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Analysis for Torsion in Multistorey Buildings

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ANALYSIS FOR TORSION IN MULTISTOREY BUILDINGS

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APPENDIX A

EXCERPTS FROM CODES OF PRACTICE

1. INTRODUCTION

Many structural designers in New Zealand are now using computer based analysis packages for routinely carrying out three-dimensional spectral modal analyses of building structures. The typical output from such packages does not always produce results which are consistent with the requirements of the current New Zealand Loadings Code of buildings NZS 4203:1984. There appears to be no concensus among designers as to the preferred method for handling eccentric structures which give significant torsional response using spectral modal analysis methods.

The principal objective of this study is to prepare sound and reasoned guidelines as to how analysts and designers should most appropriately use existing convenient analysis packages to determine design actions.

The current New Zealand Code NZS 4203 contains a number of requirements for designers carring out spectral modal analyses. These appear to have been developed mainly with two-dimensional analyses in mind. No special provisions are made for for three-dimensional analysis, but some of the code requirements are difficult to interpret for three-dimensional systems. For example, code requirements for certain minimum numbers of lateral or torsional modes of vibration to be considered are somewhat arbitrary when, often there is no clear distinction between what are predominantly translational and what are torsional modes.

An important feature of the results from any spectral modal analyses, which often causes concern among designers, is that the member actions output are an envelope only, not an equilibrium set of actions, and no signs are available. A check on the actions at any joint connecting several members will generally suggest that equilibrium is not satisfied. Similarly, a check on the envelope base shear can generally not be reconciled with the direction of the applied spectrum. In the past some designers have found it difficult to reconcile such results and have resorted to a process which involves deriving an equivalent static load distribution which, when applied to the three-dimensional model, gives a consistent set of structural actions which are in equilibrium. The validity of this approach requires investigation.

One question of interest in carrying out this study is therefore, how the spectral modal analysis results for earthquake loading are being used by designers, or should most sensibly be used, in conjunction with gravity and other loading cases to derive member design actions. The way in which these actions can be used as part of the New Zealand capacity design procedures also requires investigation.

In Section 2 relevant national building code requirements from New Zealand and other countries for seismic torsion effects and spectral modal analysis methods are reviewed and summarised.

Section 3 gives a summary of an extensive literature review of recent research on torsional effects in buildings.

A brief review is made in Section 4 of the spectral modal analysis procedure and, in particular, various commonly used modal combination methods are discussed.

Results of an informal survey of New Zealand spectral modal analysis practice are discussed in Section 5.

At this stage the research project is approximately half complete. In Section 6 proposed further areas of investigation and remaining objectives to be addresses in the remainder of this study are outlined.

2. PROVISIONS FOR TORSION DESIGN IN CODES OF PRACTICE

2.1 NZS 4203

Relevant sections of NZS 4203:1984 [1] pertaining to the design for seismic torsion effects are given in Appendix A.

In NZS 4203:1984 the term "regular" appears to refer to vertical regularity. A regular structure has a similar horizontal distribution of mass and stiffness at all levels. Symmetry generally seems to refer to horizontal symmetry of lateral load resisting elements.

Three design approaches are considered by the Code. The first a wholly static approach. The second is a combined approach in which the vertical distribution of of horizontal forces is determined from a two-dimensional modal analysis (clause 3.5.2.2.1) and the torsional effects are obtained from a static provisions of clause 3.4.7. The third approach is to use a full three-dimensional modal analysis (clause 3.5.2.2.2). These approaches are discussed in the following:

i Static Approach

The first approach referred to by NZS 4203 is a wholly static approach in which the distribution of vertical forces is determined based on the triangular static force distribution and the torsional moments are derived based on the shift of resultant shear at any level by 10 percent of the building width either side of the centre of mass. The code is not specific about how the designer should carry out the analysis to comply with these requirements. Presumably for structures that are not symmetrical in plan, a three-dimensional static analysis allowing for the stiffness of elements in both horizontal directions would be necessary to correctly derive the member actions.

In practice, for fairly symmetric and regular structures, it seems that most designers assess the proportion of lateral load to be assigned to each element (eg frame or wall) by a simple analysis of stiffnesses in each direction at a typical storey, and each element would then be analysed in isolation assuming that at each level a constant proportion of the total storey force on the building was applied. The proportions of lateral load carried by each element would normally be assessed taking into account an allowance for the accidental eccentricity requirements of the code by considering the centre of force at all floors to be simultaneously shifted by 0.1b from the originally estimated position toward the element being considered.

ii Combined Modal/Static Approach

The second method referred to by NZS 4203 is a combined two-dimensional modal and static analysis approach. The distribution of lateral forces on the building as a whole is determined using a two-dimensional response spectrum analysis. The torsional effects are then determined in the same way as for the static analysis.

This method is to be preferred over the completely static approach because it should give a more appropriate distribution of seismic forces over the height of the building, but only in accordance with the elastic dynamic characteristics. No account is taken of the likely inelastic dynamic response.

It is noted that this approach assumes that the designer derives a set of equivalent static horizontal storey forces from the two-dimensional modal analysis. A vector of storey forces is calculated in each mode of vibration by multiplying the modal participation factor times the mode shape vector times the response spectral displacement (giving the vector of modal displacments) times the square of the modal circular frequency (giving the vector of modal accelerations) times the mass matrix.

{Q}i = Pi {O} Sdi w2 [M]

The equivalent storey inertia forces from each mode are then combined using some assumed statistical combination method such as the SRSS or the CQC modal combination method. This represents the set of maximum inertia forces expected to be experienced by each storey.

The code apparently then assumes that the structure will be analysed subject to these forces applied simultaneously as a set of equivalent static lateral forces. This is not strictly valid as there is no reason to expect that all maximum floor inertia forces all occur concurrently in the same direction at the same instant. The theoretically correct way to apply modal analysis results is to carry out a complete structural analysis in each mode of response and then to determine the response variable of interest (displacement, member force, storey inertia force, interstorey shear, base shear etc) and only then to perform the statistical combination by whatever method deemed appropriate. The important point is that modal analysis results are statistical predictions of the maximum individual actions which are expected to occur in a structure during dynamic response.

This point about modal anaylses is not widely recognised and it would appear to be often overlooked. It seems that the code writers also endorsed this adaption of the modal analysis procedure, at least for two-dimensional modal analyses.

Despite this, the procedure by which an set of equivalent storey forces is derived from the modal analysis is a convenient one, and which is apparently used often by designers. It is computationally easier as only a single analysis of the complete structure is carried out rather than one for each mode. Also the member forces from the analysis are in equilibrium whereas the full modal analysis results will not be. For example the sum of moments in members framing into any joint will not add to zero.

iii 3D Spectral Modal Analysis

This is the method preferred by the code and the one which is required to be used for structures which are highly irregular (ie significant changes in stiffness with height), or highly asymmetric (ie stiffness is not distributed symmetrically in plan). The code does not give guidance in several important aspects of the spectral modal analysis procedure, for example, how the designer actually derives the member design forces. The following paragraphs give some comments on the code clauses.

The response spectrum analysis results are scaled so that the effective base shear coefficient is not less than 90 percent of the value corresponding to the first mode. The code states in the commentary that "When a building is designed to resist the more accurate distribution of loads given by the spectral modal analysis then an improved performance will result. Base shear values are therefore reduced to 90 percent of the values given in section 3.4" The wording of this, particulary the word "loads" implies that the procedure involves derivation of a set of floor inertia forces which would then be used as a set of equivalent static forces for which the structure would be analysed using a static analysis procedure. As discussed previously for two-dimensional spectral modal analyses, this would not be a strictly correct way to carry out the analysis but evidently this is common practice.

The requirement in clause 3.5.2.2.1 requiring not less than 3 modes in each direction to be considered implies that only two-dimensional analyses are being performed, because for three-dimensional structures it is often difficult to associate any direction with any particular mode. Normally, for an irregular structure, three-dimensional mode shapes contain displacement components in all directions that are not necessarily related to the arbitrary directions used as the principal design axes even when the framing systems are arranged orthogonally. Similarly the requirement in clause 3.5.2.2.2 requiring not less than 4 modes in each direction, with two being predominantly translational and two being predominantly torsional is somewhat meaningless. A more rational requirement would be to require that the "effective mass" in each direction is a significant proportion (say 90%) of the total mass of the system in each direction.

Fortunately, with the use of modern computer analysis packages and hardware, dynamic analysis of large structural systems is becoming much easier to carry out. It is normally possible to include many more modes in a spectral modal analysis than the minimum number required by the code.

Clause 3.5.2.2 requires that the structural model shall include the effects of accidental eccentricities of $\pm 0.1b$. It is not clear how the designer should allow for this effect. There appear to be two main ways in which this is normally done. First by applying the seismic loadings as equivalent static forces (which may be derived from either two or three dimensional spectral modal analyses) and shifting the line of action of the applied forces by the required amount either way from the centres of mass, and second by shifting the centres of mass by the specified amount at each floor level in a spectral modal analysis.

There are limitations and advantages with both methods. The application of equivalent static forces derived from a spectral modal analysis is, as previously discussed, not strictly valid although it leads to a convenient method and a set of internal actions which are in equilibrium. The second method of shifting the centre of mass at each floor level is also convenient as it is simple to apply with most spectral modal analysis computer programs. Tso (ref LC5 of DZ 4203 submissions to SANZ) has pointed out that this may not be a valid way of allowing for accidental torsion as the shift of the centre of mass leads to a change in the dynamic properties of the structure, and therefore different mode shapes and periods. (This author's opinion is that this would generally be an acceptable approximation, as no properties of a real structure will ever be known with certainty and this is part of the reason for the requirements for accidental torsional loadings to be considered. In effect this gives some idea of the model's sensitivity.)

Clause 3.5.2.2 also requires that "for moderately unbalanced buildings the torsional effect shall be not less than that calculated by the static method of clause 3.4.7". In order to demonstrate compliance with this requirement a designer must carry out the torsional analysis using both methods.

Clause 3.5.2.3.1 requires that "the shear at any level shall be taken as the square root of the sum of the squares of the modal shears at that height". This implies that only the square root of sum of squares (SRSS) method is valid. With the widespread use of the Complete Quadratic Combination (CQC) method this requirement would seem to be adequate but not necessary. It is generally believed that for modes of vibration with close frequencies of vibration, a better estimate of the combined actions is obtained by taking the sum of the absolute values of all modal quantities. It would therefore seem reasonable to ignore this clause if a rational alternative method was used.

The code requires, in Clause 3.5.2.5.1 that "at any level the shear derived in accordance with 3.5.2.3 shall be taken as not less than 80 percent of the the values computed by by the equivalent static forces method specified in section 3.4." The code commentary explains that spectral modal analysis might give load values much lower than those given by the static force method and that these low local values are obtained partly as a result of neglecting the effects of inelastic deformations. Some designers apparently ignore this requirement, believing that the spectral modal analysis must have given them a more appropriate distribution of actions than the static analysis would have. To ignore this requirement may be unconservative.

This requirement also means that the designer is obliged to carry out the equivalent static load analysis to determine the resultant shear forces at all levels. A problem arises for designers in knowing how to adjust the distribution of structural actions if they find that the spectral modal analysis results do not comply with this requirement. If the analysis is done by determining equivalent static storey forces from a spectral modal analysis for subsequent application of these to a three-dimensional static load analysis of the structure, then this requirement can be complied with. The analyst simply adjusts the distribution of lateral forces to be applied. However, if the analysis is done using a full spectral modal analysis (the theoretically more correct method), then there is no obvious way to adjust the results to comply with this requirement. One method would be to scale all results up so that the requirement is met. It is apparent that some designers faced with this problem resort to the former method using a 3D static structural analysis.

Clause 3.5.2.6.1 requires that the horizontal forces and overturning moments shall be derived from the shears given by clauses 3.5.2.1 to 3.5.2.5 inclusive. This implies that horizontal forces and overturning moments can be determined uniquely from the horizontal shear at any level based on the principles of equilibrium. However, this is only the case when the spectral modal analysis procedure is used to determine a set of equivalent static forces. The actions derived from a full spectral modal analysis will not be a set of equilibrium actions. For example, the distributions of spectral modal interstorey shears and total overturning moment can not be derived from the distribution of spectral modal floor inertia forces.

2.1.1 Comments on NZS 4203 Eccentricity Formulae

Elms [2] discussed the background to the seismic torsional provisions of the New Zealand Loadings Code NZS 4203:1976 edition. The paper discussed how the design eccentricity equations were derived to account for the effects of accidental eccentricity (accidental variations in centres of rigidity and mass), torsional ground motion and coupling between torsional and translational modes of vibration. The first two effects lead to an assumed eccentricity of 10 percent of the width of the building, and the third effect lead to a parabolic function of the calculated eccentricity.

The accidental eccentricity term 0.1b was assumed to be made up of 0.04b due to accidental variations in the centre of rigidity, 0.01b due to variations in centre of mass and 0.03b due to torsional ground motion.

The term allowing for coupling between the torsional and translational modes of vibration was based on published work by Rosenblueth predicting the amplification of static eccentricity in single storey buildings. This may have been doubtful for multistorey structures.

It is noted that a subsequent amendment to the code equations simplified the parabolic function to current linear relationship.

2.1.2 Further Comments on NZS4203

A paper by Poole [3] paper was the result of deliberations of the NZNSEE discussion group on the Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames. In it, the torsional provisions of NZS 4203:1976, the proposed 1978 amendments to the static analysis procedure and the use of modal analysis were all discussed.

The Static Method was explained and discussed in some detail. The method of calculating the centre of mass was defined to be the centroid through which the resultant of all inertia forces above the level under consideration act. It noted that generally the centre of rigidity at a particular storey was taken to be the centroid of the stiffness of elements in the storey, but it described a more general method for determining the positions of centre of rigidity at all floors simultaneously. This was based on a static plane frame analysis computer program to analyse all frames in each direction placed side by side and linked with rigid pin-ended horizontal members. The design eccentricity formulae in NZS 4203:1976 were restated and the background paper by Elms [2] was referred to. The means of calculating the design storey torques and from these the elastic column shear forces were described. The column actions associated with the storey torques are dependent on the stiffness of each vertical element in both horizontal directions. The paper suggested that a full analysis could be completed using a plane frame computer program in a similar way to the method described for determining the centres of rigidity in each direction. This appears erroneous however, as a three dimensional (ie space) frame analysis would seem to be required. Alternatively a Muto type hand analysis could be performed.

The paper gave some comments on the application of three-dimensional modal analysis methods to the determination of torsional effects. It pointed out that three-dimensional mode shapes will in general have components in all directions simultaneously. It was recommended that 8 or 9 modes of vibration be included in analyses to ensure that all significant response is accounted for. It recommended that the accidental eccentricity effects required by the code could be adequately accounted for by shifting the centres of mass at each level by the ±0.1b in each direction. This would imply that 5 analyses are required for a single structure.

Finally some example building structures with varying degrees of vertical and horizontal regularity were discussed and recommendations were made as to the most suitable method of lateral load and torsional analysis.

2.2 2/DZ 4203/2 (1991)

Sections of the current draft New Zealand Loadings Code DZ 4203 [4], dated February 1991 relating to design for torsional effects are included in Appendix A.

Clause 3.6.3.2 specifies the requirements for accidental eccentricity effects but does not specifically require the designer to consider the torsional effects due to static eccentricity on lateral load resisting elements at any level. The term "static eccentricity" means the eccentricity between the centre of mass and the static centre of rigidity. The clause is satisfactory for determining the required loading including the effects of accidental eccentricity, but some additional clauses should require the analysis of the structure to allow for both the static eccentricity and accidental eccentricity effects in the structural analysis. This will generally require the designer to determine where the centres of rigidity are located in the building. As in the current NZS 4203, there is no modification factor applied to the static eccentricity to account for dynamic magnification due to coupling between translational and torsional modes of vibration. This may be unconservative.

The commentary in this draft gives only a very brief discussion of torsional effects.

There is a significant difference in the Spectral Modal Analysis method given in the existing NZS 4203 Code. The draft code does not require that the results of the analysis are scaled in any way. NZS 4203 and other codes such as UBC, SEAOC and NBCC require the spectral modal analysis actions to be scaled so that the base shear is at least equal to either 90 percent or 100 percent (depending on the code) of the base shear derived from the equivalent static analysis method (effectively considering a triangular first mode only).

(This needs to be updated to reflect the latest revision of the draft code)

2.3 Uniform Building Code (1988)

Sections of the Uniform Building Code (UBC) 1988 edition [5] relating to design lateral forces and dynamic analysis for lateral loads are included in Appendix A.

The provisions of the UBC code for analysis for lateral load analysis and analysis for torsional effects are similar to the present New Zealand Code NZS 4203:1984. Lateral load analysis is permissible by either a static analysis method, or a response spectrum analysis or a time history analysis. The code prescribes requirements for structures for which each of the methods is admissable and how they should be carried out.

For the equivalent static load method, horizontal torsional moments are specified by requiring the designer to consider the centre of mass at each level to be displaced in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. That is, a torsional moment is applied about the centre of mass at each floor level equal to the storey interia force times 5 percent of the lateral dimension.

This differs from the current New Zealand code NZS 4203 in two main respects. The accidental eccentricity given by UBC is based on 5 percent of the building dimension perpendicular to the direction of loading at each level compared with 10 percent required by the New Zealand code . Also, NZS 4203 requires the line of action of the resultant shear at any level to be displaced laterally by the specified amount. For buildings that have the same plan dimensions at all floors the methods are the same, but for buildings with varying plan dimensions the methods are not the same. For typical buildings the dimensions reduce with incresing height and the NZS 4203 method would be more severe.

The UBC code static method also requires the accidental torsional moments at each level to be increased by an amplification factor of up to 3.0 where a structure exhibits certain torsional irregularity characteristics. Such characteristics include significant rotation of floors about a vertical axis under the specified loading, presence of re-entrant corners, diaphragm discontinuity, out of plane offsets of vertical elements or load resisting elements that are not parallel to or symmetric about the orthogonal axes of the structure.

2.4 National Building Code of Canada (1985)

Sections of the National Building Code of Canada NBCC 1985 edition [6] relating to design lateral forces and dynamic analysis for lateral loads are included in Appendix A.

The NBCC code appears to allow the use dynamic spectral modal analysis methods only for determining the vertical distribution of lateral seismic force on a building. It does not specifically refer to the use of spectral modal analysis methods for determining actual member forces. Presumably, the code writers expected that once a spectral modal analysis had been completed and a distribution of lateral force had been derived, then the static analysis procedure would be followed.

For static analysis the design torsional moments about a vertical axis at each storey level are computed by multiplying the resultant horizontal shear by each of the design eccentricities given by the following formulae:

$$e_x = 1.5e + 0.1D_n$$

or

$$e_x = 0.5e - 0.1D_n$$

The eccentricity e is defined as the distance between the resultant of all forces at and above the level being considered and the centre of rigidity (not defined) at the level being considered. (similar to the static eccentricity effect considered in NZS 4203)

The design eccentricities are more severe than those required by NZS 4203, due to the 0.5 and 1.5 coefficients applied to the static eccentricities. The accidental eccentricity terms, being 10 percent of the building lateral dimension, are identical to the NZS 4203 eccentricities.

NBCC also states that where the centroids of mass and the centres of stiffness (not defined) of the different floors do not lie approximately on vertical lines, then a dynamic analysis shall be carried out to determine the torsional effects.

2.5 SEAOC (1990)

The Structural Engineers Association of California (SEAOC) Recommended Lateral Force Requirements [7] are the same as those in the Uniform Building Code 1988 edition.

2.6 FEMA NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings

The United States Federal Emergency Management Agency National Earthquake Hazards Reduction Programme (FEMA NEHRP) has published a document on earthquake hazard reduction [8]. This provides for a static distribution of horizontal earthquake loading on structures and torsional moments resulting from the assumed displacement of the building masses each way by a distance of 5 percent of the building dimension perpendicular to the direction of applied forces.

2.7 Mexico (1977)

The 1977 Mexico design code [9] allowed either static or dynamic analysis of structures. For static analysis it was required that a torsional moment at each level should be considered equal to the shear force at the level times the less favourable of the two eccentricities: 1.5es + 0.1b or es - 0.1b, where es is defined as the distance between the shear centre (not defined) and the shear force at the level under consideration.

For a dynamic analysis the code allows that torsional response may be ignored and computed in accordance with the static analysis procedure.

2.8 Mexico (1987)

In addition to the 1977 requirements, the 1987 Mexico design code [10] refers to the concept of a "resistance eccentricity", defined as the distance from the centre of resistance to the centre of mass. The centre of resistance is assumed to be located at the centroid of the ultimate lateral resisting forces of all elements. This is also referred to by others as the "centre of strength" or the "plastic centre". The code requires that where a force reduction factor greater than 3 from elastic response is assumed in design, then the resistance eccentricity er shall be at least equal to es-0.1b and also that er and es shall have the same sign. These additional requirements were introduced as a result of the 1985 Mexico earthquake in which many buildings appeared to have suffered damage due to torsional response.

The additional requirements relating to the position of the centre of resistance appear to be aimed at trying to ensure that the distribution of strength is not too disimilar from the distribution of stiffness and so that the torsional response of the structure should not be excessive even if significant inelastic deformations occur.

2.9 Japan (1980)

The only Japanese document reviewed was "Standards for Aseismic Civil Engineering Constructions in Japan - Earthquake Resistant Regulations for Building Structures in Japan" 1980 edition [11]. This code requires that the eccentricity stiffness ratio Re of each storey shall be less than 0.15, where:

$$Re = e/r_e$$

where
e = the eccentricity of the centre of stiffness from the centre of gravity
r_e = the elastic radius defined as the square root of the torsional stiffness divided by the lateral stiffness

Where this is not met at any particular level, the code apppears to require that the ultimate lateral shear strength is increased at that level.

Although not specifically defined, presumably the centre of stiffness is calculated considering only the lateral load resisting elements at the level being considered.

The above method makes no allowance for any accidental eccentricity effects. It is believed [12] that current Japanese building codes still do not require accidental eccentricity effects to be considered in design of buildings. This is evidently considered reasonable because any errors are expected to be compensated for by the large earthquake design loads which are specified.

3. LITERATURE SURVEY OF RESEARCH ON TORSION IN BUILDINGS

3.1 Summary

A review of published literature on research into the combined lateral and torsional behaviour of building structures indicates that the full problem is far from being well-understood. Theoretical studies of the elastic and inelastic response of single storey structures are now quite numerous and the behaviour of those systems should now be able to be dealt with with some confidence. For multistorey structures theoretical studies have been reported dealing with elastic torsional response, but information on the inelastic torsional response is very sparse. The problems in studying these effects are directly related to the number of variables involved, which increase by orders of magnitude as multistorey structures or inelastic response is considered. The inelastic response of multistorey structural systems appears to be at least one order of magnitude more complex than has been adequately dealt with so far and is likely to take a considerable future research effort to come to a good understanding of. A definitive understanding of the problem and adequately codified recommendations are necessary, but these are not likely to be available in the near future.

The following sections discuss some research information which is available.

3.2 Elastic Response of Single Storey Structures

During the 1970s and early 1980s a number of researchers conducted theoretical studies of the elastic response of coupled lateral torsional response of single storey structures (eg Tso and Dempsey 1980 [13] and 1982 [14]). These have generally been based on a simple structural models employing rigid floor diaphragms with translational and rotational mass supported on a number of elements with elastic lateral stiffness properties. The elements are arranged so that there is an asymmetry caused by an eccentricity between the centres of mass and of stiffness. Studies have been conducted to determine the response of these simple systems to digitised ground motion records from real earthquakes.

The response of such a system is found to be a function of the degree of eccentricity and the ratio of the uncoupled torsional and translational frequencies of vibration. The largest lateral displacements normally occur on the edge situated furthest from the centre of stiffness or closest to the centre of mass, depending on whether the system has stiffness or mass eccentricity.

When the structure has a torsional mode of vibration with a frequency close to that of a translational mode, coupling takes place between the modes and an amplified torsional response occurs. It has been shown in several similar studies that torsional coupling can give up to 100 percent increase in the elastic lateral load demand in the element on the most severely affected side.

3.3 Inelastic Response of Single Storey Structures

Recently, a number of publications have dealt with the next level of complexity, namely the inelastic coupled torsional/translational response of single storey structures (eg Irvine and Kountouris 1980 [15], Tso and Sadek1985 [16], Goel and Chopra 1990 [17] and 1991 [21], Tso and Ying 1990 [18], Maheri et. al. 1991 [19], Chandler and Duan 1991 [20]. These have been theoretical studies based on similar models used in the elastic response studies except that inelastic behaviour of the lateral load resisting elements below the rigid floor diaphragm is assumed.

These studies have investigated the effect of the distribution of strength, stiffness and mass on the inelastic response of single storey models. In particular the effects of the following have been studied:

- the presence of resisting elements perpendicular to the direction of input ground motion,
- the number of resisting elements in the direction of input ground motion,
- the distribution of strength as specified by various earthquake resistant building codes,
- the relative values of stiffness and strength eccentricities,
- system asymmetry due to stiffness or mass eccentricity.

These studies have generally used as measures of response parameters such as the the maximum floor edge displacement or maximum ductility demand compared with the demands in the corresponding symmetric system.

In general the resistance (or strength) eccentricity is defined as the distance from the centre of resistance (plastic centroid) and the centre of mass of the system.

Irvine and Kountouris (1980,[15]) carried out a parametric study of the inelastic seismic repsonse of a simple torsionally unbalanced building. The building model consisted of a two degree of freedom mass eccentric rigid diaphragm supported by two identical elasto-plastic frames. The parametric study consisted of approximately 3500 separate analyses to identify the effect of variations in the input earthquake record, translational period, torsional to lateral frequency ratio, static eccentricity and yield force level on the peak ductility ratio in the supporting frames. The most interesing result of this study was that there did not appear to be a strong correlation between the peak ductility demand and the static eccentricity. The peak ductility in the most severely affected frame of eccentric structures (static eccentricity $e < 0.25$ times building dimension) was rarely found to be more than 30 percent greater than the ductility demand in a similar symmetric structure.

The summary of findings from some of the most recent papers is given in the following.

Tso and Ying (1990, [18]) used a single mass, three element, monosymmetric model to examine the additional seismic inelastic deformations and displacement caused by structural asymmetry of the model. Stiffness eccentricity and resistance eccentricity (that is eccentricity between centre of mass and centre of resistance, or strength) are used as measures of asymmetry in the elastic and inelastic ranges respectively. Seven ways of specifying strength distribution among resisting elements were considered, including code provisions from Canada, Mexico, New Zealand and the United States. Those specifications were related to the model resistance eccentricity. An ensemble of real earthquake records was chosen which gave a reasonable match to a chosen design response spectrum.

It was shown that when torsional shears are included in deriving the distribution of strength of the elements, the structure will in general have a small resistance eccentricity, even if it has a large stiffness eccentricity in the elastic range. For structures which are designed with allowance for torsional shears, the ductility demands on the elements are similar to those where the structure is symmetrical. However, the edge displacements can be up to three times those of the equivalent symmetrical system. It was pointed out that this finding has significant implications in providing adequate separations between buildings to avoid the problem of pounding during earthquakes.

Goel and Chopra (1990, [17]) evaluated the plan-wise distribution of stiffness and strengt, as determined by the number, location, orientation and yield deformations of resiting elements, on the inelastic response of one-storey systems. Various sytems were investigated for a wide range of parameters, with the objective of establishing how the response was influenced by: (i) the presence of resiting elements perendicular to the direction of ground motion; (ii) the number of resisting elements along the direction of ground motion; (iii) the overstrength typical of code designed buildings; (iv) the relative values of strength and stiffness eccentricities; and (v) whether asymmetry of the system was due to eccentricity of stiffness or mass. The results presented made it possible to explain the inconsistencies in conclusions from various earlier investigations, and to evaluate their applicability to actual buildings.

It was concluded that the presence of perpendicular elements, which are present in real buildings, significantly reduce the effects of torsional coupling for short period, acceleration sensitive systems. The number of resisting elements oriented along the direction of ground motion was found to have little influence on the response. Although the elastic response of mass eccentric and stiffness eccentric systems is identical provided that the elastic parameters are the same for both systems, it was found that the inelastic response of the two may be different even if the inelastic response parameters were the same. It was also recommended that since the asymmetry in most buildings arises from the distribution of stiffness rather than the distribution of mass, then the mass eccentric system should not be used in estimating the the inelastic response of such buildings. It was also concluded that the recent amendment to the Mexico Building Code, requiring a minimum value of strength eccentricity in order to reduce the ductility demands on asymmetric buildings may not be necessary, as this was based on studies of mass eccentric systems. Code designed asymmetric plan buildings have larger lateral strength than corresponding symmetric plan structures and this overstrength in short period acceleration sensitive systems was found to significantly reduce the lateral deformation at the centre of stiffness and the maximum ductility demands. Strength symmetric systems and systems with strength eccentricity much smaller than stiffness eccentricity, which are typical of most code designed buildings, generally experience smaller effects of torsional coupling than do systems with equal strength and stiffness eccentricities.

Chandler and Duan (1991, [20]) addressed some contradictory conclusions between an earlier paper of their's and the findings of Tso and Ying (1990, [18]) regarding the additional ductility demand in asymmetric building structures and the adequacy of certain design code torsional provisions. They clarify a number of issues which had arisen between the two different research groups. They also present further results based on the torsion design requirements of the Mexico 1976 and 1987 codes.

They disputed the validity of including the code specifed accidental eccentricity in determining the lateral strength of resisting elements to be used in the dynamic analysis models. It was argued that the accidental eccentricity term in the code design eccentricity equations is intended to allow for uncertainties in the position of the centre of mass or stiffness and possible torsional ground motion in real structures, but that that no such uncertainties exist in the models and torsional ground motions are not considered. Based on a 3 element inelastic dynamic analysis model with yield strength distributed according to the code requirements but ignoring the accidental eccentricity term, they conducted further dynamic analyses. They demonstrated that the structural element at the stiff edge is more critical in terms of increased ductility demand than the element on the flexible edge as was claimed by Tso et. al.

The authors also noted that results obtained from such dynamic analyses are somewhat dependent on the frequency content of the input ground motion used. Their results showed that the amount of increased ductility demand was related to both the period of the structure and the frequency content of the earthquake record. They pointed out that Tso and Ying's predictions for additional ductility imposed on the stiff side element did not show the overall trend as only a particular structural period was used in conjunction with the chosen ensemble of earthquake records which all had similar frequency contents.

It was contended by Chandler and Duan, that the provisions of the 1976 Mexico code and the similar requirements in some other countries are inadequate and substantially underestimate the strength demand on the stiff side element. By contrast, the Mexico 87 code torsional provisions were said to be over-conservative in that the strength of the element on the stiff side of the structure is required to be increased more than is necessary. They recommend that the additional requirements of the Mexico 87 code be deleted and that the code eccentricity equations be changed to be the same as is currently used in the Canadian code NBCC 1990, namely:

$$e_d = 1.5\, e_s + 0.1b$$

$$e_d = 0.5\, e_s - 0.1b$$

They pointed out that application of these formulae automatically ensures that the resistance and stiffness eccentricities have the same sign.

Goel and Chopra (1991, [21]) published a further paper looking at the effects of system parameters and yielding on the inelastic seismic response of one-storey, asymmetric-plan systems. Some useful definitions are given in this paper. The "centre of rigidity" of a linearly elastic one storey system is defined as the point on the deck through which application of a static horizontal force in any direction causes no rotation of the deck. For such a system the centre of rigidity coincides with the "centre of stiffness", which is the point in the plane of the deck about which the first moment the resisting element stiffnesses becomes zero. The distance between the centre of mass and the centre of stiffness is defined as the "stiffness eccentricity" of the system (sometimes known as the "static eccentricity"). The "plastic centre" or "centre of strength" is defined as the location of the resultant of yield forces of the resisting elements.

In the results of their study, Goel and Chopra found that yielding lead to reduced torsional deformation of medium period, velocity-sensitive and long period, displacement-sensitive systems, regardless of their stiffness eccentricity. For short period, acceleration-sensitive systems a range of behaviour was found depending on parameter values. For the case where the yield strength of the system was about half of that required to remain elastic, yielding caused a slight increase in deformations. For the case where the yield strength of the system was only about one quarter of that required to remain elastic, the torsional deformation of short period inelastic systems with equal but large stiffness and strength eccentricities was larger compared with elastic systems; for systems with small stiffness eccentricity, yielding resulted in reduced torsional deformation; torsional deformation of strength symmetric systems became smaller as a result of yielding, regardless stiffness eccentricity.

It is important to note from these results that systems which are strength symmetric tend to reduce the torsional deformations when inelastic action occurs. As the paper points out, the strength symmetric case is representative of code designed buildings, which typically possess strength eccentricities much smaller than stiffness eccentricities. Although this result is strictly on applicable to single storey structures, this is a reassuring finding from a designer's point of view, and it is to be hoped that further research will show that this is also true for multistorey structural systems.

3.4 Elastic Response of Multistorey Structures

Tso and Meng (1982, [22]) conducted a study on the accuracy of the Canadian Building Code 1977 edition static torsional provisions. A uniform frame type monosymmetric 12 storey building was used as an example. The design static storey torques were compared with the maximum dynamic storey torques computed using the response spectrum technique as outlined in Commentary K of the Canadian Building Code. It was found that for a building with uniform eccentricity over the height, the code static torque estimate was in good agreement with the maximum dynamic torque if the effect of sympathetic coupled torsional lateral resonance was small. The effect of sympathetic coupled resonance was found to be significant only if (a) the buildings have small eccentricity and (b) the uncoupled torsional and lateral periods are close to each other. It was claimed that coupled resonance could be neglected if the uncoupled torsional period was not within 25 percent of the fundamental lateral period.

It was recommended that, if the effect of sympathetic coupled resonance was expected to be significant then three approaches could be taken:

(a) The static torque as computed by the code could be doubled,

(b) An additional factor could be applied the design static eccentricity to estimate the dynamic eccentricity. Expressions by Muller and Keintzel (1978, [23]) or Tso and Dempsey (1980, [13]) were found to be viable. The design eccentricity should then be taken as the sum of the estimated dynamic eccentricity and the code specified accidental eccentricity.

(c) A dynamic analysis approach could be used based on the response spectrum technique. It was recommended that a modal combination method be used which takes into account cross-modal modal effects.

For buildings with large static eccentricities, the static torques as computed using the NBCC 1977 code formulae were found to adequately estimate the maximum dynamic torques.

Doubling the computed static torques, as specified in the NBCC code at that time was found to be a very conservative requirement.

The effect of the revised expression in the 1980 NBCC code for calculating the structural (static) eccentricity was also considered. The revised formula calculated the eccentricity e at any level x as follows:

$e = \Sigma F_i \, e_{ix} / \Sigma F_i$ (summations on i from x to N in num. and denom.)

where F_i is the static force to be applied at level i and e_{ix} is the distance between centre of mass at level i and the centre of stiffness at level x.

This expression was intended to account for the effects of eccentric offsets in buildings. The authors of the paper warned that no generally accepted method existed at that time for determining the locations of the centres of stiffness, but that common practice was to calculate locations on a floor by floor basis. That method was used for three example buildings with setbacks and it was found that the static floor torques conservatively estimated the dynamic values for the main part of the building, but that dynamic torques in the upper offset portions were grossly underestimated.

Finally, Tso and Meng concluded that the most reliable means of estimating the distributions of torque for irregular buildings was by means of a dynamic analysis.

Tso (1983, [24]) summarised findings of research he had completed with co-workers over the previous several years in a paper which made proposals to improve the static torsional provisions of the National Building Code of Canada 1980 edition. Four changes to the code static provisions were proposed as follows:

(1) That the design eccentricity expressions should be changed to:

$e_x = 1.5e + 0.1D$

and $e_x = 0.5e - 0.1D$

where e = structural eccentricity at each floor (that is, the distance between the centre of mass and the centre of stiffness).

It was noted that these expressions were similar to the then Mexican seismic code for buildings.

(2) That the NBCC 1980 code requirement to double the torsional effect, if the design eccentricity exceeded 25 percent of the building dimension D, should be deleted.

(3) That the NBCC 1980 code expression used to define the structural eccentricity e should be deleted.

(4) That a statement should be added to the NBCC 1980 code to clarify the limitations of the static torsional provisions. It was recommended that such a statement would point out that the provisions were only applicable to buildings in which the locus of mass centres and locus of stiffness centres at each level each lay on approximately

vertical lines. Further, for buildings that did not satisfy these conditions then a dynamic modal analysis should be carried out to determine the torsional effects.

Cheung and Tso (1986, [25]) studied the effects of eccentricity in irregular elastic multistorey building structures. They discussed how the concepts of eccentricity developed in single storey structures could be extended to multistorey structures by defining the centres of rigidity at each floor, for which they noted no universally accepted definition existed at that time. The authors defined these as the set of points located at floors levels such that when the given distribution of lateral loadng is applied through them, no rotational movement of any floor the building would occur about a vertical axis. It was noted that, even for buildings with identical floors, the centres of rigidity may not be determined based on examination of a typical floor. For example, in an asymmetric wall-frame structure the internal forces can vary substantially over the height due to strong interaction effects between the walls and frames. Also the positions are dependent on the distribution of applied lateral load.

A practical procedure for locating the centres of rigidity was presented based on the use of a plane frame computer analysis program. Since it was assumed that no rotation could occur at any floor when the loading was applied through the centres of rigidity, only the frames in the direction of applied earthquake loading needed to be considered. For a building with orthogonally oriented framing, all the frames aligned along the direction of applied loading could be modelled simultaneously, connected together by rigid pin-ended linking members at each floor. From the analysis results, the interstorey shear in each frame could be found, and then, considering the actual plan locations of frames, the centroids of the frame forces (the "load centres") give the locations of the centres of rigidity. That is, the first moment of forces in all vertical elements at a given floor about the centre of rigidity is zero.

(An alternative way of determining the centres of rigidity, not discussed by the authors, would be to use a three-dimensional analysis with floors locked against rotation. This would give identical results.)

The current Canadian code suggests that the torsional provisions are strictly applicable only to buildings in which the centres of mass and rigidity are located on two vertical lines. For eccentric buildings that do not satisfy this requirement the code torsional provisions may or may not be valid to determine the torsional effects. In using the Canadian code, the designer would therefore be obliged to determine the positions of the centres of rigidity. Having determined these it could then be assessed how well the code requirement was satisfied and the design eccentricities, and hence the design floor torques for which the building is to be designed, could be determined.

It was noted that when the centres of rigidity are defined in this way, they are in general load dependent and not aligned vertically. There are however some special circumstances for which the centres of rigidity are independent of the distribution of applied loads and where they do lie on a vertical line. In particular, where the framing is "proportional", that is where the stiffness matrices of separate frames across the building are scalar multiples of each other, then both of these hold. In that case the location of the centres of rigidity could be determined based on examination of a single typical floor. It was shown by example that even a slight deviation from proportional framing could give a wide scatter of centres of rigidity.

Examples for three different eccentric buildings were presented to show the locations of centres of rigidity. For each building the Canadian code static procedure for determining the required strength distributions were compared with the distributions determined using dynamic response spectrum analyses. It was shown that for buildings with centres of rigidity located along a vertical axis, the Canadian code procedure provided a good estimate of the torsional shear distributions. For buildings with centres of rigidity scattered about a vertical axis, it was found that the code procedure may or may not give good estimates of the distribution compared with the response spectrum analysis, depending on the degree of irregularity in the framing. It was concluded that if the centres of rigidity did lie on a vertical axis the NBCC 85 torsional provisions were applicable, but that they may also be applicable in other cases. For structures with significant eccentricity, it was concluded that the evaluation of torsional effects is very complicated and that the determination of strength demands on elements allowing for torsion is best done by means of dynamic analysis.

Maheri et. al. (1991, [19]) reported a very interesting study in which a series of parametrically defined experimental models were tested on a shaking table and for the responses were also predicted theoretically using response spectrum techniques. The 4 storey high models were designed with variable ratios of torsional to lateral stiffness and with either symmetric or asymmentric mass distributions (mass eccentric structures). A number of different model configurations were chosen to give a range of dynamic parameters.

Comparison of free vibration studies showed very good agreement between measured and theoretically predicted frequencies and mode shapes of vibration.

The models were subjected to shaking table excitation using scaled real earthquake records as the input motion. Response of the models was limited to the elastic range. The responses were also theoretically predicted using the Response Spectrum Method of analysis. It was found that the response both theoretically and experimentally was dominated by the first two modes of vibration, one being predominantly a translational mode and the other being predominantly a torsional mode, depending on the degree of eccentricity. However, the significance of translational and torsional components of motion was found to be different between theory and experiment. The theoretical results for asymmetric structures showed that the response was dominated by the first coupled mode, with a less significant contributions from the second mode. By contrast, the experimental results showed that the second mode was much more significant than the theory predicted. It was found that the peak displacement response on the side nearest to the centre of mass was quite well predicted, but the peak response displacements of the edge furthest away from the centre of mass were not very well predicted and in some cases were substantially less than found experimentally. It was noted that elements on the side furthest away from the centre of mass are generally assumed to be affected beneficially by the torsional coupling in comparison with the equivalent symmetric case. Therefore it was concluded that current code design procedures may be quite unconservative at predicting the force demands on these elements.

The maximum displacement at the so-called critical edge, closest to the centre of mass were found to be in some cases up to 2.6 times the displacement in the corresponding symmetric model (This author notes that this is generally in agreement with predictions of other theoretical studies).

It was concluded that the theory underestimated the significance of the torsional component of response but overestimated the significance of the translational component. These effects compensated for each other to a large extent on the side of the structure which is most severely affected by torsional response, but produce large and unconservative inaccuracies on the side of the building which is commonly assumed to be affected beneficially by torsional coupling.

An extension of this study to investigate inelastic torsional coupling effects by allowing certain key structural members to form plastic hinges, is reported to be currently underway.

This author believes that the paper by Maheri et. al. is significant research towards a better understanding of the coupled lateral-torsional behaviour of multistorey building structures. However, there are some aspects which may cast doubt over the general validity of the conclusions. First, the levels of equivalent viscous damping measured in the experimental models were only about 0.5 percent, which is substantially lower than is expected in full-scale buildings responding to ground motions with significant amplitude. Second, the models were all of the mass-eccentric type. Previous researchers have identified differences between behaviour of mass and stiffness eccentric structures, even where they otherwise have the same dynamic response properties. It has been stated by some that most real buildings are more like to be stiffness eccentric rather than mass eccentric, although inevitably real buildings could be either stifness or mass eccentric. Third, the discrepancies identified between theory and experiment may actually highlight possible inadequacies of the Response Spectrum Analysis method and modal combination methods for predicting the maximum experimental results. These discrepancies may also be somewhat exaggerated because of the very low levels of damping found in the experimental structure and thereby used in the theoretical model. It is noted by this author that, instead of using the Response Spectrum Method of analysis, it would have been possible to employ elastic time history analyses to predict the experimental dynamic response.

Maison et. al. (1983, [26]) carried out a study of the comparative performance of seismic response spectrum combination rules in building analysis. Four response spectrum combination methods were discussed: the Square Root of Sum of Squares (SRSS) method, the Double Sum Combination (DSC) method, the Complete Quadratic Combination (CQC) method and the Absolute Sum method (ABS). These were used to compare the response spectrum analysis predictions with elastic time history analysis results for two 15 storey steel moment resisting frame buildings, one symmetric and the other asymmetric.

The DSC method is similar to the CQC method but uses different cross-modal correlation coefficients. The DSC method includes a parameter related to the duration of strong shaking. It is possible to select a value for this parameter if the earthquake response spectrum is based on an actual ground motion (real or artificial). However, for design spectra this parameter is meaningless.

It was concluded that both the DSC and CQC modal combination methods provide good estimates of peak response for both regular and irregular building models, irrespective of modal coupling. The SRSS method was found to give accurate response predictions for for the regular building but was recommended as suitable only for structures where coupled modes with closely spaced periods do not dominate the response. However, for irregular buildings with coupled modes having closely spaced periods, the SRSS method was found to underestimate the response in the direction parallel to the earthquake ground motion by not adequately allowing for cross-modal reinforcement, and to overestimate response in the direction perpendicular to the earthquake ground motion by neglecting cross-modal cancellation. Therefore, it was recommended that the SRSS rule should not be used in those situations.

For the examples studied, the DSC method was able to give slightly better response predictions for the orthogonal response than the CQC method, but only by a somewhat arbitrary selection of the paremter related to duration of strong ground motion. It was stated that for design applications which allow for idependent designs in both principal directions, the orthogonal response component does not normally govern the design. Therfore, it was concluded that the DSC and CQC methods may be considered to yield results of equivalent accuracy.

3.5 Inelastic Response of Multistorey Structures

There is very little published information related to the torsional response of inelastic multistorey building structures. Although the inelastic behaviour of simple single storey structural systems has been quite extensively investigated in the past few years, the response of multistorey structures is an order of magnitude more complex. No researchers seem to have yet attempted to undertake systematic parametric type studies of the inelastic response of three-dimensional multistorey structures. The only analysis method suitable for inelastic dynamic response analysis is the step-by step time history analysis which is normally a computationally large and expensive procedure. Such analyses are now used routinely to predict the earthquake response of large or unusual structures such as offshore platforms, but only a limited number of such analyses are reported for typical building systems.

Gillies (1979, [27]) developed a three-dimensional inelastic time history computer program for the analysis of frame structures. The method of analysis was based on the principle of modal superposition, extended into the post-elastic domain. To demonstrate the implementation of the program, the response of two three-dimensional structures, a two span bridge and a six storey reinforced concrete moment resisting frame, was investigated.

The six storey building was analysed for both uni-directional and concurrent orthogonal earthquake loading. The uni-directional response displayed bands of plastic hinging migrating up the structure, and with the effective modal period becoming elongated from the initial elastic period. This verified the behaviour predicted from two-dimensional inelastic analyses.

A study was made of the effect of a 10 percent static eccentricity of the centre of mass from the centre of stiffness at each level, compared with the response of a similar symmetric building. During the initial part of the response the response of the eccentric model was similar to the response of the symmetric model. However, after the onset of inelastic actions, the responses of the two structures began to differ markedly. The complex variation of the instantaneous centre of stiffness in the eccentric model made it impossible to predict the response based on the response of the symmetric model.

The response characteristics of the symmetric model were analysed for ground motions such that significant inelastic action resulted. Concurrent orthogonal components of ground motion were input to the analysis and it was found that the resulting asymmetric distribution of yielding in the structure caused a torsional response to develop. It was found that the degree to which torsional response was excited was related to the duration of yielding which took place in the structure. As the earthquake motion approached its peak intensity the torsional deformation was observed to become increasingly dominant.

The pattern of plastic hinging observed in the three dimensional model was not the same as the migrating bands of damage observed in the planar model analyses. This was attributed to the markedly different element yield response time-history which occurred when a biaxial yield surface was used to account for concurrent loading effects.

The results of the analyses gave a useful qualitative insight into the inelastic response of three-dimensional building structures.

4. SPECTRAL MODAL ANALYSIS METHODS

This section is based mainly on selected extracts from a paper by Maison et al [26].

4.1 Spectral Modal Analysis Methods

It is not the intention of this report to give extensive details of spectral modal analysis methods. In essence the method is a combination of response spectrum techniques with the modal analysis method. For details of the spectral modal analysis method the reader is referred to well known texts such as Clough and Penzien [28].

The response spectrum method is a widely used procedure for performing elastic dynamic seismic analysis. The response spectrum, by definition, represents the set of the maximum acceleration, velocity or displacement responses of a family of single degree of freedom (SDOF) damped oscillators, resulting from excitation by a specific earthquake ground motion. The application of response spectrum analysis procedures to structures which cannot be adequately described as SDOF systems requires modal analysis techniques to transform the coupled multi-degree of freedom equations of motion to a set of uncoupled equations in the normal co-ordinates. This transformation allows the response of each mode to be evaluated as a SDOF system. The response spectrum can be used to predict the individual modal response maxima, but lacks modal time phasing information. Therefore the relative times at which each peak modal response occurs are unknown. To estimate the total peak response, techniques which combine the individual maximum modal responses are required. Numerous response spectrum modal combination rules have been proposed with the intent of minimising the total peak response prediction errors when compared to the elastic time history analysis values. The most common rule is the square root of the sum of squares (SRSS) method, which has been widely recommended for use in nuclear power, offshore oil and building industries in the past. However, it is generally recognised that the SRSS method can be a poor estimator of peak responses when applied to systems with closely spaced natural periods. For these cases, various other rules have been suggested. No single method has gained universal acceptance although a candidate may be the complete quadratic combination (CQC) method. This method accounts for the influence of modes with closely spaced periods using the principles of random vibration theory, and is relatively easy to use.

4.2 Modal Combination Rules

In the following section details of some of the best known and widely used modal combination schemes are given.

Square Root of Sum of Squares (SRSS) Method [29]

Double Sum Combination (DSC) Method [30]

It should be noted that there is also a (different) Double Sum method proposed by the Nuclear Regulatory Commission.

Complete Quadratic Combination (CQC) Method [31,32]

Absolute Sum of Modal Maxima (ABS) Method

The accuracy of each of the above modal combination rules in predicting the peak elastic time history response depends upon the characteristics of the earthquake record and the structure's dynamic properties. The SRSS, DSC and CQC methods are based upon the theory of random vibrations and are therefore statistical or probabilistic estimates of peak response quantities. Two of the major assumptions used in the development of these rules were: (1) the excitation is a sample of a wide frequency band (covering the structure's natural frequencies) stationary Gaussian random process; and (2) the vibration responses of the structure's normal modes are also stationary. In general, these assumptions are reasonably accurate if the earthquake has a time segment with extreme irregular accelerations of roughly equal intensity which is several times longer than the fundamental period of the structure. The simple form of the SRSS rule as compared with the DSC and CQC rules is a consequence of the additional assumption that the modal vibrations are statisitically independent; that is, the vibration of any mode is not correlated to any other mode.

In systems with closely spaced periods, the SRSS rule may be a poor estimator of the actual maximum response. By introduction of a modal cross-correlation coefficient matrix P, the DSC and CQC rules account for the mutual reinforcement and/or cancellation of modes with closely spaced periods. In particular, the important quality of retaining the signs when combining the cross-modal components (allowing cancellation) can be most significant. Elements of the modal cross-correlation coefficient matrix can assume values ranging from zero to unity (where zero represents no modal cross-correlation) depending primarily upon the relative proximity of the natural periods (refer Fig. 1). If the periods are well separated, the off-diagonal cross-modal terms of the matrix P become small and the DSC and CQC methods approach equivalence with the SRSS rule.

Both the DSC and CQC modal cross-correlation coefficient matrices are functions of the modal frequencies and damping ratios. In addition, the DSC formulation includes a paramater for the duration of strong motion. To contrast the two methods, the effects of these parameters on the modal cross-correlation coefficient relating two modes are shown on Fig. 1. For both the DSC and the CQC methods, modal cross-correlation coefficients increase as adjacent modal periods approach the same value, and as the modal damping increases. In addition, for the DSC method, as the ratio of the natural period to the earthquake strong motion duration parameter increases, then the modal cross-correlation coefficients increase. Therefore, for a given period ratio Ti/Tj, modes with the longest periods will have the largest cross-modal effects. When the DSC duration of strong motion parameter is set to infinity, the DSC and CQC methods become virtually identical.

As a guide to the approximate natural vibration period range in which random vibration theory based rules (ie SRSS, DSC and CQC) are most appropriate, it has been suggested that structures having their most significant natural periods in the range bounded by the intersections of the peak acceleration, velocity and displacement lines on a tripartite logarithmic repsonse spectrum earthquake plot are best suited for these types of combination rules. For earthquake records associated with firm ground sites and moderate distances from the earthquake hypocentre (El Centro 1940 record type), the corresponding period range is from about 0.5 s to 4 s. An example where a combination rule not based on random vibration theory would be more appropriate is in the analysis of very short period (very stiff) structures where the spectral accelerations approach the peak ground acceleration. For this case, an algebraic sum of the modal responses will yield the best accuracy in a response spectrum analysis. This approach is equivalent to a static analysis using the peak ground acceleration times the structure's mass to develop external forces. In the analysis of high-rise buildings, the modes contributing significantly to the response generally have periods grater than 0.5 s; therefore, the algebraic combination rule is not normally appropriate. However, it should be noted that situations can arise where other special rules are more appropriate.

The ABS rule is an upper bound estimate of the response. It assumes that all modes reach their maxima with the same sign at the same instant in time. In general, this method results in response estimates that are very conservative and is usually not used for design purposes.

In the application of the four above modal combination rules, several properties reqarding the peak response quantity estimations should be noted. First, the sign of the response quantity is lost; that is, the peak response quantity may be either positive or negative. When combining the results with load cases of known signed responses (eg static gravity load cases) judgement must be exercised to formulate the the appropriate loadings for design purposes. Second, a collection of response quantities produces an estimated maximum response envelope. When considering an envelope of maximum response quantities, it should be recognised that they do not necessarily occur at the same time, consequently if additional response parameters are generated from combinations of these envelope values, inconsistencies are introduced. For example, the use of a storey inertia force envelope to calculate cumulative storey shears results in values larger than the combined modal storey shears. In addition, the use of a storey displacement envelope to calculate storey drifts results in values smaller than the combined modal drifts. Regarding design applications, the former case may be considered conservative whereas the latter case is unconservative. In a similar way, it is not appropriate to calculate total base overturning moment on a structure from the storey inertia force envelope. Thus it is concluded, that to arrive at the best estimates of the peak response values, modal combinations should be performed separately for each of the response quantities that are to be considered.

Singh and Maldonado [32] have very recently (1991) presented a new response spectrum method which is claimed to give better results than those which are currently used. "Missing mass" effects caused by truncation of higher modes are corrected for by the use of a pseudo-static response term. It is stated that traditional response spectrum approaches require the earthquake input to be defined in terms of a relative acceleration response spectrum. However, it is noted that the current practice of seismic structural analysis is oriented toward the use of pseudo-acceleration spectra rather than relative acceleration spectra. It is implied that the use of pseudo-acceleration spectra in conjunction with currently used response spectrum procedures is inappropriate. By means of examples it was demonstrated that the new method could give significantly less error than either the CQC method or Double Sum Combination method of Rosenblueth and Elorduy, particularly for stiff structural systems with a period in the vicinity of 0.05 seconds. However, it is noted by this author that the errors in say the CQC method for flexible structures are only a few percent; much less than for maximum errors claimed for stiff structures. Also, the so-called flexible example structure had a fundamental period of about 0.5 seconds. Therefore it is possible that for typical multistorey building structures, with periods in the order of 1 second or more, the errors in using the current procedures based on say the CQC combination method may be quite small.

5.0 NZ PRACTICE FOR 3D SPECTRAL MODAL ANALYSIS

An informal survey of common practice by NZ designers was recently undertaken as part of the consultative process of preparing a revision to the New Zealand Loadings Code for Buildings NZS 4203:1984. A number of consulting structural engineers were asked to give a summary of the procedures they use in carrying out three-dimensional spectral modal analyses of multistorey building structures.

The majority of designers were using the ETABS software package, although other programs (such as EASE2, a derivative of the SAP family of programs, or in-house programs) were being used by some. With the relatively low cost and high computional power offered by modern computer analysis equipment and software, most designers are using commercially available packages rather than developing their own. However, these analysis packages are not specifically designed to allow designers to work in compliance with the spectral modal analysis and torsional loading provisions of NZS 4203. There are therefore a number of techniques that are being used by New Zealand designers to adapt theses analyses to comply as much as possible with the code.

Most of the comments that follow are related to the use of the ETABS program. It would appear that this software is being used routinely for the design of multistorey buildings in New Zealand and that designers feel quite comfortable with the results they get from the program.

It would seem that some analysts prefer to use the full spectral modal analysis capability of the program, while others prefer to first perform a spectral modal analysis to determine a set of floor inertia forces and then apply these back on the three dimensional model using a static load analysis. However, some designers have reported difficulty reconciling results for unusual structures using their prefered method (both methods seem to have caused concern for different designers) and that they had resorted to the alternative analysis procedure.

Designers generally attempt to comply with all requirements of NZS 4203, but where unusual cases have occured, some requirements have not been complied with, presumably as they have been found too onerous, and non compliance is justified on the basis of "engineering judgement".

The Complete Quadratic Combination (CQC) modal combination method is most commonly used for combining modal actions. It would appear to be generally accepted that the CQC method is the best available and to be used in preference to say the SRSS method.

It appeared to be general practice to scale the results to ensure that the resultant base shears from the analysis were equal to 90 percent of the base shear given by the code static method. Some noted that they determine the required correction factor and then rerun the analysis with this as a scale factor so that all the actions can then be taken from the output without the need for further scaling. The initial analysis to determine the period of the structure would normally be run assuming the centres of mass at the best estimated positions at each level.

The multiple load case capabilities of ETABS can be used to carry out equivalent static and spectral modal lateral load analyses simultaneously so that the 90 percent base shear requirement and the 80 percent interstorey shear requirement at any level could be easily checked. In some instances designers have found that the spectral modal results did not satisfy the requirement that shear at any level should not be less than 80 percent of the value obtained using the equivalent static method. Some choose to disregard this requirement claiming that the spectral modal analysis results must be more appropriate than any static load distribution. This of course misses the exact point that the requirement was set down to guard against. The alternative to this has been to resort to a three-dimensional static load analysis using the storey inertia forces (including moment about the vertical axis) as applied loads. This is easily done as ETABS gives as part of the results a set of maximum floor forces at each storey and these can be re-input and a three-dimensional static load analysis carried out.

Separate dynamic load cases are normally used to determine the seismic effects of earthquake excitation in two orthogonal directions. Up to three independent dynamic analysis load cases are available within the ETABS analysis in each of which an arbitrary direction of input of the horizontal earthquake spectrum can be specified. Using the options for combination of load cases the spectral modal analysis load cases can be combined together or with static load cases.

Allowance for the so-called accidental eccentricity effects is most often made by shifting the centres of mass of each floor by $\pm 0.1b$ to give the most adverse effect on each element. To the author's knowledge this is the only reasonable way that the accidental eccentricities can be allowed for where a full spectral modal analysis is performed. Normally the centres of mass at all floors would be shifted in the same direction in one single analysis, on the assumption that this would give the worst possible effect. It is necessary to carry out separate analyses, relocating the centres of mass either side of the expected positions in each direction. In order to reduce the number of separate analyses required, analysts sometimes shift the centres of mass in both horizontal directions simultaneously, that is, along a diagonal to the assumed orthogonal directions. In this way the positive and negative accidental eccentricity effects can be analysed for earthquake inputs in both orthogonal directions with just two separate analyses. This may not pick up the worst loading on every member. It would be advisable to consider separate analyses in each direction with the centres of mass shifted either way for each. Also, it may be found that the worst loading in some members is imposed by not shifting the centres of mass from the calculated position. Therefore it would seem reasonable to carry out five separate analyses for each structure.

After the structural actions have been scaled so that the resultant spectral modal analysis base shear matches 90 percent of the code static method, they are then taken as the unfactored design earthquake actions. The normal procedure would then be to determine the strength load combination actions by combining the earthquake actions with other load types, dead, live etc. Forces would be redistributed as allowed by the appropriate design codes to determine the final strength design actions. For ductile structures capacity design, procedures would then be followed to ensure that the chosen heirarchy of failure mechanisms could be sustained.

6. SPECTRAL MODAL ANALYSIS AND CAPACITY DESIGN PROCEDURES

6.1 Introduction

Structural design procedures currently used in New Zealand for the ductile design of building structures rely heavily on capacity design methodoligies to ensure that a satisfactory and predictable performance can be expected from structures responding well into the inelastic range during moderate to large earthquake excitation. These procedures are well understood by designers and only a brief summary of the capacity design philosophy is necessary here.

The capacity design procedures are intended to encourage ductile failure mechanisms to develop and to minimise the likelihood of unpredictable modes of failure. Modes of failure that are preferred generally include those associated with ductile flexural plastic hinging, while undesirable modes which are guarded against include those associated with shear failures in members. In structures with moment resisting frames, overall frame failure mechanisms involving beam sidesway mechanisms are preferred because they better distribute ductility demands throughout the structure and to locations which are more easily detailed to provide inelastic deformation capacity. In structures where the lateral load resistance is provided by structural walls, a failure mechanism involving ductile flexural plastic hinging at the the critical base region of the wall is generally designed for.

All locations selected for inelastic deformation are carefully detailed in accordance with the appropriate materials code (for example by the provision of transverse confining reinforcement in reinforced concrete structures) to ensure that they can sustain the inelastic deformation demands expected. Other parts of the structural system are designed to be protected from inelastic actions by having strength in excess of that required to maintain the assumed failure mechanism allowing for the effects of likely member overcapacity at the damage regions. For example, in the design of reinforced concrete frames, beams are designed assuming that plastic hinges will form within each span, generally at the ends adjacent to columns. These end regions are provided with confining reinforcement to ensure that they can provide the required rotational ductility capacity. The shear capacity of beams is made at least equal to the shear demand when plastic hinging occurs. Columns are also deliberately designed to have strength in excess of that required to sustain the required beam sidesway mechanism allowing for overstrength in the beam plastic hinges and for dynamic magnification effects.

The New Zealand Concrete Design Code NZS 3101 (Appendix to Commentary on Section 3) contains a recommended method for the evaluation of column actions in ductile multistorey frames. This is a capacity design procedure, specifically for reinforced concrete frames, but which can also be adapted, for example, to structural steel frames. The method is widely accepted and used in New Zealand.

The code capacity design method for columns in frames was developed by Paulay and others based on the analysis of a limited number of two-dimensional example frame structures of different sizes. The procedures for determining column design actions so that columns are protected against inelastic deformations were substantiated by first designing the frames according to the assumed design procedure and then subjecting the frames to a number of (two-dimensional) inelastic time history dynamic analyses to confirm adequate response behaviour.

The current code procedure for the capacity design of columns is based on using the results of the equivalent static force analysis of the New Zealand Loadings Code NZS 4203 and does not include provision for the use of a spectral modal analysis. The spectral modal analysis actions are an envelope; they do not occur simultaneously and they are not an equilibrium set of actions. A major question is therefore whether the present capacity design procedures can still be used, or how they should be modified, for the case where a spectral modal analysis has been carried out.

While it is beyond the scope of this study to provide detailed modifications to the existing capacity design procedures, suggestions for simple adaptions of the existing methods are made.

6.2 Details of the NZS 3101 Capacity Design Procedure for Frames

The following is a summary of the procedure incorporated in the code. It is based on the assumption of a beam-sidesway mechanism in the frame.

1. The earthquake induced actions in the frame are determined based on the code specified equivalent static force method. This requires an elastic static analysis of the frame.

2. The earthquake induced actions are combined with the actions from gravity loading in accordance with the required code loading combinations for dead, live and earthquake load. Moment redistribution in accordance with the limits in the code may then be carried out.

3. All critical beam sections are then designed for the flexural strength demands determined above and actual numbers and sizes of longitudinal reinforcing bars are allocated.

4. Beam flexural overstrengths at the critical sections where plastic hinging is anticipated (normally at the column faces) are estimated assuming a beam flexural overstrength factor depending on the Grade of reinforcement used. This factor is typically in the range of 1.25 to 1.40 and allows for likely increases in the yield strength above the specified minimum (or characteristic) strength and for the effects of strain-hardening in the reinforcement. The corresponding moments at the column centrelines are then estimated, based on extrapolation from moments at the plastic hinge locations. The overstrength shear induced in each beam consistent with plastic hinging is also calculated and capacity design of the beams for shear can then be completed.

5. A beam overstrength factor at each column centreline and at each level (ie each beam column joint) is then determined. This is taken as the sum of the beam overstrength

moments (at the column centreline) divided by the sum of the beam moments (at the column centreline) due to the code earthquake loading.

6. Dynamic magnification factors are determined for each level. These are tabulated for one-way and two-way frames as functions of the fundamental period of the frame. The factors allow for the effects of higher mode dynamic effects in the frame which result in deviation of the bending moment patterns in the columns from the simple pattern determined by the static analysis. The dynamic magnification factors vary from a minimum of 1.0 to a maximum of 1.9 at various locations and depending on the characteristics of the frame.

7. The axial loads induced at all levels in each column due to plastic hinging in the beams are calculated. This is done by summing beam overstrength shears for all beams framing into the column above the level being considered. Dynamic analysis of multistorey frames has shown that it is unlikely that all beams in a frame simultaneously attain full plastic hinging. Therfore a reduction factor is applied to reduce the calculated axial load depending on the number of floors above the level being considered. Interior columns in frames have only small earthquake induced axial loads because the shears from beams in adjacent bays tend to cancel. By contrast exterior columns have beams on only one side and so they sustain large tensile or compressive axial loads. It is also required that, in calculating the earthquake induced axial forces for columns in two-way frames, overstrength shears from beams in both directions are considered. This means that corner columns must be designed for axial load effects from simultaneous plastic hinging in adjacent beams in both directions; this is a severe requirement and is sometimes difficult to design for.

8. The column design axial loads are determined based on the dead, live and earthquake induced axial forces derived above, using the required loading combinations.

9. The magnified column moments at the beam centreline are determined by multiplying the static analysis bending moments by the beam overstrength factor at each joint and the dynamic magnification for each level.

10. Design shear forces for the column, both above and below each joint, are determined by multiplying the static analysis shear forces by the beam overstrength factor at each joint and the dynamic magnification for each level. For a given column between two levels, two design shear forces would be derived, one from the capacity actions at the joint below and one from the joint above. Obviously, the shear design of the column must be based on the larger of the two forces.

11. The column design moment at the face of the beam is determined by reducing the column moment calculated at the centreline of the beam to the face of the column according to the beam depth and design column shear. To allow for higher mode effects the moment is reduced by only 60 percent of the design shear force times half the depth of the beam.

The next question is how can this procedure be adapted to the common case where spectral modal analysis frame actions are available rather than static load analysis actions. This situation arises quite often as the Loadings Code specifically forbids using equivalent static analysis for structural systems which do not meet the specified regularity requirements. In that case the designer is required to use dynamic analysis procedures to determine the earthquake design actions, but little guidance is available as to how the capacity design actions on columns should be determined.

The following section discusses the way in which the above capacity design procedure can be modified for the case where the seismic analysis is completed using a spectral modal analysis.

6.3 Modified Procedure Based on Spectral Modal Analysis Actions

Steps 1 to 5

These steps in the above capacity design procedure can be followed identically with no difficulties.

Step 6 - Dynamic Magnification Factors

In determining the dynamic magnification factors, there is some uncertainty as to whether the values given by the code method are still appropriate. The factors allow for the expected deviation and incease in column actions above those predicted by simple static analysis of the frame. These increases arise due to dynamic response of the structure and could be attributed to various factors including: the response of higher modes of the structure, changes in response caused by inelastic action in the structure (mainly in the beams) and the effects of ground motions with different characteristics.

The dynamic magnification factors given in the code were derived from a limited number of research studies for frames which were designed assuming the equivalent static loads specified by the NZS 4203 (1976 edition) code and "tested" to confirm satisfactory response behaviour using inelastic time history dynamic analysis.

When a spectral modal analysis is carried out to determine the earthquake actions in the frame, it is expected that higher mode effects are already allowed for in the column actions. It should also be noted that the beam actions also include effects from higher modes, therefore no additional protection to the columns is implied by the distribution of actions. Overall however, it is expected that the spectral modal analysis actions should represent a better distribution of strength demands in the structure during earthquake shaking than the distribution of actions based on the static analysis simple triangular lateral loading pattern. It is also reasonable to expect that spectral modal analyses using code type acceleration response spectra allow for a wide range of earthquake ground motion types by virtue of the derivation procedure and smooth nature of the spectra. The main unknown is that spectral modal analysis results do not allow for the additional effects of inelastic action.

Consideration of the above suggests that there is some doubt about the applicability of the dynamic magnification factors presently given in the code for the case where analysis is carried out using the spectral modal analysis procedure. If anything, the use of the same values is likley to be conservative. It is possible that a detailed review of this show that dynamic magnification factors used for determining column actions could be greatly reduced. Sufficient protection against inelastic action may in fact be available to columns if little more than the beam overstrength actions are designed for without large additional magnification.

It is noted that in the list of limitations noted the original method presented in NZS 3101, it is pointed out that certain numerical values, particulary those of the dynamic magnification factors, are based only on a limited study of the inelastic response of ductile frames, and that these may change as more information comes to hand.

A further point which may be important is that the shape of the acceleration response spectra in the draft loadings code DZ4203 are significantly different from the rather flat spectra used in the current code. The new spectra are significantly more peaked at shorter periods and will result in a greater contribution of response from higher modes than at present. Any capacity design procedures developed specifically for use with spectral modal analysis should be based on the proposed new code design spectra.

Until further research is undertaken to clarify these points, it is recommended that the same dynamic magnification factors should be used as proposed for the original static analysis based method. However, it is noted that the presently used factors may be quite conservative for structures analysed using a spectral modal analysis.

Step 7 and 8

The column axial loads can be determined in exactly the same way as for the method based on the static analysis approach.

Step 9 - Magnified Column Moment at Beam Centreline

In the original method, Step 9 amounts to magnifying the total overstrength moment from beams at each joint, and then distributing this amongst the column members framing into the joint in accordance with the distribution from the original code loading moment pattern. If this method is followed exactly using the spectral modal analysis results, the effect is somewhat different because the bending moments at the joint are not in equilibrium. The sum of the column moments may be greater or less than the sum of the beam moments. The main intent of providing protection to the column members may not then be achieved. Therefore a suggested procedure is to simply take the total beam centreline overstrength moment input to the joint from step 4 and distribute this between the columns. Some sensible means of effecting this distribution is required. One method would be to use the column flexural stiffness (I/L) values with some reasonable allowance for fixity at the far end of each member. Alternatively, an easier and possibly adequate method, is simply to distribute the total moment input in proportion to the column spectral modal analysis moments. A possible additional requirement would be to ensure that the moment obtained is not less than the overstrength factor at that joint (step 5) times the spectral modal moment at the column centreline (derived from step 1).

Step 10 - Design Column Shear Force

The maximum column shear from the spectral modal analysis already allows for elastic dynamic effects, therefore the application of the dynamic magnification factors given by the original method may be overly conservative (see above discussion of Step 6). However, until proved otherwise, it is suggested that the same method is used.

Step 11 - Design Column Moment at Beam Face

This step, reducing the magnified column moment at the beam centreline to the beam face can be done in the same way as in the original method. Again, it is assumed that the shear force acting simultaneously with the moment is only 60 percent of the maximum value and therefore the moment reduces accordingly over the half-depth of the beam.

6.4 Final Comments

The above modifications to the well known procedure are suggestions only and are not based on any analytically based research. What really needs to be done is to "test" and refine this procedure in a similar way to which the present code method was developed more than a decade ago. This is beyond the scope of the present project and should be undertaken as a separate research topic. Any such research work to refine the suggested procedure using spectral modal analysis results should be based on the acceleration response spectra in the new (currently draft) loadings code, as the shape of the response spectra are quite different from the present code spectra and are expected to give significantly greater structural response contributions from higher modes.

The same limitations which applied to the original method also apply to this modified method. The reader is referred to the NZS 3101 code document.

Two important limitations of the procedure are: i) that a point of contraflexure exists in each column within every storey and ii) a requirement for the relative stiffness of adjoining beams and columns in any storey bent to meet a certain criterion. The requirements ensure that the frame is regular and can be expected to behave in a normal frame-like manner. If the frame satisfies these limitations, it is possible to reasonably assume that reversed curvature occurs in both beams and columns. Therefore, even though no signs are available for the actions output from a spectral modal analysis, consistent signs can be determined for the actions in every member of the frame for earthquake loading from any particular direction. However, if the frame did not meet the limitations (possibly because of significant irregularity) then determining a consistent set of signs for the various member actions would be much more difficult.

Although results from any spectral modal analysis are an envelope of actions only, and can not be assumed to occur simultaneously, the capacity design procedure is still able to determine a reasonable and consistent set of frame actions for design purposes. The spectral modal analysis results are used mainly only for determining the distribution of beam flexural demands. Other actions are then calculated as functions of the actual beam strengths provided.

Many of the spectral modal analysis actions (for example, the column axial forces or the beam shear forces) are not actually used by the proposed capacity design method.

7. PROPOSED FURTHER INVESTIGATIONS

Work completed so far in this research project includes an extensive literature review of the research on torsion in building structures and also a review of various international building design codes.

During the remainder of this Research Project it is also intended to undertake further investigations to demonstrate points and/or to justify recommendations for design practices.

It is noted that the New Zealand National Society for Earthquake Engineering presently has a study group investigating dynamic anlysis and spectral modal analysis methods. Many of the issues raised in this report are likely to be dealt with by the NZNSEE study group and the present research will not attempt to deal with them fully.

Some areas which are being considered for attention in the remainder of this research project are as follows.

i Recommendations for Designers Using Spectral Modal Analysis Packages.

One aim of this project is to assemble a brief guide to the practicising New Zealand structural engineer using commonly available computer analysis packages such as ETABS with recommended procedures for designing structures for seismic torsional effects.

ii 3D Inelastic Response of Simple Framed Structures

It would be possible to undertake a limited number of 3D inelastic response analyses of simple 3D frame structures to "test" frames designed using current code provisions. An extensive study is beyond the scope or resources of this project. BCHF has computer facilities and analysis packages which could be used for this.

iii Concurrency of Member Actions

The present literature gives several examples of comparisons between elastic time history analyses and spectral modal analyses using various modal combination methods including the CQC method. They generally conclude that spectral modal analyses using CQC give good predictions of the peak individual actions. However, the actions calculated by these procedures are not simultaneous. For example, in a column member the interaction of axial load and bending moment is important. SMA may give good estimates of both the (elastic response) maximum axial load level and maximum moment demand. Generally the designer has no choice but to assume that these occur simultaneously, each having either positive or negative signs. Clearly, this may be quite conservative, as in general the maxima of each action will not occur concurrently. While the concurrency assumptions may be conservative for strength load combinations, this may not be such a problem for the case where overstrength actions are considered as part of a capacity design procedure.

iv Absence of Signs in Spectral Modal Analysis Results

No signs are assigned to actions which are derived by Spectral Modal Analysis procedures. However, it is noted that some modal combination procedures do allow for the effects of cross-modal cancellation and hence the resultant response quantity may actually be less than the response quantity from any single mode. The maximum resultant actions can be positive or negative. This makes it difficult for a designer to determine whether, for example, it should be assumed that a point of contraflexure occurs along a member. This apparent difficulty will be examined and commented on. It is believed that this may not be too important when capacity design procedures are employed.

Items iii and iv are also likely to be addressed by the NZNSEE study group.

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