

## DRAFT REVISION OF NZS 4203 : 1984 : SEISMIC PROVISIONS

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**ABSTRACT:** A draft revision to NZS4203:1984 [1] is now available for public comment. Significant changes to the existing seismic provisions are proposed. The basis for some of these are discussed and, where appropriate, resulting values are compared with the current provisions.

### 1. INTRODUCTION

Since NZS 4203:1976 [1] first offered New Zealand designers rules for accounting for structural characteristics when determining seismic loading, two national codes with rather different types of seismic load formulation have been published, namely the Highway Bridge Design Brief [2] and the Seismic Design of Petrochemical Plants [3]. These codes also have provisions which recognise some of the improvements that have become available in recent years in seismicity description and in structural appreciation, but neither is entirely suitable for general building design. There have been other significant developments in engineering seismology and in our understanding of structural response to earthquake excitation which ought to be recognised in our national codes.

At the same time, and principally overseas (though New Zealand people have contributed), efforts have been made to provide more rational bases for design rules which control element and structure responses to static and dynamic loads.

The present code [1] was last changed in 1984, when the most significant of its three amendments was ratified. This shortened the code a little and simplified some parts that had proven troublesome in practice; but essentially the document is still the 1976 one. Patching has achieved about as much as patching can achieve. The time has come for a fresh look.

In the preparation of the draft, many individuals and institutions with specialist skills and experience were canvassed for their views and for contributions to the work. The drafting committee's debt to these people for their helpful responses is acknowledged. Two SANZ committees, the Relative Risk Sub-Committee, which prepared a seismicity model of New Zealand and used it to establish uniform risk horizontal acceleration response spectra, and the Probabilistic Design Advisory Committee, which helped with the selection of factors

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appropriate for applying to loadings and with other matters, were significant contributors.

The draft requires designers to consider two limit states, serviceability and ultimate, with appropriate load factors specified for each. The load factors for the ultimate state are smaller than New Zealand designers are accustomed to, and definitions of some loadings have been altered. These changes should make design appraisals more realistic. They follow trends in other countries, notably in Canada and the United States of America, but studies to support them have been made in New Zealand, especially by the Probabilistic Design Advisory Committee of SANZ. To make the design process properly consistent throughout, changes will be needed in materials codes.

The drafting committee was constantly aware of the demand for simple codes for designing simple buildings. The draft is written in such a way that users can edit it for themselves, removing the complexities when it is justifiable to do that. But every competent designer knows that earthquake loadings are structure generated, so the complexity of the design and assessment provisions must reflect structural complexity if consistent strength and ductility is to be achieved.

The draft consists of four parts. A fifth (Part D: Wind Loadings) has yet to be written. Part A contains the General Provisions, Part B deals with gravity loadings and Part E with environmental loadings (including snow, thermal, soil, hydraulic and ice loadings).

The following discusses only Part C: Earthquake Loadings. Comparisons with the present requirements [1] are made and reasons for some of the requirements are given.

### 2. GENERAL REQUIREMENTS

The NZS4203 [1] provision that main elements resisting seismic forces be disposed symmetrically "as nearly as is practicable" has been abandoned in the draft for reasons that are well known to practitioners; but, of course, the superiority of symmetrical framing continues to be recognised.

The draft also recognises that

building frames with limited capacity to respond in a ductile manner will be built, and it makes provision for them. Although a continuous tradeoff between strength and displacement ductility could justifiably have been made, design processes are more easily devised and controlled by establishing discrete ductility levels. For each of these levels, all materials codes should specify design and detailing rules. Cooperation between the Loadings Code Committee and committees responsible for materials codes is needed to formulate both loading and design controls. Tentative provisions have been included in the draft, anticipating the kinds of controls that might result from this exercise.

When the response modifying effect of displacement ductility is not invoked, there is a diminished prospect that contributions to any internal force from earthquake excitation in one direction will significantly reinforce the peak probable value of that internal force generated by the orthogonal earthquake excitation. On that account, concurrency provisions in the draft do not apply to elastically responding systems.

Studies indicate that the response modifying effect of displacement ductility may owe as much to period shift as it does to energy dissipation [4]. NZS4203 references to the energy dissipating mode of response abatement have accordingly been changed.

### 3. FOUNDATIONS

No rational basis for the existing NZS4203 provision limiting SM to 2 for the design of foundations for buildings has been uncovered, so there is no corresponding limit in the draft. Thus a philosophical difficulty that has been troubling many designers for some years is removed. A designer may elect to invoke load limiting properties of the foundation material as part of the ductile system if he is prepared to make a special study. This would normally involve him in a programme to establish a reliable soils model, followed by an inelastic dynamic analysis.

### 4. STRUCTURAL REGULARITY AND TORSION

The draft requires buildings to be classified according to the uniformity of their framing systems, defining the boundary separating "regular" from "irregular" by referring to apparently reasonable but somewhat arbitrarily chosen measures of the properties of structures which are determined from consideration of deflection responses to prescribed lateral loads. Design forces for the components of regular buildings may be obtained from static load analyses. Modal analyses (at least) are required for irregular buildings. Tall or flexible buildings, those for which the fundamental period of vibration exceeds 1 sec, must be analysed for earthquake response by the modal method even when regular, so need not be classified. This requirement is to ensure that significant higher mode

components that might be generated in these buildings are not overlooked.

"Vertical" regularity controls are intended to ensure that the lateral displacement response to the prescribed static lateral load is sufficiently smooth to be accepted as a reasonable representation of the prime mode shape. When it is not, a modal assessment is needed to correct the pattern. Fig. 1 describes the checking for vertical regularity. It is evident that one of the deflection patterns does not fall within the criterion.

"Horizontal" regularity controls are intended to separate buildings for which the assessment of torsional response by static procedures is considered to be too unreliable from those for which static procedures will do.

The problem of dealing with torsional response deserves more consideration than it is customarily given. Many of the methods that have been devised for it are often quite inadequate, having deficiencies well known to practical designers even though they seem to be largely unrecognised by researchers. Concepts based on "centre of resistance" or of "rigidity" and on the distance between that usually ill-defined and often physically meaningless place and the mass centre through which a floor inertia acts, the so-called "eccentricity" assume, usually tacitly, that a vertical axis of rotation exists and can be defined, and that it passes through the "centre of resistance" on each floor. Unfortunately this assumption is, in general, a false one.

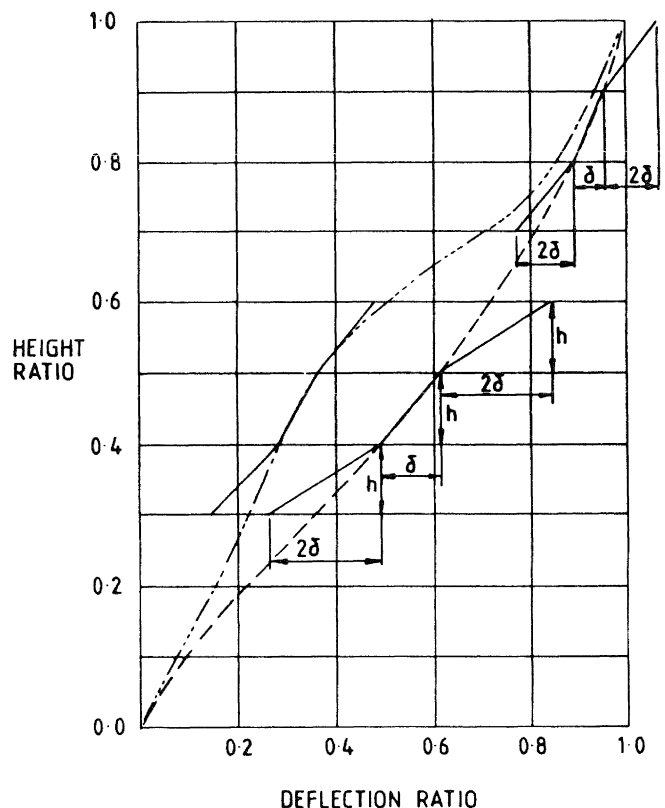


FIG. 1 : CHECKING FOR VERTICAL REGULARITY.

For buildings which are assembled from plane frames all of which have geometrically similar lateral deflection responses to any given lateral load pattern, the eccentricity idea will produce a result, as it will for buildings which are absolutely symmetrical about both principal axes. But for real buildings, which astonishingly rarely have either of these qualities, any result that is obtained is, at best, a flawed one. Not surprisingly, application of rules for dealing with torsion in design offices has been tentative, groping and non-uniform. Buildings of the type now called hybrid in which walls and frames cooperate to resist lateral load need be only modestly asymmetrical to show alarming centre of resistance shifts between floors near the ground and near the tops of the buildings, even when the resisting frames and walls are quite uniform over the entire heights.

Notwithstanding its defects, the eccentricity idea (with appropriate warnings and limits) has been retained in the draft, to serve when essentially two dimensional analyses are considered to be sufficiently reliable to furnish design data. (It is also used in defining the boundary separating horizontally regular from irregular buildings.) It is expected that, for the limited classes of buildings that qualify to be assessed for torsional response by static methods, designers will use the method either to define factors to amplify the internal forces that lateral load generates in each frame when the building is constrained to translate without rotation, or to provide information for modifying the properties of frame components so that a separate two-dimensional analysis can be made to supply torsional information. (This latter option will only occasionally be the more attractive one.) To this end, the drafting group will recommend that the project committee alter the draft by inserting a provision that will be useful for hybrid buildings, permitting instability that cannot be avoided in the position of the centre of resistance near the top and near the bottom of structures in certain controlled cases to be ignored. It is proposed that the centre of resistance to be used throughout should be that calculated as the average for the floors in the middle third of the building height. Designers who use the method should be aware that a resistance centre set is not simply a property of a structure, as it is commonly supposed to be, but of the structure and its loading.

Modal analysis, available now to most designers, is required for more structures than it was hitherto, so that some of the difficulties can be resolved. Designers should be aware that texts which discuss torsional and translational modes for unconstrained structures are limiting their consideration to symmetrical cases. Real buildings do not usually behave so obligingly that such an absolute classification of modes is justifiable.

New in this draft are provisions to recognise resonant interaction between translational and torsional displacement response in certain structures. There are

possibilities for very significant amplification which ought not be ignored.

Much work remains to be done before there can be any complacency concerning torsional provisions in the design rules. Those who contribute should concentrate on the problems of real structures, remembering how rarely these are, or reasonably can be, symmetrical.

##### 5. BASE SHEAR COEFFICIENT

The seismic "weight" of the building as defined in the draft consists of two components - dead load and "serviceability" live load (Ls). The latter quantity is separately tabulated in Part B of the draft, alongside the maximum live load. The ratio of the two live loadings varies with floor usage and has been estimated on the basis that the difference is greatest for office loadings, for which survey information is available, and that there is no difference for storage usage. In some literature, Ls is referred to as the live load "at an arbitrary point in time" - it may be regarded as a snatch sample value.

The elastic response spectrum for a 5% damped oscillator on alluvium as derived by the SANZ Seismic Risks Subcommittee [5] forms the basis of the suite of design spectra proposed.

The elastic response spectrum ( $u=1$ ) is a "risk spectrum", that is, each acceleration response ordinate is derived on the basis that it has the same probability of exceedance as has every other ordinate. The process is described in Matuschka et al [5]. The spectrum is therefore not an average of "real" earthquake spectra.

Fig. 2 compares the risk spectra with elastic response spectra in two other design documents [1,6] for zones of highest intensity. In addition, two averaged spectra for some Japanese classes of earthquake [7] have been computed with attenuation modified to suit New Zealand conditions [8]; finally, some averaged spectra for earthquakes in the Western US [9] have been included.

The spectra from ATC-3-306 [6] have frequencies of exceedance of 1/475 and 1/150 per annum, and so may be directly compared with the risk spectra [5]. It is evident that the ordinates of both risk spectra are significantly greater than the ATC3-06 ordinates for periods up to about 1 sec - by up to 30%.

The 1/150 per annum risk spectrum has similar ordinates to the present [1]  $SM=4.8$  ordinates between period values of 0.4 sec and about 1.4 sec after which it continues to reduce below current values. It was hoped to recommend the use of the 1/450 per annum risk spectrum as the basis for design loadings in the Draft. However, as is evident from Fig. 2, the increase in ordinates above those presently applying was felt to be unacceptably high; this applies particularly at periods below about 0.5 sec where the effects of enhancement due to uncertainty [8] have

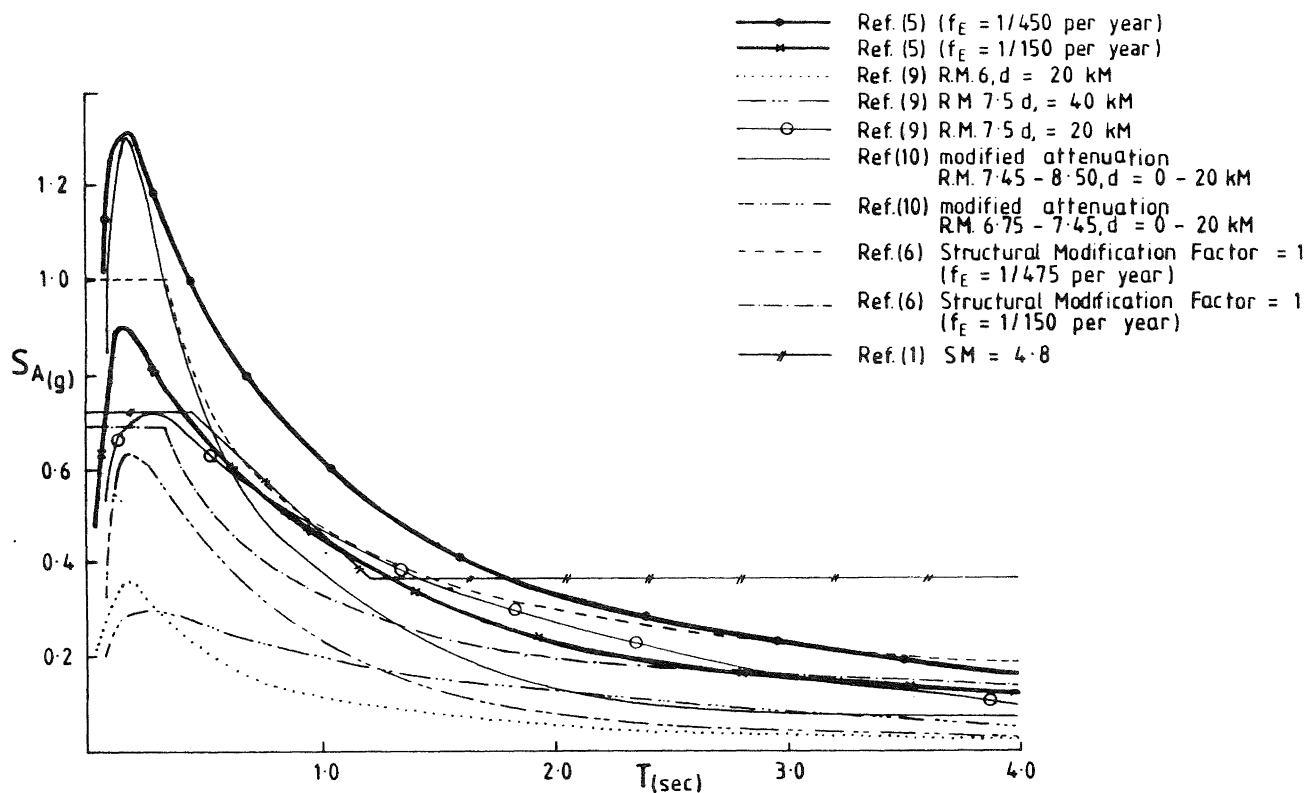


FIG. 2 : COMPARISON OF SMOOTHED ELASTIC RESPONSE SPECTRA (5% OF CRITICAL DAMPING) .

the greatest effect. The reduction to about 70% of the ordinates for the 1/450 per annum event therefore leads to a three-fold increase in the probability of exceedance of the design earthquake for normal buildings.

The table of ductility factors proposed in the draft is influenced by the present code [1] and ATC3-06 [6]. Following receipt of public comment on the draft, these factors will be reviewed and it is expected that input from the SANZ Design Code Liaison Committee will be requested. This committee includes representatives from committees associated with structural materials codes.

Fig. 3 describes the relation between response ordinate and return period. It is evident, for example, that a 170% increase in these ordinates describes a 1000 year event and a 100% increase describes a 450 year event.

The Draft includes a proposal to increase the ordinates of some densely populated areas of moderate seismicity. It was felt that concentrations of national investment should have a margin over surrounding areas. Attempts were made to quantify the margin, but these have been inconclusive. One approach was to consider the expected damage cost times total population (i.e. sum of (frequency of earthquake shaking within a narrow band  $\times$  expected damage ratio)  $\times$  population for other cities compared to Wellington.

While most margins proposed seem plausible when the rate of change of

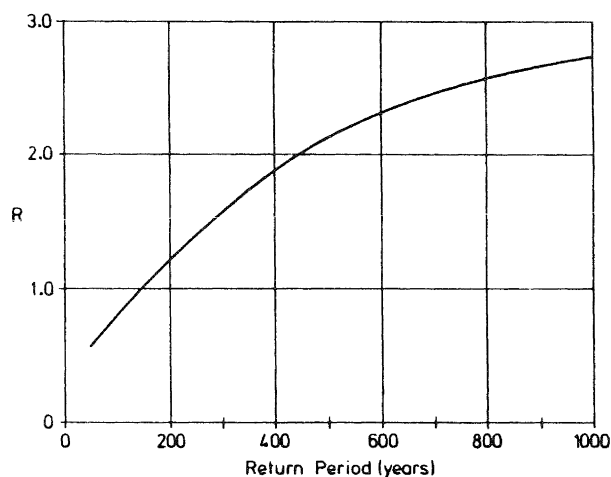


FIG. 3 : RELATIONSHIP BETWEEN RISK FACTOR AND RETURN PERIOD.

seismicity in surrounding areas is considered, the proposed value of 0.5 g for Auckland is perhaps a little high - a value of 0.4 g might be more appropriate for the 1/150 per annum exceedance probability.

The design spectra proposed in the draft comprise sets for varying levels of overall deflection ductility factor,  $\mu$ . The relationship of the ordinates of the inelastic spectra (i.e.  $u > 1.0$ ) to those of the elastic spectrum proposed by the SANZ Seismic Risk Sub-committee [5] for respective values of natural period (T) was obtained as follows:

(1) Single degree of freedom structures with 5% equivalent viscous damping were subjected to shaking of six earthquake records, namely Hachinohe NS, Tohoku NS, Sendai Basement, El Centro 1940 NS, Parkfield N65E and Orien EW. Thus, three Japanese and three US earthquakes were used.

(2) The simple resonators were analysed at various yield strengths (estimated as a proportion of peak base moment after the elastic response was obtained) of 0.167, 0.25 and 0.5 times peak response, and natural periods of 0.1, 0.2, 0.5, 0.8, 1.0, 1.5, 2.0, 3.0, 4.0 and 6.0 sec. Deflection ductility factors (ratio of maximum deflection to deflection at yield) were calculated, and were found to range widely at the 0.1 sec value of natural period. (It is expected that the effect of numerical inaccuracy will be more apparent when computations involve high frequency response coupled with the effect of dividing the response deflection by a small value (yield deflection) to obtain ductility factor.) The 0.1 sec values were not used in the process, therefore. A set of acceleration responses versus natural period were plotted for each earthquake record and the calculated ductility factor noted. Sets of ordinates at the above natural periods were thus obtained by interpolation for ductility factors of 1.25, 2.0, 4.0 and 6.0.

(3) The six curves for each value of  $\mu$  were superimposed with ordinates calculated as a proportion of elastic response ordinate for the risk spectrum at respective natural periods. Fig. 4 shows the resulting set at  $\mu = 4$  against a background of the curves appearing in the draft for normal soils. The scatter is most pronounced at low values of natural period. It is apparent that smoothing as well as averaging of ordinates has been involved in producing the proposed curve which harmonise with the form of the remainder of the family of curves.

It has been assumed that the peak ground acceleration (i.e.  $T = 0$  sec) is 0.4 times the peak response acceleration at  $\mu = 1$ , after reference [6]. Elastically responding structures of period less than 0.2 sec are permitted to be designed for a value of response acceleration which rises from 0.4C ( $T=0.4$ ,  $\mu=1$ ) to C ( $T=0.2$ ,  $\mu=1$ ). This is not permitted for ductile structures, because the increase in effective period following yield will result in an increase in response acceleration. It is further assumed that acceleration response for ductile structures of zero period matches peak ground acceleration. This allows the spectrum end point at  $T=0$  to be defined.

Separate spectra for soft soils are proposed in the draft. The same definitions of soft soils as in the present code [1] are retained. Katayama's [7] Class IV ("usually soft alluvial layer or reclaimed land") has been taken as equivalent and the ordinates have been calculated from

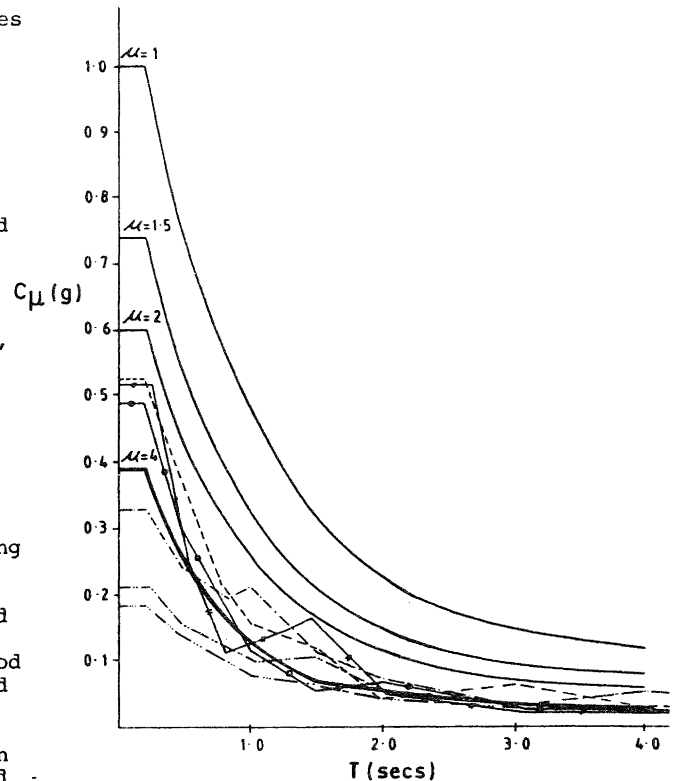


FIG. 4 : FORM OF DESIGN SPECTRA PROPOSED AND SET OF SOURCE SPECTRA FOR  $\mu = 4$ .

$$C(T, \mu)_{\text{soft soil}} = C(T, \mu)_{\text{normal soil}} \times$$

$$SA(T)_{\text{class IV}} / SA(T)_{\text{class III}}$$

(10). Potentially liquifiable soils are required to be the subject of a special study.

Fig. 5 compares present and proposed spectra for elastically responding and ductile reinforced concrete buildings founded on normal soils. While there is similarity between the  $\mu = 1.25$  (elastically responding) case, the effect of the so-called "equal energy" criterion at periods less than 1 sec becomes increasingly pronounced as the period reduces for ductile buildings, when compared to the present [1] "equal displacement"-based approach.

#### 6. SEISMIC LOADING FOR PARTS AND THEIR CONNECTIONS

The treatment of parts (or "non-structural elements") is based on the notion that a part may be regarded as a structure within a structure: that is, the part may itself respond inelastically to seismic motion which is input from the ground via an inelastically responding supporting structure.

Accordingly, no equivalent of Table 9 has been included in the present draft [1]. As with the building structure, so with the part: the designer selects the ductility factor appropriate to the form of the element and obtains a value of basic acceleration coefficient,  $C_{up}$ . This is divided by the

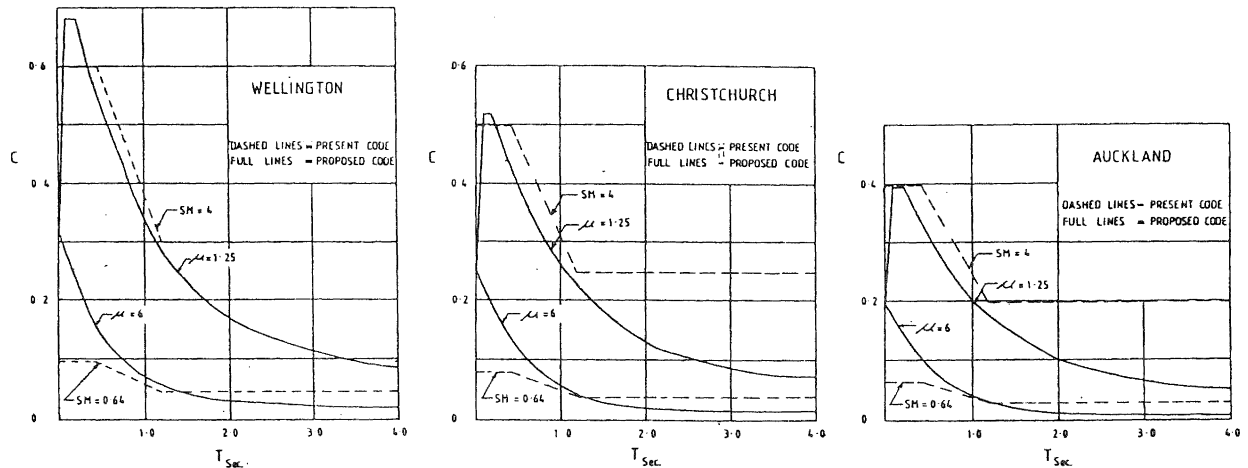


FIG. 5 : COMPARISON OF PRESENT AND PROPOSED LOADINGS FOR REINFORCED CONCRETE BUILDINGS.

basic coefficient for an elastically responding part,  $C_p$ , and the quotient multiplies the response acceleration as described by Kelly [1] for elastically responding parts.

Table 1 compares design values for parts with those in the present code [1]. Values from reference [1] are shown in brackets. Two items of parts from Table 9 [1] have been selected because the method of determining  $C_p$  (response acceleration for parts) differs between them (there is no difference in approach in the draft): diaphragms (item 6 of Table 9) are shown at left of each bracketed pair, and the right hand value is for walls etc. not adjacent to exitway (item 1(b)(ii) of Table 9 [1]).

In order to achieve reasonable comparison, the following assumptions were made:

	Draft	Present Code
Ductile part	$C_{up} = 6^*$	$S_p = 1.0$
Limited ductility part	$C_{up} = 2$	$S_p = 1.2$
Elastic part	$C_{up} = 1$	$S_p = 2.0$
	$= C_p$	$MP = 0.8$

\* The draft limits  $C_{up}$  to 3.

Inertia load on parts, as it is assessed by the rule in the draft, is less sensitive to height above ground than it is by the rule in the present code, so direct comparison is difficult. However, Table 1 shows that loadings for ductile parts (walls and diaphragms) are smaller in the draft than they are in the present code, whatever the building ductility. There are similar provisions for limited ductility parts in ductile buildings. But elastically responding parts in all buildings and limited ductility parts in elastically responding buildings have higher prescribed load levels in the draft than in the present code.

The draft recognises that response acceleration for flexible parts is less than it is for rigid parts unless there is

resonant amplification. Connections will generally be designed by capacity methods. If they are not, then:

- \* a ductility factor of unity must be used for load assessment, and
- \* if people are in jeopardy from a potential failure of the connection of the part to the structure, the connection must have capacity to resist 20% more load than is specified for the part.

A cautious upper limit of 3 was set for the ductility factor of parts because there is a relative paucity of information about response of highly ductile parts. There is a suggestion that this limit should be relaxed for stiff parts, say for those for which the natural period is shorter than about 0.5 seconds, and an amendment to this effect is proposed.

7. SPECTRAL MODAL ANALYSIS

Spectral modal analysis is an alternative to the equivalent static load method from which it differs principally by using the lateral response characteristics of subject buildings to determine the distributions of loads within static load patterns. According to both methods total load magnitudes are functions of vibration periods.

Either spectral modal analysis or explicit dynamic analysis is mandatory when requirements for regularity are not satisfied, or when any prime mode vibration period exceeds 1 second. In three-dimensional modal analysis torsional provisions of the draft can be satisfied by appropriate shifts of the mass centres.

A modified "square root of the sum of squares" (SRSS) method of combining modal components to establish probable peaks of responses is specified. By this method, as compared with others, the significance of "off-diagonal" terms ( $m \neq n$ ) is increased [12].

TABLE 1 : SEISMIC DESIGN COEFFICIENT FOR PARTS

BUILDING		PART								
		DUCTILE (STOREYS)			LIMITED DUCTILITY (STOREYS)			ELASTICALLY RESPONDING (STOREYS)		
		6	10	20	6	10	20	6	10	20
DUCTILE	MID-HEIGHT	0.14 (0.15) (0.15)	0.12 (0.15) (0.20)	0.11 (0.15) (0.20)	0.26 (0.20) (0.30)	0.23 (0.20) (0.65)	0.22 (0.20) (0.65)	0.44 (0.30) (0.30)	0.39 (0.30) (0.65)	0.36 (0.30) (0.65)
	TOP	0.17 (0.30) (0.30)	0.14 (0.30) (0.40)	0.12 (0.30) (0.40)	0.32 (0.40) (0.60)	0.27 (0.40) (1.30)	0.22 (0.40) (1.30)	0.54 (0.60) (0.60)	0.45 (0.60) (1.30)	0.37 (0.60) (1.30)
LIMITED DUCTILITY	MID-HEIGHT	0.19 (0.15) (0.21)	0.16 (0.15) (0.20)	0.13 (0.15) (0.15)	0.36 (0.20) (0.30)	0.31 (0.20) (0.65)	0.25 (0.20) (0.30)	0.60 (0.30) (0.42)	0.51 (0.30) (0.65)	0.42 (0.30) (0.30)
	TOP	0.28 (0.30) (0.42)	0.22 (0.30) (0.40)	0.16 (0.30) (0.30)	0.52 (0.40) (0.60)	0.41 (0.40) (1.30)	0.31 (0.40) (0.60)	0.87 (0.60) (0.83)	0.69 (0.60) (1.30)	0.51 (0.60) (0.60)
ELASTICALLY RESPONDING	MID-HEIGHT	0.25 (0.15) (0.20)	0.21 (0.15) (0.20)	0.16 (0.15) (0.20)	0.47 (0.20) (0.62)	0.40 (0.20) (0.43)	0.31 (0.20) (0.43)	0.78 (0.30) (0.65)	0.66 (0.30) (0.65)	0.51 (0.30) (0.65)
	TOP	0.38 (0.30) (0.40)	0.24 (0.30) (0.40)	0.21 (0.30) (0.40)	0.73 (0.40) (1.24)	0.46 (0.40) (0.86)	0.40 (0.40) (0.69)	1.21 (0.60) (1.30)	0.77 (0.60) (1.30)	0.67 (0.60) (1.15)

For a framed structure, a set of peak probable shears in all vertical resisting elements is to be found for earthquake excitation along each of the building's axes. The results, which include the effects of the mass shifts that generate torsions, are used to construct a load system for each axis. These two systems are used in analyses to establish the required capacities for ductile elements. Capacity protection is to be provided for all other elements.

Design load effects thus obtained might not be adequate when design is dominated by base moment, as in a shear wall. However, the increase in ductility demand should be modest, and would normally be covered adequately by capacity design. Of course the peak probable value of base moment can be obtained from modal analysis and should be used in design whenever it is appropriate to do that. There is discussion in the commentary to the draft.

#### 8. NUMERICAL INTEGRATION TIME HISTORY ANALYSIS

This method of obtaining design load effects involves modelling an assemblage of structural members, generally including their inelastic properties, subject to input of an earthquake motion. Normally, two-dimensional models are used, and these can be adjusted to include some representation of three-dimensional response [13].

In practice, such analysis would normally be a "special study." The draft notes that, in such circumstances, "some or all of the requirements of this Standard or of other Standards may be waived" if agreed by the Engineer, Designer and Design Checker. However, it is likely that the input earthquake would

be related to that proposed in the draft so that a return period could be associated with the event.

The draft proposes that three different earthquake records be used, with ordinates scaled according to Housner's spectrum intensity method, that is, that areas under the velocity spectrum from periods between 0.1 and 2.5 seconds be the same for the elastic spectrum in the draft (appropriate to the level or equivalent viscous damping for the structure) and for the (scaled) velocity of the earthquake record. Thus, the scaling factor for acceleration ordinates is

$$F = \frac{\int_{T=0.1}^{T=2.5} C_e(T) \cdot T \cdot \Delta T}{\int_{T=0.1}^{T=2.5} S_a(T) \cdot T \cdot \Delta T}$$

since  $S_v(T) = S_a(T)/w = S_a(T) \cdot T/2\pi$

where  $S_a(T)$  and  $S_v(T)$  are acceleration and velocity spectra for the motion to be scaled, and  $C_e(T)$  is the code elastic response spectrum.

The method appears relevant if:

- the structure responds at an equivalent first mode period of less than 2.5 seconds
- higher modes are significant.

However, where a single mode at effective period  $T$  (i.e. the "period" of inelastic response) is strongly dominant, then the need for requiring other ordinates at very different values of  $T$  to contribute to the scaling factor becomes questionable. It would seem not unreasonable to scale on the basis of equal spectrum intensity over  $(T-0.5)$  to  $(T+0.25)$ , say. (The 0.25

sec increase in T recognises the likelihood of the inelastic "period" being dependent on scaling factor. A prior inelastic run is needed to establish T.)

It is expected that, generally, two-dimensional analysis would be carried out. Three dimensional analysis is much more expensive. The effects of torsion or of asymmetry can be approximated in two-dimensional analyses by factoring masses for some structures.

Inelastic computer programs give measures of ductility demand at yielding sections of structural members. The tolerability of these can usually be assessed by comparison with the performance of laboratory specimens loaded to similar levels of deformation. Additional strength may have to be provided to reduce ductility demand. Recourse to laboratory results is necessary because materials codes do not give information on member section ductility capability. Limits on the value of this, rather than control on minimum base shear would seem a justified means of reducing structural strength. This approach means that there is no need to check base shear against that associated with an overall ductility factor. Accordingly, we do not require such a check. However, interstorey deflections are constrained by values stipulated in Section 11.

#### 9. STRUCTURAL DEFLECTION

The draft prescribes deflection limits for both ultimate and serviceability limit state. The two limits are measured differently: The former is concerned with both loss of building stability through p-delta effects as well as the possibility of collision with an adjacent structure. The limit is simply the horizontal deflection difference between floors in the case of building stability; this is limited to 2% of the interstorey height at maximum response to earthquake. This value is equivalent to the present [1] limit of 1% at SM=2, and justification is given in [14].

Where deflections have been obtained using inelastic spectra, as in the case of the equivalent static method or when spectral modal analysis is used, scaling up is required to obtain actual maxima for the ultimate limit state. This is not achieved on the basis of the ratio of elastic to inelastic loadings but in the ratio of inelastic to elastic displacement ductilities. The use of a loadings ratio would apply if the "equal displacement rule" applied: inelastic loading proportional to overall ductility factor. However, this holds only for long period structures.

The draft requires all parts in categories I and II structures (i.e. "Buildings dedicated to the preservation of human life or whose loss of function would have a severe impact upon society" and "Buildings which as a whole contain people in crowds") whose impact with the structures would result in a loss of functionality to be separated to accommodate the ultimate limit state earthquake.

In the case of serviceability, a reduced level of earthquake loading is appropriate, a "moderate" earthquake [15]. In the draft, this has been set at a frequency of exceedance of three times that of the ultimate limit state earthquake and corresponds (Fig. 3) to about 55% of the load level.

The deflection of interest will often not be the horizontal component of nodal movement but rather the shear deflection between levels as, for example, when estimating clearance needed for glazing. The component of nodal movement due to the rotation of the structure needs to be subtracted in this case.

#### 10. CONCLUSION

Writing the draft has been the task of the first named author whose contribution was, for the most part, generously funded by the Ministry of Works and Development by arrangement with a former Commissioner, Mr. Bob Norman. All the remaining authors were a committee assisting Dr. Hutchison.

The draft is available to people who may wish to comment now. There will be a SANZ project committee appointed to review the draft and commentators' contributions and to prepare the new standard.

#### 11. REFERENCES

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