SEISMIC DESIGN OF BRIDGES

SECTION 2

DESIGN EARTHQUAKE LOADING AND DUCTILITY DEMAND

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2.0 NOTATION

$A_{H\mu}$ = intermediate force coefficient used in Zone B

$A_{\Delta\mu}$ = intermediate displacement coefficient used in Zone B

$C_{H\mu}$ = basic horizontal force coefficient

C_{HE} = basic horizontal displacement

$C_{\Delta\mu}$ = basic horizontal displacement coefficient

D_ζ = correction coefficient for non-standard degrees of damping

E = Young's modulus

H = seismic base shear force

I = moment of inertia

M = total mass assumed to participate in horizontal motion

T = fundamental natural period of structure

Z_H = return period coefficient

a_v = peak vertical acceleration response

f_y = yield stress of steel

g = acceleration due to gravity

ℓ = span length

m = mass per unit length

n = an integer index

p = probability

p_1 = probability of annual exceedance

p_n = probability that motion s will be exceeded n times in t years

s = ground motion with return period t_s

t = design life, in years

t_s = return period, in years

β = geographic coefficient

Δ_μ = seismic displacement of at centre of mass relative to ground

μ = displacement ductility factor

2.1 SEISMIC BASE SHEAR FORCE

2.1.1 Base Shear Force Expression and Seismic Zones

The minimum horizontal seismic base shear force H should be derived from the expression

$$H = C_{H\mu} Z_H M g \qquad (2.1)$$

where $C_{H\mu}$ = basic horizontal force coefficient, and depends on the chosen design value of structure displacement ductility factor μ, the fundamental natural period of the structure, and on the seismic zone defined in figure 2.1.

Z_H = coefficient from Table 2.1,

depending on design return period.

M = total mass assumed to participate

in the horizontal degree of freedom. This should normally exclude the mass due to super-imposed live load.

g = acceleration due to gravity.

2.1.2 Basic Force Coefficient $C_{H\mu}$

For Zones A and C, defined in figure 2.1, values of $C_{H\mu}$ may be obtained directly from figures 2.2 and 2.4 respectively. For the transition Zone B, $C_{H\mu}$ is the product of a geographic coefficient β from figure 2.1 and the coefficient $A_{H\mu}$ from figure 2.3. That is, for Zone B

$$C_{H\mu} = \beta A_{H\mu} \qquad (2.2)$$

The design value of the structural displacement ductility factor μ, is defined as the ratio of maximum displacement under the design earthquake to the theoretical yield displacement, both measured at the centre of mass. Design values of μ should not exceed six for any structure, unless special studies are carried out to justify them.

In assessing the appropriate value

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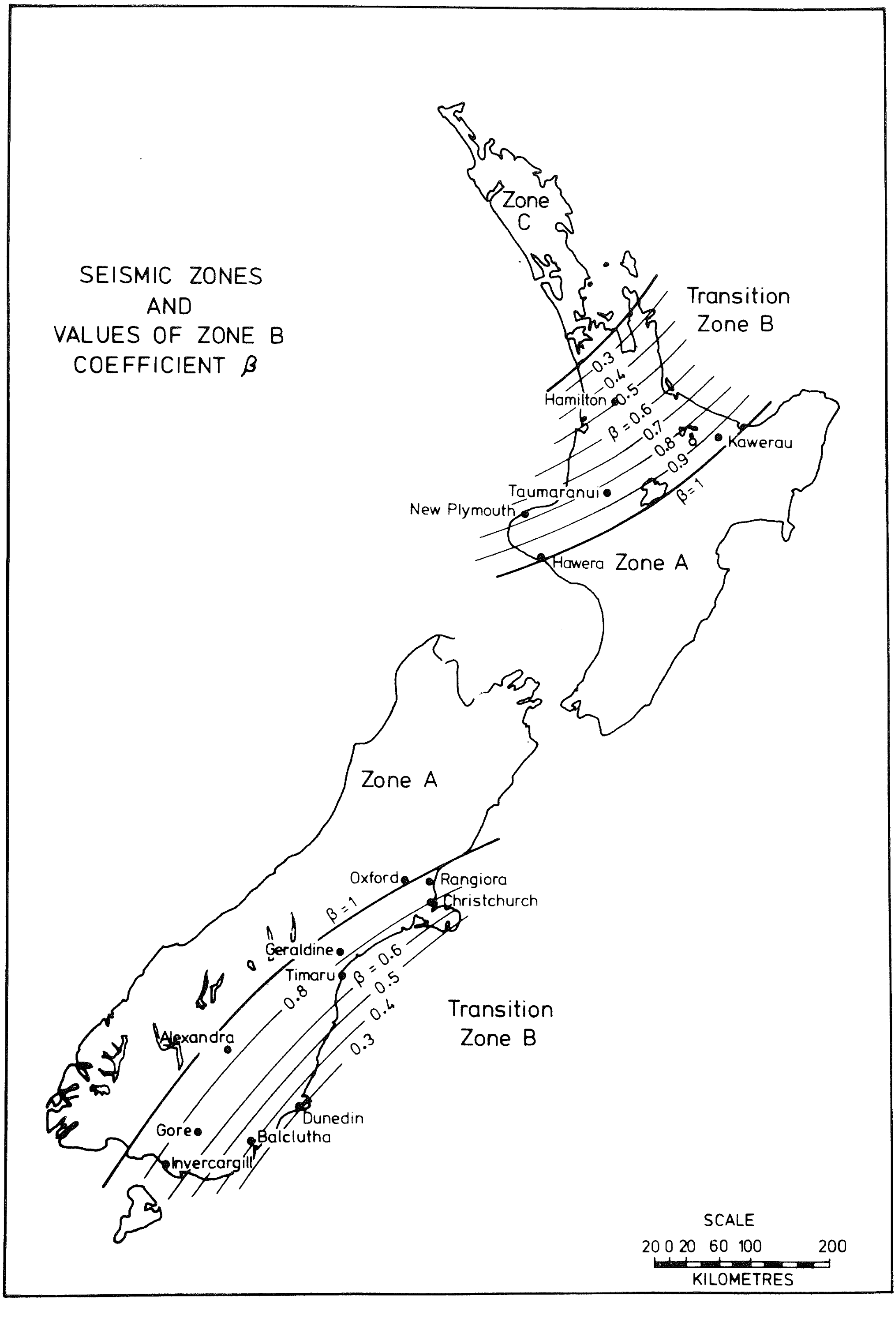

FIG. 2.1: SEISMIC ZONES AND GEOGRAPHIC COEFFICIENT B

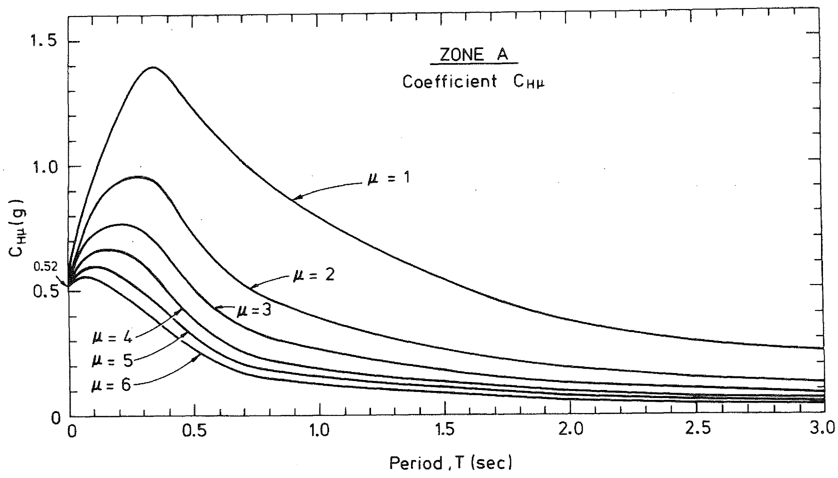

FIG. 2.2: BASIC FORCE COEFFICIENT CHμ, ZONE A

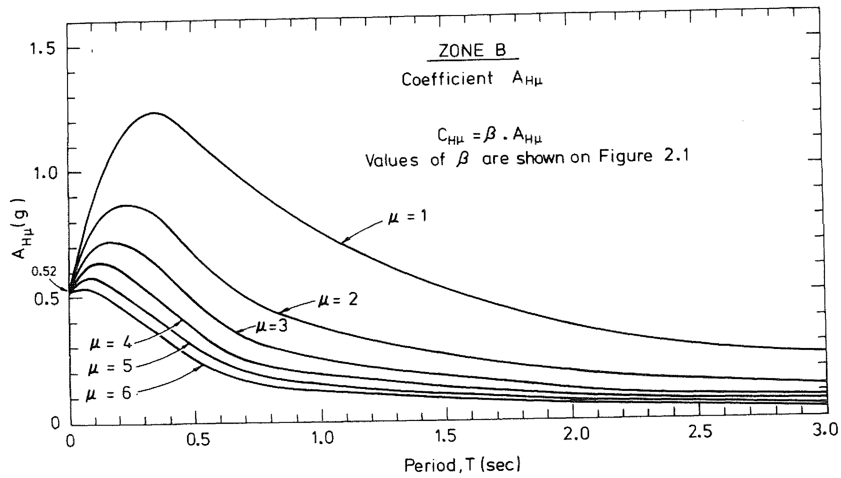

FIG. 2.3: INTERMEDIATE FORCE COEFFICIENT AHμ, ZONE B

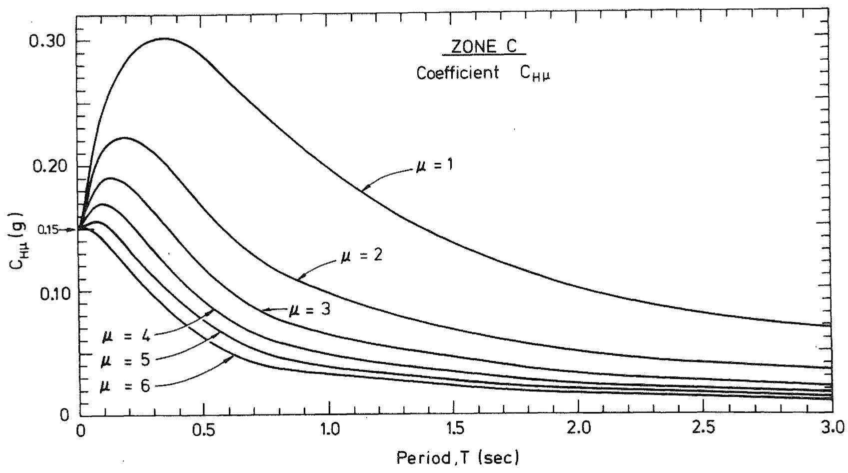

FIG. 2.4: BASIC FORCE COEFFICIENT CHμ, ZONE C

for μ, account should be taken of the inherent ductility capacity of the materials adopted, the extent to which ductile response is assured by the adoption of special detailing provisions, and the relationship between structural and member ductility factor, including effects of foundation and bearing flexibility.

Calculation of the fundamental natural period for obtaining the value of $C_{H\mu}$ should be based on cracked-section moment of area of piers, as appropriate, and should include effects of additional flexibility resulting from foundation and bearing deformations.

2.1.3 Return Period Coefficient Z_H

The return period chosen for determining Z_H from Table 2.1 should be based on the design life of the bridge and the acceptable risk of occurrence of the design level earthquake during the design life of the bridge.

TABLE 2.1 - COEFFICIENT Z_H

Return Period (yrs)	Z_H
5	0.17
10	0.24
20	0.35
50	0.56
100	0.80
150	1.00
250	1.33

2.1.4 Angle of Seismic Attack

The design level earthquake should be considered to act in any direction in the horizontal plane. However, simultaneous shaking in two orthogonal horizontal directions at design intensity need not be considered in assessing the strength required of energy dissipating elements.

2.2 SEISMIC DISPLACEMENTS

2.2.1 General

Consideration should be given to displacements induced by response of the foundation/pier/superstructure system to ground shaking, and to the consequences of relative ground displacements between supports.

2.2.2 Displacement Response

Where the seismic structural system can reasonably be simulated as a single degree-of-freedom oscillator, the maximum seismic displacement of the centre of mass, in mm, may be derived from the expression:

$$\Delta_\mu = C_{\Delta\mu}\ Z_H \qquad (2.3)$$

where $C_{\Delta\mu}$ = basic displacement coefficient from figures 2.5 or 2.7 for Zones A and C respectively, for the chosen value of design structure displacement ductility factor μ

Z_H = coefficient from Table 2.1 corresponding to the earthquake return period

For Zone B, $C_{\Delta\mu}$ for use in (2.3) is the product of the intermediate displacement coefficient $A_{\Delta\mu}$ from Figure 2.6, and the geographic coefficient β from figure 2.1. That is, for Zone B, Δ_μ is given by the expression:

$$\Delta_\mu = \beta\ A_{\Delta\mu}\ Z_H \qquad (2.4)$$

2.2.3 Relative Ground Displacements Between Supports

Attention is drawn to section 10.4 where the possibility of relative displacements of piers due to out of phase ground motions is discussed. This effect should be considered when span lengths exceed 200 m.

2.3 VERTICAL SEISMIC RESPONSE

2.3.1 General Considerations

The response of bridge superstructures to vertical ground motions during seismic attack should be investigated in the design. Bridge superstructures should be designed to ensure that such response remains within the elastic range of material behaviour.

In calculating maximum stresses during vertical response, neither live load nor concurrent vertical and horizontal response need be considered.

2.3.2 Vertical Acceleration Response

Peak vertical absolute acceleration response, a_v, for regular structures may be taken as 0.67 times the peak horizontal acceleration response. That is:

$$a_v = 0.67\ C_{HE}\ Z_H\ g \qquad (2.5)$$

where C_{HE} = elastic horizontal force response coefficient (i.e. $C_{H\mu}$ for μ = 1) from Figures 2.2, 2.3 and 2.4 for Zones A, B and C respectively.

Z_H = coefficient from Table 2.1 corresponding to the earthquake return period.

g = acceleration due to gravity.

In computing vertical accelerations, natural periods of vertical vibration should be used in obtaining values of C_{HE}.

COMMENTARY - SECTION 2

C2.1.1 Base Shear Force and Seismic Zones

The design value of horizontal earthquake base shear force depends on the

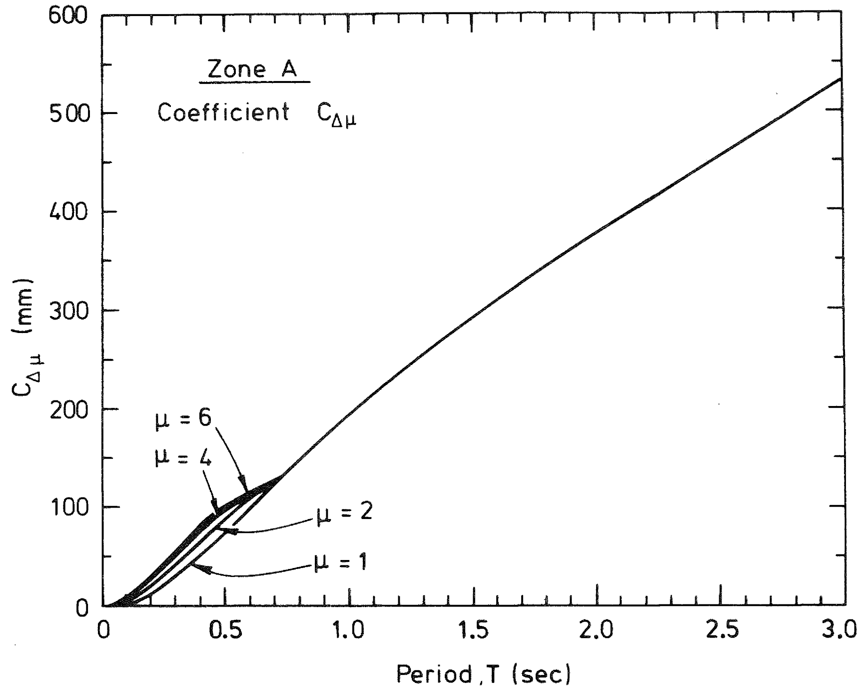

FIG. 2.5: BASIC DISPLACEMENT $C_{\Delta}\mu$, ZONE A

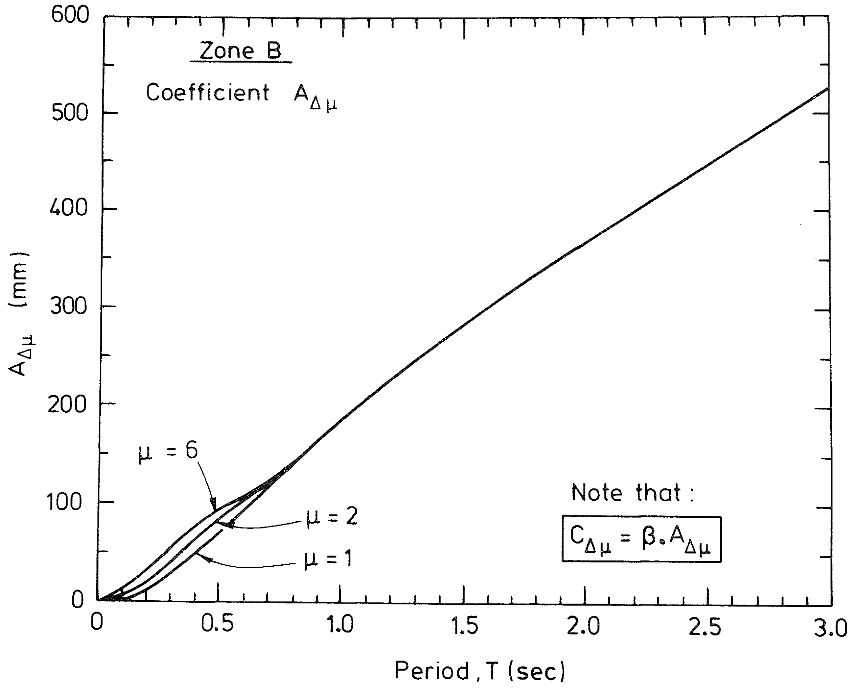

FIG. 2.6: INTERMEDIATE DISPLACEMENT COEFFICIENT $A_{\Delta}\mu$, ZONE B

seismic zone, the return period of the earthquake, the fundamental natural period of the structure and the design value of ductility.

The seismic zones are based on both seismicity observed over the short period of European settlement [C2.1, C2.2] and geologic and tectonic evidence of earthquake occurrence [C2.3, C2.4, C2.5, C2.6]. The elastic response spectra underlying the seismic coefficients are based principally on Smith's[C2.1, C2.2] study of modified Mercalli intensities, on unpublished work by Matuschka [C2.7], and on analyses[C2.8, C2.9, C2.10, C2.11] of strong-motion data recorded mainly in North America and Japan. They are intended to estimate the average response at alluvial sites. Because of the random nature of earthquakes and since the spectral estimates are obtained from empirical relationships based on sparse statistical data, there are large uncertainties associated with the base shear forces given by equation (2.1). The uncertainties can be divided into two types: those arising from scatter about an expected, or average, spectrum and uncertainty in the average spectrum itself. It is felt that uncertainty of the first type is fairly well described by the probability distribution underlying Table 2.1 and Figure C2.2. But the mean spectra, based on much scantier data, may be substantially in error, possibly by as much as a factor of two.

Zone B is intended to provide a smooth transition, approximating relative risk, between Zones A and B. However, there are downward steps of up to 15 percent in the force coefficient $C_{H\mu}$ at some periods in crossing from Zone A to B and from B to C. These are small compared with the uncertainties in the absolute values themselves and do not warrant the use of a more complicated scaling procedure.

C2.1.2 <u>Seismic Coefficient</u> $C_{H\mu}$

Figures 2.2, 2.3 and 2.4 give elastic horizontal acceleration response spectra (C_{HE}), in units of g, estimated for 150 year return period and 5 percent critical damping, at alluvial sites for Zones A, B and C respectively, and the corresponding inelastic spectra for design displacement ductility values of $\mu = 2$ to $\mu = 6$. These curves ($C_{H\mu}$) have been derived from the elastic response spectrum shape as follows:

(1) For T > 0.7 sec, the equal displacement principle is applied. Namely:

$$C_{H\mu} = \frac{C_{HE}}{\mu} \qquad (2.7)$$

(2) For T < 0.7 sec, the following empirical equation is used:

$$C_{H\mu} = \frac{C_{HE}}{\left\{\frac{(\mu-1)T}{0.7} + 1\right\}} \qquad (2.8)$$

Thus, for short period structures the design curves result in greater design forces than would be obtained on the basis of the equal displacement principle, which is recognized as being non-conservative for short period structures. At T = 0, the seismic design force is independent of the chosen value of design ductility.

It is felt that the value of 5 percent equivalent viscous damping assumed in figures 2.2 to 2.4 is a reasonable, average value for concrete bridges. However, when response is expected to remain elastic, or with low ductility demand, and material damping is expected to be low, values predicted by this approach may be nonconservative. Design base shears for other values of damping may be estimated by multiplying the value of H obtained from equation (2.1) by the factor D_ζ taken from Table C2.1.

The definitions of yield displacement and structural ductility factor

TABLE C2.1 - FACTOR D_ζ

Percentage of Critical Damping	D_ζ
2	1.4
5	1.0
10	0.8

are illustrated in figure C2.1, where point A is the idealized yield point and point B is the point at which the tensile reinforcement first reaches yield stress. In assessing the yield displacement, elastic stiffness of concrete piers should be based on the cracked-section moment of inertia, calculated in accordance with the guidelines below. Experimental evidence indicates that simple reinforced concrete bridge columns with fixed bases, detailed in accordance with the draft N.Z. Concrete Code DZ3101, can sustain member displacement ductilities in excess of $\mu = 8$. Therefore, such structures can be designed with confidence for the lowest value of coefficient $C_{H\mu}$.

The value of overall structure ductility is limited to six to allow for uncertainties in relationships between structural and curvature ductility, and to avoid damage under frequent minor earthquakes.

Where reinforced concrete bridge columns do not comply with the confining requirements of the draft N.Z. concrete code, DZ3101, or where foundations or bearing flexibility increases the ratio of required local curvature ductility to required overall structural displacement ductility, a lower value of structure ductility should be adopted.

The structural stiffness, EI, adopted in period calculations should be such that it produces a close estimate of actual yield displacement of the centre of mass

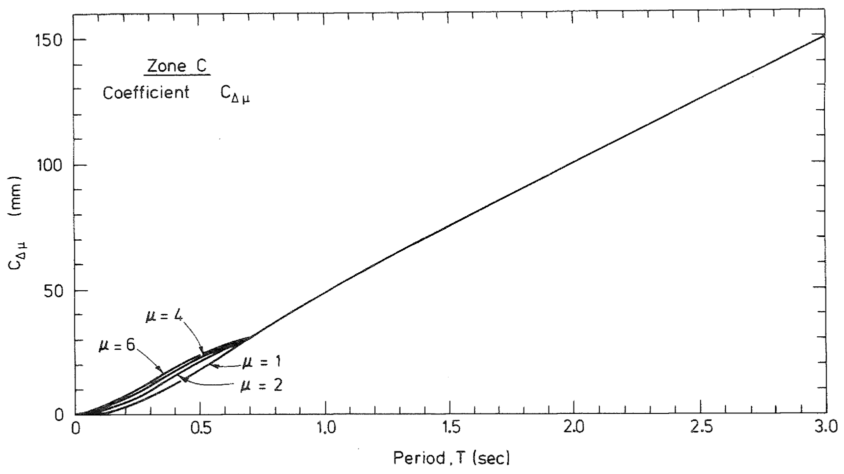

FIG. 2.7: BASIC DISPLACEMENT $C_{\Delta}\mu$, ZONE C

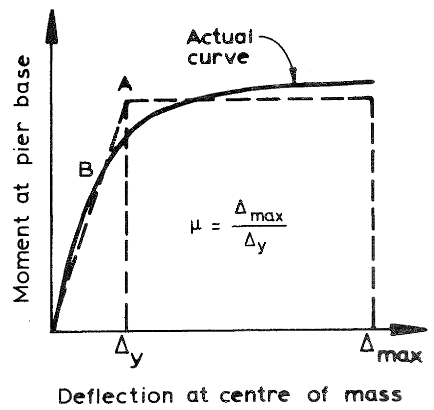

FIG. C2.1: DEFINITION OF YIELD DISPLACEMENT

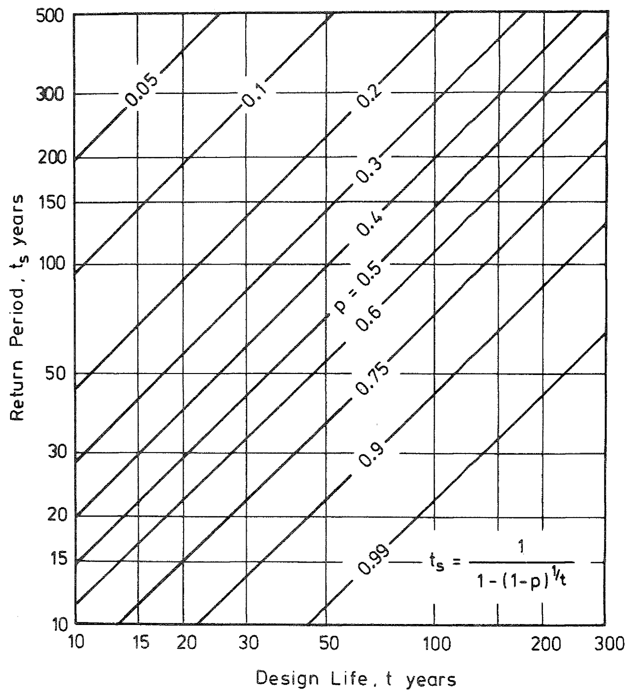

FIG. C2.2: RELATION BETWEEN RETURNED PERIOD t_s, DESIGN LIFE t, AND PROBABILITY OF EXCEEDENCE P.

of the equivalent simple oscillator. Consequently the moment of inertia of bridge piers subject to flexural action should be based on cracked-section properties, and contributions to the yield displacement resulting from foundation and bearing compliance must be included. It is recommended that, in the absence of special studies, the following values should be used for calculating member stiffnesses:

(a) For members in which primary plastic hinging is intended to occur (e.g. pier stems), the EI value is found from the curvature in the member at first yield of the tensile reinforcement. Design aids are available in CDP 810/A[C2.13].

(b) For members intended to remain elastic during severe seismic motions (for example, foundation cylinders), the EI value is taken as the mean of the value found from the curvature in the member at first yield of the tensile reinforcement and the value equivalent to the gross uncracked section of the member.

The response spectra in figures 2.2 to 2.4 estimate shaking at sites on deep alluvial soils, typical of most New Zealand bridge sites. Sites on hard rock, particularly crystalline basement rock, may be shaken more strongly at short periods than comparable alluvial sites. As a rough guide, the coefficient $C_{H\mu}$ may be increased by, say, 20 percent for sites on well-cemented sedimentary rock and by 40 percent for crystalline rock sites, at periods less than 0.4 seconds. At longer periods on hard rock, lower coefficients may be justified by special studies, although the possibility of topographic amplification should also be considered.

Designers should also be aware of the possibility of local amplification in uniform layers of very soft soils such as unconsolidated esturine sediments. Again, in these circumstances the state of the art is not well defined, and special studies, involving engineering judgement, are advised.

C2.1.3 Return Period Coefficient, Z_H

The coefficient Z_H from Table 2.1 scales the basic force coefficient from figures 2.2 to 2.4 to produce the design value of seismic acceleration, as a fraction of the acceleration of gravity, for different return periods of seismic attack.

It is expected that in most cases the bridge owner will specify the design return period to be used; part 1.3 of Section 1 : Design Philosophy, makes some general recommendations. The following comments are given to help the designer select a return period when one has not been prescribed.

The return period t_s associated with a given strength of ground motion s is defined as follows:

If p_1 (s) is the probability that motion s will be equalled or exceeded during a time interval of one year, then the return period of motion s is defined as the inverse of p_1 (s), that is

$$t_s = \frac{1}{p_1\ (s)} \text{ years} \qquad (2.9)$$

Alternatively, it is expected that motion s will be equalled or exceeded on average once every t_s years.

To obtain further information about the likelihood of the ground motion s occurring during the design life t of the structure it is necessary to know the distribution of occurrence times. Statistical data are sparse, but provided large regions are considered, they are consistent with a Poisson arrival process [C2.14, C2.15]. The assumption that recurrence intervals are Poisson distributed leads to the following relationship between return period, design life and probability of occurrence:

$$t_s = \{1 - (1- p)^{\frac{1}{t}}\}^{-1} \qquad (2.10)$$

where p is the probability that motion s, with return period t_s will be equalled or exceeded in t years. Expression (2.10) is plotted in figure C2.2.

It should be noted that the consequence of adopting a very high probability (say 95 percent) of the design earthquake being equalled or exceeded during the design life is that significant damage, requiring structural repair, can be expected to result from moderate ground shaking several times during the life of the bridge. If such an approach is adopted, careful study of the full economic and social consequences should be made. The probability p_n (t_s, t) that the motion with return period t_s will be equalled or exceeded exactly n times in an interval of t years may be estimated roughly by the expression

$$p_n\ (t_s,\ t) \simeq \frac{(1 - \frac{1}{t_s})^t\ (\frac{t}{t_s})^n}{n!}, \text{ for } t_s > 10 \qquad (2.11)$$

which follows from the assumption of a Poisson distribution of earthquake occurrence.

Finally, it is impossible to estimate maximum "credible" motions precisely, but the few existing recordings from epicentral regions serve as a guide. The Zone A elastic spectrum ($C_{H\mu}$ with μ = 1) multiplied by a factor of 2.25 just envelops that of the strongest recorded motions available at present, and it is considered that these are possible anywhere in New Zealand. Thus the maximum "credible" base shear force for a structure with 5 percent damping may be estimated by using $Z_H = 2.25$ in Equation (2.1) together with the appropriate value of $C_{H\mu}$ from figure 2.2, for all zones.

C2.1.4 Direction of Seismic Attack

The design earthquake intensity is considered to represent the worst ground shaking along any horizontal axis. It would therefore be inconsistent to consider vector addition of concurrent attack in two orthogonal directions. However, structural design must be based on the most disadvantageous direction of peak seismic attack, and consequences of flexural yield and displacement in diagonal directions (rather than simply in longitudinal and transverse directions) should be considered. Two examples are listed below:

(1) A group of four foundation cylinders arranged in a square pattern is likely to be most sensitive to horizontal loading along a diagonal rather than a major axis.

(2) A shear key between superstructure and supports, designed to resist transverse relative motion while allowing sliding longitudinally, can result in concurrent loading on the support. The longitudinal force would then be a function of the transverse acceleration force and the appropriate friction coefficient for the shear key. When any shear keys located eccentric to the pier centreline can be loaded in the above manner, a check should be made of the torsional capacity of the pier to resist the eccentric seismic forces transferred through those keys.

C2.2.1 Seismic Displacements : General Considerations

The ductility demand of column plastic hinges in bridge piers will be affected by both response displacements of the centre of mass and by relative displacements of the ground between piers, resulting from out-of-phase ground motions. Where span lengths are large, and column stiffnesses are high, the increase in ductility demand from this second cause can be substantial.

C2.2.2 Computation of Seismic Displacements

Maximum expected displacements of the centre of mass are important in designing bearings, abutment clearances, and seating details. The maximum displacement is found in similar fashion to the maximum seismic force, by reference to figures 2.5 to 2.7 for the basic displacement coefficient. This is then modified according to the design return period, by using the coefficient Z_H in Table 2.1. The curves in figures 2.5, 2.6 and 2.7 have been derived directly from the appropriate curves in figures 2.2, 2.3 and 2.4 observing that for simple yielding oscillators

$$C_{\Delta\mu} = \mu . C_{H\mu} \frac{T^2}{4\pi^2} g \qquad (2.11)$$

C2.3.1 Vertical Response : General Considerations

In general it is expected that vertical response of bridge super-structures to vertical components of seismic ground motion will be satisfactory. However, with long span structures, particularly prestressed concrete bridges where prestress 'overbalances' dead load, it is possible that vertical response could cause superstructure distress or even failure. Although superstructure seismic forces for short spans are unlikely to be severe, large transient variations in reactions at supports may cause problems in foundation performance, and possible lift-off at supports.

Unless special ductility detailing is incorporated in the super-structure, it is recommended that maximum actions induced by the combination of dead load plus vertical response should not exceed the following limits:

structural steel and mild steel reinforcing - 0.9 f_y

prestressing reinforcing stress - 0.02% proof stress

concrete strain - 0.002

C2.3.2 Computation of Vertical Acceleration Response

The method adopted for calculating seismic response in the vertical plane parallels that outlined for horizontal response. Equation (2.5) assumes elastic response with 5 percent of critical damping. Acceleration responses at other values of damping may be estimated by multiplying the value of a_v from equation (2.5) by the appropriate value of D_ζ taken from Table C2.

For structures with regular span lengths ℓ, approximately constant distributed mass/unit length m, and stiffness/unit length EI, the fundamental period of vertical vibration is approximately given by the expression:

$$T_v = 0.64\ell^2\sqrt{\frac{m}{EI}} \qquad (2.12)$$

Equation (2.12) assumes the common situation in which the vertical stiffness of bearings is high compared with that of the super-structure. Where this is not the case, bearing flexibility should be considered when calculating the vertical period of vibration. The mode shape corresponding to Equation (2.12) is sinusoidal with points of inflexion at supports. Consequently, and since vertical response is taken as two thirds horizontal response, maximum moments occur at midspan and are approximately given by the expression:

$$M_{max} = \pm 0.101 \left(\frac{2}{3} C_{HE}\right) m g \ell^2 \qquad (2.13)$$

Reaction changes at each end of a simple span will be

$$R = \pm \frac{\left(\frac{2}{3} C_{HE}\right) m g \ell}{\pi} \qquad (2.14)$$

Care should be exercised in extrapolating results of equations (2.12) to (2.14) to continuous bridges with irregular spans. In this case higher mode response may be significant, resulting in maximum moments at supports.

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