

COMPARATIVE ANALYSES OF A MULTI—STOREY FRAME AND WALL USING THE CURRENT AND DRAFT LOADINGS CODES

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SYNOPSIS

The two draft loadings codes, DZ 4203-86 and 89, introduced major changes in both the response spectra used for seismic design and the methods of analysis. In this paper a wall and a frame, which have been proportioned to provide the seismic lateral load resistance for a 24 storey structure, are analysed by the methods given in these codes to highlight the differences. To simplify the comparison torsional effects have been omitted. The technique of finding a set of lateral loads, by taking the differences of the combined modal storey shears and using these to find the design structural actions, is shown to be inappropriate for structures analysed for the response spectra given in the draft loadings code (DZ 4203, 86 or 89), if the second or higher modes make a significant contribution to the combined modal storey shears.

INTRODUCTION

There are major differences in the response spectra given for seismic design in the current loadings code (NZS 4203-84) [1] and the two draft codes of practice (DZ 4203-86) and DZ 4203-89) [2,3]. The response spectra in the draft codes were derived from an elastic response spectrum, which was developed by the SANZ Seismic Risks Sub-committee [4]. The changes are illustrated in Fig. 1, where the lateral force coefficients from the current and draft loadings codes for the Wellington region are compared. It can be seen that with the draft codes for structures with a period in excess of 1.5 seconds these coefficients are reduced, while for structures with periods of less than 0.6 seconds the values are significantly increased. In the second draft a cut off was proposed, so that buildings with a first mode period of less than 0.4 seconds could be proportioned for the lateral force coefficient corresponding to the value at 0.4 seconds.

For multi-storey structures the proposed changes in the seismic response spectra have a number of effects. The first group of these arise from the marked decrease in lateral force coefficient for the longer period structures. This leads to the following changes:

(a) For strength considerations wind loading becomes more critical in relation to earthquake actions than was previously the case.

(b) Due to the reduced lateral load resistance required for earthquake actions less strength is available in some structures for resisting secondary actions and consequently considerably more attention must be given to P-delta effects.

(c) The reduction in lateral strength required for earthquake actions means that more flexible buildings may meet the code requirements for seismic inter-storey deflections than was previously the case. As a result attention may need to be given to the dynamic performance of buildings under wind excitation to ensure a satisfactory performance occurs in serviceability conditions.

The second major cause of change arises from the different shape of the response spectra. With the draft code the ratio of the lateral force coefficients in the short period range to those in the long period range is much greater than the corresponding ratio in the current loadings code. As a result when the contributions of different modes are combined the higher mode effects become much more significant with the draft code than was previously the case. This can lead to distributions of combined modal actions which can in some cases be very different from those found in comparable analyses using the current loadings code. With this change some approximations, which have been used for a number of years by many designers, need to be re-assessed before they are used with the draft code spectra.

There are a number of distinct differences in the methods of analysis for earthquake actions in the current loadings code (NZS 4203-84) and the two draft loadings codes

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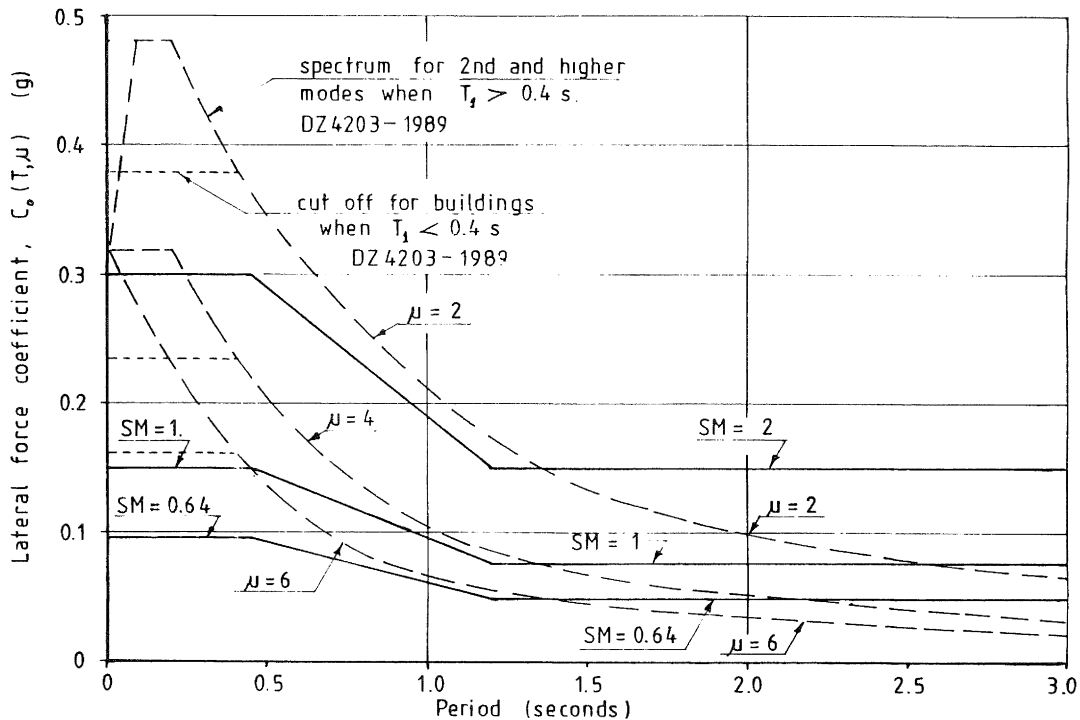


FIGURE 1 RESPONSE SPECTRA FOR SEISMIC DESIGN AS GIVEN IN NZS 4203 - 84 AND DZ 4203 - 89 FOR THE WELLINGTON REGION

(DZ 4203-86 and 89). In the current loadings code the design structural actions may be found from either an equivalent static or a modal response spectrum analysis. However, limitations are placed on the modal analysis values to ensure that seismic actions are not very different from the corresponding equivalent static values. This is achieved by not allowing the base shear to be less than 90 percent, or the storey shears to be less than 80 percent, of the corresponding values in an equivalent static analysis.

The first draft of the loadings code, DZ 4203-86 [2] restricted the use of the equivalent static method to regular buildings which had a fundamental period of a second or less. For other structures a modal or a time history analysis was required. However, the 90 and 80 percent limits to the equivalent static base and storey shears, which were required in NZS 4203-84, were dropped from the draft code (DZ 4203 - 86). The modal analysis was carried out in two steps. In the first step the modes were combined to find the storey shears. In the second step a set of lateral forces were calculated from the differences in the storey shears and these were applied to the structure to determine the design actions.

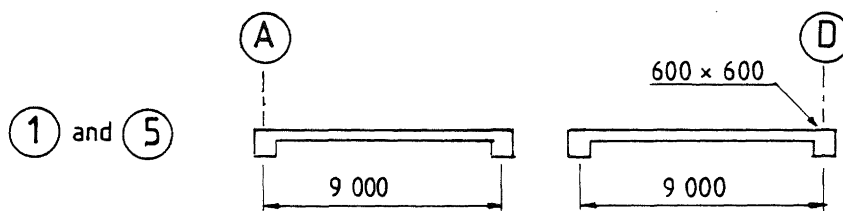
As a result of carrying out a series of comparative analyses using the equivalent static and modal response spectrum methods, on both regular and irregular wall and frame structures [5], it was considered that the one second period restriction on

the use of the equivalent static method could be liberalised. Consequently in the second draft of the loadings code the suggested period limits for frame and wall structures were raised to 2.5 and 1.5 seconds respectively. In the modal method of analysis the second step required in the earlier draft code was abandoned as a theoretical basis for this approach could not be found and the technique yielded misleading results in some cases.

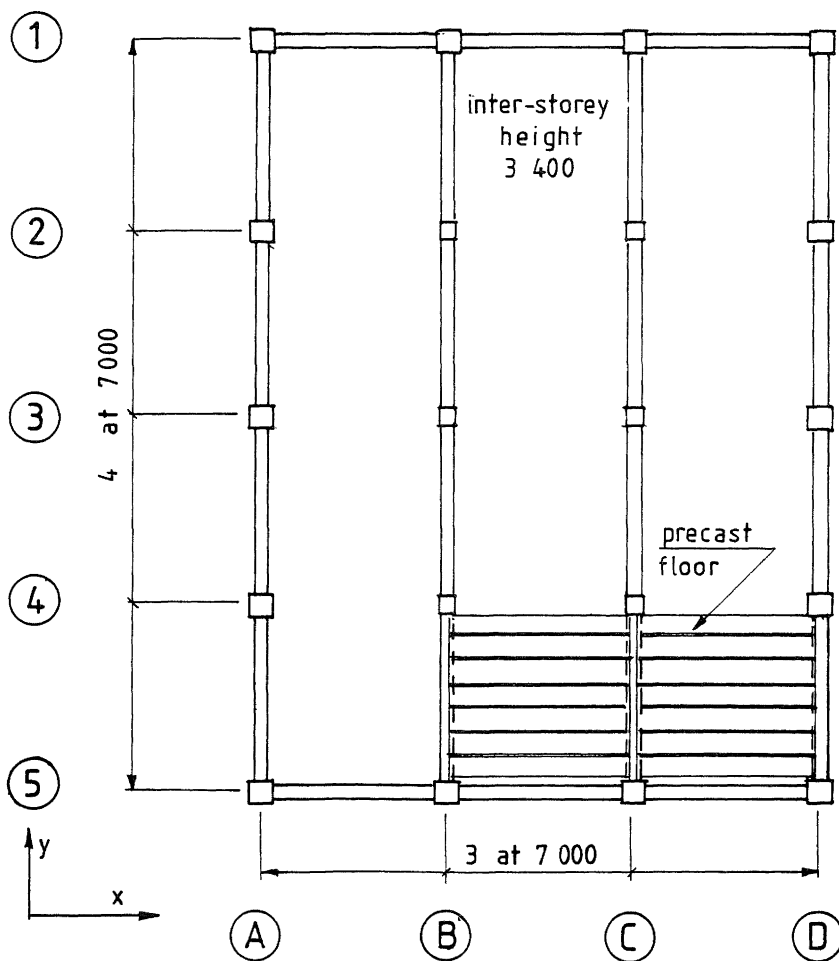
To illustrate the differences which arise from the use of these codes a series of analyses were made of an external frame and a wall that were proportioned to provide the lateral load resistance for a 24 storey building. The form of the structures and the results of the analyses are described in the following sections of this paper.

DETAILS OF STRUCTURES ANALYSED

The analyses were based on a 24 storey building with the rectangular plan shown in Fig. 2. The seismic actions (or forces) in the x direction are resisted by either external frames or walls. The seismic weight at each level was taken as 3 400 kN and to simplify the results of the analyses torsional actions were excluded. In the first option the lateral load resistance was provided by the external frames on lines 1 and 5, while the second option each of these frames was replaced by two structural walls. With this arrangement the seismic weight associated with each level



Structural wall alternative



Typical floor plan with external frame

FIGURE 2 TYPICAL FLOOR PLAN OF 24 STOREY BUILDING

was 1 700 kN for each frame and 850 kN for each wall.

The structural dimensions, which are given in Fig. 2, were chosen to comply with the stiffness criterion given in the draft loadings code DZ 4203-86. The analyses were carried out assuming the structures were to be built in the Wellington region. With the draft code a zone factor of 0.8 was used while with the current loadings code the values corresponding to zone A were used. Both the wall and the frame were sized to have the same fundamental period of 3.38 seconds.

METHODS OF ANALYSIS

To satisfy the minimum requirements of the codes, structural ductility factors of 6 and 5 were used for the frame and walls respectively with the draft code analyses, while the corresponding SM values for the current loadings code were taken as 0.64 and 0.8.

The methods of analysis employed are given in the following list. The number 1 to 5 are used to define the methods in the remainder of the text.

(1) The equivalent static method as defined in the draft codes, DZ 4203-86 and 89.

(2) The modal equivalent static method, which is defined in the 1986 draft code (DZ 4203-86) as the spectral modal analysis method. With this approach the envelope of storey shears is found by combining the modal contributions. From this envelope a set of equivalent static forces is found and these are applied to the structure to generate the design actions.

(3) The modal response spectrum method as defined in the draft code DZ 4203-89. In this approach the structural actions are found from the modal contributions (combined by the square root of the sum of the squares or other appropriate technique as in the first stage of method [2]).

(4) The equivalent static method as outlined in NZS 4203-1984.

(5) The modal response spectrum method as defined in NZS 4203-84, but with the omission of the 90 and 80 percent limits on the base and storey shears respectively, which tie this approach to the equivalent static method.

RESULTS OF ANALYSES

The periods of vibration and the proportions of the weight acting in each mode are given in Table 1 for both the wall and the frame. From this information and the lateral load coefficients for the different design spectra the base shear for each mode can be found. In the table these values are listed together with the ratios of the base shear in each mode to that in the first mode. Comparing these values for the different response spectra it can be seen that with the draft code the ratios for the second and higher modes are typically 3 to

4 times the corresponding values found from the current loadings code spectra. For the wall with the draft code spectrum the base shear in the second mode is nearly twice that sustained in the first mode. However, for the frame the first mode still dominates.

The two draft loadings codes recommend against the use of the equivalent static method for structures with fundamental periods in excess of 2.5 seconds, as this approach does not have a reliable means of allowing for the higher mode contributions. Because of this the results obtained using this approach are reproduced in this paper for comparative purposes only.

The base shears, overturning moments and roof deflections for the frame and one of the walls found using the five methods of analysis are listed in Table 2. The most striking feature of these is the large difference in values which occurs due to the change in the response spectra between the current and draft codes. Compare the results of analyses (1), (2) and (3) derived from the methods in the two draft codes with those of (4) and (5).

The results of the three methods of analysis ((1), (2) and (3)), which are made using the response spectra ($\mu = 6, 5$) given in the draft code [3] are shown in Fig. 3 for the frame and Fig. 4 for the wall. For the purposes of comparison the shear values found from the equivalent static (4) and modal (5) methods with the current loadings code spectrum (SM = 0.8) are included in Fig. 4(a) for the wall.

From Table 2 and Figs. 3 and 4 it can be seen that the results of the equivalent static method (1) of DZ 4203-89 are on the conservative side of the modal response spectrum (3) values in all aspects except the shear forces in the wall. As previously noted the equivalent static method does not comply with the fundamental period limits recommended in the draft code.

The difference in the results of the modal analysis (3) carried out to the draft DZ 4203-89 code and the modal equivalent static method (2) specified in the 1986 draft (DZ 4203-86) are particularly striking for the wall. The deflections and overturning moments associated with the latter method are of the order of 90 percent in excess of the corresponding method given in the later draft code. This difference arises from the requirement in the first draft (1986) to determine structural actions on the basis of an equivalent set of lateral loads found by differencing the storey shears, while in the 1989 draft the design values are found directly from the combined modal values.

When the modal actions are combined by the square root of the sum of squares, or other suitable method, equilibrium is lost. The envelope in storey shears obtained in this way does not correspond to either the envelope of moments or deflections in the structure. In theory the combined modal values for any action represent an envelope of values, which may be expected to occur over the period of earthquake excitation.

TABLE 1 - DYNAMIC CHARACTERISTICS OF FRAME AND WALL STRUCTURES

Mode "n"	Period (seconds)	Proportion "p" of W_t	Sum of p Σp	DZ 4203 - 86 (89)		NZS 4203 - 84		Ratio V_n / V_1	
				C_d	base shear " V_n "	C_d	base shear	DZ 4203	NZS 4203
FRAME ($\mu = 6, SM = 0.64$)									
1	3.38	0.758	0.758	0.020	0.0152 W_t	0.048	0.0364 W_t	1.00	1.00
2	1.16	0.117	0.874	0.062	0.0073 W_t	0.051	0.0060 W_t	0.48	0.16
3	0.67	0.039	0.914	0.101	0.0039 W_t	0.082	0.0032 W_t	0.26	0.09
4	0.46	0.021	0.936	0.146	0.0037 W_t	0.095	0.0020 W_t	0.24	0.05
5	0.35	0.012	0.948	0.176	0.0021 W_t	0.096	0.0012 W_t	0.14	0.03
WALL ($\mu = 5, SM = 0.8$)									
1	3.38	0.628	0.628	0.025	0.0157 W_t	0.060	0.0377 W_t	1.00	1.00
2	0.55	0.195	0.823	0.156	0.0304 W_t	0.112	0.0218 W_t	1.94	0.58
3	0.20	0.069	0.891	0.272	0.0188 W_t	0.120	0.0083 W_t	1.20	0.22
4	0.11	0.036	0.926	0.294	0.0106 W_t	0.120	0.0043 W_t	0.68	0.11
5	0.07	0.022	0.948	0.320	0.0070 W_t	0.120	0.0026 W_t	0.45	0.07

p = proportion of seismic weight participating in mode

V_n = base shear of mode n

V_n / V_1 = ratio of base shear in mode n to 1st mode value

TABLE 2 - PRINCIPAL RESULTS OF ANALYSES

Method No. *	Code	METHOD OF ANALYSIS				
		1 DZ 4203-86 (89) Eq. static	2 DZ 4203-86 *** Modal eq. st.	3 DZ 4203-89 Modal	4 NZS 4203-84 ** Eq. static	5 NZS 4203-84 Modal
24 Storey Frame, $T_1 = 3.38$ secs, $\mu = 6, SM = 0.64$						
Base shear	kN	827	734	734	1 960	1 510
Overturning moment	kNm	47 300	40 800	35 100	112 000	82 100
Deflection at roof level	mm	82	94	79	257	186
24 Storey Wall, $T_1 = 3.38$ secs, $\mu = 5, SM = 0.8$						
Base shear	kN	516	846	846	1220	911
Overturning moment	kNm	29 700	40 000	22 700	70 500	47 100
Deflection at roof level	mm	164	217	110	389	259

* Method number, see text

** Equivalent static

*** Modal equivalent static

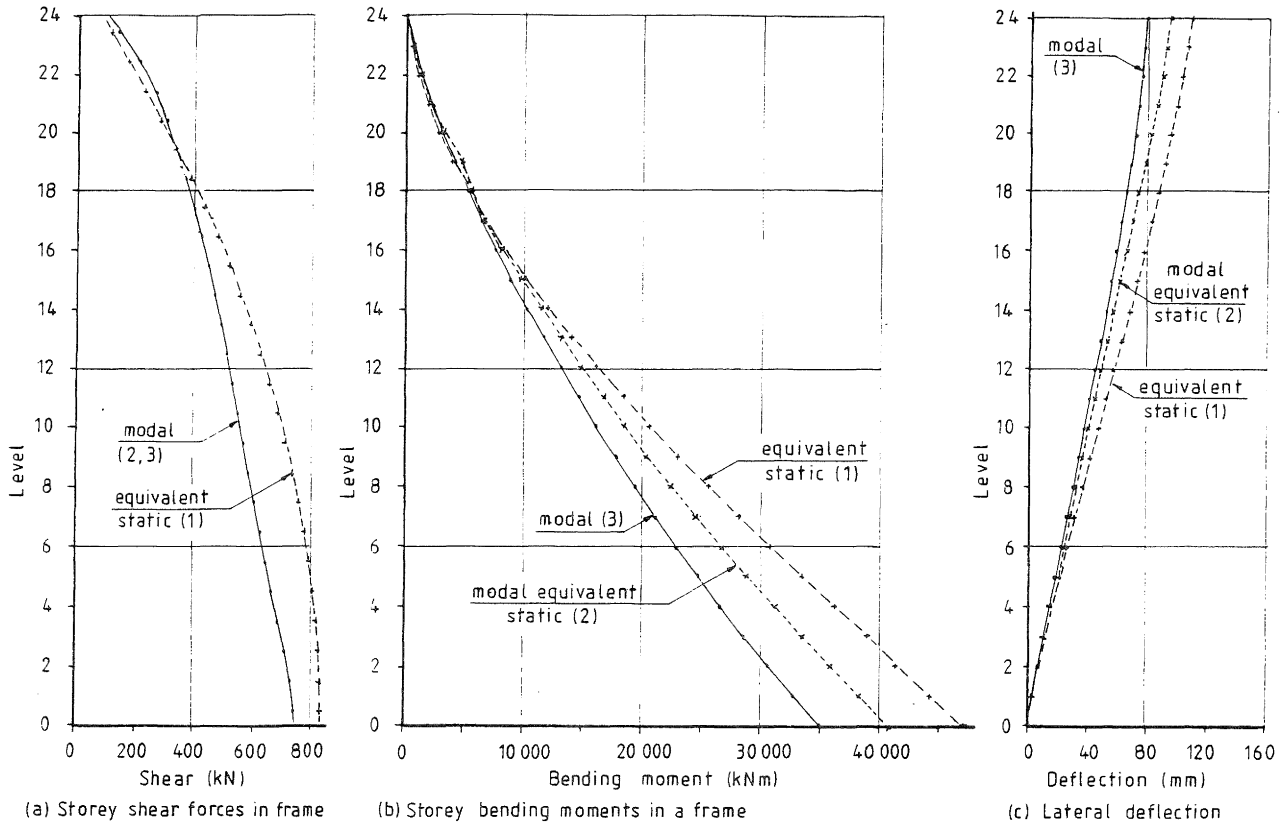


FIGURE 3 RESULTS OF EQUIVALENT STATIC AND MODAL ANALYSES IN ONE EXTERNAL FRAME (methods of analysis (2) and (3) in DZ 4203-86 and -89)

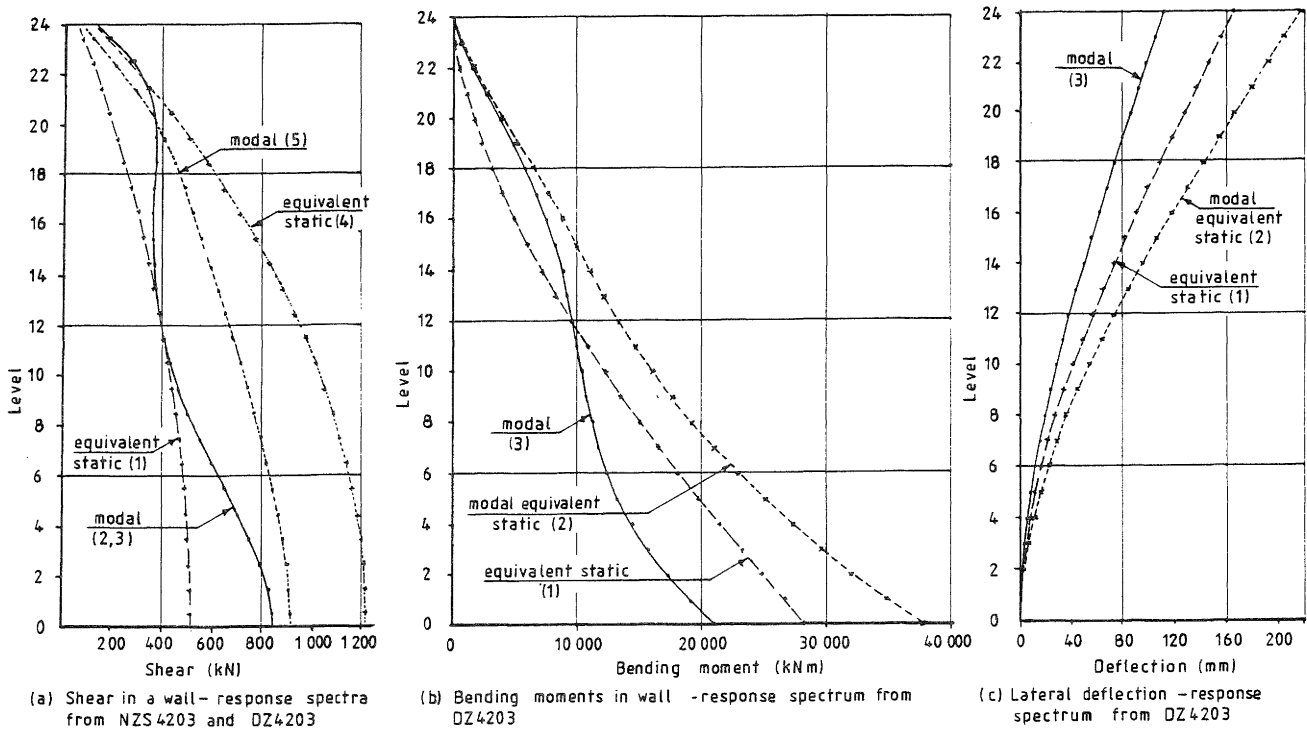


FIGURE 4 RESULTS OF EQUIVALENT STATIC AND MODAL METHODS OF ANALYSIS FOR ONE EXTERNAL WALL (methods of analysis (1), (2) and (3) in DZ 4203-86 and -89, and (4) and (5) in NZS 4203-84)

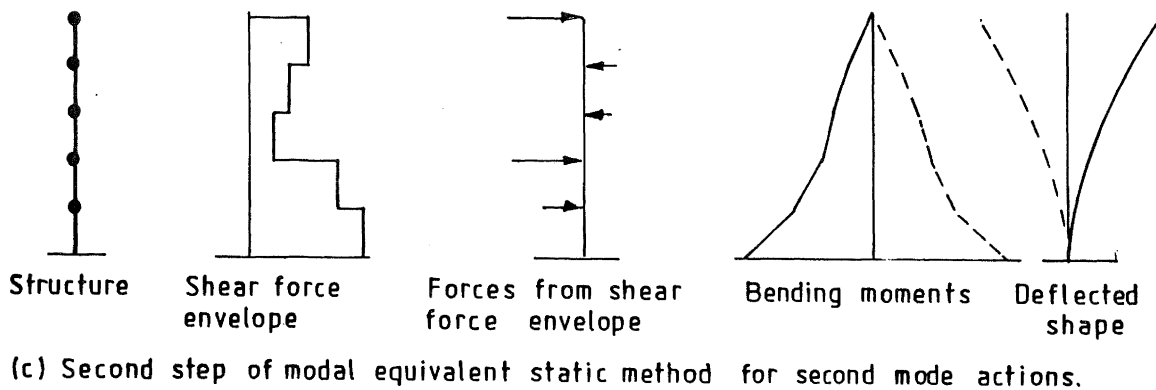
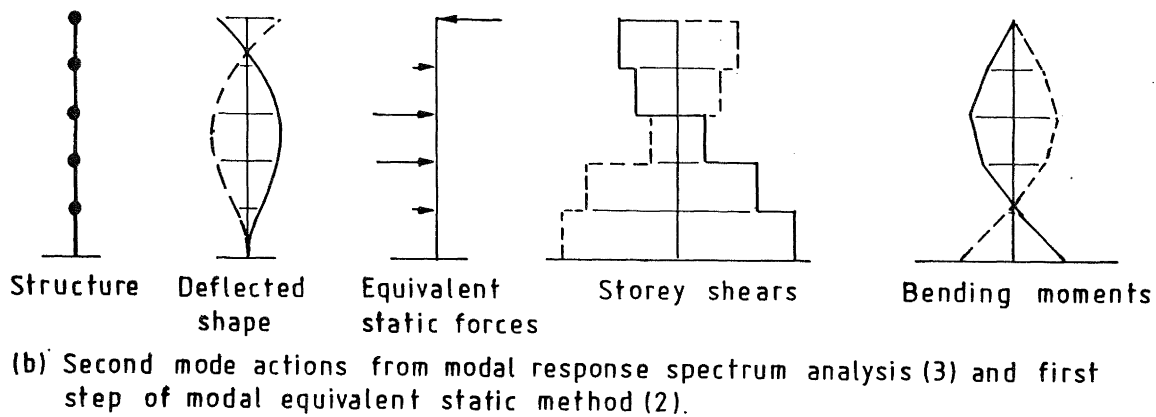
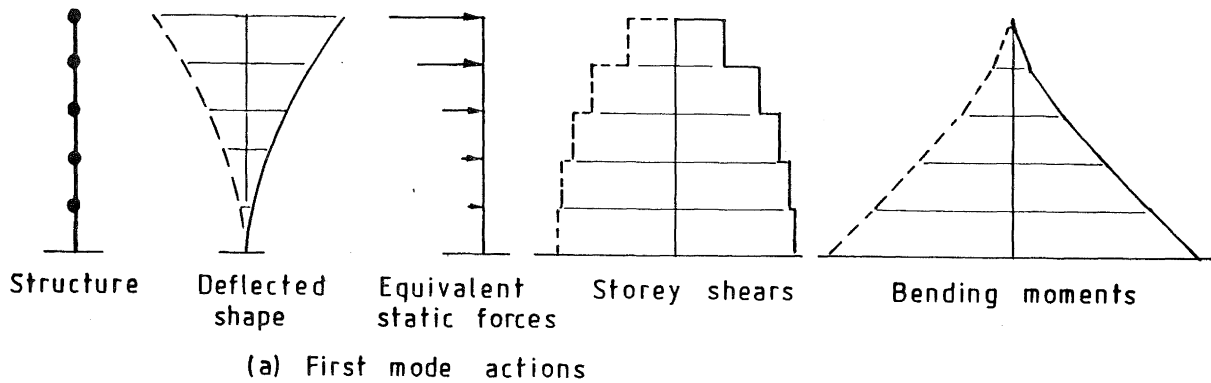


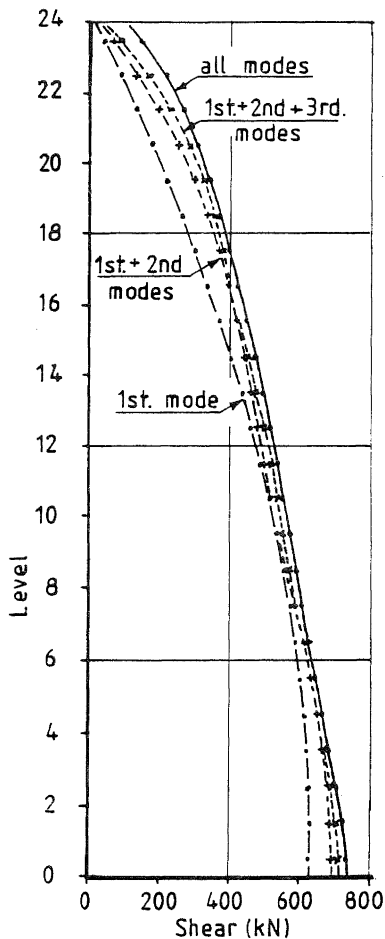
FIGURE 5 FIRST AND SECOND MODE ACTIONS IN A WALL AND RESULTS OF MODAL RESPONSE SPECTRUM ANALYSIS (3) AND MODAL EQUIVALENT STATIC ANALYSIS (2) FOR SECOND MODE ACTIONS BY THEMSELVES

The maximum values of the storey shears, or for that matter any other action, are not sustained simultaneously at any stage.

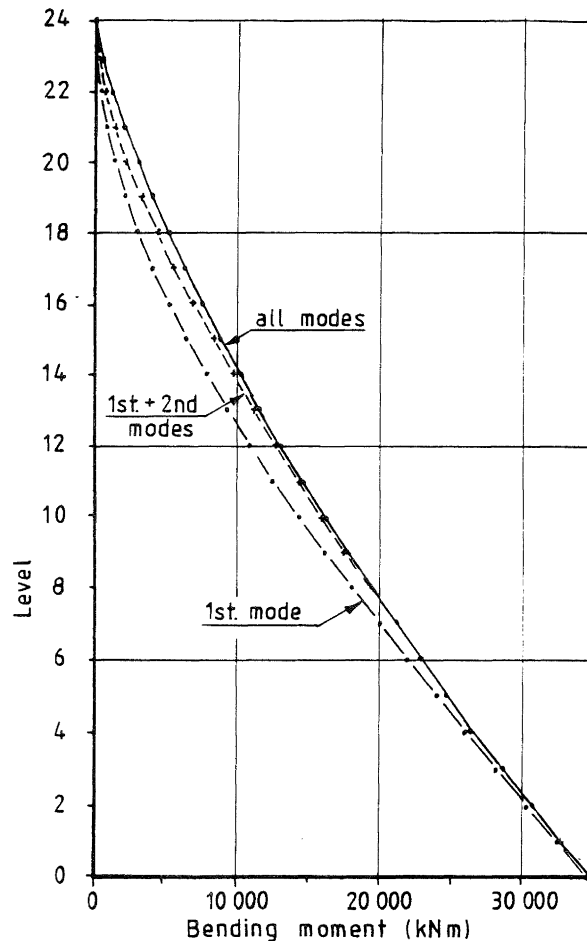
To illustrate how the modal equivalent static method (2) gives rise to larger bending moments and deflections than the modal response spectrum method (3), the modal actions for a simple five level wall are described. In Fig. 5 the first and second mode shapes are shown. The maximum lateral inertial force each weight applies to the wall is proportional to the maximum acceleration of the weight, which in turn

is proportional to its deflection. Consequently for each mode a set of equivalent static forces can be found, as illustrated in the figure.

The difference between the modal equivalent static (2) (DZ 4203 - 86) and the modal response spectrum (3) (DZ 4203 - 89) methods is evident if the structure shown in Fig. 5 is subjected to excitation that activates the second mode by itself. The deflected shape and the equivalent static forces corresponding to this mode, together with the storey shears and bending moments



(a) Storey shear in frame - modal analysis (3) - DZ 4203 spectrum



(b) Storey bending moments in frame - modal analysis (3) - DZ 4203 spectrum, $\mu=5$

FIGURE 6 CONTRIBUTIONS OF DIFFERENT MODES TO STOREY SHEARS AND BENDING MOMENTS IN AN EXTERNAL FRAME

are shown in Fig. 5(b) for the modal response spectrum method of analysis (3). For the modal equivalent static approach (2) finding the storey shear force envelope forms the first step in the analysis. The second step is shown in Fig. 5(c). From the shear force envelope found in the 1st step the equivalent static lateral forces are determined. However, as indicated the sign of the shears is lost in the envelope and as a result the shear force has the same sign from the top to the bottom of the wall. This creates a bending moment in the wall which continuously increases from the top to the base.

From the illustration shown in Fig. 5 it can be seen that the modal equivalent static method fails to correctly represent the higher mode actions. The second step in this method replaces the true deflected shapes and bending moment diagrams from the higher mode contributions by diagrams which have similarities to first mode actions.

In Figs. 6 and 7 the different modal

contributions to the combined modal storey bending moments and shears in the frame and wall are shown. In these diagrams modal contributions have been combined by the square root of the sum of squares. For both the frame and the wall the storey bending moments in the lower portion of the structure are dominated by the first mode contribution, with the higher modes making a significant contribution in the higher zones of the building. The modal deflection components are not plotted as the first mode dominates this action. The total contribution of the second and higher modes to the deflection at any level in either structure does not exceed 2 percent of the deflection at the roof level.

Fig. 7(b) shows that for the 24 storey wall the second and third modes make a major contribution to the combined modal shear values, while the corresponding contribution to the frame structure is much less, see Fig. 6(a). The first mode dominance in the frame structure explains why there are only small discrepancies

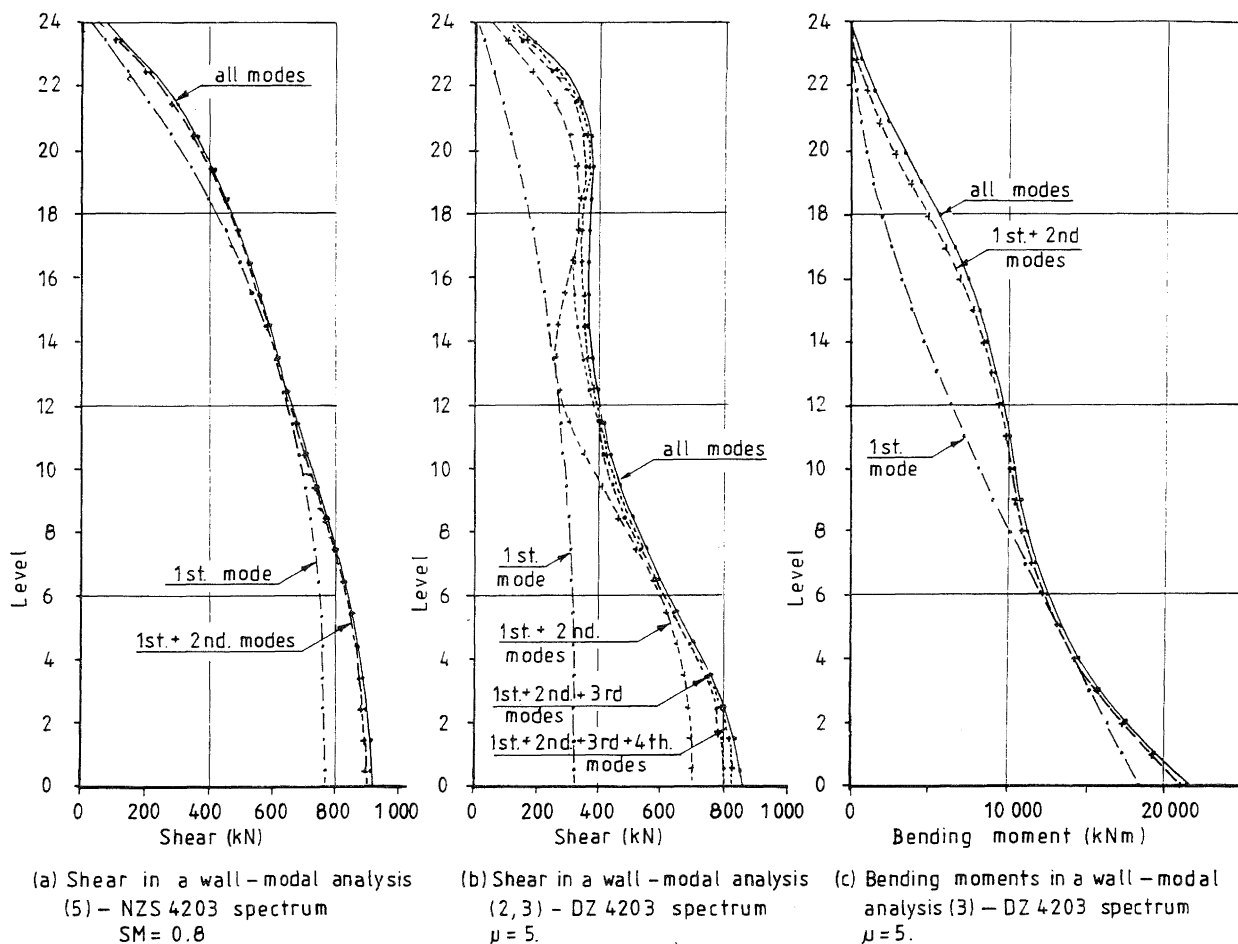


FIGURE 7 CONTRIBUTIONS OF DIFFERENT MODES TO STOREY SHEARS AND BENDING MOMENTS IN ONE END WALL

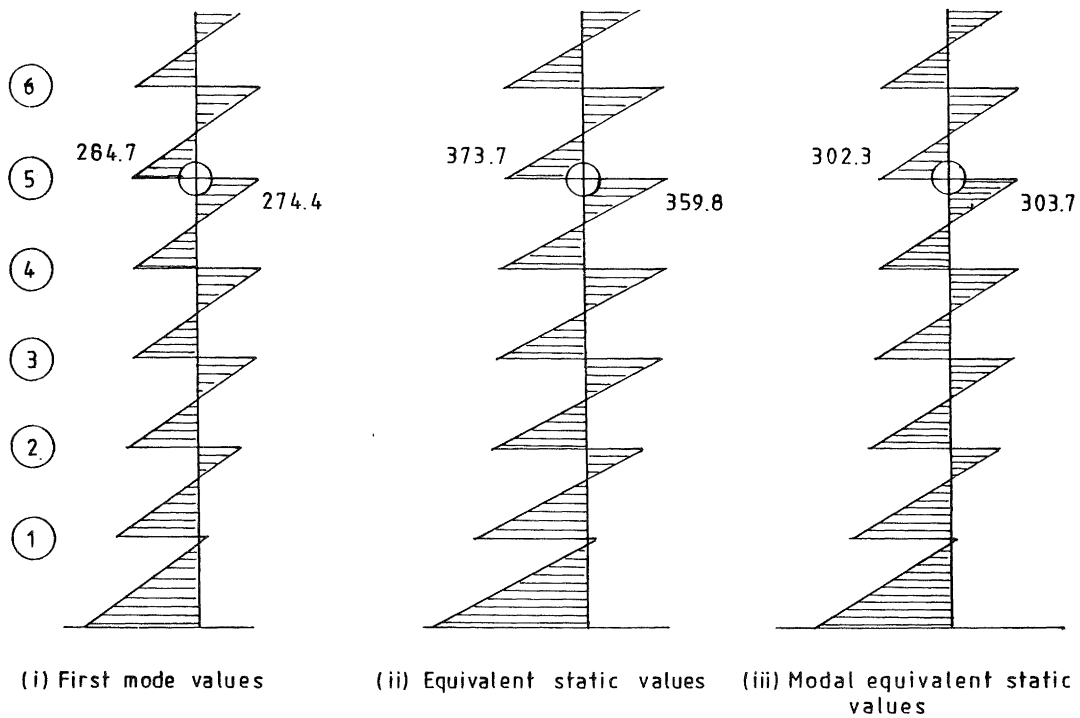
between the modal equivalent static (2) and modal response spectrum (3) values. Similarly the major difference between these two approaches in the structural wall arises from the significant contributions that the second and third modes make in this case.

From Figs. 6 and 7, and from Table 1, it can be seen that the total combined modal actions can be predicted with negligible error if the first three modes are considered for the frame and four modes considered for the wall. This suggests that sufficient modes should be combined so that the sum of the weight participating in each mode reaches or exceeds 90 percent of the total weight. No further scaling of actions is required to compensate for the missing 10 percent of the mass.

There are two reasons why the modal equivalent static method (2) should not be used in the design of structures in which the second and higher mode effects make a significant contribution to the behaviour. The first of these is related to the increased structural costs associated with

the higher design bending moments and deflections. The second arises from the incorrect moment to shear ratio that the approach predicts. For example, with the wall that was analysed a structural ductility of 5 was assumed in determining the modal storey shears. The method over assesses the bending moments corresponding to these actions and as a result the implied ductility demand, if this is to be limited by flexural yielding at the base of the wall, is reduced from 5 to 2.8. However, with the 2.8 value the corresponding shear forces would need to be increased by approximately a further 60 percent to correspond to this level of ductility if a premature shear failure is to be avoided.

The modal equivalent static method (2) has been used extensively by a number of structural engineers in New Zealand without difficulties often becoming apparent. The reason for this is that it gives acceptably accurate predictions for structures where the first mode dominates. With the response spectra in the current loadings code this condition is generally satisfied. However,



(a) The bending moments in an internal column corresponding to the first mode, the equivalent static and the modal equivalent static values.

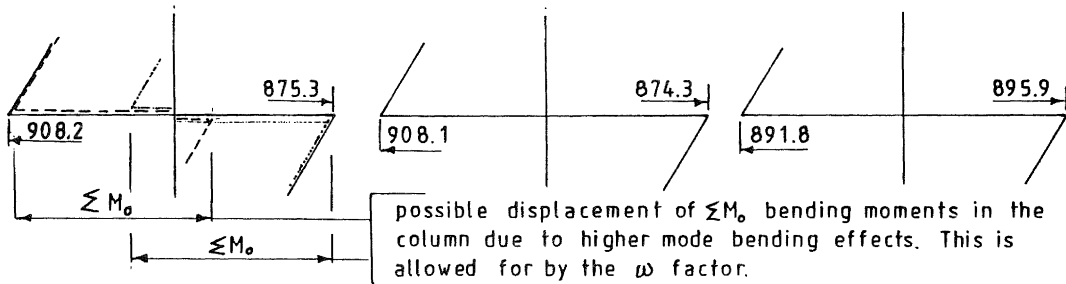
Assuming $\sum M_o$ at joint 5 is equal to 1100 kNm then for the different cases

$$\phi_o = \frac{1100}{(264.7 + 274.4)} = 1.96 \qquad \phi_o = 1.50 \qquad \phi_o = 1.82$$

The dynamic magnification factor $\omega = 1.62$ (one way frame $T_f = 3.4$ s)

$$\omega \phi_o = 3.19 \qquad \omega \phi_o = 2.43 \qquad \omega \phi_o = 2.95$$

hence for the 3 cases the capacity design bending moments at level 5 are



(i) Based on first mode values. (ii) Based on equivalent static values. (iii) Based on modal equivalent static values.

(b) Capacity design moments in a column predicted from different elastic analyses.

FIGURE 8 FINDING CAPACITY DESIGN MOMENTS IN A COLUMN

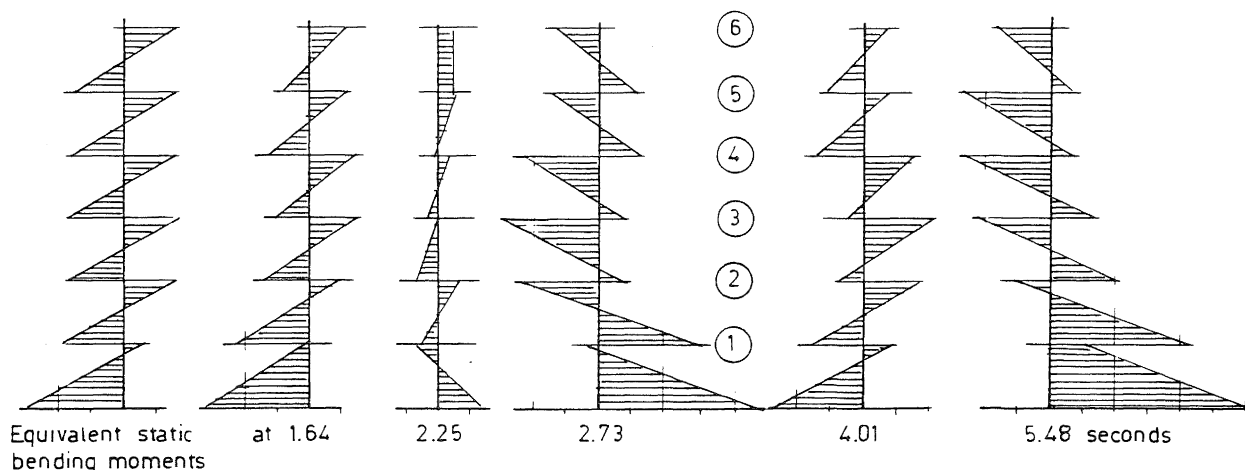


FIGURE 9 THE EQUIVALENT STATIC BENDING MOMENTS IN A COLUMN COMPARED WITH VALUES FOUND IN A TIME-HISTORY ANALYSIS OF A MULTI-STOUREY FRAME (FROM REFERENCE 8)

as previously noted the proposed response spectra in the draft codes greatly amplifies the second and higher mode contributions due to the increased lateral force coefficients for short periods and the reduced values for long periods (see Table 1.). The difference is apparent in Fig. 4a, where it can be seen the form of the storey shear envelope in the wall derived from a modal analysis (5) carried out to the response spectrum in the current loadings code is very similar in form to the corresponding equivalent static values (4). The comparable values found from the draft code spectra are very different in shape.

With the current loadings code some designers determine the combined modal base shears and if the value exceeds 90 percent of the corresponding equivalent static value they scale down their actions to correspond to the 90 percent level. This scaling is based on the clause 3.5.2.4 in NZS 4203, which can be subjected to different interpretations. With the second draft code, DZ 4203-89, scaling down is not permitted. With the enhanced second and higher mode contributions, which arise with the draft code spectrum, such scaling could lead to serious under estimates in the storey bending moments. For example, if the modal actions obtained in the wall are scaled to 90 percent of the corresponding equivalent static value the overturning moment would be reduced to 42 percent of the corresponding equivalent static analysis value!

CAPACITY DESIGN

With current practice capacity design of the columns in frames is required to ensure that the primary plastic hinge zones are located in the beams during a major earthquake. The recommended procedure for concrete columns is contained in appendix C 3A of Part II of the concrete design code

[7]. With this approach the basic bending moments in the columns are found by scaling the column moments found in an equivalent static analysis of the frame by an overstrength factor (ϕ_0) and a dynamic magnification factor (ω). With the 1989 draft loadings code (DZ 4203 - 89) an equivalent static method of analysis is no longer required, and consequently some other set of moments have to be found to work the design procedure.

In the draft code (DZ 4203 - 89) it was suggested that the equivalent static analysis values could be replaced by the first mode actions found in a modal analysis. The method of finding the capacity design moments in the columns at the beam intersections is illustrated in Fig. 8 for an internal column at level 5 in the 24 storey frame. In Fig. 8(a) the first mode, the equivalent static and the modal equivalent static bending moments in the first six levels are shown. For the purposes of this exercise each set of bending moments is used to calculate capacity design moments in the columns. It is assumed the sum of the overstrength moments in the beams intersecting with the column is 1 100 kNm. The first step is to scale the bending moments in the columns by the ϕ_0 factor so that equilibrium is maintained between the column and beam bending moment at the joint when the overstrength moments are applied. As indicated the ϕ_0 values range from 1.5 to 1.97 for the three different cases considered. The second step is to multiply the resultant bending moments by the dynamic magnification factor " ω ", which is 1.62 in this case. This allows for the lateral displacement of the bending moment diagram associated with high mode bending of the column. Fig. 9, which is taken from reference 8, illustrates how the column bending moments change during earthquake excitation due to higher mode effects.

It can be seen from Fig. 8 that the capacity design moments in the columns predicted from the first mode, the equivalent static and the modal equivalent static values, are for all practical purposes identical. Consequently the most readily available set of moments, namely the first mode values, can be used. Minor changes need to be made. One of these is to the upper limit of ϕ_0 that is used. With the equivalent static method of analysis a value of 4 implies near elastic response and consequently this is a reasonable upper limit. With actions based on the first mode values this should be increased to 5 to retain a similar margin of safety.

CONCLUSIONS

The change in the shape of the seismic design response spectra from those existing in the current loadings code to those proposed in the draft loadings code has a number of important implications for designers of multi-storey structures.

(i) The reduction in the lateral load coefficient for long period structures results in wind loading effects becoming more critical than was previously the case both in terms of the required strength and the dynamic characteristics for serviceability conditions.

(ii) In assessing the earthquake actions the second and higher mode effects assume more importance than was previously the case. As a result of this change for many multi-storey structures the error involved in determining the actions by applying a set of lateral loads, which are found by differencing the modal storey shears, is no longer acceptable. This approach should only be used on structures where the second and higher mode contributions are small.

(iii) Scaling down of modal values so that the combined modal base shear is equal to the equivalent static or 90 percent of the equivalent static value can lead to a serious underestimate of the required flexural strength for a given ductility.

(iv) From the modal response spectrum analyses reported in the paper and in reference 5, it appears that accurate values of structural actions can be obtained by using sufficient modes to ensure that 90 percent of the mass of the structure is participating. This is in agreement with conclusions of other work [6].

(v) In the capacity design of columns as described in reference 7, the design moments may be obtained by scaling the first mode moments by the overstrength factor of ϕ_0 and the dynamic magnification factor ω .

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