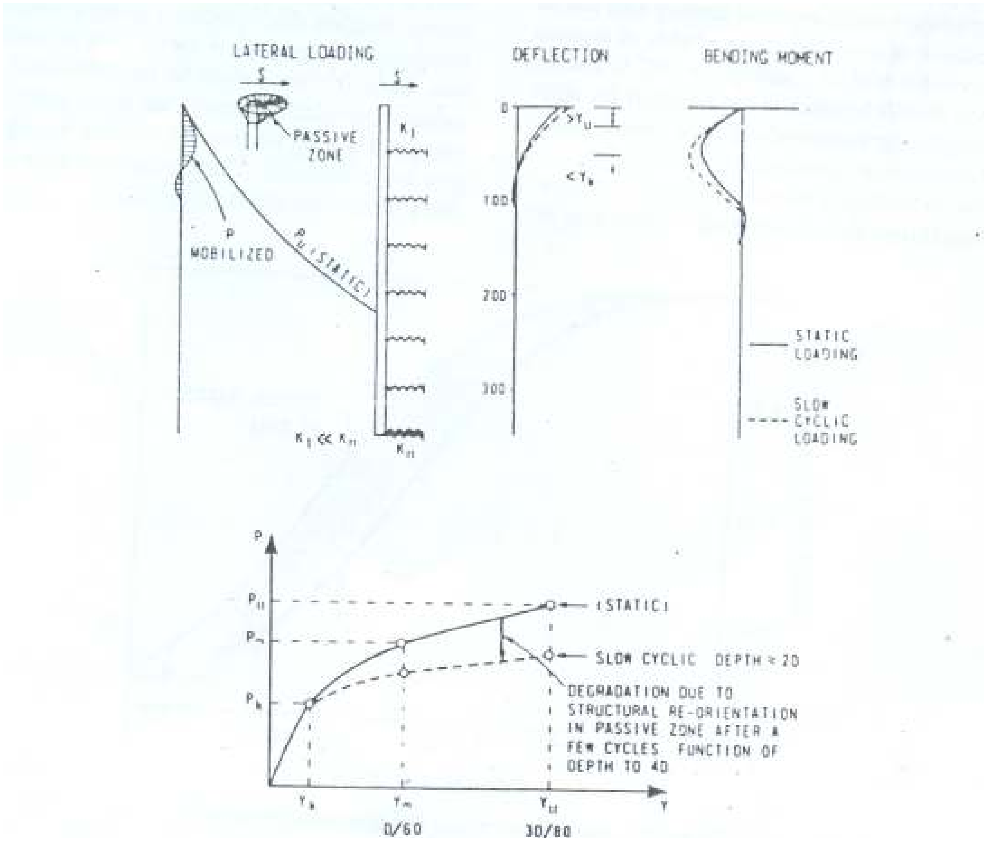

Figure 26 : Lateral Loading or Piles in Sand. Using API Criteria

Only axial loads are induced. Hence large relative lateral displacements of the more flexible surrounding soil may occur during the free-field earthquake response of the site (particularly, if large changes in soil stiffness occur over the pile length); and these relative displacements may in turn induce high pile bending moments.

For this reason, more flexible vertical pile systems where lateral load is resisted by bending near the pile heads, are recommended. However, such pile systems must be designed to be ductile, because large lateral displacements may be necessary to resist the lateral load. A compromise design using battered \ piles spaced some distance apart may provide a system which has the benefits of limited flexibility and the economy of axial load resistance to lateral load.

Soil-Pile Interaction. The use of pile stiffness characteristics to determine earthquake-induced pile bending moments based on a pseudo-static approach, assumes that moments are induced only by lateral loads arising from inertial effects on the bridge structure. However, it must be remembered that the inertial loads are generated by interaction of the free-field earthquake ground motion with the piles, and that the free-field displacements themselves can influence bending moments. This is illustrated in an idealized manner in Figure 27. The free-field earthquake displacement time histories provide input into the lateral resistance interface elements which in turn transfer motion to the pile. Near the pile heads, bending moments will be dominated by the lateral interaction loads generated by inertial effects on the bridge structure. At greater depth (e.g. greater than 10d) where soil stiffness progressively increases with response to pile stiffness. The pile will be constrained to deform in a similar manner to that of the free field and pile bending moments become a function of the curvatures induced by free-field displacements.

To illustrate the nature of free-field displacements reference is made to Figure 28. which shows a 200 ft (61 m) deep cohesionless soil profile subjected to the El Centro Earthquake.

The free-field response was determined using a nonlinear one-dimensional response analysis from the displacement profiles shown at specific times. Curvatures can be computed and pile bending moments calculated if it is assumed that the pile is constrained to displace in phase with the free-field response.

Large curvatures could develop at interfaces between soft and rigid soils and, clearly, in such cases emphasis should be placed on using flexible ductile piles, iii Margason 19 suggests that curvatures of up to 6 x 10-4 '!'} in.-l (15.24 x 10-3 mm-l) could be induced by strong earthquakes but these should pose no problems to well designed steel or prestressed concrete pile.

Studies incorporating the complete soil-pile-structure interaction system as presented by Figure 27. have been described by Penzien[20] for a bridge piling system in a deep soft clay. A similar but somewhat simpler soil-pile structure interaction system (SPASM) to that used by Penzien, has been described by Matlock et al.[21] The model used is, in effect, a dynamic version of the previously mentioned BMCOL program.

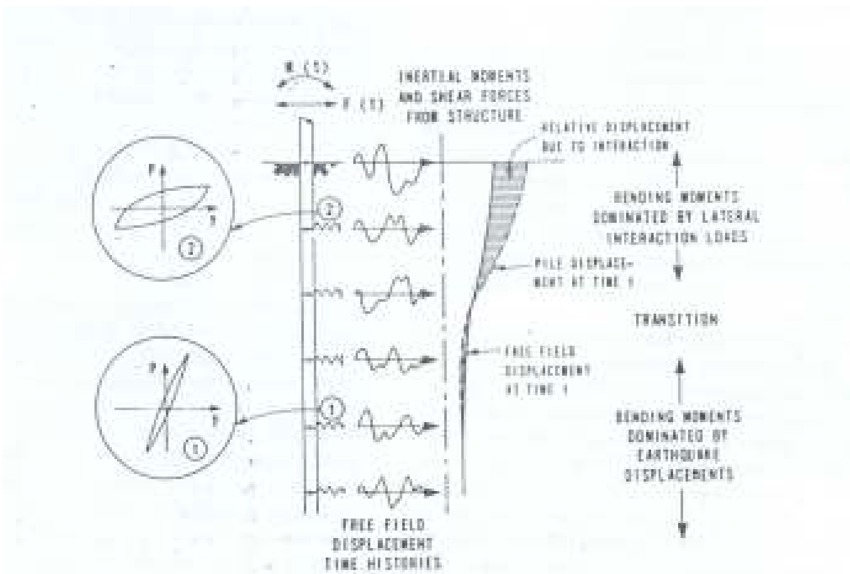

Figure 27 : Mechanism or Soil Pile Interaction During, Seismic Loading

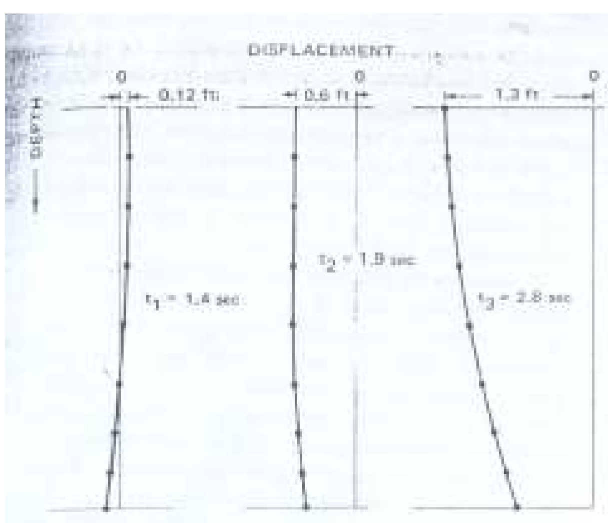

Figure 28 : Typical Earthquake Displacement Profiles

C6.3.1(C) Special Pile Requirements

The uncertainties of ground and bridge response characteristics lead to the desirability of providing tolerant pile foundation systems. Toughness under induced curvature and shears is required and hence piles such as steel H-sections and concrete filled steel-cased piles are favored for highly seismic areas. Unreinforced concrete piles are brittle in nature, so nominal longitudinal reinforcing specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie elements together and to assist in load transfer from the pile to the pile cap.

Experience has shown that reinforced concrete piles tend to hinge or shatter immediately below the pile cap. Hence tie spacing is reduced in this area so that the concrete is better confined. Driven precast piles should be constructed with considerable spiral confining steel to ensure good shear strength and tolerance of yield curvatures should these be imparted by the soil or structural response. Clearly, it is desirable to ensure that piles do not fail below ground level, and that flexural yielding in the columns is forced to occur above ground level.

C6.3.2 Abutments

The numerous case histories of damage to, or failure of, bridges induced by abutment failure or displacement during earthquakes have clearly demonstrated the need for careful attention to abutment design and detailing in seismic areas. Damage is typically associated with fill settlement or slumping, displacements induced by high seismically-induced lateral earth pressures or the transfer of high longitudinal or transverse inertia forces from the bridge structure itself. Settlement of abutment backfill, severe abutment damage or bridge deck damage induced by the movement of abutments may cause loss of bridge access, and hence abutments must be considered as a vital link in the overall seismic design process for bridges.

The nature of abutment movement or damage during past earthquakes has been well documented in the literature. Evans[22] examined the abutments of 39 bridges within 30 miles (48.3 km) of the 1968m M7 Inangahua earthquake in New Zealand, of which 23 showed measurable movement and 15 were damaged. Movements of free standing abutments followed the general pattern of outward motion and rotation about the top after contact with and restraint by the superstructures. Fill settlements were observed to be 10 to 15% of the fill height. Damage effects on bridge abutments in the M7.1 Madang earthquake in New Guinea reported by Ellison[23] were similar; abutment movement as much as 20in, (500 mm) were noted. Damage to abutments in the 1971 San Fernando earthquake is described by Fung et al.[20]. Numerous instances of abutment displacement and associated damage have been reported in publications on Niigata and Alaskan earthquakes. However, these failures were primarily associated with liquefaction of foundation soils.

Design features of abutments vary tremendously, and depend on the nature of the bridge site, foundation soils, bridge span length and load magnitudes. Abutment types include free-standing gravity walls, cantilever walls, tied back walls, and monolithic diaphragms. Foundation support may use spread footings, vertical piles or battered piles, while connection details to the superstructure may incorporate roller supports, electrometric bearings or fixed bolted connections. Considering the number of potential design variables together with the complex nature of soil-abutment-superstructure interaction during earthquakes, it is clear that the seismic design of abutments necessitates many simplifying assumptions.

C6.3.2(A) Free Standing Abutments

For free-standing abutments such as gravity or cantilever walls, which are able to yield laterally during an earthquake (i.e., superstructure supported by bearings which are able to slide freely) the well-established- Mononobe-Okabe pseudo-static approach outlined below, is widely used to compute' earth pressures induced by earthquakes.

For free-standing abutments in highly seismic areas design of abutments to provide zero displacement under peak ground accelerations may be unrealistic, and design for an acceptable small lateral displacement may be preferable. A recently developed method for computing the magnitude of relative wall displacement during a given earthquake is outlined in this subsection. Based on this simplified approach, recommendations are made for the selection of a pseudo-static seismic coefficient and the corresponding displacement level for a given effective peak ground acceleration.

Mononobe-Okabe Analysis

The method most frequently used for the calculation of the seismic soil forces acting on a bridge abutment is a static approach developed in the 1920s by Mononobe[25] and Okabe[26]. The Mononobe-Okabe analysis is an extension of the Coulomb sliding-wedge theory taking into account horizontal and vertical inertia forces acting on the soil. The analysis is described in detail by Seed and Whitman[27] and Richards and Elms[28]. The following assumptions are made:

1. The abutment is free to yield sufficiently to enable full soil strength or active pressure conditions to be mobilized. If the abutment is rigidly fixed and unable to move. The soil forces will be much higher than the predicted by the Mononobe-Okabe analysis.)
2. The backfill is cohesionless with a friction angle of ϕ.
3. The backfill is unsaturated so that liquefaction problems will not arise.

Equilibrium considerations of the soil wedge behind the abutment (Figure 29) then lead to a value. E_{AE} of the active force exerted on the soil mass by the abutment (and vice versa), when the abutment is at the point of failure: E_{AE} is given by the expression:

$$E_{AE} = ½ \gamma H2 (1 - K_v) K_{AE} \qquad \text{(C6-3)}$$

where the seismic active pressure coefficient K_{AE} is

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta \cos^2\beta \cos(\delta + \beta + \theta}$$

$$X \left| 1 + \sqrt{\frac{(\sin\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}} \right|^2 \qquad \text{(C6-4)}$$

and where

γ = unit weight of soil
H = height of soil face
ϕ = angle of friction of soil

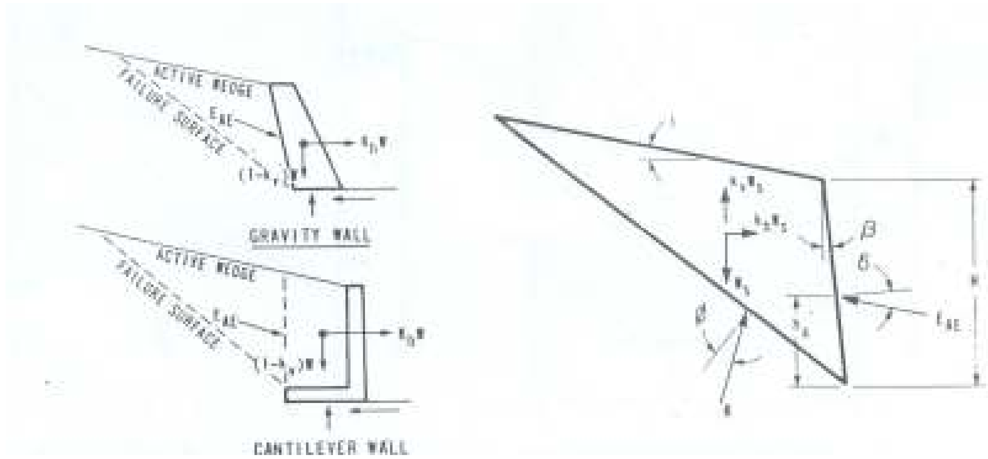

Figure 29. Active Wedge Force Diagram

θ	=	arc tan (k_h/l -K_v)
δ	=	angle of friction between soil and abutment
K_h	=	horizontal acceleration coefficient
K_v	=	vertical acceleration coefficient
i	=	back fill slope angle
β	=	slope of soil face.

The equivalent expression for passive force if the abutment is being pushed into the backfill is:

$$E_{pE} = 1/2\gamma H^2(l - k_v)K_{pE} \qquad \text{(C6-5)}$$

where the seismic passive pressure coefficient K_{PE} is

$$K_{pE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta \cos^2\beta \cos(\delta - \beta + \theta}$$

$$x\left(1 - \sqrt{\frac{(\sin\phi + \delta)\sin(\phi - \theta + i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right)^2 \qquad \text{(C6-6)}$$

As the seismic inertial angle θ increases, the values of K_{AE} and K_{PE} approach each other and, for vertical backfill, become equal when $\theta = \phi$

Despite the relative simplicity of the approach, the accuracy of Eq. C6-3 has been substantiated by model tests[27] and back calculation from observed failures of flood channels walls.[29] In the latter case. However, the displacements were large: and this, as will be seen, can modify the effective values of k_h at which failure occurs.

The value of h, the height at which the resultant of the soil pressure acts on the abutment, may be taken as Hl3 for the static case with no earthquake effects involved. However, it becomes greater as earthquake effects increase. This has been shown by tests and theoretically by Wood, 3D who found that the resultant of the dynamic pressure acted approximately at mid height. Seed and Whitman have suggested that h could be obtained by assuming that the static component of the soil force (computed from Eq. C6-3 with 8 = kv = 0) acts at H/3 from the bottom of the abutment, while the additional dynamic effect should be taken to act at a height of 0.6H. For most purposes it is sufficient to assume h = H/2, with a uniformly distributed pressure.

Although the Mononobe-Okabe expression for active thrust is easily evaluated for any particular geometry and friction angle, the significance of the various parameters is obvious. Figure 30 shows the variation of K_{AE} against k_h for different values of and k_v; K_{AE} is obviously very sensitive to the value of ϕ. Also, for a constant value of ϕ, K_{AE} doubles as K_h increases from zero to 0.35 for zero vertical acceleration, and thereafter it increases more rapidly.

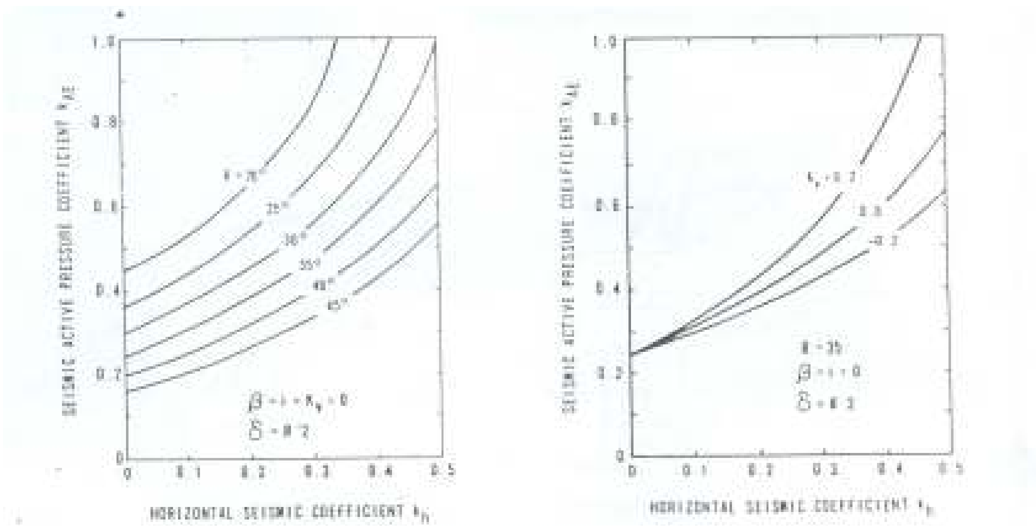

Figure 30 : Effect or Seismic Coefficients and Soil Friction Angle on Seismic Active Pressure Coefficient

due to earthquake effects more easily, K_{AE} can be normalized by dividing by its static value KA to give a thrust factor;

$FT = K_{AE}/K_A$ (C6-7)

Whereas Figure 30 shows that K_{AE} is sensitive to changes in the soil friction angle <p, the plots of FT against <P in Figure 31 indicate that the value of <P has little effect on the thrust factor until quite suddenly, over a short range of <P, FT increases rapidly and becomes infinite for specific critical values of ϕ. The reason for this behavior may be determined by examining Eq. C6-4. The contents of the radical must be positive for a real solution to be possible, and for this it is necessary that

$$\phi \geq i + \theta = i + \arctan\left(\frac{K_h}{1 - K_V}\right) \quad \text{(C6-8)}$$

This condition could also be thought of as specifying a limit to the horizontal acceleration coefficient that could be sustained by any structure in a given soil. The limiting condition is that

$K_h < (1 – K_v) \tan(\phi - I)$ (C6-9)

For zero vertical acceleration and backfill angle and for a soil friction angle of 35°, the limiting value of k_h is 0.7. This is a figure of some interest in that it provides an absolute upper bound for the seismic acceleration that can be transmitted to any structure whatsoever built soil with the given strength characteristics.

Figure 32 shows the effect on F^T of changes in the vertical acceleration coefficient k_v. Positive values of k have a significant effect for values of k_h greater than 0.2. The effect is greater than 10% above and to the right of the dashed line.

As is to be expected from Eq. C6-6, K_{AE} and Fr are also sensitive to variations in backfill slope, particularly for higher values of horizontal acceleration coefficient when the limit implied by Eq. C6-6 is approached. This, effect is shown in Figure 33.

The effects of abutment inertia are not taken into account in the Mononobe-Okabe analysis. Many current procedures assume that the inertia forces due to the mass of the abutment itself may be neglected in considering seismic behavior and seismic design. This is not a conservative assumption, and for those abutments relying on their mass for stability it is also an unreasonable assumption. In that to neglect the mass is to neglect a major aspect of their behavior. The effects of wall inertia are discussed further by Richards and Elms, [28] who show that wall inertia forces should not be neglected in the design of gravity retaining walls.

Design for Displacement

If peak ground accelerations are used in the Mononobe-Okabe analysis method, the size of gravity retaining structures will often be excessively great. To provide a more economic structure, design for a small tolerable displacement rather than no displacement may be preferable.

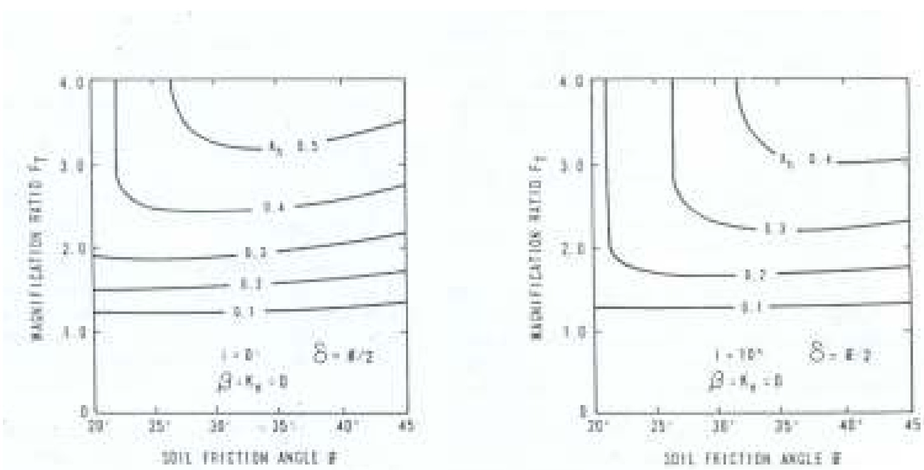

Figure 31 : Influence of Soil Friction Angle on Magnification Ratio

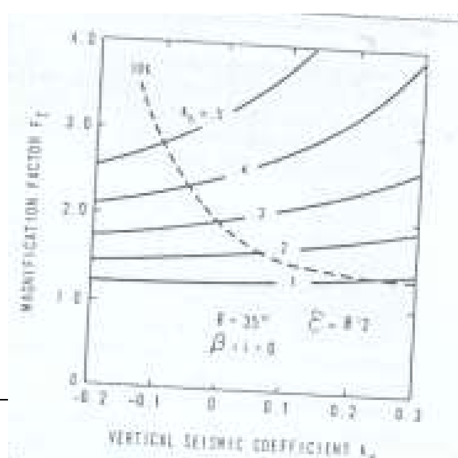

Figure 32 : Influence of Vertical Seismic Coefficient on Magnification Ratio

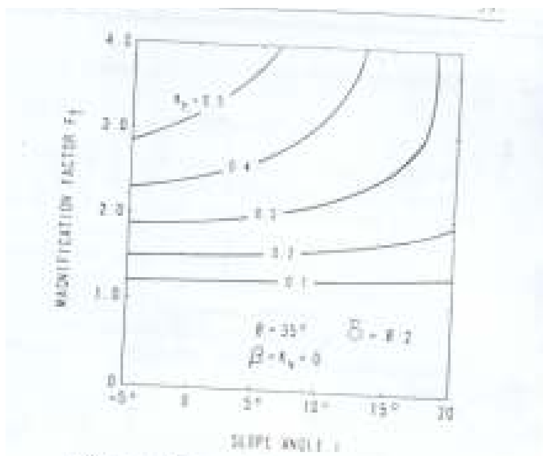

Figure 33 : Influence of Backfill Slope Angle on Magnification Ratio

Tests have shown that a gravity retaining wall fails in an incremental manner in an earthquake. For any earth quake ground motion, the total relative displacement may be calculated using the sliding block method suggested by Newmark.[31] The method assumes a displacement pattern similar to that of a block resting on a plane rough horizontal surface subjected to an earthquake, with the block being free to move against friction Resistance in one direction only. Figure 34 sh0ws how the relative displacement relates to the acceleration and velocity time histories of soil and wall. At a critical value of K_h, the wall is assumed to begin sliding; relative motion will continue until wall and soil velocities are equal. Figures 35 and 36 show the results CS of a computation of wall displacement for k_h = O. I for the El Centro 1940 N-S record.

Newmark computed the maximum displacement response for four earthquake records, and plotted the results after scaling the earthquakes to a common maximum acceleration and velocity. Franklin and Chang[32] repeated the analysis for a large number of both natural and synthetic records and added their results to the same plot. Upper bound envelopes for their results are shown in Figure 37. All records were scaled to a maximum acceleration coefficient of 0.5 and a maximum velocity V of 30 in./sec (762 mm/sec). The maximum resistance of coefficient N is the maximum acceleration coefficient sustainable by a sliding block before it slides. In a case of a wall designed using the Mononobe-Okabe method, the maximum coefficient is, of course, k_h.

Figure 37 show that the displacement envelopes for all the scaled records have roughly the same shape.

An approximation to the curves for relatively low displacements is given by the relations, expressed in any consistent set of units

$$d = 0.087\frac{v^2}{Ag}(\frac{N}{A})^{-4} \qquad \text{(C6-10)}$$

where d is the total relative displacement of a wall subjected to an earthquake ground motion whose maximum acceleration coefficient and maximum velocity are A and V respectively. This is drawn as a straight line on Figure 37. Note (hat as this expression has been derived from envelope Curves. it will overestimate d for most earthquakes.

One possible design procedure would be to choose a desired value of maximum wall displacement d together with appropriate earthquake parameters, and to use Eq. C6-IO to derive a value of the seismic acceleration coefficient for which the wall should be designed. The wall connections, if any, could then be detailed to allow for this displacement.

By applying the above procedure 10 several simplified examples, Elms and Martin have shown that a value of k_h = A/2 is adequate for most design purposes provided that allowance is made for an outward displacement of the abutment of up to 10A in. (254A mm).

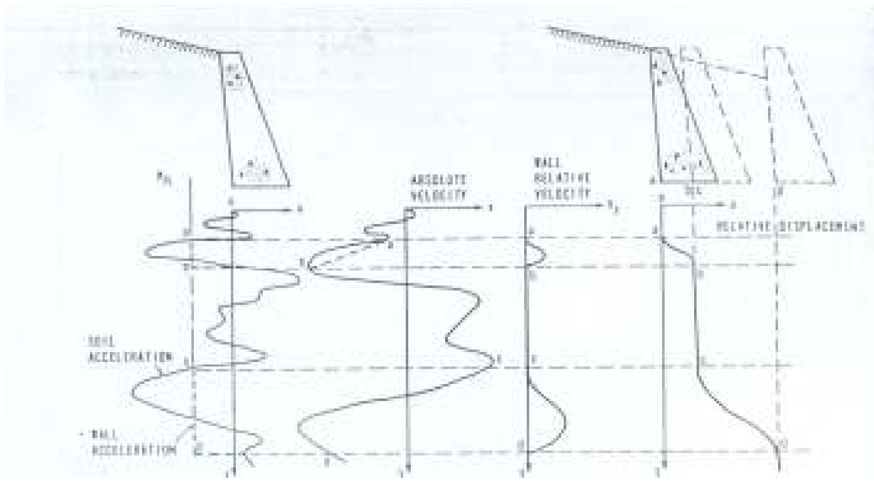

Figure 34 : Relation Between Relative Displacement and Acceleration and Velocity Time Histories of Soil and Wall

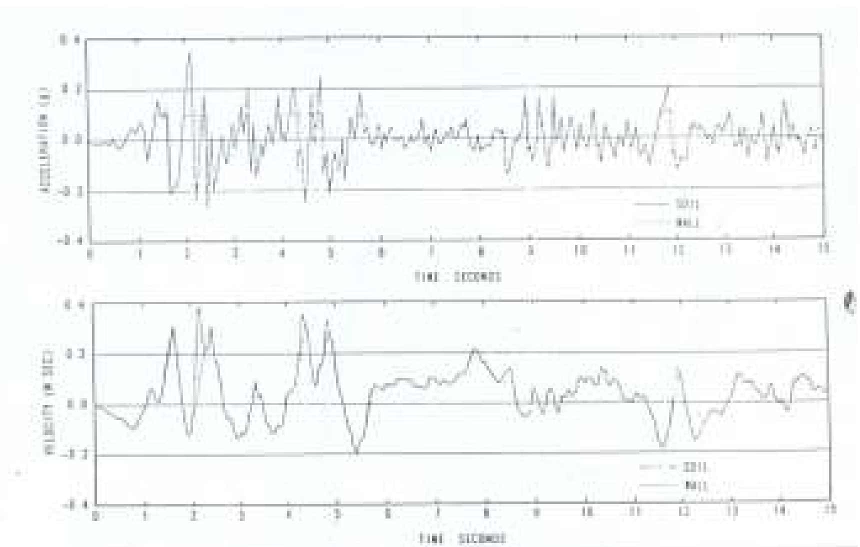

Figure 35 : Acceleration and Velocity Time Histories of Soil and Wall (El Centro 1940 N-S record)

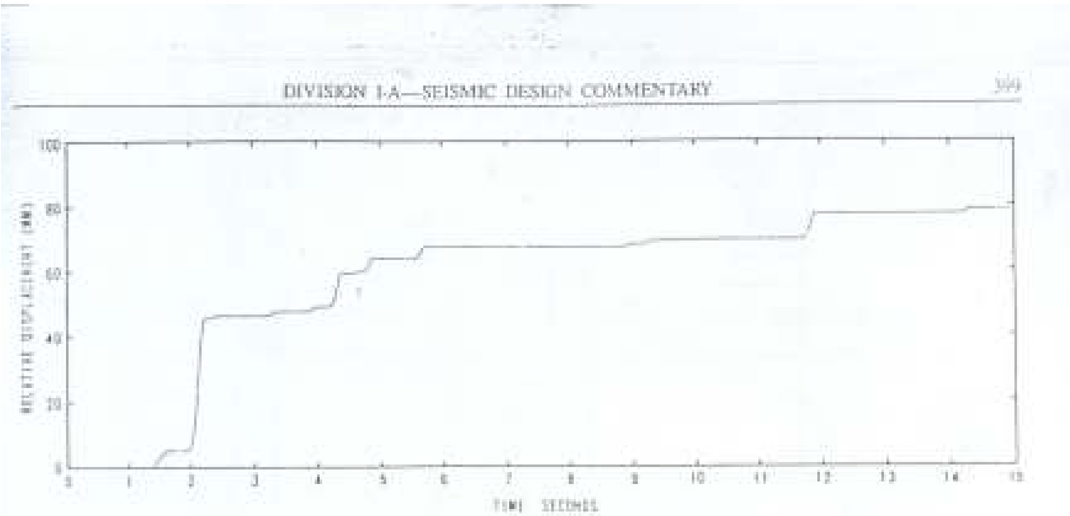

Figure 36 : Relative Displacement of Wall (EO Centro 1940 N-S record)

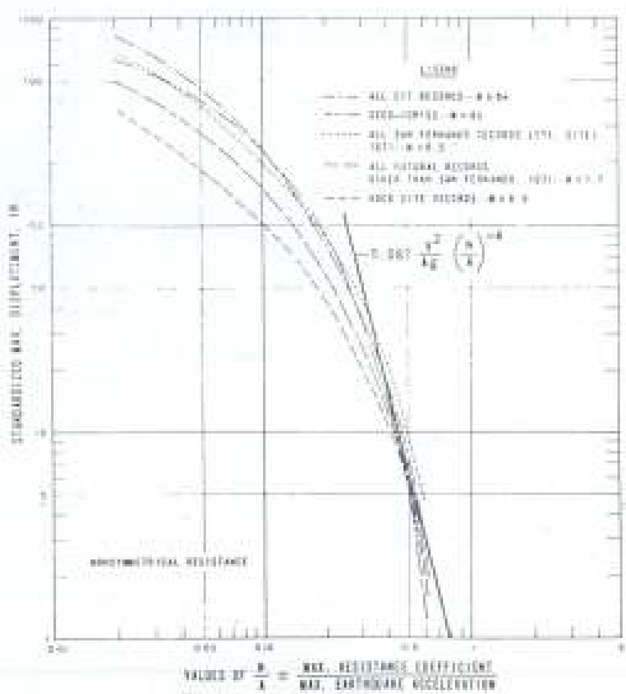

Figure 37. Upper Bound Envelop Curves of Permanent Displacements for All Natural and Synthetic Records Analyzed by Franklin and Change (I in = 25.4mm)

For sliding steel bearings or pot bearings, force diagrams describing limiting equilibrium conditions for a simple abutment are shown in Figure 38. Where bearings comprise unconfined elastomeric pads, the nature of the forces transferred to the abutment becomes more complex. Since such bearings are capable of transferring significant force. The magnitude of the force initially depends on the relative movement between the superstructure and the abutment and force magnitudes can become quite large before slip will occur.

The use of a settlement or approach slab in Figures 39 and 40 which has the effect of providing bridge access in the event of backfill settlement is also noted. The slab also provides an additional abutment friction anchorage against lateral movement.

Non – Yielding Abutments

As previously noted, the Mononobe – Okabe analysis assumes that the abutment if free to yield laterally a sufficient amount to mobilize peak soil strengths in the soil backfill. For granular soils, peak strengths can be assumed to be mobilized if deflections at the top of the wall are about 0.5% of the abutment height. For abutments which are restrained against lateral movement by tie backs and batter piles, lateral pressures induced by inertia forces in the backfill will be greater than those given by a Mononobe – Okabe analysis. Simplified elastic solutions presented by Wood[30] for rigid non-yielding walls, also indicate that pressures are greater than those given by Mononobe –Okabe. The use of a factor of 1.5 in conjunction with peak ground accelerations is suggested for design where doubt exists that an abutment can yield sufficiently to mobilize soil strengths.

C6.3.2(B) Monolithic Abutments

Monolithic or end diaphragm abutments such as shown in Figure 40 are commonly used for single and for two span bridges in California. As shown, the end diaphragm is cast monolithically with the superstructure and may be directly supported on piles, or provision may be made for beam shortening during post-tensioning. The diaphragm acts as a retaining wall with the superstructure acting as a prop between abutments.

Such abutments have performed well during earthquakes and avoid problems such as backwall and bearing damage associated with yielding abutments, and reduces the lateral load taken by columns or piers. On the other hand, higher longitudinal and transverse superstructure inertia forces are transmitted directly in to the backfill and provision must be made for adequate passive resistance to avoid excessive relative displacements.)

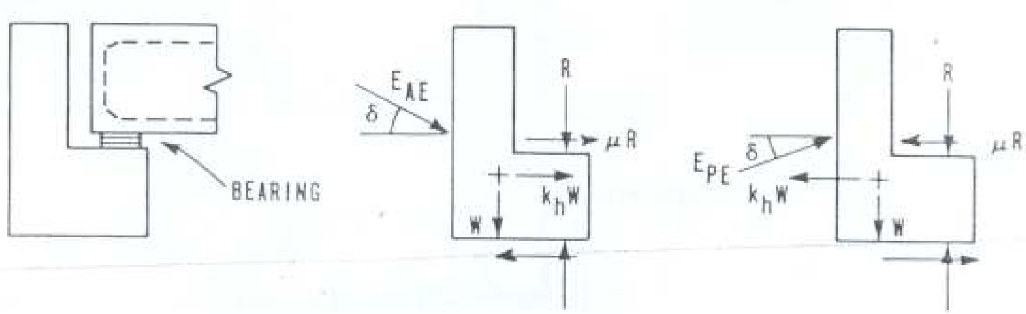

Figure 38 : Force Diagrams Including Bearing Friction

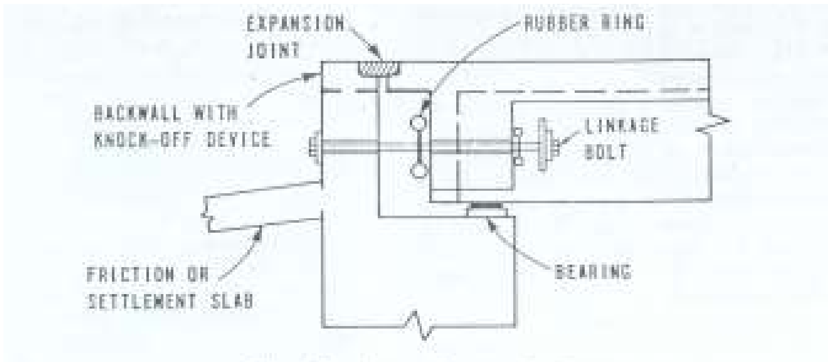

Figure 39 : Abutment Support Detail

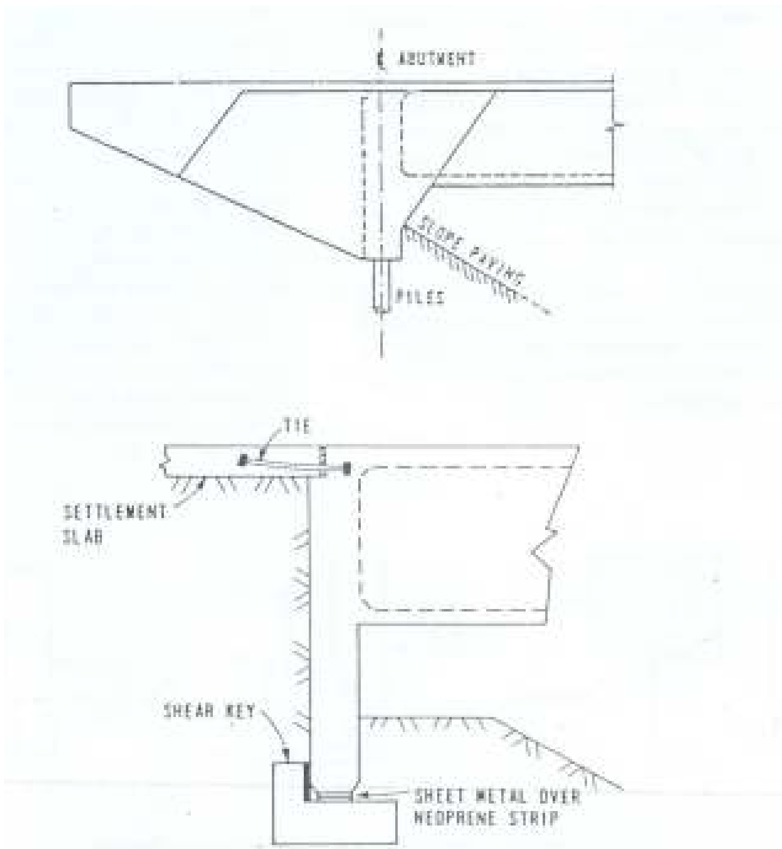

Figure 40. Typical Monolithic Abutments

Whereas free-standing or seat type abutments allow the engineer more control over development of soil forces, the added joint introduces a potential collapse mechanism into the structure. In making estimates of monolithic abutment stiffness and associated longitudinal displacements during transfer of peak earthquake forces from the structure, it is recommended that abutments be proportioned to resist displacements to 0.3 ft (91.4 mm) or less in order to minimize damage.

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COMMENTARY

SECTION 7 -STRUCTURAL STEEL

C7.1 GENERAL

The 50% increase in allowable stresses for service load design is based on the following:

I. The margin of safety between the yield strength and allowable stress of short columns.
2. The margin of safety between the yield strength and allowable tensile stress.
3. The margin of safety of compression members, which varies between 1.7 and 1.9,[1,2]

C7.3 SEISMIC PERFORMANCE CATEGORIES 3

This subsection provides modifications to the interaction equations when the P-delta effects are explicitly determined. In columns, the reductions to the allowable stresses are in part a result of the consideration of member P-delta effects. The selection of the value of Cm where joint translation is permitted was an approximation applicable primarily to designs for which significant applied horizontal forces are not present. Since the advent of computer analysis the solution of the interaction equations when secondary effects resulting from deflection are taken into account, has become much easier. In most cases with significant horizontal displacements the first iteration of deflection) is sufficient. It is possible that for some members such' as weak axis columns depending on end support conditions, critical stress may occur at the mid-height rather than the column ends. Thus the stress limits specified when joint translation is prevented should not be exceeded.

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COMMENTARY

SECTION 8-REINFORCED CONCRETE

C8.1 GENERAL

The purpose of the additional design requirements of this section is to ensure, that the design of the components of a bridge are consistent with the overall design philosophy and that the potential for failures observed in past earthquakes is minimized. The additional design requirements for piers provide some inelastic capacity; however, the R-factor specified for piers is such that anticipated inelastic capacity is significantly Jess than that of columns.

The actual ductility demand on a column or pier is a complex function of a number of variables 1.2 including the earthquake characteristic, design force level. period of the bridge, shape of the inelastic hysteresis loop of the columns, elastic damping coefficient, contributions of foundation and bearing compliance to structural flexibility and plastic hinge length of the column. The damage potential of a column is also related to the ratio of the duration of strong motion shaking to the natural period of the bridge. This ratio will be an indicator of the number of yield excursions, and hence of the cumulative ductility. There are some grounds for considering the cumulative ductility to be a more useful index than the peak ductility level; for example, 10 cycles at a curvature ductility factor of 8 might be more damaging than one yield excursion at a curvature ductility factor of 10 or 12. However, there is little experimental evidence to support or contradict this view.

Both Service Load and Load Factor methods of design are permitted although it is recommended that the Load Factor method of design be used since it is consistent with the ultimate load capacity concept used in determining the design force levels. An increase in allowable stresses of $33^{1/3}$% is permitted to fix Service Load design.

C8.2 SEISMIC PERFORMANCE CATEGORY 1 & 2

Consistent with the overall philosophy for bridges classified as SPC 1 & 2, special seismic design requirements were eliminated because of the low level of seismic and the low probability that a column would be subjected to seismic forces that would cause yielding.

C8.3 SEISMIC PERFORMANCE CATEGORY 3

Bridges classified as SPC 3 have a reasonable probability of being subjected to seismic forces that will cause yielding of the columns. Thus it was deemed necessary that columns have some ductility capacity. The most important requirement to ensure some level of ductility is the transverse reinforcement requirement specified. [1.2] This will prevent buckling of the longitudinal steel and provide confinement for the core of the column. The maximum spacing for the transverse reinforcement was increased to 6 in. (152 mm) because of the anticipated lower ductility demand.

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Appendix A

Basis of HA and HB highway loading

Type HA loading is the normal design loading for Great Britain, where it represents the effects of normal permitted vehicles * other than those used for the carriage of abnormal indivisible loads.

For loaded lengths up to 30 m, the loading approximately represents closely spaced vehicles of 24 t laden weight in each of two traffic lanes. For longer loaded lengths the spacing is progressively increased and medium weight vehicles of 10 t and 5 t are interspersed. It should be noted that although normal commercial vehicles of considerably greater weight are permitted in Great Britain their effects are restricted, so as not to exceed those of HA loading, by limiting the weight of axles and providing for increased overall length.

In considering the impact effect of vehicles on highway bridges an allowance of 25 % on one axle or pair of adjacent wheels was made in deriving HA loading. This is considered an adequate allowance in conditions such as prevail in Great Britain.

This loading has been examined in comparison with traffic as described for both elastic and collapse methods of analysis, and has been found to give a satisfactory correspondence in behaviour.

HB loading requirements derive from the nature of exceptional industrial loads (e.g. electrical transformers, generators, pressure vessels, machine presses, etc.) likely to use the roads in the area.

Appendix B

Recommendations for the protection of piers by safety fences

The space available for deflection determines the arrangement of the safety fences. If the clearance between the pier and the guard rail is 0.6 m or more, the guard rail should be mounted on posts to form a free-standing fence. If the clearance is less than 0.6 m, the guard rail should be mounted on the traffic face of the member by means of energy absorbing brackets. Whatever the arrangement the protection afforded should be such that when a car of 1.5 t strikes the safety fence at 110 km/h, and at an angle of 20°, the wheels of the car will only just reach the member.

Appendix C

Vibration serviceability requirements for foot and cycle track bridges

C.1 General. For superstructures where f_o, the fundamental natural frequency of vibration for the unloaded bridge, exceeds 5 Hz, the vibration serviceability requirement is deemed to be satisfied.

For superstructures where f_o is equal to, or less than, 5 Hz, the maximum vertical acceleration of any part of the superstructure shall be limited to $0.5\sqrt{f_o}$ m/s^2. The maximum vertical acceleration shall be calculated in accordance with **C.2** or **C.3**, as appropriate.

C.2 Simplified method for deriving maximum vertical acceleration. This method is valid only for single span, or two- or three-span continuous, symmetric superstructures, of constant cross section and supported on bearings that may be idealized as simple supports.

The maximum vertical acceleration a (in m/s^2) shall be taken as

$$a = 4\pi^2 f_o^{\,2} y_s k \psi$$

where

f_o is the fundamental natural frequency (in Hz) (see C.2.3)
Ys is the static deflection (in m) (see C.2.4)
K is the configuration factor (see C.2.5)
Ψ is the dynamic response factor (see C.2.6)

For values of f_o greater than *4* Hz the calculated maximum acceleration may be reduced by an amount varying linearly from zero reduction at *4* Hz to 70% reduction at 5 Hz.

C.2.1 ***Modulus of elasticity***. In calculating the values of f_o and y_s, the short-term modulus of elasticity shall be used for concrete (see Parts 7 and 8 of this standard), and for steel as given in Part 6 of this standard.

C.2.2 ***Second moment of area***. In calculating the values of f_o and y_s, the second moment of area for sections of discrete concrete members may be based on the entire uncracked concrete section ignoring the presence of reinforcement. The effects of shear lag need not be taken into account in steel and concrete bridges.

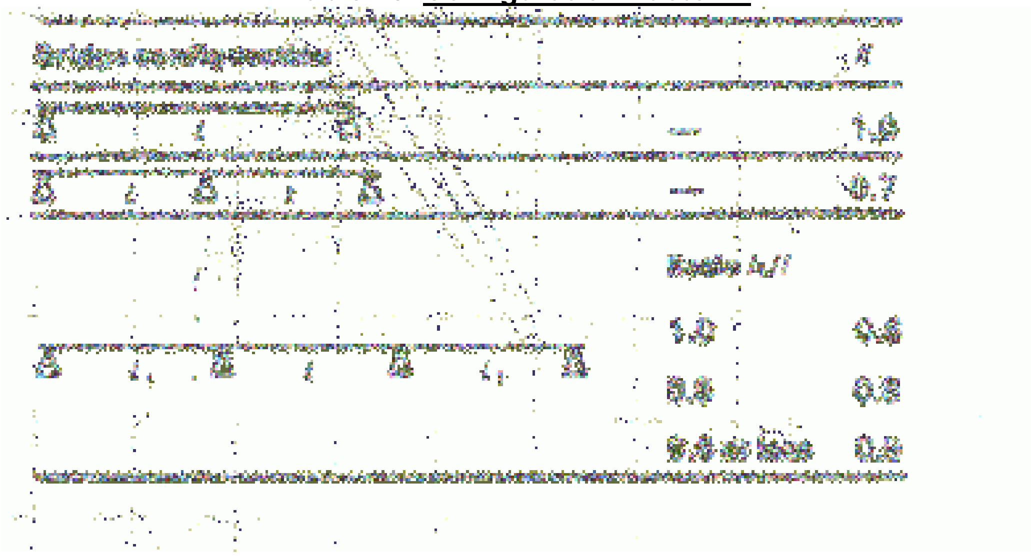
C.2.3 ***Fundamental natural frequency f_o***. The fundamental natural frequency f_o is evaluated for the bridge including superimposed dead load but excluding pedestrian live loading.

The stiffness of the parapets shall be included where they contribute to the overall flexural stiffness of the superstructure.

C.2.4 ***Static deflection y_s***. The static deflection y_s is taken at the midpoint of the main span for a vertical concentrated load of 0.7 kN applied at this point. For three-span superstructures, the centre span is taken as the main span.

C.2.5 ***Configuration factor K***. Values of *K* shall be taken from table 18.

Table 18. Configuration Factor K



Bridge configuration		K
	—	1.0
	—	0.7
	Ratio l_1/l	
	1.0	0.6
	0.8	0.8
	0.6 or less	[illegible]

For three-span continuous bridges, intermediate values of K may be obtained by linear interpolation.

C.2.6 ***Dynamic response factor Ψ.*** Values of ψ are given in figure 41. In the absence of more precise information, the values of δ (the logarithmic decrement of the decay of vibration due to structural damping) given in table 19 should be used.

Table 19. Logarithmic decrement of decay of vibration δ.

Bridge superstructure	
Steel with asphalt or epoxy surfacing	0.03
Composite steel/concrete	0.04
Prestressed and reinforced concrete	0.05

C.3 General method for deriving maximum vertical acceleration. For superstructures other than those specified in C.2, the maximum vertical acceleration should be calculated assuming that the dynamic loading applied by a pedestrian can be represented by a pulsating point load f, moving across the main span of the superstructure at a constant speed V_t as follows.

$f = 180 \sin 2\pi f_o T$ (in N), where T is the time (in s),
$V_t = 0.9 \; f_o$ (in m/s).

For values of f_o greater than 4 Hz, the calculated maximum acceleration may be reduced by an amount varying linearly from zero reduction at 4 Hz to 70 % reduction at 5 Hz.

C.4 Damage from forced vibration. Consideration should be given to the possibility of permanent damage to a superstructure by a group of pedestrians deliberately causing resonant oscillations of the superstructure. As a general precaution, therefore, the bearings should be of robust construction with adequate provision to resist upward or lateral movement.

For prestressed concrete construction, resonant oscillation may result in a reversal of up to 10 % of the static live load bending moment. Providing that sufficient unstressed reinforcement is available to prevent gross cracking, no further consideration need be given to this effect.

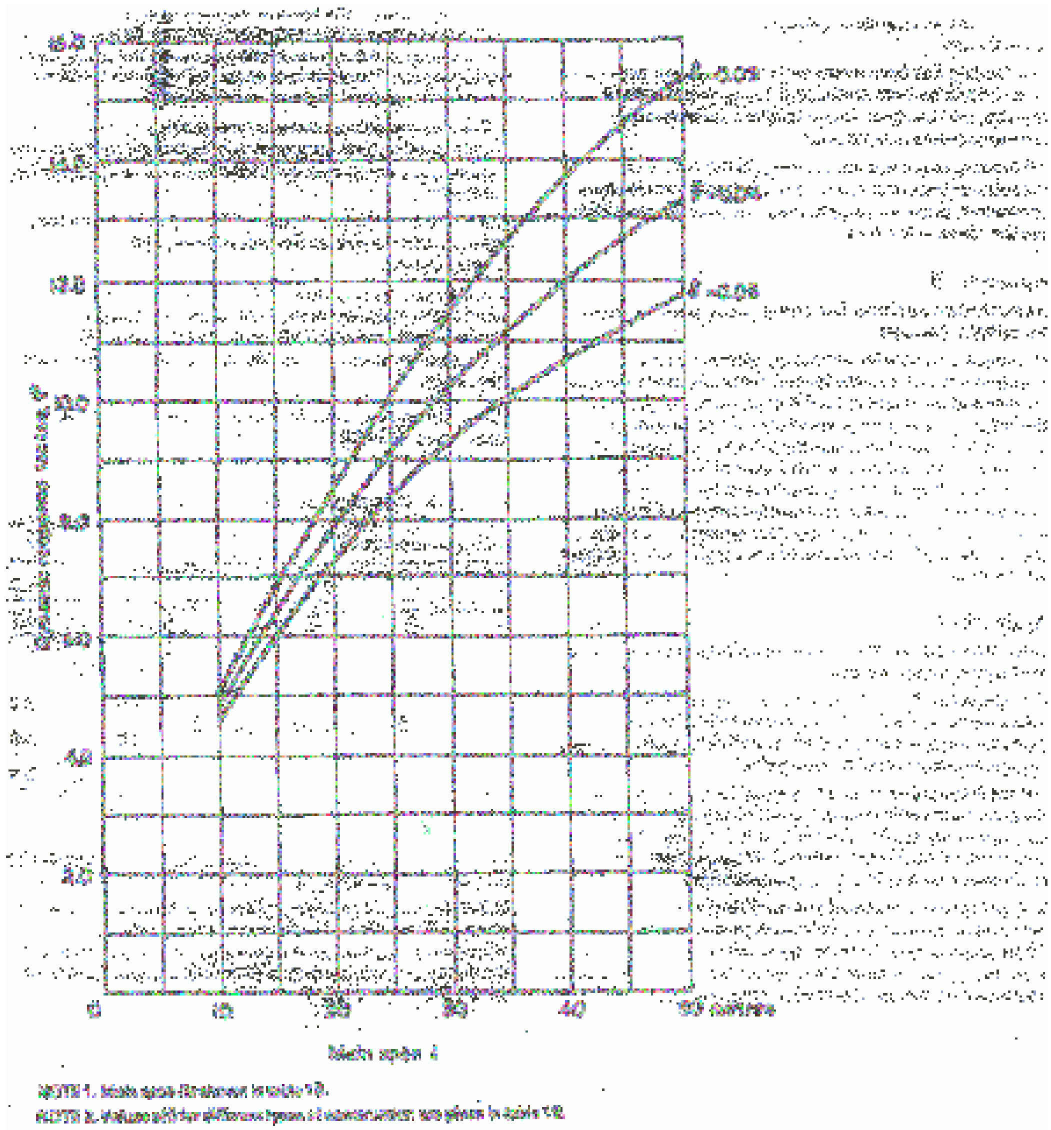


Figure 41 - Dynamic Response Factor Ψ

Appendix D

Derivation of RU and RL railway loadings

D.1 RU loading. The loading given in 8.2.1 has been derived by a Committee of the International Union of Railways to cover present and anticipated future loading on railways in Great Britain and on the Continent of Europe. Motive power now tends to be diesel and electric rather than steam, and this produces axle loads and arrangements for locomotives that are similar to those used for bogie freight vehicles, freight vehicles often being heavier than locomotives. In addition to the normal train loading, which can be represented quite well by a uniformly distributed load of 8 t/m, railway bridges are occasionally subject to exceptionally heavy abnormal loads. At short loaded lengths it is necessary to introduce heavier concentrated loads to simulate individual axles and to produce high end shears. Certain vehicles exceed RU static loading at certain spans, particularly in shear but these excesses are acceptable, because dynamic factors applied to RU loading assume high speeds whereas those occasional heavy loads run at much lower speeds.

The concentrated and distributed loads have been approximately converted into equivalent loads measured in kN when applying RU loading in this British Standard.

Figure 42 shows diagrams of two locomotives and several wagons all of which, when forming part of a train, are covered by RU loading. Double heading of the locomotives has been allowed for in RU loadings.

The allowances for dynamic effects for RU loading given in 8.2.3.1 have been calculated so that, in combination with that loading, they cover the effects of slow moving heavy, and fast moving light, vehicles. Exceptional vehicles are assumed to move at speeds not exceeding 80 km/h, heavy wagons at speeds of up to 120 km/h, passenger locomotives at speeds of up to 250 km/h and light, high speed trains at speeds of up to 300 km/h.

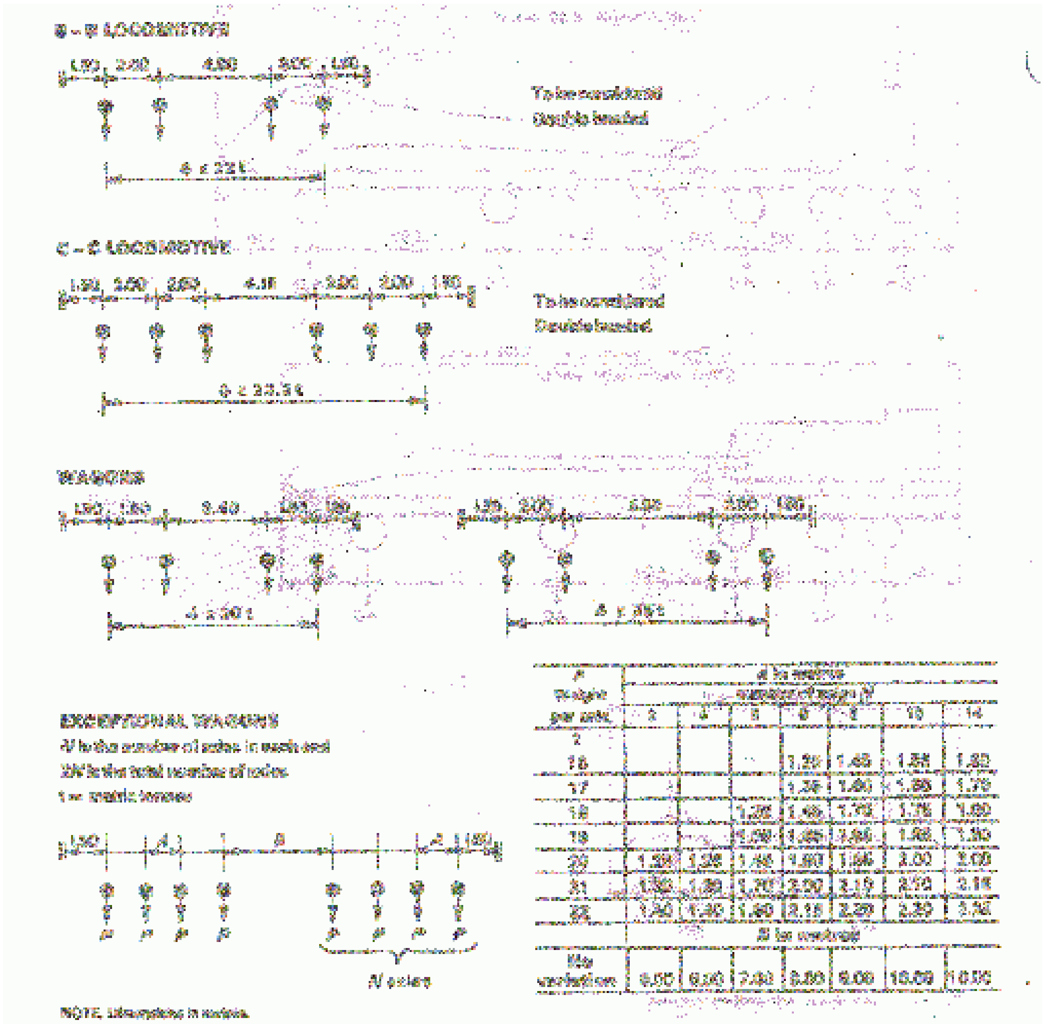


Figure 42 - Wagons and locomotives covered by RU loading

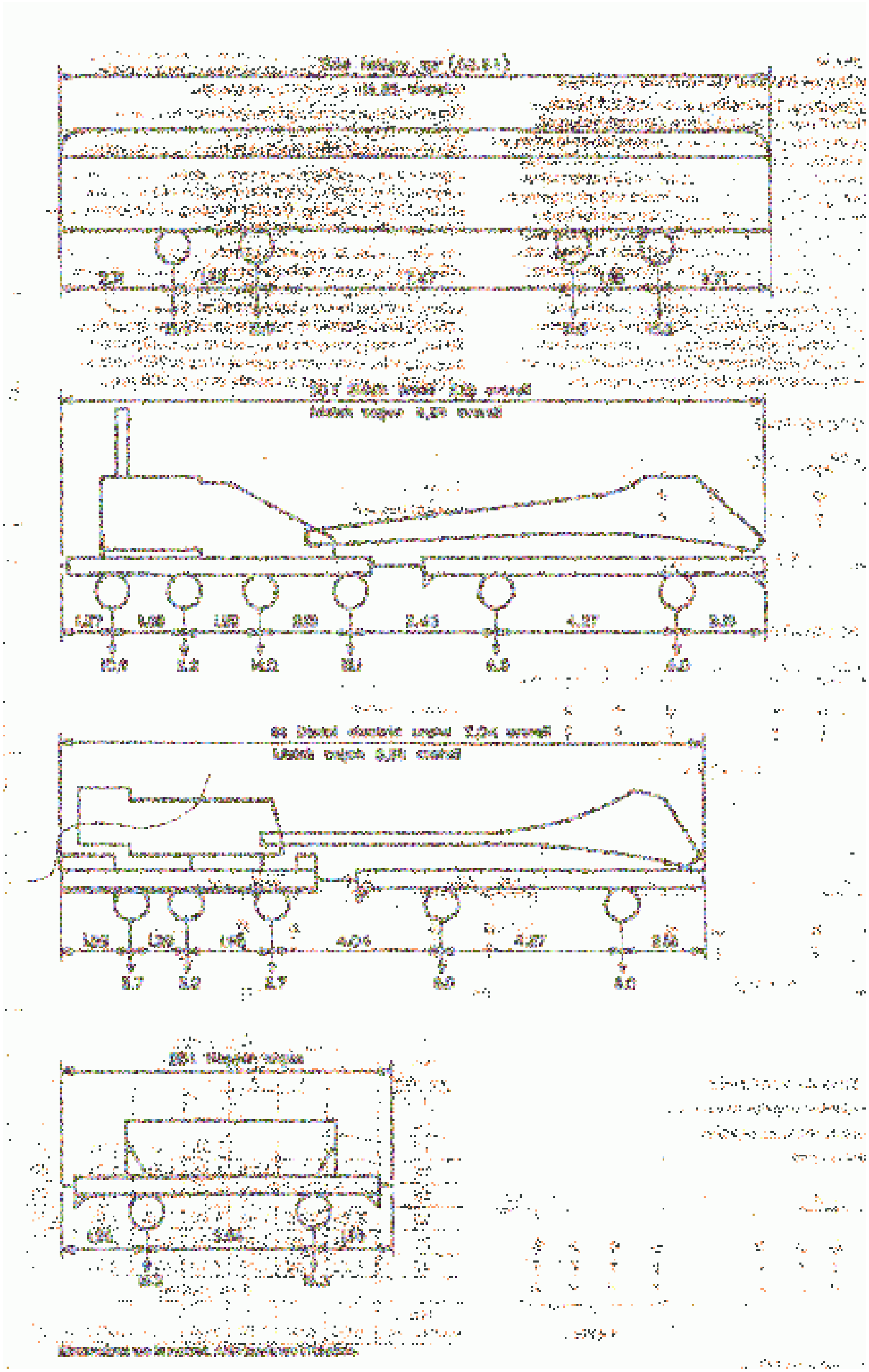


Figure 43 – Works trains vehicles covered by RL loading

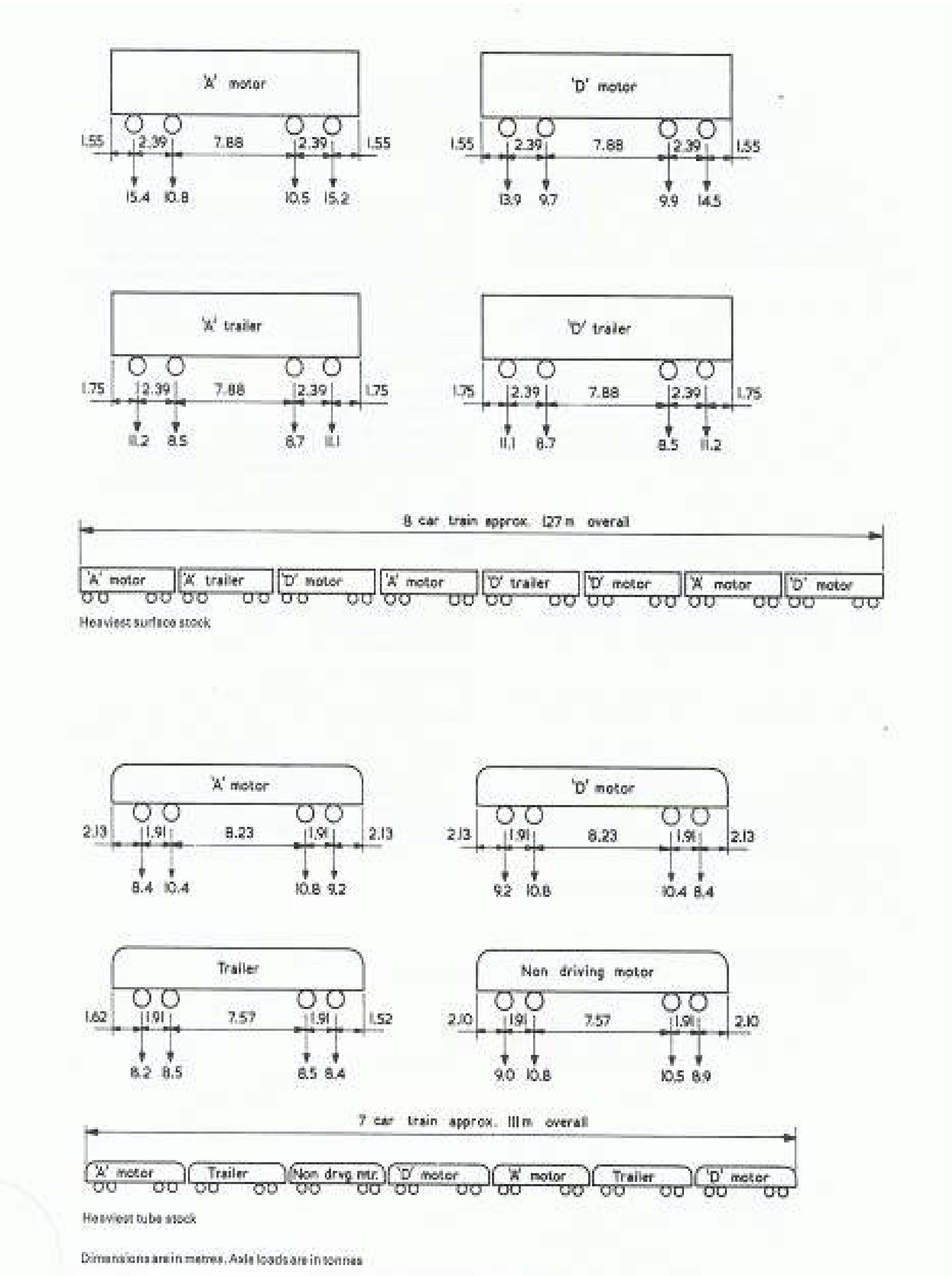


Figure 44 – Passengers vehicles covered by RL loading

The formulae for the dynamic effects are not to be used to calculate dynamic effects for a particular train on a particular bridge. Appropriate methods for this can be found by reference to a recommendation published by the International Union of Railways (UIC), Paris *.

Similar combinations of vehicle weight and speed have to be considered in the calculation of centrifugal loads. The factor f given in 8.2.9 allows for the reduction in vehicle weight with increasing speed above certain limits. The greatest envisaged speed is that which is possible for the alignment as determined by the physical conditions at the site of the bridge.

D.2 RL loading. The loading specified in 8.2.2 has been derived by the London Transport Executive to cover present and anticipated loading on lines that only carry rapid transit passenger stock and light engineers' works trains. This loading should not be used for lines carrying 'main line' locomotives or stock. Details are included in this appendix to allow other rapid transit passenger authorities to compare their actual loading where standard track of 1.432 m gauge is used but where rolling stock and locomotives are lighter than on the main line UIC railways.

RL loadings covers the following conditions, which are illustrated in figures 17 and 18.

(a) Works *trains.* This constitutes locomotives, cranes and wagons used for maintenance purposes. Locomotives are usually of the battery car type although very occasionally diesel shunters may be used. Rolling stock hauled includes a 30 t steam crane, 6 t diesel cranes, 20 t hopper wagons and bolster wagons. The heaviest train would comprise loaded hopper wagons hauled by battery cars.

(b) *Passenger trains. A* variety of stock of different ages, loadings and load gauges is used on surface and tube lines.

The dynamic factor has been kept to a relatively low constant, irrespective of span, because the heavier loads, which determine the static load state, arise from works trains which only travel at a maximum speed of about 32 km/h. The faster passenger trains produce lighter axle loads and a greater margin is therefore available for dynamic effects.

Loading tests carried out in the field on selected bridges produced the following conclusions.

(a) *Main girders*

(1) Works trains produce stresses about 20 % higher than static stresses.

(2) Passenger trains produce stresses about 30 % higher than static stresses.

(b) *Cross girders and rail bearers* (away from rail joints). *All* types of train produce stresses about 30% higher than static stresses.

(c) *Cross girders and rail bearers at rail joints*

(1) With no ballast, one member carrying all the joint effect (e.g. rail bearer or cross girder immediately under joint with no distribution effects), all trains can produce an increase over static stress of up to 27 % for each 10 km/h of speed.

(2) With no ballast, but with some distribution effects (e.g. cross girder with continuous rail bearers or heavy timbers above), all trains can produce an increase over static stress of up to 20 % for each 10 km/h of speed.

(3) With ballasted track, the rail joint effect is considerably reduced, depending on the standard and uniformity of compaction of ballast beneath the sleepers. The maximum increase in poorly maintained track is about 12 % for each 10 km/h of speed.

The equivalent static loading is over generous for short loaded lengths. However, it is short members that are most severely affected by the rail joint effect and, by allowing the slight possibility of a small overstress under ballasted rail joints, it has been found possible to adopt a constant dynamic factor of 1.2 to be applied to the equivalent static loading.

For the design of bridges consisting of independently acting linear members, the effects of trains are adequately covered by the effects of the basic RL loading system. Recent trends, however, are towards the inclusion of plate elements as principal deck members, and here the load representation is inadequate. *A* reinforced concrete slab deck between steel main girders, for example, will distribute concentrated loads over a significant length of the main girders and in consequence suffers longitudinal stresses from bending, shear and torsion.

To cater for this consideration, a check loading bogie has been introduced. This should be used only on deck structures to check the ability of the deck to distribute the load adequately. To allow for dynamic effects, an addition of 12 % per 10 km/h of speed has been made to the heaviest axle, assumed to be at a rail joint, and an additional 30 % has been made to the other axle of the bogie.

D.3 Use of tables 20 to 23 when designing for RU loading

D.3.1 ***Simply supported main girders and rail bearers.*** Bending moments in simply supported girders are to be determined using the total equivalent uniformly distributed load given in the tables for the span of the girder, assuming a parabolic bending moment diagram.

End shears and support reactions for such girders shall be taken from the tables giving end shear forces.

Shear forces at points other than the end shall be determined by using the static shear force from table 21 for a span equal to that of the length of shear influence line for the points under consideration. The static shear thus calculated shall be multiplied by the appropriate ratio (figure 45) and the result shall be multiplied by the dynamic factor for shear in which L is taken to be the span of the girder.

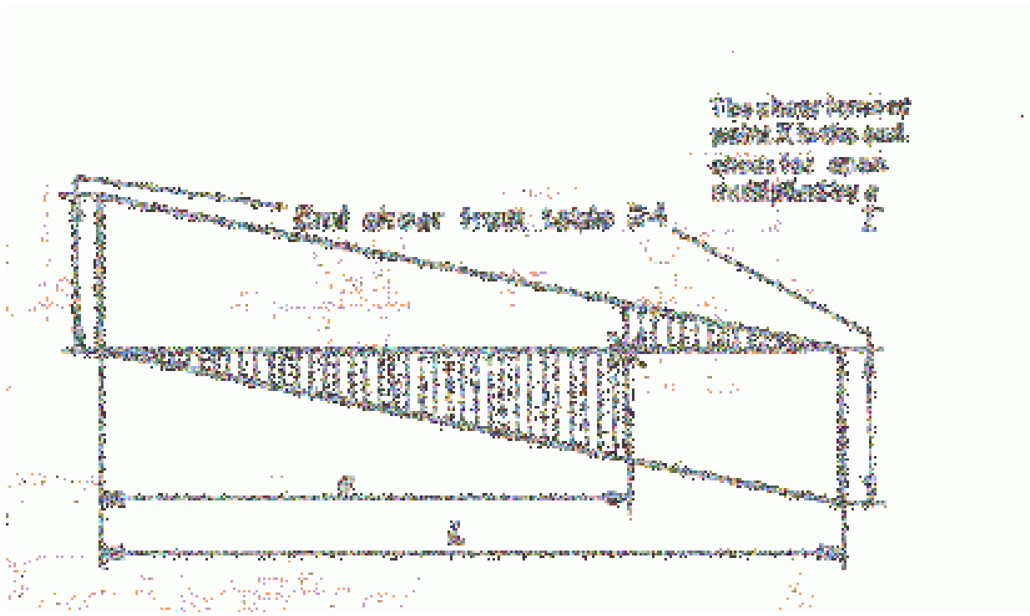


Figure 45 – Shear forces determination

D.3.2 ***Cross girders loaded through simply supported rail bearers.*** The cross girders shall be designed to carry two concentrated point loads for each track. Each of these loads is to be taken as one-quarter of the equivalent uniformly distributed load for bending moments shown in table 20 for a span equal to twice the cross girder spacing, multiplied by the appropriate dynamic factor.

Table 20. Equivalent uniformly distributed loads for bending moments for simply supported beams (static loading) under RU loading

Span m	Load kN	Span m	Load kN	Span m	Load kN
1.0	500	8.0	1 257	50.0	4 918
1.2	500	8.2	1 282	52.0	5 080
1.4	500	8.4	1 306	54.0	5 242
1.6	500	8.6	1 330	56.0	5 404
1.8	501	8.8	1 353	58.0	5 566
2.0	504	9.0	1 376	60.0	5 727
2.2	507	9.2	1 399	65.0	6131
2.4	512	9.4	1 422	70.0	6 534
2.6	518	9.6	1 444	75.0	6 937
2.8	523	9.8	1 466	80.0	7 340
3.0	545	10.0	1 488	85.0	7 742
3.2	574	11.0	1 593	90.0	8144
3.4	601	12.0	1 695	95.0	8 545
3.6	627	13.0	1 793	100.0	8 947
3.8	658	14.0	1 889	105.0	9 348
4.0	700	15.0	1983	110.0	9 749
4.2	738	16.0	2075	115.0	10151
4.4	773	17.0	2165	120.0	10 552
4.6	804	18.0	2 255	125.0	10 953
4.8	833	19.0	2 343	130.0	11354
5.0	860	20.0	2 431	135.0	11754
5.2	886	22.0	2 604	140.0	12155
5.4	910	24.0	2 775	145.0	12 556
5.6	934	26.0	2 944	150.0	12 957
5.8	956	28.0	3112	155.0	13 357
6.0	978	30.0	3 279	160.0	13758
6.2	1 004	32.0	3 445	165.0	14158
6.4	1 036	34.0	3 610	170.0	14 559
6.6	1 067	36.0	3 775	175.0	14 959
6.8	1 097	38.0	3 939	180.0	15 360
7.0	1 126	40.0	4103	185.0	15 760
7.2	1 154	42.0	4 267	190.0	16161
7.4	1 181	44.0	4 430	195.0	16 561
7.6	1207	46.0	4 593	200.0	16 961
7.8	1 232	48.0	4 755		

Table 21. End shear forces for simply supported beams (static loading) under RU loading

Span m	Force kN	Span m	Force kN	Span m	Force kN
1.0	252	8.0	729	50.0	2 529
1.2	255	8.2	740	52.0	2 610
1.4	260	8.4	752	54.0	2 691
1.6	266	8.6	763	56.0	2 772
1.8	278	8.8	774	58.0	2 852
2.0	300	9.0	785	60.0	2 933
2.2	318	9.2	795	65.0	3134
2.4	333	9.4	806	70.0	3 336
2.6	347	9.6	817	75.0	3 537
2.8	359	9.8	827	80.0	3 738
3.0	371	10.0	837	85.0	3 939
3.2	383	11.0	888	90.0	4139
3.4	397	12.0	937	95.0	4 340
3.6	417	13.0	984	100.0	4 541
3.8	434	14.0	1 030	105.0	4 741
4.0	450	15.0	1 076	110.0	4 942
4.2	465	16.0	1 120	115.0	5142
4.4	479	17.0	1 165	120.0	5 342
4.6	492	18.0	1208	125.0	5 543
4.8	505	19.0	1 252	130.0	5 743
5.0	520	20.0	1 295	135.0	5 944
5.2	538	22.0	1 380	140.0	6144
5.4	556	24.0	1 464	145.0	6 344
5.6	571	26.0	1 548	150.0	6 544
5.8	586	28.0	1 631	155.0	6 745
6.0	601	30.0	1 714	160.0	6 945
6.2	615	32.0	1 796	165.0	7145
6.4	629	34.0	1 878	170.0	7 345
6.6	642	36.0	1960	175.0	7 545
6.8	656	38.0	2042	180.0	7 746
7.0	668	40.0	2123	185.0	7 946
7.2	681	42.0	2205	190.0	8146
7.4	693	44.0	2 286	195.0	8 346
7.6	705	46.0	2 367	200.0	8546
7.8	717	48.0	2 448		

Table 22. Equivalent uniformly distributed loads for bending moments for simply supported beams, including dynamic effects, under RU loading

Span m	**Load** kN	**Span** m	**load** kN	**Span** m	**Load** kN
1.0	1 000	8.0	1951	50.0	5136
1.2	1 000	8.2	1975	52.0	5 273
1.4	1 000	8.4	1999	54.0	5411
1.6	1 000	8.6	2022	56.0	5 547
1.8	1 002	8.8	2044	58.0	5 684
2.0	1 007	9.0	2066	60.0	5 820
2.2	1 015	9.2	2088	65.0	6160
2.4	1 024	9.9	2109	70.0	6 534
2.6	1 035	9.6	2130	75.0	6 937
2.8	1 047	9.8	2150	80.0	7 340
3.0	1 089	10.0	2171	85.0	7 742
3..2	1 148	11.0	2 268	90.0	8144
3.4	1203	12.0	2 359	95.0	8 545
3.6	1 255	13.0	2 447	100.0	8 947
3.8	1 293	14.0	2 531	105.0	9 348
4.0	1 351	15.0	2 613	110.0	9 749
4.2	1 401	16.0	2 694	115.0	10151
4.4	1 444	17.0	2 773	120.0	10 552
4.6	1 481	18.0	2 851	125.0	10 953
4.8	1 512	19.0	2 927	130.0	11 354
5.0	1 541	20.0	3 003	135.0	11 754
5.2	1 567	22.0	3153	140.0	12155
5.4	1 591	24.0	3 301	145.0	12 556
5.6	1 613	26.0	3 447	150.0	12 957
5.8	1 633	28.0	3 592	155.0	13 357
6.0	1 652	30.0	3 736	160.0	13 758
6.2	1 680	32.0	3 878	165.0	14158
6.4	1 717	34.0	4 020	170.0	14 559
6.6	1 753	36.0	4162	175.0	14 959
6.8	1 785	38.0	4 302	180.0	15 360
7.0	1 817	40.0	4 442	185.0	15 760
7.2	1 846	42.0	4 582	190.0	16161
7.4	1 874	44.0	4 721	195.0	16 561
7.6	1900	46.0	4 860	200.0	16 961
7.8	1926	48.0	4 998		

Table 23. End shear forces for simply supported beams, including dynamic effects, under RU loading

Span m	Force kN	Span m	Force kN	Span m	Force kN
1.0	421	8.0	997	50.0	2 604
1.2	427	8.2	1 007	52.0	2 676
1.4	435	8.4	1 018	54.0	2 748
1.6	445	8.6	1 028	56.0	2 821
1.8	464	8.8	1 037	58.0	2 893
2.0	501	9.0	1 047	60.0	2 965
2.2	532	9.2	1 057	65.0	3144
2.4	557	9.4	1 066	70.0	3 336
2.6	579	9.6	1 076	75.0	3 537
2.8	601	9.8	1 085	80.0	3 738
3.0	621	10.0	1 094	85.0	3 939
3.2	640	11.0	1 138	90.0	4139
3.4	663	12.0	1 181	95.0	4 340
3.6	695	13.0	1 223	100.0	4 541
3.8	714	14.0	1 264	105.0	4 741
4.0	729	15.0	1 304	110.0	4 942
4.2	743	16.0	1 343	115.0	5142
4.4	756	17.0	1 383	120.0	5 342
4.6	768	18.0	1 421	125.0	5 543
4.8	780	19.0	1 460	130.0	5 743
5.0	794	20.0	1 498	135.0	5 944
5.2	815	22.0	1 574	140.0	6144
5.4	832	24.0	1 649	145.0	6 344
5.6	849	26.0	1 724	150.0	6 544
5.8	864	28.0	1 799	155.0	6 745
6.0	878	30.0	1 873	160.0	6 945
6.2	892	32.0	1947	165.0	7145
6.4	905	34.0	2021	170.0	7 345
6.6	917	36.0	2094	175.0	7 545
6.8	930	38.0	2167	180.0	7 746
7.0	942	40.0	2 240	185.0	7 946
7.2	953	42.0	2 313	190.0	8146
7.4	965	44.0	2 386	195.0	8 346
7.6	976	46.0	2 459	200.0	8546
7.8	987	48.0	2 531		

Appendix E

Temperature differences T for various surfacing depths

The values of T given in figure 9 are for 40 mm surfacing depths for groups 1 and 2 and 100 mm surfacing depths for groups 3 and 4. For other depths of surfacing, the values given in tables 24 to 26 may be used. These values are based on the temperature difference curves given in Transport and Road Research Laboratories (TRRL) Report LR 765 'Temperature differences in bridges', which may be used in preference. Methods of computing temperature difference are to be found in TRRL Report LR 561 'The calculation of the distribution of temperature in bridges.

Table 24. Values of T for groups 1 and 2

Surfacing thickness	Positive temperature difference				Reverse temperature difference
	T_1	T_2	T_3	T_4	T_1
mm	°C	°C	°C	°C	°C
unsurfaced	30	16	6	3	8
20	27	15	9	5	6
40	24	14	8	4	6

Table 25. Values of T for group 3

Depth of slab (h)	Surfacing thickness	Positive temperature difference	Reverse temperature difference
		T_1	T_1
M	mm	°C	°C
0.2	unsurfaced	16.5	5.9
	waterproofed	23.0	5.9
	50	18.0	4.4
	100	13.0	3.5
	150	10.5	2.3
	200	8.5	1.6
0.3	unsurfaced	18.5	9.0
	waterproofed	26.5	9.0
	50	20.5	6.8
	100	16.0	5.0
	150	12.5	3.7
	200	10.0	2.7

Table 26. Values of T for group 4

Depth of *slab (h)*	Surfacing thickness	Positive temperature difference			Reverse temperature difference			
		T_1	T_2	T_3	T_1	T_2	T_3	T_4
m	mm	°C	°C	°C	°C	°C	°C	°C
< 0.2	unsurfaced	12.0	5.0	0.1	4.7	1.7	0.0	0.7
	waterproofed	19.5	8.5	0.0	4.7	1.7	0.0	0.7
	50	13.2	4.9	0.3	3.1	1.0	0.2	1.2
	100	8.5	3.5	0.5	2.0	0.5	0.5	1.5
	150	5.6	2.5		1.1	0.3	0.7	1.7
	200	3.7	2.0		0.5	0.2	1.0	1.8
0.4	unsurfaced	15.2	4.4	1.2	9.0	3.5	0.4	2.9
	waterproofed	23.6	6.5	1.0	9.0	3.5	0.4	2.9
	50	17.2	4.6	1.4	6.4	2.3	0.6	3.2
	100	12.0	3.0	1.5	4.5	1.4	1.0	3.5
	150	8.5	2.0	1.2	3.2	0.9	1.4	3.8
	200	6.2	1.3	1.0	2.2	0.5	1.9	4.0
0.6	unsurfaced	15.2	4.0	1.4	11.8	4.0	0.9	4.6
	waterproofed	23.6	6.0	1.4	11.8	4.0	0.9	4.6
	50	17.6	4.0	1.8	8.7	2.7	1.2	4.9
	100	13.0	3.0	2.0	6.5	1.8	1.5	5.0
	150	9.7	2.2	1.7	4.9	1.1	1.7	5.1
	200	7.2	1.5	1.5	3.6	0.6	1.9	5.1
0.8	unsurfaced	15.4	4.0	2.0	12.8	3.3	0.9	5.6
	waterproofed	23.6	5.0	1.4	12.8	3.3	0.9	5.6
	50	17.8	4.0	2.1	9.8	2.4	1.2	5.8
	100	13.5	3.0	2.5	7.6	1.7	1.5	6.0
	150	10.0	2.5	2.0	5.8	1.3	1.7	6.2
	200	7.5	2.1	1.5	4.5	1.0	1.9	6.0
1.0	unsurfaced	15.4	4.0	2.0	13.4	3.0	0.9	6.4
	waterproofed	23.6	5.0	1.4	13.4	3.0	0.9	6.4
	50	17.8	4.0	2.1	10.3	2.1	1.2	6.3
	100	13.5	3.0	2.5	8.0	1.5	1.5	6.3
	150	10.0	2.5	2.0	6.2	1.1	1.7	6.2
	200	7.5	2.1	1.5	4.8	0.9	1.9	5.8
> 1.5	unsurfaced	15.4	4.5	2.0	13.7	1.0	0.6	6.7
	waterproofed	23.6	5.0	1.4	13.7	1.0	0.6	6.7
	50	17.8	4.0	2.1	10.6	0.7	0.8	6.6
	100	13.5	3.0	2.5	8.4	0.5	1.0	6.5
	150	10.0	2.5	2.0	6.5	0.4	1.1	6.2
	200	7.5	2.1	1.5	5.0	0.3	1.2	5.6

Appendix F deleted. Superseded by part 9 of this standard.

Standards Publication referred to

BS 153 Steel girder bridges
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* In course of revision.

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British Standards Institution - 2 Park Street London W1A 2BS - Telephone 071-629 900 F_t 0 - Telex 266933