

SEISMIC DESIGN OF BRIDGES

SECTION 4 – BRIDGE FOUNDATIONS

F.D. Edmonds*, A.J. Carr**, P.R. Goldsmith***, P.J. North****, J.H. Wood*****,
R.L. Preston*****

4.0	<u>NOTATION</u>	k_s	= soil spring constant; units F/L
			$k_s = k_h \times \text{pile frontal area}$
C_u	= soil undrained shear strength		= k_{hDL}
$C_{H\mu}$	= basic horizontal seismic coefficient, see Section 2		= $K_H L$
$C_{H\mu}(T=0)$	= basic horizontal seismic coefficient for period T=0 seconds	$K_{YY}, K_{Y\theta}, K_{\theta Y}, K_{\theta\theta}$	= foundation stiffness matrix coefficients, see Section C4.3.2.1(a)
	$C_{H\mu}(T=0) = 0.52$ (Zone A)	L	= length
	= 0.15 (Zone C)	L'	= embedded length of pile
D	= pile width or diameter	L_s	= cantilever length for stiffness, see Figure C4.2
EI	= pile flexural rigidity (pile modulus of elasticity x second moment of area)	L_m	= cantilever length for moment, see Figure C4.2
E_s	= soil modulus of elasticity	M	= bending moment
F	= force	n_h	= constant of horizontal subgrade reaction (rate of increase in K_h with depth); units F/L ³
H	= horizontal force		$n_h = d(K_h) / dz$
I_A, I_B, I_C	= equivalent MM intensities for the design earthquake in seismic Zones A, B, and C respectively.	N	= standard penetration test resistance (blows per 0.3m)
k_h	= coefficient of horizontal subgrade reaction (ratio of soil pressure to lateral pile deflection); units F/L ³	\bar{N}, N_{crit}	= specific values of N used in the determination of the liquefaction potential of sands, see Equation 4.4
	$k_h = p/y$	R	= $\sqrt[4]{\frac{EI}{K_h}}$ Relative stiffness factor for piles in uniform cohesive soils, see Section C4.3.1 (a)
K_h	= modulus of horizontal subgrade reaction (ratio of soil pressure to pile lateral diametric strain); units F/L ²	R'	= $\sqrt[5]{\frac{EI}{n_h}}$ Relative stiffness factor for piles in uniform cohesionless soils, see Section C4.3.1 (a)
	$K_h = \frac{p}{y/D}$	P	= soil pressure; units F/L ²
	= $k_h D$	W	= weight
		y	= pile lateral deflection
		z	= depth from ground line
		z_w	= depth of water table below ground surface
		β	= seismic zone B geographic coefficient, see Section 2
		θ	= rotation
		ϕ	= strength reduction factor
		ϕ'	= effective soil internal angle of friction
		μ	= structural ductility factor
		μ'	= coefficient of friction

* Design Engineer, Ministry of Works and Development, Wellington

** Senior lecturer in Civil Engineering, University of Canterbury, Christchurch

*** Consulting Engineer, Auckland

**** Consulting Engineer, Hamilton

***** Laboratory Services Engineer, MWD, Central Laboratories, Lower Hutt.

***** Assistant Chief Designing Engineer, MWD, Wellington.

4.1 FOUNDATION/STRUCTURE INTERACTION

4.1.1 In assessing the response of a bridge structure to earthquake excitation, the foundation flexibility resulting from the interaction between the bridge foundation and the soil should be taken into account.

4.1.2 In the design of minor bridge structures or as a first approximation in the design of a major structure, the effect of foundation flexibility on the design horizontal seismic coefficient may be disregarded if the rigid base period (the natural period of the structure assuming fixity at the ground line) is greater than 0.25 second.

4.1.3 In assessing the seismic loading and the curvature ductility demand at each bridge pier, account should be taken of differences in foundation flexibility at the various bridge piers and abutments.

4.1.4 The effect of foundation flexibility should be taken into account in assessing the seismic displacements at the bridge piers and abutments.

4.2 FOUNDATION DESIGN

4.2.1 Because of difficulties associated with the repair of foundation damage resulting from a severe earthquake, it is desirable to design the bridge foundations to remain elastic below the ground line at the design earthquake loading. In the case of flooded foundations it is similarly desirable to design the pier stem below the mean high water line to remain elastic.

4.2.2 In general, bridge foundations should be designed to resist the imposed seismic loading in accordance with the principles of capacity design as outlined in Section 3.

4.2.3 Where yielding of the bridge foundations at the design earthquake loading is unavoidable, the foundations should be designed to behave in a ductile manner by using the detailing guidelines given in Section 5.

4.3 PILE OR CYLINDER FOUNDATIONS

4.3.1 General Considerations

4.3.1.1 In assessing the 'effective ground line' (the level below which soil interaction can be expected to influence the seismic response of a bridge structure) the following considerations should apply:

- (a) Cohesive soils. The effective ground line should be assumed at a depth of 1.5 to 2 pile diameters below the actual ground line.
- (b) Cohesionless soils. Where the surface materials have a standard penetration resistance, $N < 2$, the effective ground line should be assumed at a depth underlying such materials.

4.3.1.2 The lateral resistance from friction along the underside of a pile cap should be disregarded for design purposes as it may be effectively eliminated by soil settlement during an earthquake.

4.3.1.3 In cohesionless soils, additional lateral restraint from passive pressure mobilised against an embedded pile cap may be used in the design provided scour or excavation around the pile cap is not likely to render it ineffective.

4.3.2 Analysis of Pile Foundations

4.3.2.1 A number of methods are available to analyse pile foundations.

These include:

- (a) Approximate methods using soil stiffness parameters which are assumed to be independent of the soil strains:
 - (i) Equivalent cantilever methods
 - (ii) Analytical solutions based on a soil/pile system analogous to a beam on an elastic foundation.
 - (iii) Analytical solutions based on the assumption that the soil acts as an elastic half space.
 - (iv) Computer solutions using soil springs attached at discrete intervals along the embedded length of the pile to model soil/pile interaction.
- (b) Methods using soil load/deflection characteristics which are compatible with the soil strains mobilised by the applied loading. Such methods normally employ soil springs as in (iv) above, the spring stiffness being adjusted to match the secant stiffness corresponding to the pile deflection at each soil spring location.
- (c) Refined soil foundation interaction models. Refined models which may be more appropriate for use in time-history dynamic analyses, using soil springs or a finite element mesh to represent the soil.

4.3.2.2 Where preliminary analysis indicates that the seismic response is significantly affected by soil stiffness, the design should be based on analysis in which the soil stiffness parameters are compatible with the pile deflections produced by the design loading.

4.3.2.3 Rational analysis should be used to determine the manner in which raked piles resist lateral loading.

4.3.2.4 The effect of interaction between piles in groups on the foundation stiffness should be taken into account. In the absence of more specific information, the

effective modulus of horizontal subgrade reaction for each pile may be assumed to vary linearly from 25% of the K_h value for a single pile when spacing in the direction of load is 3 pile diameters to the K_h value for a single pile when the spacing is 8 pile diameters.

4.3.2.5 Unless otherwise substantiated, the equivalent viscous damping of the bridge soil/structure system should be taken to be 5% of critical damping. In a time-history analysis in which soil damping is taken into account by modelling the soil as an inelastic material or as an inelastic material with viscous damping, the damping in the structural system should be taken as 3% of critical, see Section 10.

4.3.3 Design Loading for Pile Foundations

4.3.3.1 Pile foundations designed in accordance with the principles of capacity design. The piles should be designed to resist the loading imposed by the formation of plastic hinges in the bridge piers, in accordance with the requirements of Section 3. In the case of 'rigid' piles (L'/R or $L'/R' < 4$), the lateral load resistance of the pile foundation as governed by the soil in which the piles are embedded, should also comply with:

$$\begin{aligned} & \phi \times (\text{lateral load resistance of the} \\ & \quad \text{foundation*}) \\ & > (\text{loading imposed by 'overstrength'} \\ & \quad \text{plastic hinging in the piers}) \end{aligned} \quad (4.1)$$

where ϕ is the appropriate soil strength reduction factor listed in Table 4.1.

4.3.3.2 Bridges with limited ductility where yielding of the piles is expected to occur under the design seismic loading. The piles should be designed to resist the seismic loading (corresponding to the appropriate ductility factor μ listed in Section 7.5.2) in accordance with the requirements of Section 3. In the case of 'rigid' piles (L'/R or $L'/R' < 4$), the lateral load resistance of the pile foundations, as governed by the soil in which they are embedded, should also comply with:

$$\begin{aligned} & \phi \times (\text{lateral load resistance of the} \\ & \quad \text{foundation *}) \\ & > (\text{design seismic loading}) \end{aligned} \quad (4.2)$$

where ϕ is the appropriate soil strength reduction factor listed in Table 4.1.

4.4 SPREAD FOOTINGS

For spread footings, conventional design techniques should be used. Where the overturning moments exceed the economical limits for conventional capacity designed footings it may be practical to consider a pier design in which rocking of the pier and its footing on the soil becomes

*Methods by which the lateral load resistance of single and group pile foundations may be evaluated are referenced in the Commentary.

the lateral load limiting mechanism.

4.4.1 Rocking Foundations

4.4.1.1 Bridge piers may be designed to rock on their foundations at a design seismic coefficient corresponding to $\mu = 3$ when it is uneconomic to provide sufficient stability to prevent rocking. However the advantages of a rocking response indicated by recent research are recognised and may be taken advantage of at a lower level of response, corresponding to $3 < \mu \leq 6$, provided special studies are carried out to ensure satisfactory behaviour.

4.4.1.2 Piers on rocking foundations are not required to meet the special seismic requirements for reinforcing outlined in Section 5.

4.4.1.3 The design vertical load on a bridge pier intended to rock under the design seismic loading should be taken as 0.8 times the dead load and overstrength contributions from superstructure or other members adjacent to the pier which may yield during rocking of the pier should be taken into account.

4.4.1.4 To protect against excessive plastic deformation of the soil imposed by rocking, which may result in misalignment of the pier after an earthquake, it is suggested that the bearing pressure on the soil under the design lateral loading should not exceed the ideal bearing pressure divided by 1.8 as recommended in Reference 4.2.

4.5 FRICITION SLABS

4.5.1 Friction slabs may be used to provide seismic anchorage at bridge abutments in accordance with the capacity design principles outlined in Section 3. The dependable resistance provided by the friction slab should be calculated as follows:

$$H = \phi W (\mu' - C_{H\mu}(T=0)) \quad (4.3)$$

where W is the dead weight of the friction slab and the overlying fill, $C_{H\mu}(T=0)$ is the basic horizontal seismic design coefficient for period $T = 0$ seconds, μ' is the coefficient of friction between the friction slab and the cohesionless material against which it has been cast and ϕ is an appropriate strength reduction factor.

4.5.2 Friction slabs should be founded on cohesionless material. Where friction slabs are to be used in a cohesive material, a layer of coarse granular material of at least 100 mm thickness should be placed beneath the friction slab.

4.5.3 Friction slabs should not be used to provide seismic anchorage in an embankment which is likely to fail under earthquake loading.

4.6 SOIL PARAMETERS

4.6.1 Analyses should be carried out

for a range of soil parameters to test for sensitivity and to provide a design envelope for all likely soil conditions at a particular site. Suggested soil stiffness parameters for use in preliminary seismic analyses of pile foundations are given in Table 4.2.

4.6.1.1 Variation of Soil Stiffness with Depth. In an analysis in which the soil stiffness parameters are assumed to be independent of the mobilised soil strains, see Section 4.3.2.1 (a) the following may be assumed to apply:

(a) Uniform cohesive soil. Depending on the method of analysis used,

(i) the soil modulus of elasticity E_s may be assumed to be constant

or

(ii) the modulus of horizontal subgrade reaction K_h may be assumed to be constant with depth and to be independent of the pile width or diameter.

(b) Uniform cohesionless soil. The modulus of horizontal subgrade reaction K_h may be assumed to increase linearly with depth and be independent of pile width or diameter.

4.6.2 Soil stiffness parameters more appropriate to a given bridge site may be determined from the soil stress/strain characteristics or the soil resistance/deflection characteristics obtained from laboratory testing of samples or from insitu testing.

4.6.3 The effective reduction in the soil stiffness due to cyclic loading under earthquake conditions should be accounted for by using stiffness parameters equal to 70% of their static values. (Note that the suggested values given in Table 4.2 incorporate this reduction).

4.7 LIQUEFACTION

4.7.1 The liquefaction potential of soils comprising loose saturated sand and/or coarse silts should be investigated. The recommended method of assessing liquefaction potential is outlined in the commentary.

4.7.2 Important or major bridge structures should not be located at sites where the potential for liquefaction exists. In some circumstances where the potential for liquefaction is very localised, some remedial measures can be taken as discussed in the Commentary.

A check on the lateral load resistance of pile or cylinder foundations, as governed by the soil in which the piles are embedded, should be made if:

$L'/R' < 4$ Cohesionless soils

where $R' = \sqrt[5]{\frac{EI}{n_h}}$

or $L'/R < 4$ Cohesive soils

where $R = \sqrt[4]{\frac{EI}{K_h}}$

TABLE 4.1 Strength Reduction Factors for the Evaluation of the 'Dependable' Lateral Load Resistance of Pile or Cylinder Foundations.

Data Available	Strength Reduction Factor*
Test loaded piles, plate bearing tests or insitu pressuremeter tests	0.70 - 0.90
Measured soil parameters	0.60 - 0.70
Visually assessed soil parameters from bore logs	0.50 - 0.60

*From Table 3, Reference 4.1

TABLE 4.2 Suggested Soil Stiffness Parameters for use in a Preliminary Seismic Analysis

SOIL TYPE	SITE DATA			DESIGN PARAMETERS			
	N (blows/0.3m)	C_u (kPa)	ϕ' (degrees)	n_h (kN/m ³) Dry Submerged		K_h (kN/m ²)	E_s (kN/m ²)
Cohesionless (dense soils)	30 - 50		45°	15x10 ³	9x10 ³		
	(loose)	4 - 10	30°	2x10 ³	1x10 ³		
Cohesive soils	(hard)	20 - 60				18x10 ³	25x10 ³
	(stiff)	8 - 15				6x10 ³	8x10 ³
	(soft)	2 - 4				1.5x10 ³	2x10 ³

SITE DATA:

- N = Standard penetration test resistance (blows/0.3m penetration)
- C_u = Soil undrained shear strength
- ϕ' = Effective soil internal angle of friction

DESIGN PARAMETERS

- (a) Cohesionless soils
- n_h = constant of horizontal subgrade reaction
- = $d(K_h)/dz$
- = $d(k_h D)/dz$
- (b) Cohesive soils
- (i) K_h = modulus of horizontal subgrade reaction
- = $k_h D$
- or (ii) E_s = soil modulus of elasticity

COMMENTARY:

C4.1.1 To assess the seismic loading and displacement of a bridge structure it is first necessary to establish the stiffness (and in more rigorous analyses, the damping) that the structure derives from its foundations.

C4.1.2 A ductile structure of given natural period of vibration and flexural yield strength will respond to the design earthquake with a certain structural ductility demand. An increase in foundation flexibility will increase the natural period of vibration and increase the yield displacement of the bridge structure. For a given curvature ductility

at the pier hinge, an increase in the yield displacement due to an increase in foundation flexibility will reduce the available overall structural ductility. For a rigid base period greater than that corresponding to peak seismic response, the increase in C_{Hu} due to the reduction in the available structural ductility is almost matched by the reduction in C_{Hu} due to the increase in period. It can be shown from the design response spectra (Section 2) that for each Seismic Zone, C_{Hu} for a flexible foundation is less than about 1.25 times the rigid base value provided that the rigid base period is greater than 0.25 second.

C4.1.3 Unequal foundation stiffness in an otherwise symmetrical structure will influence the seismic shear load and curvature ductility demand at each bridge pier. The piers with the stiffer foundation will resist a relatively greater proportion of the lateral load and have a greater curvature ductility demand placed upon them.

C4.1.4 Although C_{Hu} is relatively insensitive to foundation flexibility, increased flexibility of the foundations will significantly increase the seismic deflections.

C4.2.1 The bridge designer would normally be expected to make a preliminary cost assessment of foundation repair following a severe earthquake as compared to the additional cost involved in designing for an elastic foundation, taking into account the design life of the bridge.

C4.3.1 In estimating the foundation stiffness, only that part of the soil that can be relied upon to provide lateral restraint under seismic conditions should be taken into account. A low estimate of the likely foundation stiffness will in general result in a conservative design, i.e. an overestimate for the seismic displacements and, as discussed in Section C4.1.2, an overestimate for the design seismic loading.

The recommendation concerning cohesive soils is made because of the potential for pile/soil separation at the pile top under cyclic loading and is based on the findings of Davisson and Prakash^{4.3} and Matlock^{4.4}. Cohesionless soils have a greater ability to flow into the separation gap. The recommendation concerning cohesionless soils with $N < 2$ is based on the suggestion of ACI Committee 336^{4.5}. Where great depth of such cohesionless material exist, special consideration of their stiffness and strength may be appropriate though such soils would not normally be suitable for bridge foundations in seismic areas because of their potential for liquefaction, see Section 4.7.

Shaking table tests by Kubo^{4.6} have confirmed that the passive pressure acting on the vertical face of embedded pile caps in cohesionless soils significantly reduces the moments induced in the foundation piles.

C4.3.2 The use of a more refined soil foundation interaction model for pile foundations will not necessarily lead to a more reliable prediction of foundation behaviour as the accuracy of the prediction will depend as much on the reliability of the soil data as upon the refinement of the model. Confidence in the soil data implies knowledge of the following:

- Modification to the undisturbed characteristics of the soil caused by the change in the stress state of the soil during and subsequent to the installation of the foundation.
- Time dependent changes in the soil properties depending on the number of cycles of loading and the amplitude of each cycle.

The development of methods to solve the soil/foundation interaction problem have been the subject of considerable research effort in recent years, as discussed by Seed et al^{4.7} and Berger and Pyke^{4.8}. However because of the complexity of the physical problem advances that have been made in predicting soil parameters suitable for use in seismic soil foundation interaction analyses from standard soil tests have not kept pace with advances made in the analytical techniques. Caution should thus be used in relying too heavily on the results of soil testing, however extensive, as relating the test results to the physical conditions in the soil around a pile foundation under seismic loading will inevitably involve considerable judgement.

C4.3.2.1 (a) For preliminary seismic analyses of a bridge structure or for the purpose of design where it can be shown that the seismic response is relatively insensitive to changes in foundation flexibility, use may be made of the simplifying design assumption that the soil stiffness properties are independent of the strain level. For seismic design

purposes, the elastic soil stiffnesses used in such analysis should relate to the secant stiffness of the soil resistance/deflection relationship at approximately one half of the ultimate lateral soil resistance. As the lateral response of a pile foundation is largely governed by the interaction of the piles over the top 8 to 10 diameters, the representative soil stiffness in relatively uniform soils may be based on the soil characteristics at a depth $4D$ from the ground line. A suggested range of soil stiffness parameters for use in preliminary seismic analyses of pile foundations is given in Table 4.2.

For the purpose of analysis, an artificial division is often made between the structure above the ground line and the bridge foundation. Using dummy members whose load/deflection relationship at the ground line approximates that of each foundation group and the surrounding soil (see fig C4.1(a)) the seismic response of the bridge structure can be obtained from the design response spectra (Section 2) or by means of time-history dynamic techniques (Section 10).

The peak pile bending moments, or if required the distribution of bending moment along the length of the piles, corresponding to the design seismic loading can be subsequently obtained from standard solutions available in the literature. In the case of a Winkler soil spring model (see fig. C4.1 (b)), the pile bending moments can be obtained directly by applying the design seismic loading.

(i) Equivalent cantilever methods. This method, which is based on beam on elastic foundation theory, may be used as a rough approximation for free head piles in uniform cohesive soils, where the soil stiffness can be reasonably represented by a constant modulus of horizontal subgrade reaction, or in uniform cohesionless soils, where the modulus of horizontal subgrade reaction can be assumed to vary linearly with depth. Details of this method, which involve the evaluation of the relative stiffness factors:

$$R = \sqrt[4]{\frac{EI}{K_h}} \quad (\text{for cohesive soils}) \quad (C4.1)$$

$$\text{or} \quad R' = \sqrt[5]{\frac{EI}{n_h}} \quad (\text{for cohesionless soils}) \quad (C4.2)$$

are given in fig C4.2. Note that different cantilever lengths are used depending on whether foundation stiffness (for the purpose of evaluating superstructure displacement) or peak pile moment is to be modelled. The cantilever lengths for foundation stiffness are based on Reference 4.9 and are approximations for a range of ground line moment to shear ratios. More stringent elastic theory^{4.10}

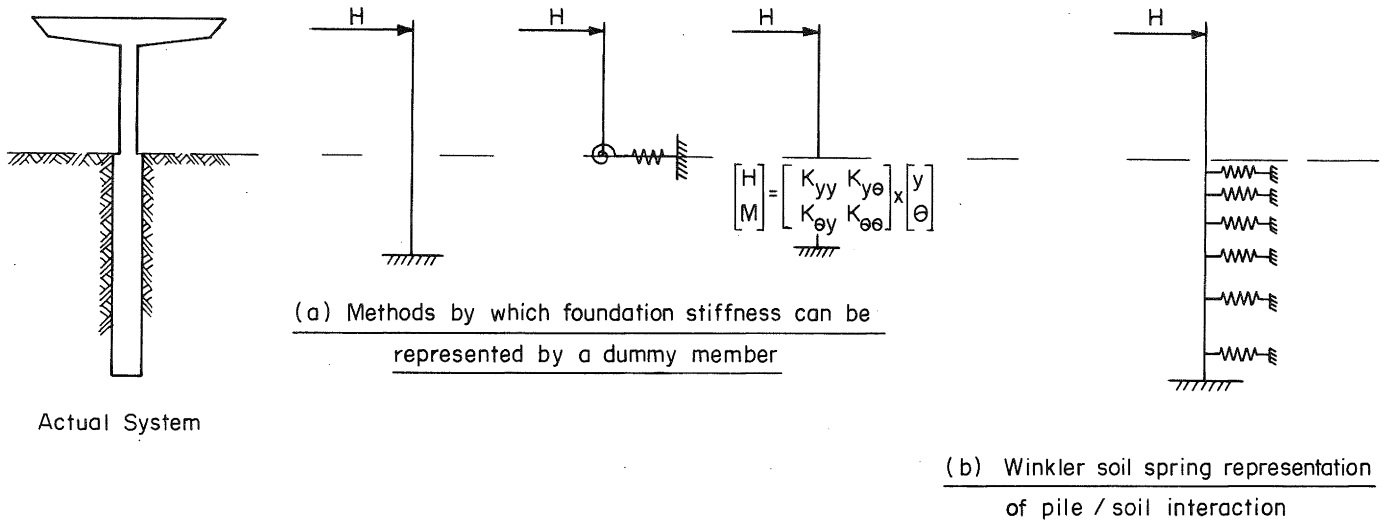
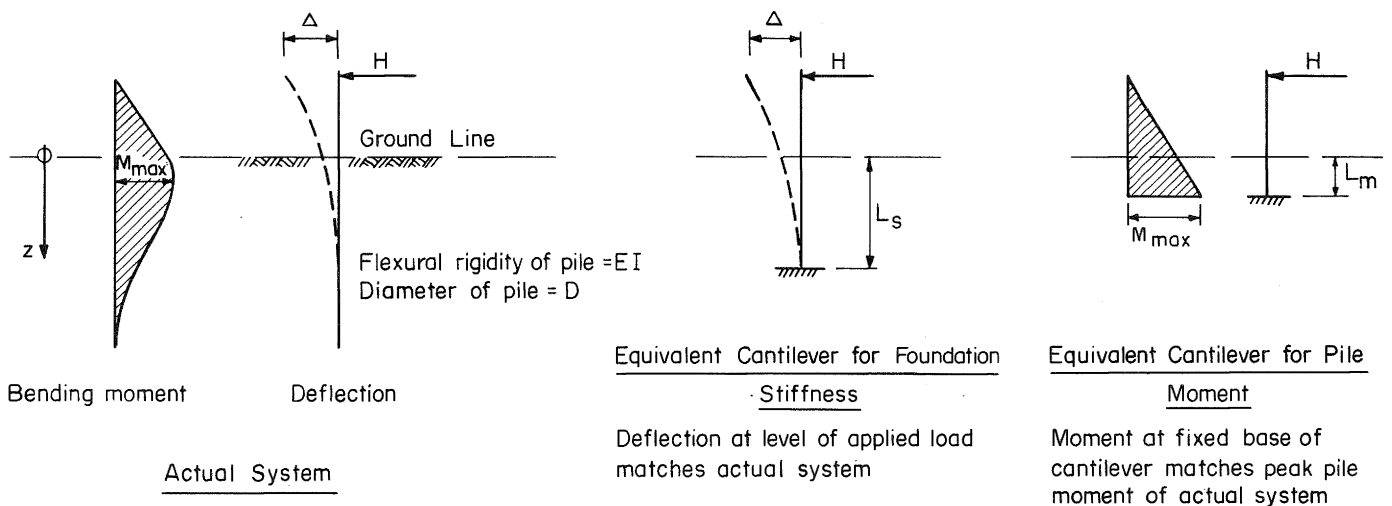


FIG. C4-1 METHODS OF REPRESENTING PILE FOUNDATION STIFFNESS



Cohesive Soil (Constant K_h)	$R = \sqrt[4]{\frac{EI}{K_h}}$	$L_s = 1.4R$	$L_m = 0.44R$
Cohesionless Soil (Constant n_h)	$R' = \sqrt[5]{\frac{EI}{n_h}}$	$L_s = 1.8R'$	$L_m = 0.78R'$

FIG. C4-2 EQUIVALENT CANTILEVER METHOD

indicates that these cantilever lengths will vary to some extent with the ground line moment to shear ratio. The cantilever lengths for pile moment are from Reference 4.9 for the case of applied shear at the ground line and are conservative for the more general case of shear and moment at the ground line.

(ii) Analytical solutions based on elastic foundation theory.

Pile foundations can be generally categorised as 'rigid' or 'flexible' in the following way:

$$L'/R \text{ or } L'/R' < 4 \quad (\text{rigid piles}) \quad (\text{C4.3})$$

$$L'/R \text{ or } L'/R' > 4 \quad (\text{flexible piles}) \quad (\text{C4.4})$$

where L' is the embedded length of the pile. Beam on elastic foundation theory indicates that the solution for L'/R or $L'/R' > 4$ is essentially the same as the solution for L'/R equal to infinity. Graphical solutions for deflection and moment along the length of flexible piles are presented by Davisson^{4.9}. These solutions cover a range of pile head fixity (from free through to fixed). Note that the majority of practically proportioned piles will be in the flexible category.

(iii) Analytical solutions based on the assumption that the soil acts as an elastic half space.

Many of the available structural analysis computer programs, e.g. STRUDL (reference 4.11), have the facility to accept an input stiffness matrix in place of input stiffness parameters. Foundation stiffness matrices of the form:

$$\begin{bmatrix} H \\ M \end{bmatrix} = \begin{bmatrix} K_{yy} & K_{y\theta} \\ K_{\theta y} & K_{\theta\theta} \end{bmatrix} \times \begin{bmatrix} y \\ \theta \end{bmatrix} \quad (\text{C4.5})$$

which relate ground line shear H and moment M to ground line deflection y and rotation θ can be derived from the work of Poulos^{4.12} or may be obtained for free head piles from the following equations:

$$K_{yy} = 16 \frac{EI}{D^3} \left(\frac{E_s}{E} \right) 0.75 \quad (\text{C4.6})$$

$$K_{y\theta} = K_{\theta y} = -4.8 \frac{EI}{D^2} \left(\frac{E_s}{E} \right) 0.50 \quad (\text{C4.7})$$

$$K_{\theta\theta} = 3.2 \frac{EI}{D} \left(\frac{E_s}{E} \right) 0.25 \quad (\text{C4.8})$$

where EI is the flexural rigidity of the piles and (E_s/E) is the ratio of the soil to pile modulus of elasticity. The above equations are empirical approximations provided by Blaney et al^{4.13} to a parametric study conducted by Novak^{4.14} and are suitable for use in preliminary analyses where more detailed study is not

required. Graphical solutions for peak pile moment in terms of the design ground line bending moment and shear force are provided by Poulos^{4.12}.

The assumption of a constant soil elastic modulus normally applies to uniform cohesive soils only and should not be used for cohesionless soils or for layered cohesive soils.

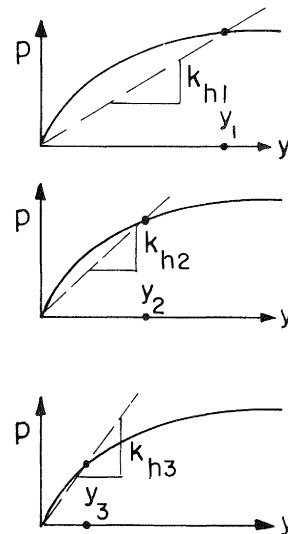
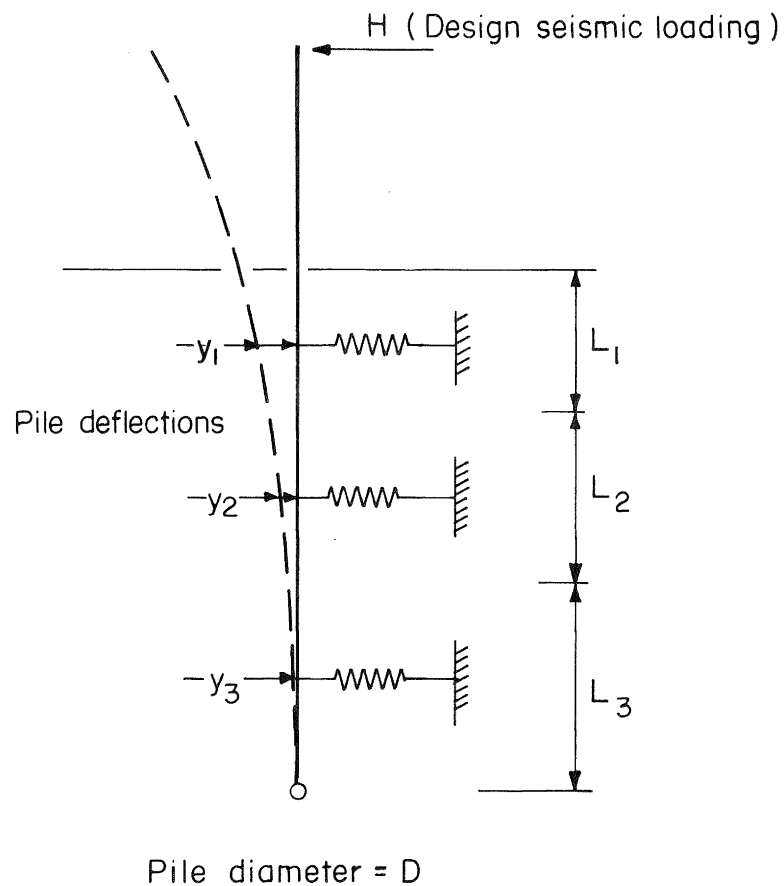
(iv) Soil spring models. Computer models which use soil springs attached at discrete intervals along the length of the pile are similar in theory to analytical solutions based on a beam on an elastic foundation. However the computer models enable the theory to be applied to layered soils where the soil stiffness is not characterised by constant or linear variation with depth. Margason and Holloway^{4.15} point to the importance of accounting for layering effects as rapid changes in soil stiffness with depth can induce local bending moment maxima in the piles.

C4.3.2.1 (b) Account of the amplitude dependent nature of soil stiffness can most readily be achieved by using a soil spring computer model in which the soil stiffness associated with each spring is appropriate to the peak seismic pile displacement at each particular soil spring location. Each soil spring constant

$$k_s = k_h DL \quad (\text{C4.9})$$

should be based on the secant stiffness of the soil p/y curve at the appropriate pile displacement corresponding to the design loading, see Fig C4.3. Methods by which the soil p/y curves (the soil resistance/deflection characteristics) can be obtained from laboratory or in-situ tests are referenced in Section C4.6.2. The analysis will normally require an initial estimate of the deflected pile shape followed by a number of iterations to achieve compatibility between the initially assumed displacements and the computed displacements at each soil spring location. The number of iterations will depend on the precision with which the solution is required. In general the computed pile displacements are more sensitive than pile bending moments to changes in soil stiffness.

C4.3.2.1 (c) Where it is warranted, e.g. for use in a time-history analysis, a more refined model may be used to represent soil foundation interaction. This may be done using a computer model incorporating inelastic soil springs or elastic soil springs with dashpots to model hysteretic soil damping and incorporating lumped masses along the embedded depth of the pile to represent soil inertia effects. Alternatively a finite element mesh can be used to represent the soil. For a time-history analysis, equal seismic excitation should be input at each of the soil spring support points or in the case of a finite element mesh, at an appropriate soil layer



Family of p/y curves for a specified pile diameter

The p/y curves relate the soil resistance at a point to the pile deflection at the same point.

The Winkler soil spring constants are obtained from the secant stiffness of the p/y curve at each soil spring location:

$$k_s = k_h \times \text{frontal area}$$

$$= k_h \times D \times L$$

FIG.C4.3 DETERMINATION OF WINKLER SOIL SPRING CONSTANTS FROM SOIL P/Y CURVES

boundary at or below the base of the pile. A comparison of these two methods is given in Reference 4.8.

C4.3.2.3 Raked piles resist the seismic loading imposed by the bridge superstructure partly by axial load and partly by bending. As shown by Patterson-Kane^{4.16}, it is not sufficiently accurate to assume that raked piles resist the lateral load purely by axial load, even in situations where the line of action of the horizontal load coincides with the intersection of the axes of the raked piles.

C4.3.2.4 The recommendation to account for the effective reduction in soil stiffness resulting from interaction between piles in a group is based on the work of Prakash^{4.17} and Davisson and Salley^{4.18}. They found the interaction existed between piles at spacings of up to 8 pile diameters in the direction of load and 3 pile diameters in the direction normal to the load. Recent research at Auckland University by Hughes et al^{4.19} indicates that the 8 pile diameter spacing is non-conservative and that interaction in cohesionless soils occurs at spacings of up to 13.5D in the direction of the loading. This work has also investigated load sharing between piles in a group and has indicated that a disproportionate share of the lateral load is resisted by the front piles.

C4.3.2.5 Damping in a dynamically loaded soil/structure system occurs as energy is dissipated in a hysteretic manner in the structure and the soil and from energy radiated from the structure into the soil. The concept of equivalent viscous damping to represent the overall effect of damping in the system is used for mathematical convenience rather than on the basis of its correctness.

Traditionally the damping ratio for bridge structures under design seismic excitation has been taken as 5% of critical though it has been thought that this value would depend to some extent on the soil type. Present understanding of the relationship between an overall viscous damping factor for the system and the actual damping mechanisms does not permit any rational modification to this assumption.

C4.3.3 Piles resist applied lateral load by the combined action of their inherent structural strength and rigidity and the resistance mobilised by the surrounding soil as the pile deforms. In general terms, the lateral load resistance of a pile or group of piles is governed by either the flexural strength of the piles (in the case of 'flexible' piles) or by the soil strength (which may govern in some circumstances for 'rigid' piles). Thus in addition to designing piles to have sufficient strength to withstand the imposed bending moments and shear forces in accordance with the requirements of Section 3, a check on the overall lateral load resistance as governed by the soil in which the piles are embedded should also

be made in the case of 'rigid' piles (L'/R or $L'/R' < 4$) where soil strength considerations may apply. Note that for the case of pile foundations designed in accordance with the principles of capacity design, the 'dependable' rather than the 'ideal' lateral load resistance is matched to the loading imposed by 'overstrength' plastic hingeformation in the piers, in contrast to the strength requirements of the pile foundation covered in Section 3.4.1.2. Matching the 'dependable' lateral load resistance to the design seismic loading for the foundation piles has been recommended for both ductile design and for the design of bridges of limited ductility as the \emptyset values listed in Table 4.1 relate to the certainty with which the soil strength parameters can be estimated rather than to an assessment of the dependable strength of the soil.

Methods by which the lateral load resistance for single piles (Kuthy et al^{4.20}) and for groups of piles (Pidgeon and Toan^{4.21}) may be evaluated are readily available. Reference 4.21 gives details of a generalised computer program which may be used for raked or vertical pile groups and can also be used for single piles. These methods pertain to static loading but may be used to approximate the lateral load resistance of pile foundations under seismic conditions.

C4.4 An outline of the different design considerations for spread footings and rocking foundations is given in Reference 4.2.

C4.4.1.1 It is generally accepted that bridge foundations need not be designed for forces larger than those corresponding to $\mu = 3$. In some circumstances (e.g. in the design of the footings of slab piers subjected to lateral loading in the transverse bridge direction) it may be difficult or impossible to design a spread footing to resist the pier moment capacity. Should the pier moment capacity exceed the seismic overturning moment corresponding to $\mu = 3$ and the natural period of vibration (prior to rocking), rocking of the bridge pier and its footing on the soil may be assumed to occur.

Because of the complete absence of experience with rocking piers in earthquakes, the design of a rocking foundation for a load level less than that corresponding to $\mu = 3$ should be based on special studies, including appropriate dynamic analyses^{4.22}, to verify the behaviour of the rocking system. The lower limit on the load at which rocking may be permitted to commence should not be less than that corresponding to $\mu = 6$.

C4.4.1.2 A rocking pier may be considered sufficiently protected against overload and hence failure if it possesses the ideal strength to resist the seismic forces corresponding to the chosen ductility factor μ . Piers on rocking foundations are thus exempted from the

special seismic requirements for reinforcing. However they should be detailed for limited ductility to ensure some protection in the event that the base does not rock until a load greater than that corresponding to $\mu = 3$ is reached.

C4.4.1.3 In assessing the size requirements for the rocking foundation pad, a design vertical load of 0.8 times the dead load (to account for the effect of vertical acceleration under earthquake conditions) should be used to minimise the pier restoring moment while over-strength contributions of any yielding connections should be taken into account to maximise the overturning moment. In addition, a thorough analysis should be carried out to determine the ductility demands on components of the bridge other than the rocking pier to ensure that these do not exceed the demands implied by the μ factors appropriate to those components. This implies a full assessment of the performance of both structural and non-structural components of the bridge as a consequence of the vertical and horizontal movements associated with the rocking motion of the piers. Spans which extend between rocking and non-rocking piers or abutments must be detailed to preserve their integrity for carrying the intended vertical load.

C4.5.1 The derivation of Equation 4.3 which aims to give a conservative estimate of the dependable seismic resistance provided by friction slabs is outlined in Fig C4.4. The implication is that the friction slab will not be capable of providing resistance to seismic forces in the bridge superstructure if μ' , the coefficient of friction between the friction slab and the cohesionless soil against which it has been cast, is less than $C_{Hu}(T=0)$. In the absence of more specific information, μ' may be taken as being equal to $\tan \phi'$. Until such time as the behaviour of friction slabs under earthquake conditions has been more extensively investigated, the use of a conservative strength reduction factor, $\phi = 0.60$ is recommended. On the basis of the approximate analysis presented in Fig C4.4, the effectiveness of friction slabs in Seismic Zone A is in some doubt.

The use of a sliding friction slab as an energy dissipating device in either a cohesive or cohesionless soil may be a feasible design proposition, but until this concept has been proven by test, friction slabs should be used in a design to provide positive anchorage only.

C4.5.2 The use of a coarse gravel blanket is recommended in cohesive soils to provide free drainage and as it is felt that the friction characteristics of the cohesionless material can be more reliably quantified.

C4.6.1 The soil parameters listed in Table 4.2 are suggested typical values for use in a preliminary seismic analysis only. The implied equivalence between the listed K_h and E_s values for different

types of cohesive soil is very approximate.

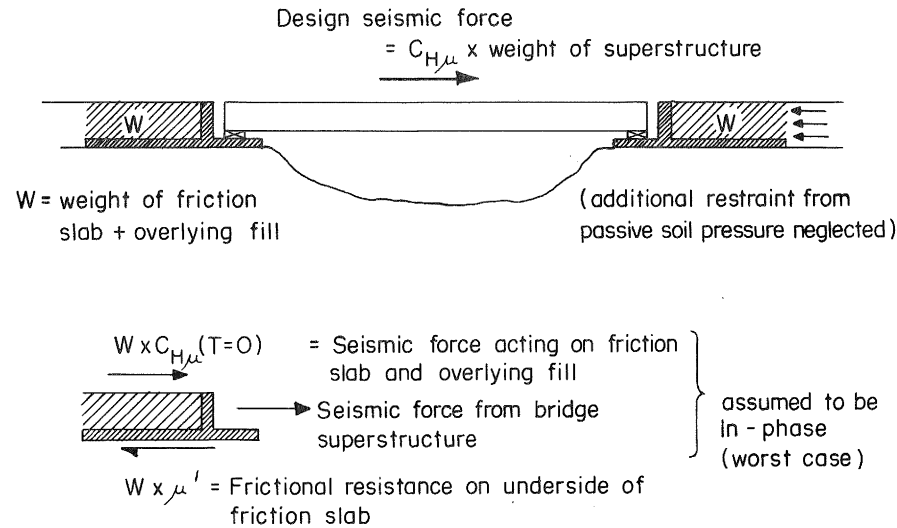
Note that the moduli of horizontal subgrade reaction, K_h values, listed in Table 4.2 are constants independent of pile diameter. It follows that the coefficient of horizontal subgrade reaction $k_h = K_h/D$, the ratio between the horizontal pressure between the pile and the soil and the deflection produced by the pressure application at that point, decreases linearly with increasing pile diameter. In his 1955 paper, Terzaghi^{4.23} points out that there is an 'erroneous conception, widespread among engineers, that the numerical value of the coefficient of subgrade reaction depends exclusively on the nature of the subgrade'. Terzaghi's 'bulb of pressure' concept, used to illustrate the effect of pile width or diameter on k_h is shown in Fig C4.5(a). In 1972 Broms^{4.24} stated that although knowledge of scale effects on the value of the coefficient of horizontal subgrade modulus was not yet complete, Terzaghi's assumption that k_h was inversely proportional to D was normally used for design purposes. Broms^{4.24} also refers to test results which show that the shape of the pile cross section (in sands) has only a very small effect on k_h .

Recent research^{4.25} has indicated that the assumption that k_h varies inversely to changes in D (in clays) is not strictly true. In a theoretical prediction of a full scale pile deflection test from self boring pressure-meter data, Hughes et al^{4.26}, while making use of the Terzaghi assumption that k_h varies as the inverse of pile diameter, emphasised that because of the non-linear nature of the soil p/y relationship, the dependence of k_h on the level of soil strain must be taken into account. In the evaluation of the soil spring constants in a Winkler model, the k_h values used were the secant gradients of the soil p/y curves appropriate to the pile lateral diametric strains, y/D (as outlined in Section 4.3.2.1(b)).

C4.6.1.1 The normally assumed variation in the modulus of horizontal subgrade reaction with depth in uniform cohesionless and cohesive soils is illustrated in Fig C4.5(b).

C4.6.2 Methods by which soil p/y curves may be obtained from laboratory tests of samples are presented by Matlock^{4.4} (soft clay) and Reese, Cox and Koop^{4.27} (sand). Methods by which the static p/y curves can be modified to more closely represent cyclic loading conditions are also discussed in these papers. A comprehensive summary of these and other methods is given by Hughes, Goldsmith and Fendall^{4.19}.

With further development, the determination of p/y curves from insitu testing methods (i.e. Menard pressuremeter or self boring pressuremeter) is likely to provide more realistic design parameters than p/y curves determined from laboratory test data. An outline of the historical background and recent advances that have



– Ideal restraint from friction slab under earthquake conditions

$$= W \times \mu' - W \times C_{H\mu} (T=0)$$

$$= W [\mu' - C_{H\mu} (T=0)]$$

– Dependable restraint provided by friction slab under earthquake condition

$$= \phi W [\mu' - C_{H\mu} (T=0)]$$

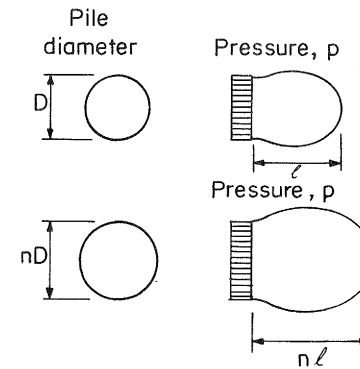
Suggested values:

– strength reduction factor, $\phi = 0.70$

– friction coefficient, $\mu' = \tan \phi'$

where ϕ' = effective soil internal angle of friction

FIG. C4-4 LATERAL RESTRAINT PROVIDED BY A FRICTION SLAB UNDER EARTHQUAKE CONDITIONS

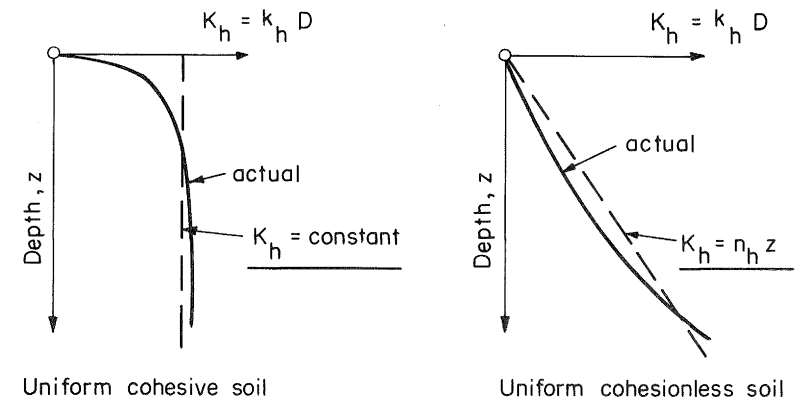


Lateral displacement	Pile lateral diametric strain	Coefficient of horizontal subgrade reaction
y_l	$\frac{y_l}{D}$	$k_{hl} = \frac{p}{y_l}$
ny_l	$\frac{ny_l}{nD} = \frac{y_l}{D}$	$k_h = \frac{p}{ny_l} = \frac{k_{hl}}{n}$

The coefficient of horizontal subgrade reaction varies as the inverse of pile diameter (for a given level of pile lateral diametric strain).
– Applies to both cohesive and cohesionless soils.

Terzaghi's "bulb of pressure" concept (Reference 4-23)

(a) Assumed Variation in the Coefficient of Horizontal Subgrade Reaction with Pile Diameter



(b) Assumed Variation in the Modulus of Horizontal Subgrade Reaction with Depth in Uniform Soil

FIG. C4-5

been made in pressuremeter development are given in References 4.28 and 4.29. It should be noted that in contrast to the Menard pressuremeter, determination of soil parameters from self boring pressuremeter test results are less dependent upon empirical correction.

C4.6.3 The suggestion that for cyclic loading the soil stiffness parameters should be reduced to 70% of their static values is based on the recommendations of ACI Committee 336^{4.5}.

C4.7.1 The following method of assessing liquefaction potential has been shown to be in extremely close agreement with data from sites where liquefaction has occurred during strong ground motion. The method is based on Chinese research (as reported by Seed^{4.30}).

The critical value of the standard penetration test resistance N_{crit} separating liquefiable from non-liquefiable conditions to a depth of approximately 15 m is determined by:

$$N_{crit} = \bar{N} (1 + 0.125 (z - 3) - 0.05 (z_w - 2)) \quad (C4.10)$$

where z is the depth to the sand layer under consideration in metres, z_w is the depth to the water table below ground surface, and \bar{N} is a function of the shaking intensity as follows:

Approximate MM intensity	N (blows per 0.3m)
9	16
8	10
7	6
6	4

Where the N value for a particular sand layer is greater than N_{crit} , liquefaction is unlikely to occur under design seismic conditions.

Equivalent Modified Mercalli (MM) intensities for bridge structures designed for given earthquake return periods in the three Seismic Zones may be estimated from the following:

	Return Period		
	50 Year	100 Year	150 Year
Zone A, I_A	8.0	8.6	9.0
Zone C, I_C	5.4	6.0	6.4

For Zone B, the equivalent MM intensity in terms of the return period and the geographic coefficient β (defined in Fig 2.1, Section 2) may be obtained from:

$$I_B = 4.3 \log_{10} \beta + I_A \quad (C4.11)$$

For most sites where evidence of liquefaction has been observed, the critical layer in which liquefaction is believed to have occurred has been located at depths of less than 15 m and the ground water level has been at depths of less than 5 m.

C4.7.2 Remedial methods, by which liquefaction potential of very localised pockets of sand might be reduced, could involve:

- (a) Increasing in-situ soil density by vibro-flotation or displacement piling.
- (b) Removal and replacement of materials with liquefaction potential.
- (c) Lowering the ground water level by draining.
- (d) Grouting.

Evidence from site observations following earthquake attack has indicated that where liquefaction has occurred it has generally involved the complete bridge structure rather than isolated pier foundations. It has also been observed that damage resulting from the liquefaction of soil overlying bedrock is not necessarily avoided by socketing the piles into the bedrock.

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