# SOME COMMENTS ON THE NEW NEW ZEALAND EARTHQUAKE LOADING PROVISIONS FOR BUILDINGS

BY: O A GLOGAU*

## 1.0 SYNOPSIS

The philosophy and some of the more unusual features of the earthquake provisions in NZS 4203(1) : "Code of Practice for general structural design and design loadings of buildings" are discussed. Seismic loads are derived from a function which includes terms reflecting the expected manner of dissipation of seismic energy and the performance of a structural system in general. The code recognises that the design load level must be intimately linked with other structural design and detailing criteria if satisfactory behaviour in earthquake is to be achieved. Consequently all ductile structures are required to be the subject of a procedure called capacity design which includes the concept of concurrency. The code establishs design criteria for architectural and services components to avoid damage in moderate earthquakes and minimise direct and indirect life risks in severe events.

## 2.0 INTRODUCTION

The purpose of NZS 4203:1976 is defined in a document NZS 1900 Chapter 8. The clauses relating to the earthquake provisions are very brief and consist of a number of general statements of objectives to the effect that the design and construction of any building shall be such that all loads likely to be sustained during the life of the building will be sustained with an adequate margin of safety; deformations will not exceed acceptable levels; in events that occur occasionally such as moderate earthquakes, structural damage will be avoided and other damage minimised; in events that occur very seldom, such as major earthquakes, collapse will be avoided and the probability of injury to or loss of life of people in and around the building will be minimised.

Setting desirable general objectives is not particularly difficult but in real life situations with its severe competitive pressures, experience has shown that the objects of a code are achieved only if minimum criteria are spelled out in considerable detail. This is particularly true in the "gray area" where the requirements for life hazard damage and other damage protection merge.

To overcome the difficulty of providing adequate restraints without being overly restrictive and thus stiffling sophisticated or innovative methods, the format of presenting mandatory and commentary clauses on opposite pages in close association was chosen. Some safe "cookbook" rules could thus be given to the busy practitioner in the comment section while allowing sufficient scope for alternatives for those willing to investigate in depth.

The commentary section also contains some information that will ultimately become part of the material codes. Pending their revision it was necessary to include this material in NZS 4203 to achieve the intended structural performances at the specified load levels.

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## 3.0 DUCTILITY REQUIREMENTS

The code requires all structures to have some ductility but structural types such as ductile frames and coupled shear walls for which the lowest seismic design loads in the code apply must be the subject of more stringent design procedures to achieve "adequate" ductility.

The code does not specify earthquake motions. Using response spectra of the "Skinner" (2) type and modal analysis with 7½% damping, ductile reinforced concrete structures of the intermediate exposure hazard class (I = 1.3) possessing yield levels just matching zone A code design load levels, would have overall ductility demands of about 5 made on them. Normal design procedures however and built-in reserves such as strain hardening (Bauschinger effects) result in actual structures of greater strength and hence reduced ductility demands. Plane frame inelastic dynamic analysis (bi-linear model, 5% damping) using a series of motions including El Centro 1940 N-S and "A1" on code designed structures of the above type and class indicate ductility demands of only 2-4, a more acceptable level for reinforced concrete.

For structure types such "squat" shear walls and braced frames which do not dissipate seismic energy in flexure, higher loadings are specified to reduce ductility demand.

## 4.0 CAPACITY DESIGN AND CONCURRENCY

Buildings designed for flexural ductile yielding must be designed by a procedure called capacity design. In the capacity design of earthquake-resistant structures, energy-dissipating elements or mechanisms are chosen and suitably designed and detailed, and all other elements are then provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained throughout the deformations that may occur. Because of the relatively high ductility demand expected of code level designed structures, simultaneous hinging of all beams framing into a column and for several storeys is a high probability. The code requires consideration of this condition called concurrency, which is not to be confused with conventional assumptions made in dynamic analysis procedures to cater for orthogonal components of earthquake motions.

The principles of capacity design are simple and reasonable but the application in practice, particularly to reinforced concrete design involves more complex considerations. Because capacity design is an important feature of the code, experience with its practical application to date will be discussed in some detail. Amongst the stated aims of capacity design procedures is avoidance of column hinge mechanisms and non-ductile forms of failure. Pioneering efforts in the application of capacity design to reinforced concrete indicated that unless some of the design decisions could be made on a semi-probabilistic basis, excessively conservative and difficult to construct designs would result. It also became obvious that much research was required to obtain the required data. The design of beam column joints and the provision of shear reinforcement in columns were areas of particular difficulties. In the absence of adequate statistical information on strength variations of New Zealand manufactured reinforcements, the probabilities of adverse seismic load combinations and the extent of accidental column hinging, designers tended to make, as is customary in normal structural design, conservative assumptions. This lead to the assumption that all moment capacity enhancing reinforcements were highly overstrength and all reinforcements resisting brittle failure were under-strength. Limited investigations on the distribution of beam moments to columns in inelastic response situations indicated significant departures from elastic response situations, and further that in some cases, all efforts to the contrary, inelastic hinges occurred at one or both ends of a column within a storey. If column hinging did occur and shear reinforcement had to be provided for this condition the question arose as to what were the appropriate end-moment capacities, which are sensitive to axial loads, of the columns. While some of the data are now available assumptions must be made where information is lacking at present. (3)

## 5.0 VERTICAL EARTHQUAKE EFFECTS

Sufficient indirect protection against vertical earthquake effects on principal members is at present expected to be provided by the gravity load factors and the capacity design procedures. Drafts of the code did include provision for vertical earthquake effects but the resulting axial loads on the columns superimposed on those from other severe code requirements seemed excessive. The code does however require consideration of vertical earthquake effects for horizontal cantilevers and in the design and anchorage of certain non-structural components.

## 6.0 METHODS OF ANALYSIS

The code allows three methods of analysis:

i) The equivalent static force analysis.
ii) The spectral modal analysis. (4)
iii) Numerical integration response analysis

Methods two and three are subject to restrictions derived from method one and generally intended to be a means of obtaining additional information on the behaviour of a structure rather than reducing seismic loads. For very irregular structures a dynamic analysis is mandatory.

## 7.0 EQUIVALENT STATIC FORCE ANALYSIS

The total horizontal seismic force is derived from a multi-term expression

$$V = CISMRW_t$$

C is a coefficient allowing for 3 seismic zones and 2 types of soil stiffness. Soil effect modifications are structure period dependent and are greatest for the lowest seismic zone where the least inelastic soil effects are expected. Range 1.65

I is an importance factor and provides for three exposure hazard classes of buildings with a range of 1.6.

S is a complex factor that is intended generally to reflect the potential seismic performance of various structural systems e.g. ductile frames, coupled shear walls, squat shear walls, braced systems. Range 3.

M is a material factor e.g. structural steel, masonry etc. Range 1.5

R allows for a risk additional to that provided for by I e.g. for large numbers of people. Range 1.1.

A reduction of up to 10% is allowed for dynamic analysis. The total range of all factors is about 11 to 14 depending on period.

The lowest S factors and hence seismic loading are those assigned to coupled shear walls with diagonally reinforced coupling beams and to ductile frames. Systems dissipating seismic energy in a (controlled) shear mode must be designed for twice the S factor of the above. Cantilever structures are penalised even more heavily.

Structural steel is accorded the lowest M factors, masonry and prestress concrete the highest.

## 8.0 SOILS AND FOUNDATIONS (5)

Soil effects are allowed for in the C factors mentioned above. An arbitrary level has been set beyond which rocking of structures or lifting of some foundations is allowed. The code makes no other provisions for soil structure interaction as insufficient information was considered to be available.

## 9.0 TORSION PROVISIONS (6)

For a number of reasons torsional situations are discouraged both by the fairly severe provisions of the code and the commentary. Even the most sophisticated analysis does not account for out of phase ground motions which may damage the junctions of L, T and U shaped buildings. Equally significant is that in torsional situations uniform dissipation of energy is rare, Some portions of torsionally unbalanced structures must be the subject of larger displacements to provide compensating energy dissipation for others close to the torsion centre. The maximum permitted non-amplified eccentricity in the case of less ductile building types, (e.g. certain shear walls) is one-third the building width. This is because stiffness degradation has the particular serious effect of further increasing torsion.

Three types of design approach are permitted: the holy static, a combined approach of static torsion analysis in combination with the two-dimensional modal analysis and the three dimensional spectral modal analysis.

The code requires consideration of torsion effects to provide for amplification effects due to interaction between torsional and translational modes, and accidental effects.

## 10.0 NON-STRUCTURAL COMPONENTS (7)

Design criteria for components were established in accordance with the stated aims of the code (refer 2.0). Hence consideration had to be given to the direct and indirect effects of damage and the consequences of functional impairment. (Loss of elevators, stairs, emergency lighting etc). Direct design criteria take into account : seismic zone, function of building, expected overload performance of the component and its attachement, consequences of failure as well as the maximum expected dynamic loads on a part in a particular structure designed to code levels. To avoid over conservative values resulting from the combination of many factors, upper limits were established. These are defined from reasonable maximum accelerations likely to be experienced in an elastically responding structure. A scaling factor is then applied allowing for risk and "toughness" of part. The upper limit values are also used as co-efficients in a simplified procedure. Protection of components and structure from induced forces due to building deformations is provided by the requirements governing separation of components from the structure and limitations in general on the flexibility of buildings. Structure deformations must be computed using a modification factor taking into account inelastic deformations. Except for essential facilities, the onset of damage is expected to be in the region of 1/3 to ½ the earthquake which has a high probability of being exceeded in the life of the structure. The provisions are aimed at giving a comparable level of protection for differing structural systems. (8)

For very rigid structures such as squat shear walls for which the specified deformations are so small that even built-in rigid exterior elements are not expected to fail, separation of elements is not required.

It is recognised that the problem of floor motions and the damage causing parameters is complex. For important or unusual cases, floor motions studies are therefore recommended. For some of the more common situations such as small towers, smoke stacks, appendices and horizontal cantilevers simple provisions for dynamic amplification due to double resonance are specified.

## 11.0 SMALL BUILDINGS

Non essential buildings of less than 1400 $m^2$ and not more than 2 to 3 storeys may be designed by either of two simplified procedures which allow also less restrictive member geometry and detailing provided significantly higher loadings are used. One method, the low inelastic demand design procedure is intended to permit certain types of small buildings commonly met with in current practice but known to perform badly if designed to low loadings. The other method, the elastic design procedure does not require consideration of ductility at all, but seismic design load levels in zone A are about 1g. The provision is intended to allow small services structures to be simply designed and constructed. Additional requirements in the material coded should be adequate to legislate against highly brittle structures.

## 12.0 COMPARISON OF NZS 4203 WITH OTHER CODES

Figures 1 and 2 compare the seismic design coefficients of NZ 4203 for a number of structural types and soil conditions with the well known SEAOC (1974) and AIJ codes. As far as possible curves have been normalised for load factors or allowable stresses. It is seen that purely from a design load level NZS 4203, seismic zone A occupancy class III (private buildings etc) is in general comparable to the SEAOC provisions but well below the AIJ requirements. It should be pointed out however that the New Zealand code capacity design and detailing procedures, which include concurrency, result in significantly more severe design requirements for columns. For reinforced concrete, which is used for the construction of most medium height buildings in NZ, the zone A seismic design load levels and other requirements are probably the maximum that can reasonably be specified. From a practical and political point of view there is little prospect of an upward revision of the present load levels unless structures meeting the new requirements have been shown to be grossly inadequate in a severe earthquake.

## 13.0 CONCLUSION

The seismic provisions in NZS 4203 are believed to provide advanced, reasonable, prudent and practical protection of life and property at an acceptable economic level in cost taking into account the relative seismisity of New Zealand as compared with other countries, and the total cost of a building's construction.

## 14.0 REFERENCES


1 NZS 4203:1976, STANDARDS Ass. of New Zealand, Private Bag,Wellington Also '1976 Supplement' to E.Q. Res. - A World List 1973, IAEE.

2 Skinner, Bull. No.166, NZ Dep. Scient. & Ind. Research

3 Park & Paulay, Reinforced Concrete Structures, Sect.11.6.11, Wiley

Further background material is given by refs. 4 to 8 below, Bull. NZNSEE Vol 9, No 1, 1976.

4 Skinner : Dynamic Aspects

5 Taylor : Soils & Foundations

6 Elms : Torsional Effects

7 Kolston : Parts & Portions

8 Glogau : Code Philosophy

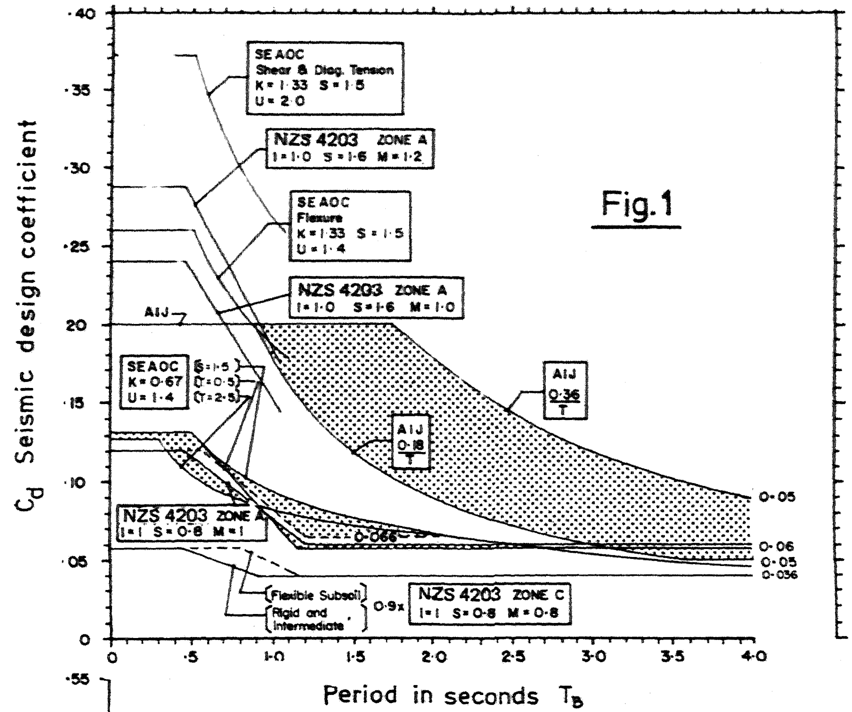
SEAOC
Shear & Diag. Tension
K = 1·33 S = 1·5
U = 2·0
NZS 4203 ZONE A
I = 1·0 S = 1·6 M = 1·2
SEAOC
Flexure
K = 1·33 S = 1·5
U = 1·4
NZS 4203 ZONE A
I = 1·0 S = 1·6 M = 1·0
AIJ
SEAOC
K = 0·67
U = 1·4
S = 1·5
T = 0·5
T = 2·5
AIJ
0·36/T
AIJ
0·18/T
NZS 4203 ZONE A
I = 1 S = 0·8 M = 1
0·066
Flexible Subsoil
Rigid and Intermediate
0·9x
NZS 4203 ZONE C
I = 1 S = 0·8 M = 0·8
0·05
0·06
0·05
0·036
$C_d$ Seismic design coefficient
Period in seconds $T_B$

Fig.1

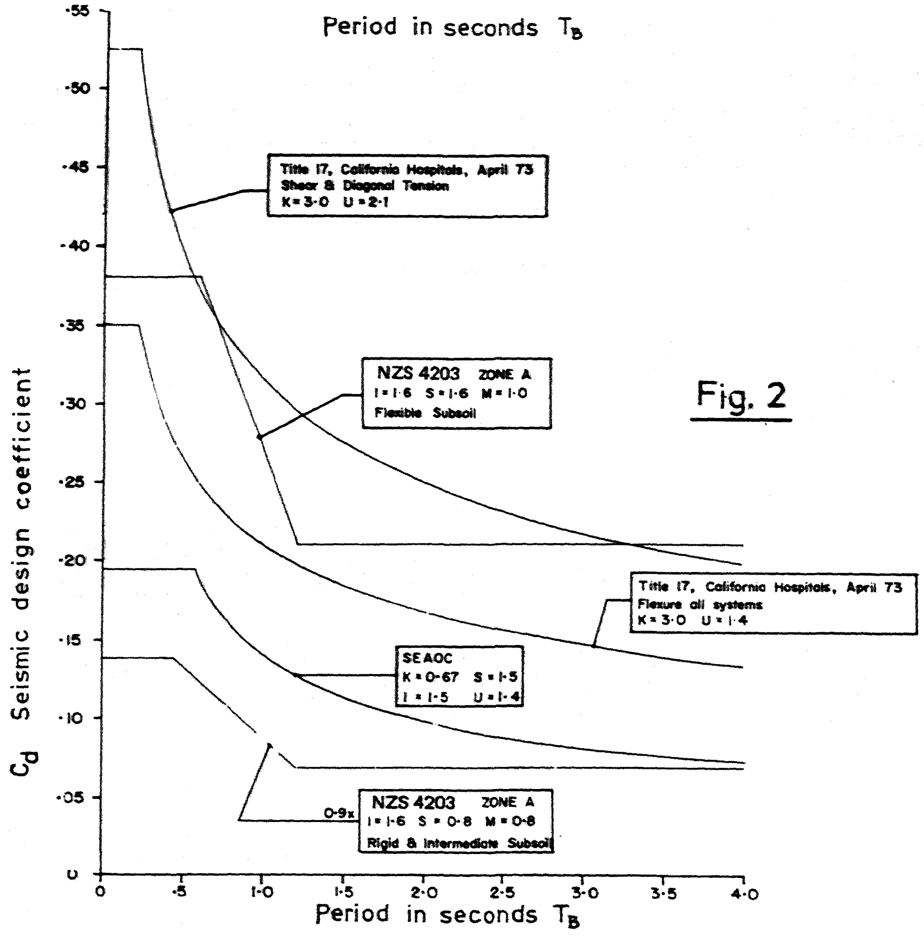
Title 17, California Hospitals, April 73
Shear & Diagonal Tension
K = 3·0 U = 2·1
NZS 4203 ZONE A
I = 1·6 S = 1·6 M = 1·0
Flexible Subsoil
Title 17, California Hospitals, April 73
Flexure all systems
K = 3·0 U = 1·4
SEAOC
K = 0·67 S = 1·5
I = 1·5 U = 1·4
0·9x
NZS 4203 ZONE A
I = 1·6 S = 0·8 M = 0·8
Rigid & Intermediate Subsoil
$C_d$ Seismic design coefficient
Period in seconds $T_B$

Fig. 2

## DISCUSSION

### P. Padmanabhan (India)

It is now an accepted principle that the development of plastic hinges in column should be avoided. But when it come to the question of translating this principle into practice, there seems to be no reliable and easily applicable design rule for this purpose. For instance the New Zealand code recommends that columns shall be designed to have adequate over capacity to avoid the formation of column hinge machanism, but gives no guidance regarding the means of ensuring compliance with this recommendation. The author has mentioned that in some of his investigations, all efforts to the contrary, inelastic hinges occurred at one or both ends of a column within a storey.

In this context, the information contained in some other papers presented at this session are also worth noting. Portillo and Ang (1) have arrived at the conclusion that in the case of a 10 storey reinforced concrete building designed in accordance with an existing code, the probability of failure of beams in flexure was of the order of 0.03 while the probability of failure of columns under combined axial load and flexure was in the range from 0.02 to about 0.10. Shibata and Sozen (2) have reported their investigation of an eight storey frame. They amplified the column moments by a factor of 1.2 to reduce the risk of column yielding, and designed the frame considering damage ratios of 6 for beams and 1.0 for columns. Inelastic dynamic analysis indicated that the beam damage ratios were generally close to the target value of 6 while the column damage ratios are generally within 1.0 (i.e. elastic range) though a few columns exhibited damage ratios upto 1.5.

From the above discussion it appears that there is an urgent need for further research for evolving some simple guidelines for the designer, regarding the additional strength for which the columns are to be designed.

References:


1. Manuel Portillo and Alfredo H.S. Ang, "Safety of Reinforced Concrete Buildings to Earthquakes", 6WCEE Preprints, 5-147.

2. Akenoiri Shibata and Mete A. Sozen, "Substitute Structure Method to Determine Design Forces in Earthquake Resistant Reinforced concrete Frames, 6WCEE, Pre-Prints 5-167.

## Author's Closure

With regard to the question of Mr. Padmanabhan, we wish to state that the author whole-heartedly agrees with the discusser that further research into the probabilities and the extent of column hinging, particularly in reinforced concrete frames, are of great importance to designers.

Handicapping researchers at present are lack of knowledge of a truely representative mathematical model of the structure in three dimensions including the slab and its reinforcement. How the model changes when responding in a non-linear manner is a further factor contributing to the difficulty. To the best of the writer's knowledge no reasonably realistic non - linear 3-dimensional numerical integration response analysis computer programme is as yet available, although significant progress has been made towards this goal.

Meanwhile some very interesting results have been obtained from 2-dimensional non-linear numerical integration analysis including the work by Kobori* et al and Kelly**.

It is important that the results should not be based on over simplified models e.g. representation of beams as infinitely stiff and/or ignoring strain hardening in the representation of the hysteretic loops, as otherwise very misleading results are obtained.

As the discussor pointed out, the formation of (at least temporary) hinges in columns is difficult to avoid that this is not necessarily disastrous provided

a the detailing is such that sudden brittle member behaviour is avoided.

b the cumulative ductility demand is low.

* Optimum design of the structural members due to ground motion. 6WCEE. Pre-prints, 5-79.

** Uniaxial dynamic analysis of a six storey reinforced concrete framed structure. Bulletin New Zealand National Society for Earthquake Engineering, Volume 10 No. 1, March 1977.

Condition (b) can be insured by amplifying column moments and shears above those obtained from an elastic code design level analysis.

Much more work will need to be done in this field but based on what is known at present, a set of relatively simple rules have been formulated for use by designers of ductile reinforced concrete frames and these will be published shortly in the 'Bulletin of the New Zealand Society for Earthquake Engineering, Volume 10 No. 2 etc. 1977.