

CODE PROVISIONS RELATED TO SOILS AND FOUNDATIONS

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1. INTRODUCTION

In the new Loadings Code, NZS 4203 "Code of Practice for General Structural Design and Design Loadings", the basic seismic coefficient (Clause 3.4.2) in addition to being related to the period of the structure, is related to the deformability of the site (section 3.4.3). The requirement for foundation interconnection (section 3.7.3) has been carried forward from the earlier code with little change. A new section on foundation design (3.3.6) has been introduced in which the principles of capacity design are applied, where appropriate, to the foundation substructure.

2. DESIGN SPECTRUM FOR DEFORMABLE SITES

2.1 Site Effects

Over the last decade or so there has been an increasing realisation among earthquake engineering specialists that the dynamic response of a structure is influenced very considerably by the nature and extent of the subsoils at the site. It is desirable to incorporate such effects in the Code.

Early Californian codes included a design spectrum (that is, the relationship between basic coefficient and building period) based on spectra of strong-motion records of earthquakes, mostly observed at sites where rock or very firm soils existed. The design spectrum showed the maximum coefficient extending from zero to 0.14 seconds period, followed by a very sharp reduction at longer periods (20% lower at 0.25 seconds period).

Since then, strong motion records on sites with less rigid subsoils have shown predominant periods (at which acceleration response is a maximum) much longer than 0.25 seconds. Provision was made in NZSS 1900 (Ref. 1) in 1965, to allow for the effects of a broader range of intermediate foundation conditions, by extending the range of periods over which the coefficient was at its maximum value to 0.45 seconds. (See Commentary on NZSS 1900, Ref. 2).

The new Code extends this concept. A definition is given (Clause 3.4.3) of the conditions under which a building "shall be considered to be on flexible subsoil". This is an attempt to define, in terms of readily obtainable data, sites with long natural period, which fall outside the rigid or intermediate conditions previously catered for.

Typical effects of different subsoil

conditions are indicated in Figure 1. In a minor earthquake, there is considerable amplification of surface motion where deep deposits of low modulus soils exist (that is, on a "soft" site) and the predominant period, as indicated by the peak acceleration response, is larger than on rock. In a major earthquake, because of greater energy loss in the soils, this amplification effect is not found. The predominant frequency is larger than in a minor earthquake, as the shear modulus of a soil is less at higher strain amplitudes.

The natural period of the subsoils at a site is dependent on modulus and thickness of the soil strata. Modulus is, at least approximately, proportional to strength in cohesive soils and this makes it possible for the definition to be in terms of strength rather than in terms of shear modulus, which is rarely measured.

For buildings on deformable sites the seismic coefficient has been increased by 10% for zone A, 20% for zone B and by 30% for zone C (approximately) but the maximum value of the coefficient has not been increased. The result is to extend the period range for which the coefficient is at its maximum to 0.6, 0.7 and 0.8 seconds in zones A, B and C respectively.

The increase in seismic coefficient for deformable sites is made greatest in zone C, where amplification of motion from a large distant earthquake is likely.

Other countries have adopted rather different code provisions to deal with this problem. Some merely increase the seismic coefficient, by a factor, but this suggests a misunderstanding of the problem. Primarily, it is the frequency content of the surface motion which is altered at a deformable site. Other codes require the estimation of the natural period of the soil profile at the site. While having a good theoretical basis, this is considered to be an impractical requirement for everyday structures, at this stage. Arguments can be put forward for reducing the coefficient at low period values on deformable sites but this also requires some estimate on the site period to determine over what range the reduction should extend.

2.2 Definition of "Flexible Subsoil"

The relevant clause states: "A building shall be considered to be on flexible subsoil if there are uncemented soils exceeding one of the following depths beneath the lowest continuous horizontal subsystem; that is, interconnecting beams or continuous slab forming a diaphragm:

6 m of cohesive soils with	50 kPa or less
undrained cohesion	

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8.5 m of cohesive soils with 100 kPa or less
undrained cohesion
12 m " " 200 kPa "
15 m of cohesionless soils"

The provisions for cohesive soils are shown graphically in Figure 2, where "flexible subsoils" are defined as lying below the line $c = 1.39 d^2$.

where c is the undrained cohesion in kPa
and d is the thickness in meters.

This relationship provides a simple method of interpolation. In practice, of course, soils are seldom uniform, and it will be necessary to use an average value of cohesion (weighted according to thickness of strata).

Unfortunately, the shear modulus for a soil, on which the natural site period is dependent, does not have a unique value but decreases with increase in shear strain amplitude, typically as shown in Figure 3. As indicated in the inset, the depth d , of the soil (assumed uniform) is one quarter of the wave length, λ , for vibration in the fundamental mode. Thus the period, $T_0 = (\text{wave length}, \lambda) / (\text{shear wave velocity } v_s)$

$$= \frac{4d}{\sqrt{G/\rho}}$$

where G is the shear modulus, and
 ρ is the mass density of the soil

Taking $G/c = 100$, a typical value in the range of strain amplitudes found in major earthquakes, then

$$T_0 = 4d \sqrt{\frac{\rho}{100c}}$$

Soil density does not have a very wide range, and if ρ is taken as 1600 kg/m^3 , $d = 12 \text{ m}$ and $c = 200 \text{ kPa}$, then $T_0 = 0.43 \text{ seconds}$, that is, about the turndown point, for rigid and intermediate soils, on the graph giving the basic seismic coefficient. A site where the depth of soft soils is greater, or the cohesion less, would have a longer natural period. The other combinations of depth and cohesion have been selected to give the same natural period. The criterion for cohesionless soils was derived in a similar manner.

The criteria for deformable sites are straightforward and practical, and should not leave too many borderline cases in which judgement must be exercised.

It should be noted that the provision of a piled foundation extending to rock does not absolve the designer from using the "flexible subsoil" seismic coefficients on deformable sites. Piles tend to move (horizontally) with the soils they penetrate so the earthquake motions on a deformable site will not be significantly reduced by the provision of a piled foundation.

3. FOUNDATION DESIGN

3.1 Tie-Beams

A clause (3.7.3) similar to that in the earlier code specifies that foundation elements should be interconnected. This has

long been regarded as good practice, and observation of damage in actual earthquake has clearly indicated the value of inter-connection. More often than not, the size of such ties will be determined by their design bending moments.

3.2 Basis of Design

The general purpose of the section on foundation design (clause 3.3.6) is to carry the philosophy (outlined in clause 3.3.2) of capacity design through to its logical conclusion - which includes estimation of and design for capacity loadings on the foundation elements. To quote from clause 3.3.2.2:

"...energy-dissipating elements or mechanisms are chosen....and all other structural elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained"

The foundation design section is divided into two parts: design for factored loads and capacity design. The latter is applied to structures which are intended to yield in a fully ductile manner while the former (factored loads) applies to all structures.

3.3 Factored Load Design

Factored load design, only, is used for categories 6 and 9 in Table 5, which have structural type factors 1.6 and 2.0 respectively. Capacity design methods are not appropriate here. In catering for the calculated foundation loadings, some reasonable factor of safety must be applied. The situation is now complicated by the use of the factored load combinations given in clause 1.3.2:

$$\begin{aligned} U &= 1.0D + 1.3 L_R + E && \text{with earth-} \\ \text{or, } U &= 0.9D + E && \text{quakes} \\ \text{and, } U &= 1.4D + 1.7 L_R && \text{for static} \\ &&& \text{loads} \end{aligned}$$

These combinations are used, in the design of steel and reinforced concrete, with a reduction factor on ultimate strength, ϕ which is taken as unity for steel and within a range (0.7 to 0.9) for concrete, depending on the element being designed.

The factors of safety traditionally applied in soil mechanics for foundation design (using a working stress method) are 3 for static loads and 2 for seismic loads. With the better site investigation practices now in use, and the observation that bearing capacity failures are rare, it is not unreasonable to reduce these factors of safety somewhat. Some factor of safety (between 1.4 and 1.7 for static load) is already inherent in the factored load combinations. Some further factor of safety, to bring this nearer the traditional value of 3, is required. An additional factor of safety of 1.8 (equivalent to a " ϕ -factor" of 0.556) is now suggested. Table 1 lists the overall effective factors of safety (as traditionally defined in the working stress method) for various load combinations.

These are marginally below the

traditional values, but should be quite adequate when used with measured soil strength parameters. The "average measured soil strength" referred to in the commentary is, of course, intended to be that appropriate for the footing or pile under consideration.

3.4 Eccentric Loadings

In the design of eccentrically loaded footings, it used to be customary to assume a triangular distribution of contact stress. Now, following the work of Meyerhof, it is considered preferable to consider a rectangular stress block (just as in ultimate strength design of concrete) as shown in Figure 4. The justification is that, if the loading were increased to failure, redistribution of stress would occur, tending towards the rectangular shape assumed. Another good reason is that it simplifies calculation.

Clause 3.3.6.1.2 deals with the case where the resultant (of gravity load and seismic load) falls outside the base of a shear wall (category 6 of Table 5). Such walls may carry a large seismic load but little gravity load, giving large eccentricities. The clause allows the design gravity load to be arbitrarily increased by up to 20% to simplify design for modest eccentricities. Some justification for this is that when a moment acts on a shear wall on spread footings, the pad may lift off the ground. The wall may rise slightly and therefore take more dead load from interconnected floors, etc.

The problem of dealing with large eccentricities still remains. For shear walls in piles, it is usually not difficult to design for tension. For spread footings, however, it may be necessary to provide adequate moment resisting capacity in the foundation interconnection beams to other footings beyond the shear wall base. In some cases, some form of "tie-down" anchors may be necessary.

It should be borne in mind that an earthquake loading which produces an apparently unstable situation does not necessarily lead to collapse. This is because we are thinking of a dynamic situation in static terms. Even if our estimate of the loading is correct, the earthquake loading is reversed after a second or less - there is no time for the structure to fall over!

3.5 Capacity Design

Most structures will be designed to be ductile, and will be subject to the capacity design procedures of clause 3.3.6.2. Although not stated in this clause (as this section of the Code deals with earthquake provisions only) foundations must, of course, be designed to carry the factored load combinations for dead and live loads with a reasonable factor of safety as outlined before. In addition, they must be capable of withstanding the capacity loads (as determined from yield moments in the beams, for a framed structure) with a minimal factor of safety. (A figure of 1.1 is suggested in the commentary.) Usually the capacity loading will control the dimensions of the foundations for

exterior columns, while, for interior columns, where the shear forces on opposite sides of the column tend to balance, gravity loading is likely to be the controlling factor. The code provisions are summarised in Figure 5 and in Table 2.

3.6 Foundation Uplift

Analysis of a framed structure may show that there is a negative foundation load on some outer column footings. In the absence of any provision for uplift forces, this means that the footing will lift off the ground. (See Figure 6.) This is not considered, in itself, to be a serious disadvantage, but it has the further effect of allowing one bay of the frame to sway without the formation of plastic hinges. That is, some of the energy-dissipating mechanisms are ineffectual. According to clause 3.3.6.2.2, only one quarter of the footings should be allowed to lift off the ground at one time.

If lift-off can only occur with seismic loading along one axis (that is N-S, but not E-W, for example), it is permissible in a 3-bay frame, but not in a 1- or 2-bay frame. 33% of the hinges will be ineffectual in the 3-bay frame. The proportion would be higher - and therefore unacceptable - in a 1- or 2-bay frame and some form of anchorage would be required.

If, on the other hand, concurrent loading (clause 3.4.1.2) may cause lift-off on two adjacent sides of a structure, the limit of not more than one quarter of the footings being allowed to lift at any one time still applies.

3.7 Load Limitation

In some cases, structures may be designed (for architectural or other reasons) with far greater strength than required to withstand Code loadings. In such cases, it is considered unrealistic to expect the designer to cater for capacity loadings on the foundations. The upper limit, with a lateral force corresponding to the product $SM=2$, then applies. For less than a large destructive earthquake, the structure will probably respond elastically. For a really major earthquake, some yielding of the foundation subsoils may occur thus providing a further "energy-dissipating mechanism". In such cases care should be taken that shear failure in the structural foundations is precluded and that any flexural yielding of the substructure occurs in a ductile manner.

REFERENCES:

1. N.Z.S.S. 1900 Standard Model Building By-Law, Chapter 8: Basic Design Loads (1965).
2. Commentary on Chapter 8 of N.Z.S.S. 1900 (1965).

TABLE 1

FACTORS OF SAFETY

Factored Loads	$L_R = 0$	$L_R = 0.5D$	$L_R = D$
$1.4D + 1.7 L_R$	2.52	2.70	2.79
$1.0D + 1.3 L_R + E$) when $E = 0.5D$)	1.80	1.93	2.02
$1.0D + 1.3 L_R + E$) when $E = D$)	1.80	1.91	1.98

TABLE 2

FOUNDATION DESIGN PROCEDURES

STRUCTURAL TYPE (Clause 3.3.6)

<u>Group A</u>	<u>Group B</u>
Fully ductile structures, where capacity design is appropriate	Structures of limited ductility where capacity design is inappropriate.
Table 5, Category 1, 3 $S = 0.8$ " 2, 4 $S = 1.0$ " 5 $S = 1.2$ " 7 $S = 2 +$ " 8 $S = 1.6$ Table 5a Category (A1) $S = 2.0$	Table 5, Category 6 $S = 1.6$ " 9 $S = 2.0$ Table 5a Category (A2) $S = 2.5$ " (A3) $S = 2.0$ " (A4) $S = 1.5$
Upper Limit of $SM = 2.0$	For foundations (Clause 3.3.6.3)

DESIGN METHODS, LOADINGS AND FACTORS OF SAFETY

<u>DESIGN METHOD</u>	<u>STRENGTH METHOD</u> (Clause 1.3.2)	<u>ALTERNATIVE METHOD</u> (Clause 1.3.3)
For all Structures: Factored Loads	$U = 1.4 D + 1.7 L_R$ $U = 1.0 D + 1.3 L_R + E$ $U = 0.9D + E$ Use additional factor of safety.	$A = D + L_R$ Use factor of safety = 3
		$A = D + L + 0.8E$ $A = 0.7D + 0.8E$ Use factor of safety = 2
For Group A structures only	For capacity design loadings, use a factor of safety = 1.1	

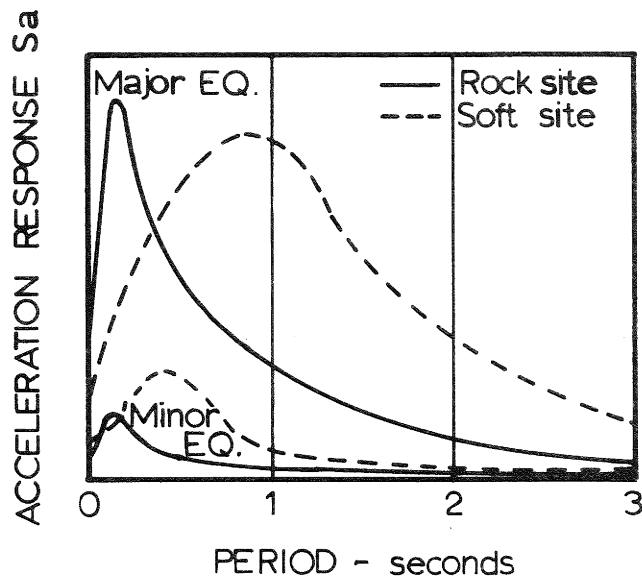


FIGURE 1: TYPICAL RESPONSE SPECTRA AS AFFECTED BY SUBSURFACE CONDITIONS.

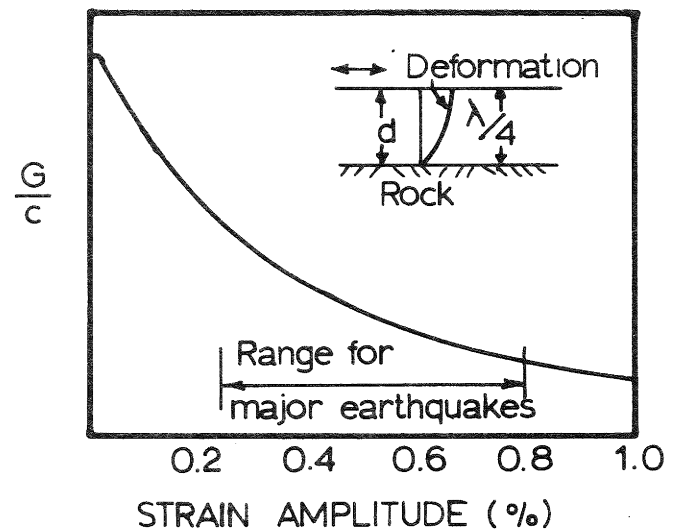


FIGURE 3: TYPICAL VARIATION IN SHEAR MODULUS WITH AMPLITUDE OF SHEAR STRAIN.

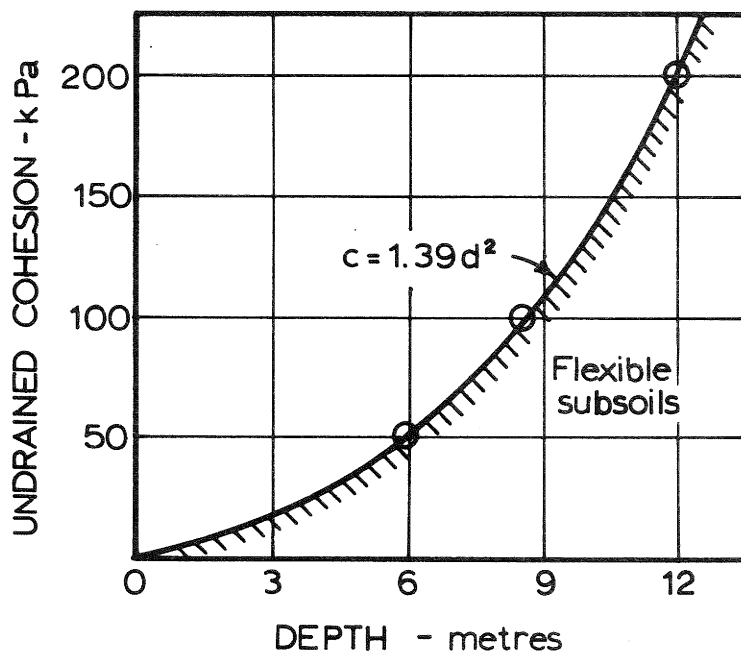


FIGURE 2: DEFINITION OF "FLEXIBLE SUBSOILS".

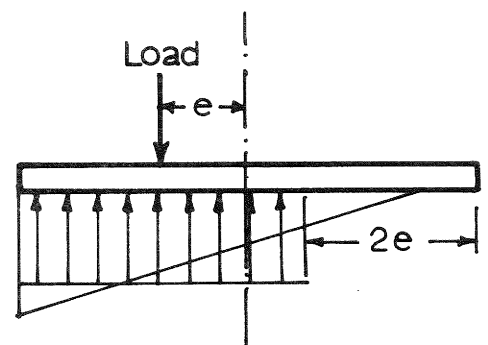


FIGURE 4: STRESS DISTRIBUTIONS ASSUMED BENEATH ECCENTRICALLY LOADED FOUNDATIONS.

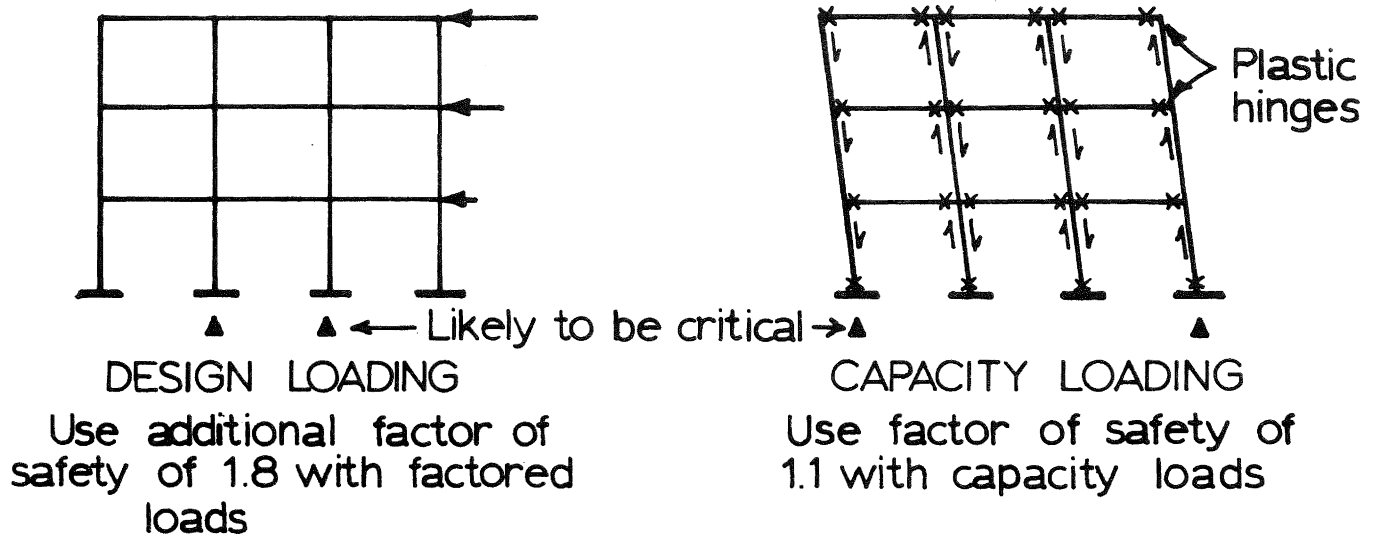


FIGURE 5: LOADINGS CONSIDERED IN FOUNDATION DESIGN

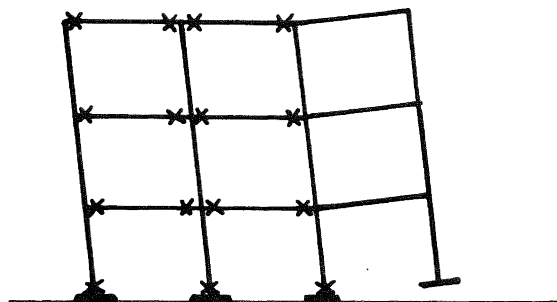


FIGURE 6: UPLIFT OF FOUNDATIONS.