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DYNAMIC BEHAVIOUR OF MULTI-STOREY BUILDINGS

by
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ABSTRACT

A series of frames and walls for multi-storey structures were sized and analysed by the different methods described in the New Zealand draft loading code DZ 4203-86. The aim was to examine the differences arising from the equivalent static and the modal response spectrum methods of analysis. Several of these structures were analysed for an earthquake ground motion in non-linear time history analyses. The effects of P-delta actions on the behaviour were investigated.

ACKNOWLEDGEMENTS

We gratefully acknowledge the financial support provided by The Earthquake and War Damages Commission for this research. We would like to thank Mr L. Robinson of Hadley and Robinson for his assistance in sizing the members of the walls and frames analysed in this report and Mr S. Sritharan for running the computer analyses. We would also like to thank Joanna Fenwick for tracing the diagrams and Eileena Sucich for typing this report.

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Chapter I Background and Scope of Project

I.1 Background

The draft loadings code DZ 4203 - 1986[1] requires more rigorous methods of analysis to be used in assessing structural actions for earthquake resistant design than is current practice. In particular the use of the familiar "equivalent static method", which has formed the basis of seismic design for some decades, is restricted to regular structures with a fundamental period of one second or less. The analysis of buildings not complying with these limits is to be based on either the modal response spectrum method or a numerical integration time history approach. If these proposals in the draft code were adopted, the designers could find that they are required to carry out a modal analysis for some buildings of only five storeys.

The committee who wrote the draft code DZ 4203 - 86 considered the different methods of analysis available to the structural engineer. It was considered that more rigorous methods could be expected to lead to improved performance of structures (both economic and structural) and they felt the use of such methods should be encouraged. Furthermore, the great increase in the availability of computers and software packages, which had occurred since the first edition of the existing loadings code[2] was written (1976), has made these methods much more readily available to the designer. The committee for DZ 4203 - 86 analysed a number of structures by both the modal response spectrum and the equivalent static methods in an attempt to establish practical limits for the second method. However, time constraints did not permit an extensive study, and the one second limit for the fundamental period of structures for which the equivalent static method was permitted could not be firmly supported. Compared with other codes of practice this limitation in DZ 4203 - 86 is restrictive[3,4].

A number of Engineers in commenting on the draft code were critical of these provisions, which require more complex methods of analysis to be carried out for seismic effects. The equivalent static method was seen as having three advantages over the modal response spectrum technique:-

(1) The equivalent static method was well known and understood by Structural Engineers, but this does not hold for the modal* method.

(2) With the equivalent static approach a defined set of forces are applied to the structure and the resultant actions are in equilibrium with these forces. With the modal response spectrum approach no clear set of forces is applied and the internal structural actions, when the different modal contributions are combined, are no longer in equilibrium with each other**. This leads to very considerable difficulties, particularly in the capacity design stages. A natural result of this is that designers tend to place a lot of reliance on computer results, and as these are not necessarily in equilibrium, the opportunity to incorporate irrational details into the structure exists.

(3) For many years it has been emphasised that the performance of a building in a major earthquake depends primarily on the detailing in the structure and the ductility, which is achieved largelyas a result of this attention to detailing, rather than on an accurate determination of minimum strength requirements to an artificial design earthquake spectrum. The increased emphasis on the importance of the analysis was seen as detracting from the emphasis on detailing.

Both the equivalent static and the modal response spectrum methods of analysis are based on assumed elastic behaviour of structures. How well such analyses represent the requirements of structures, which are expected to undergo extensive yielding in a severe earthquake, needs to be examined. The majority of inelastic time history analyses, which are made to check the theoretical performance of structures subjected to seismic ground motion, are based on a bi-linear response of yielding zones. A check with experimental test results shows that this approximation is at best a very crude representation of real behaviour of timber, steel or reinforced concrete. Time did not permit this subject to be considered in this report.

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* In this report the term modal is used as a short form for modal response spectrum.

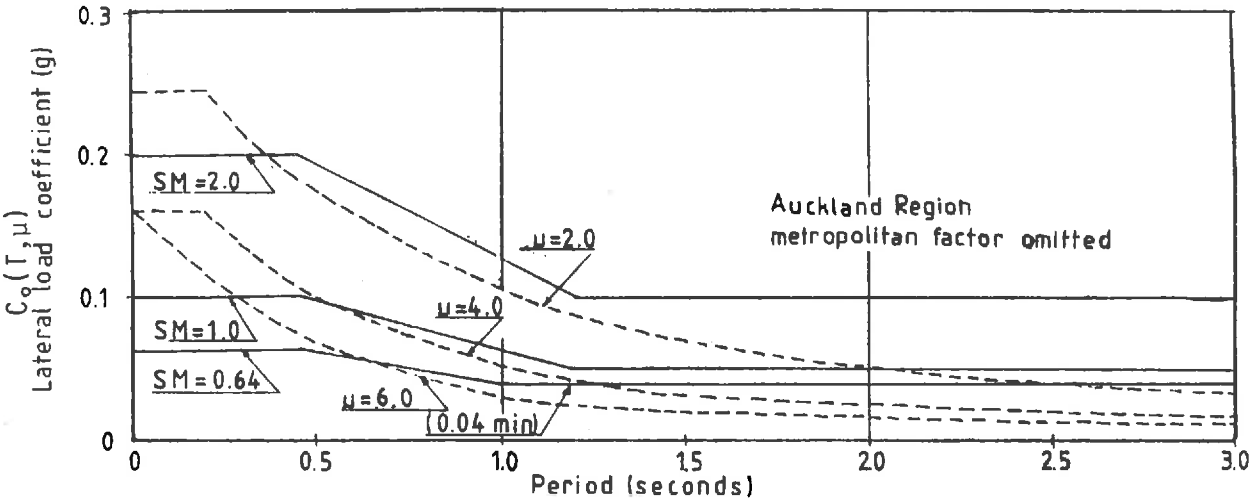
** The method given in the draft code for obtaining a set of equivalent static forces from the results of a modal response spectrum analysis was incorrect.

1.2 Major Changes in the Earthquake Design Provisions between NZS 4203 - 1984 and DZ 4203 - 1986.

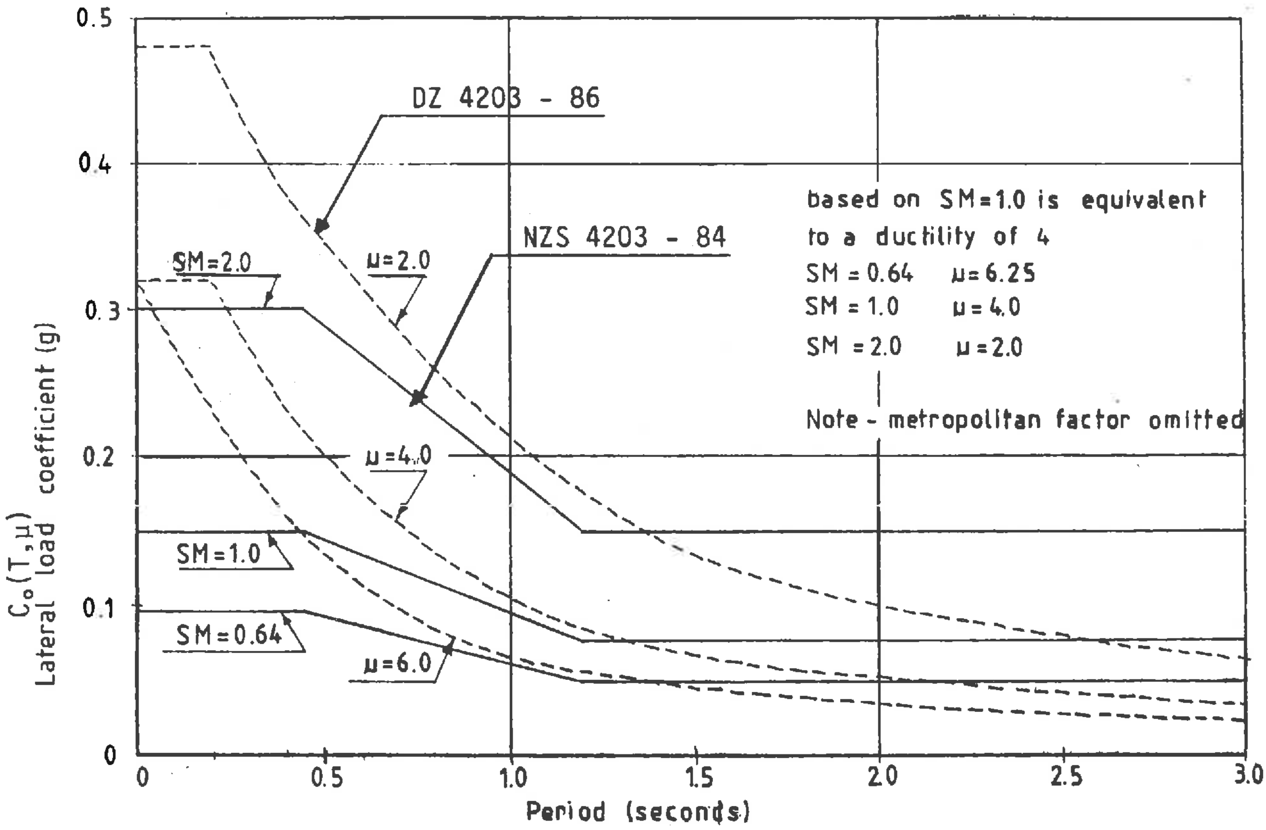
The analysis requirements for seismic actions in the current loadings code (NZS 4203)[2] are based around the equivalent static method of analysis. The modal response spectrum approach may be used, but it was required that the results from such an analysis be scaled so that the base shear was not less than 90 percent of the value corresponding to the equivalent static method. A further limitation was imposed on the minimum storey shears found in the modal analysis. These are not allowed to be less than 80 percent of the value corresponding to the equivalent static method. In effect only relatively small deviations in a storey wise sense are permitted from the results of the equivalent static method.

With the draft loadings code[1] DZ 4203-86, as previously noted, the equivalent static method of analysis is limited to reasonably regular structures in both plan and elevation, which have a fundamental period of one second or less. Unlike the current loadings code, an equivalent static analysis is not required where a modal analysis is carried out. However, the draft code does require a set of equivalent lateral forces to be found from the combined modal shears, and that these be applied to the structure to find the structural actions for design purposes. This requirement is unsound, as the modal storey shears at each level are not sustained simultaneously. Equivalent static forces derived as suggested, if applied to the structure can generate overturning moments considerably in excess (i.e. twice or more) of the corresponding modal response spectrum values.

The shape of the response spectra in the current and draft loadings code differ markedly, as illustrated in Figs. 1a and 1b, where the lateral load coefficients for the Wellington and Auckland regions as given in the two documents are compared. The metropolitan factor proposed in the draft code has been omitted for the purposes of this comparison. For the draft code the spectra corresponding to structural ductility factors of 6, 4 and 2 have been drawn together with the corresponding curves for SM products of 0.64, 1 and 2 from the current loadings[2] code. It can be seen that for the short period end of the spectra the draft code requires a marked increase in the seismic



(a) Response spectra for Auckland Region



(b) Response spectra for Wellington Region

Fig. 1.1 - A Comparison of Design Spectra in NZS 4203 - 84 and DZ 4203 - 86 for the Auckland and Wellington regions

forces, particularly in the Wellington region, while for the long period structures there is a marked decrease.

There are important implications in the proposed changes to the design spectra in the draft code. The vast majority of the buildings in New Zealand consist of low rise domestic and industrial buildings, which can be expected to have fundamental periods in the range of 0.1 to 0.45 seconds. For this group of structures the design seismic forces are greatly increased compared with existing practice. For example for ductile structures ($\mu = 6$ or SM = 0.64) the draft code design lateral force in the Wellington region varies from 1.4 to 3.5 times the corresponding value in the current code. As the proposed magnitude of the design force varies sharply with the period in this range it would be important for future designers to establish the dynamic properties of this group of structures. Clearly given the typical complexity of this group of building this would not be an easy task. This problem did not arise in the existing loadings code as the spectra are flat in this region.

For the high-rise, long-period structures, the very much lower lateral load levels proposed in the draft code mean that both P-delta and higher mode effects are potentially more severe than was the case previously.

1.3 Scope of Project

In this project a series of walls and frames sized for multi-storey buildings were analysed for earthquake actions by the equivalent static and modal response spectrum methods. The analyses were limited to two dimensions. The results of the two methods are compared so that major discrepancies are high lighted. These are followed by a number of integration time history analyses. In interpreting the results of these runs it became clear that P-delta effects were of considerable significance. For this reason a number of analyses were made with and without P-delta actions so that this effect would be quantified.

Chapter II The Structures and Methods of Analysis

2.1 Dimensions of Structures

To make the project as realistic as possible a series of frames and walls were sized for a rectangular building by Mr. L. Robinsonm, of Hadley and Robinson, Consulting Engineers, Dunedin. These, with minor modifications, were analysed for seismic actions at the University of Auckland.

The main intention behind this project was to investigate the differences in the methods of analysis. The emphasis was on the analyses rather than on structural design. Because of this a number of simplifying assumptions were made, as outlined below, to prevent possible differences in the analyses being hidden due to nominal or non-seismic effects.

(i) The sizes of the structural numbers were selected to satisfy the seismic related stiffness requirements in the draft code[1].

(ii) The strength and stiffness requirements due to wind loading on the building were neglected. In practice it is likely that wind loading would control at least the minimum strength requirements of the 12 storey and higher structures.

(iii) The torsional component of loading contained in the draft code was not considered. This enabled all the walls and frames to be analysed as two dimensional assemblages.

(iv) For the numerical intergration time history analyses the initial yield strengths of the potential plastic hinge zones had to be specified. These were based on the minimum strengths given in the appropriate modal analysis. No allowance was made for minimum strength levels that might in practice be required as a result of minimum steel contents specified in the concrete code[5].

The frames and walls analysed in this project were sized for a building which had a floor plan of 21m by 28m as shown in Fig. 2.1. Seismic actions were assumed to act in the x direction. The internal frames 2, 3 and 4 were assumed to support gravity loads but be flexible with respect to lateral forces. A precast flooring system was proposed and it was arranged so that

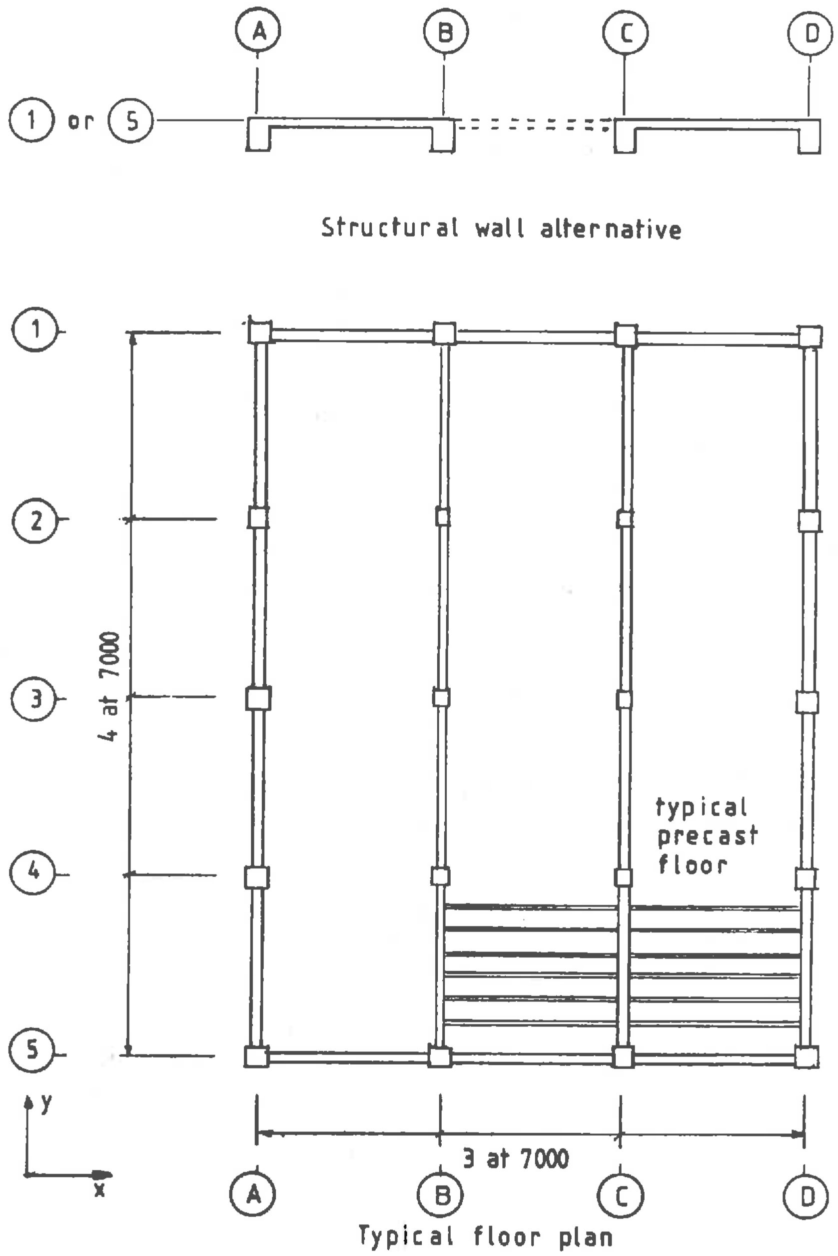


Fig. 2.1 - Typical Floor Plan of Building

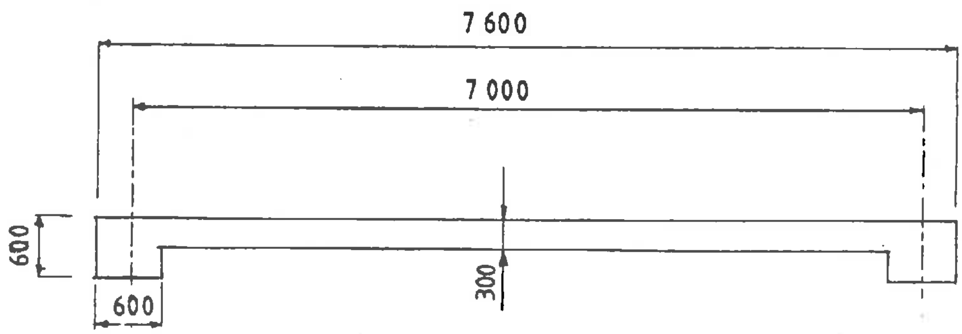


Fig. 2.2 - Dimensions of Structural Walls

negligible gravity loading was applied to the two lateral load resisting end frames, or wall pairs, located on lines 1 and 5. This assumption was made to avoid the complication of redistribution associated with plastic hinge formation in a severe earthquake[6].

In all the structures, except those with missing floors, the interstorey height was kept constant at 3.4m. Except for the podiums the seismic weight of each floor was asummed to be 3 400 kN. For the podium floors it was taken as 13 600 kN. As torsional actions were neglected half this was assumed to act with each end frame or pair of walls on lines 1 and 5.

Three different series of structures were considered.

(1) In the first series two independent (uncoupled) structural walls were placed at each end of the building (lines 1 and 5 in Fig. 2.1). The dimensions are shown in Fig. 2.2. As there were four walls in all the seismic weight associated with each wall was 850 kN per floor. These analyses were made for 12, 18, 24 and 30 storey buildings.

(2) In the second series regular lateral load resisting end frames were used for 6, 12, 18 and 24 storey structures. The elevation for the 12 storey frame is shown in Fig. 2.3, and the details of the beam and column sizes are given Table 2.1.

Table 2.1 Member sizes for regular frames

	6 Storey	12 Storey	18 Storey	24 Storey
Beams	600 x 400	600 x 400	700 x 400	800 x 400
Columns Levels				
0 - 6	450 x 450	550 x 550	700 x 700	800 x 800
6^+ - 12		"	600 x 600	700 x 700
12^+ - 18			500 x 500	600 x 600
18^+ - 24				500 x 500

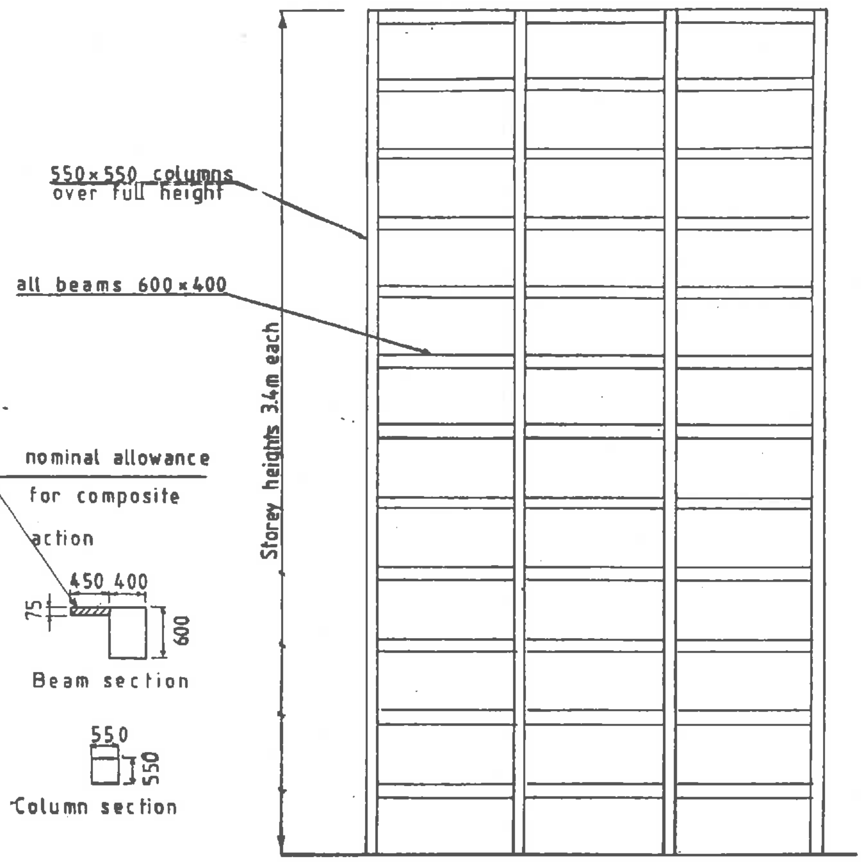


Fig. 2.3 - Typical Elevation of Frame and Beam Sections

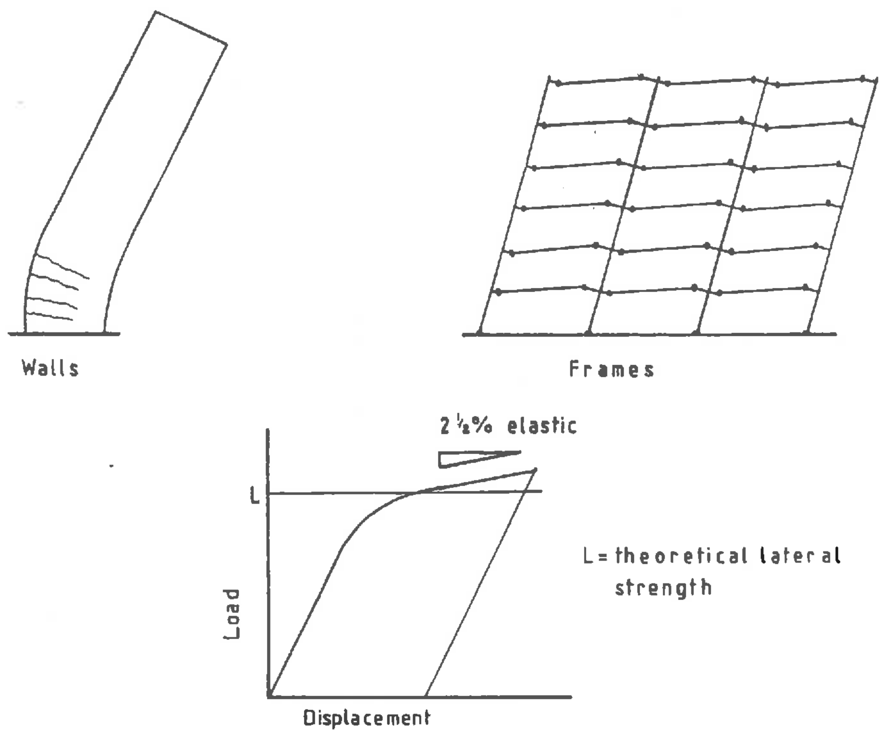


Fig. 2.4 - Strain Hardening in Model

(3) In the third series irregularities were introduced into the frame structures considered in series 2 by introducing a podium in some structures or removing the first two floors levels in others.

(a) Three storey podiums were added to the 12 and 24 storey frames, with these being laterally supported by two sets of 3 walls 0.3m wide and 15m long. The seismic mass of each podium floor was taken as 13,600kN

(b) The 12 and 24 storey frames were converted to 10 and 22 storey structures of the same height by omitting the first two floors. This gave a height to the new first floor level of 10.2m. Two alternatives were used with these missing floor structures -

(i) the beam and column dimensions were not altered, and

(ii) the columns between ground level and the new first floor were stiffened so that the fundamental period of the new structures was close to that of the original structures with no missing floors. For the 10 storey structure the ground level to first floor columns were increased to 700 x 700 mm and the first floor beam to 850 x 400 mm, while for the 22 storey structure the corresponding dimensions were 1000 x 1000 for the columns and 1100 x 400 mm for the beams.

2.2 Member Properties

The section properties used in the analyses were calculated as outlined in the following paragraphs.

Beams:
To allow for flexural cracking the moment of inertia was taken as half the value found from the uncracked gross section. Allowance was made, as indicated in Fig. 2.3, for some composite action between the beams and the precast floors and topping.

Columns:
The section properties were based on the gross section dimensions.

<u>Walls</u>:

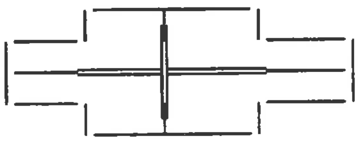
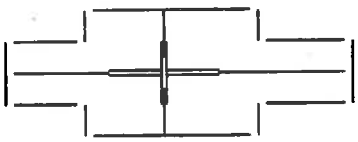
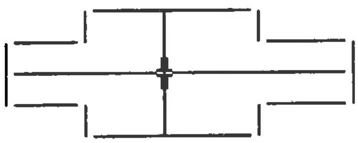
In the first series of analyses of the walls, the moment of inertia from the second floor level to the roof was based on the gross section dimensions, while half this value was used for the bottom two levels. These values were used to make a nominal allowance for the expected reduction in stiffness in the lower floor due to the greater flexural cracking in this region. However, the first inelastic analyses indicated that yielding could be expected to occur over a considerable height of the wall and in the light of this finding it was felt a uniform moment of inertia should be used. For the second and subsequent series of wall analyses the moment of inertia was taken as 0.75 times the value based on the uncracked gross section. In retrospect this value was too high[7].

In the analysis the elastic modulus was taken as 29 350 MPa and the shear modulus as 0.4 times this value. These values were based on a nominal 28 day concrete strength of 30 MPa, which was assumed to have mean value 39 MPa in the structure. This allows for the increase in strength with age and it makes a nominal allowance for the average concrete strength being greater than the specified strength.

2.3 Modelling of Joint Zones

Three different response spectrum analyses were made of the six storey frame to see how sensitive the results were to the modelling used to represent the joint zones. In the first analysis the full joint zone was represented by rigid links, in the second analysis these were reduced to two thirds of their previous lengths, while in the last analysis no allowance was made for stiffening in this zone. The results of these analyses, in the form of the periods obtained for different mode shapes, are reproduced in Table 2.2.

Table 2.2 Influence of Joint Zone Representation on Dynamic Properties of 6 Storey Frame

Joint Zone		Period (seconds)		
		T_1	T_2	T_3
Rigid Links Full Zone		1.40	0.45	0.25
Rigid Links $^2/_3$ of Zone		1.45	0.46	0.26
No Rigid Links		1.56	0.55	0.29

In the light of the relatively small differences in the dynamic properties of the structure with the different joint zone representations it was decided to use rigid zones for the remaining analyses. In terms of the magnitude of the deflections this assumption is unconservative. However, as the project is concerned with the relative differences between the predictions of the different methods of analysis this assumption is not significant.

2.4 Equivalent Static Method

To obtain structural deflections and member forces from an equivalent static analysis, lateral loads, F_i were applied at each floor of the structure. For all floor levels

$$F_i = 0.92\, V \frac{W_i\, h_i}{\Sigma\left(W_i\, h_i\right)} \tag{2.1}$$

and for the top level of the structure an additional force of 0.08V is included. In equation 2.1, W_i is the seismic weight of the i^{th} floor, h_i is the height of the i^{th} floor above ground level, and V is the base shear given by

$$V = C_o(T,\mu)\ R.Z.W_t \tag{2.2}$$

In this study R was taken as 1, Z as 0.85 (for Wellington), and $C_o(T,\mu)$ was appropriately taken from the spectra for normal soils in the draft code.

2.5 Modal Response Spectrum Analysis and the Design Spectra

In this project the normal soils response spectra from the draft code were used together with a zone factor multiplier of 0.85 (for the Wellington region) but no metropolitan factor was included. For the analyses corresponding to the strength limit state (referred as severe seismic in later code drafts) the response spectrum corresponding to a structural ductility factor of 6 was used. DZ 4203 gives μ = 6 for the frames while the corresponding, value in NZS 4203-84 is SM = 0.64 (equivalent to μ =6). For the structural walls the values are SM = 0.8 (μ = 5) for NZS 4203 and μ = 4 for DZ 4203. However, μ = 6 was used for both walls and frames. A number of analyses were repeated using the response spectrum corresponding to a structural ductility factor of 1 to enable the elastic performance for a serviceability earthquake conditions to be assessed.

In the modal response spectrum analyses the combined contributions of the different modes were found by the square root of the sum of the squares, as this method was built into the version of the ETABS program that used . A number of other combination methods were checked manually for some of the structures, but it was found that there was virtually no difference in the resultant values as the periods were well separated for the structures analysed.

In the two podium tower structures care had to be taken in finding the resultant modal combined shear at the base of the building. The shears in the podium wall tended to be in the opposite direction to those in the tower. As the "SRSS" method loses the sign of the action it was necessary to find the base shear in each mode and combine these rather than adding the combined values in the different elements.

2.6 Numerical Integration Time History Analyses

Dynamic analysis program -

The computer program DRAIN2D was used to perform the non-linear time history analyses. The program is able to analyse two dimensional finite element models of structure. The models in this project were formed from beam-column finite elements. Yielding as a result of excess moment and/or a moment-axial load combination is allowed at the ends of the elements. Yielding from large shear stresses is not explicitly modelled.

DRAIN2D performs a step by step integration of the equations of motion in a manner which does not include any equilibrium balancing iteration between time steps. It was thus important to ensure that the time steps chosen were sufficiently small so that an accurate solution was obtained. This in general was achieved by repeating analyses with the time step halved and checking the results with the previous run. If little difference was observed, then the larger time step was considered to be sufficient for the analysis.

Damping model -

Viscous damping in the analyses was assumed to take the form of Rayleigh damping. That is, the damping matrix is formed by a linear combination of the mass matrix and the initial structure stiffness matrix. This is a usual assumption made by programs which perform step-by-step integration of the equations of motion, but can lead to artificially high amounts of damping to vibrations associated with so called "higher modes". Section 4 discusses how most of the analyses were performed with the equivalent of 5% of critical damping applied to the first two modes, taking note of the implied damping applied to the higher modes.

Hysteretic model -

Yielding in the model was confined to the node points of the elements in flexure. The strength and stiffness characteristics of these points were defined so that the structure behaved in a bi-linear mode under displacements involving inelastic behaviour. Each element was given a strain hardening characteristic so that the structure as a whole, if it developed the capacity design failure mode, which is illustrated in Fig. 2.4 (see page 9), would have a displacement strain hardening ratio of 2½ percent of its elastic stiffness.

Ground motions -

The equivalent static and modal response spectrum analyses were based on the response spectra in the draft code. To make valid comparisons of these with the results of time history analyses it was felt the earthquake ground motion record should have a similar response spectra to the ones in the draft code.

To satisfy this requirement an artificial ground motion was generated with the SIMQKE program[8]. The target response spectrum for the motion was taken from the draft code[1] with a structural ductility factor of 1.0 and a zone factor of 0.85. These values give a peak ground acceleration of 0.34g. The length of the record was effectively 28 seconds. The variation in intensity of motion during this period was controlled by the intensity function, which rose from zero to unity in 3 seconds. It was then held constant at this value to 25.5 seconds, at which point it decreased rapidly.

The target velocity response spectrum is compared with the one achieved after 3 passes through the SIMQKE program in Fig. 2.5. Over most of the range the agreement between these two curves was close. The discrepancy exceeded 15 percent in the period ranges of 1.7 to 1.8, 2 to 2.6, 2.8 to 2.9 and at 4.6 to 4.7 seconds. The artificial ground motion acceleration record is shown in Fig. 2.6.

As a check on the properties of this artificial earthquake ground motion a series of single degree of freedom analyses were made. The oscillator was given bi-linear hysteretic properties with a strain hardening gradient of 2½ percent of the elastic value and an equivalent viscous damping of 5 percent of critical. In the analyses the "elastic stiffness" of the oscillator was

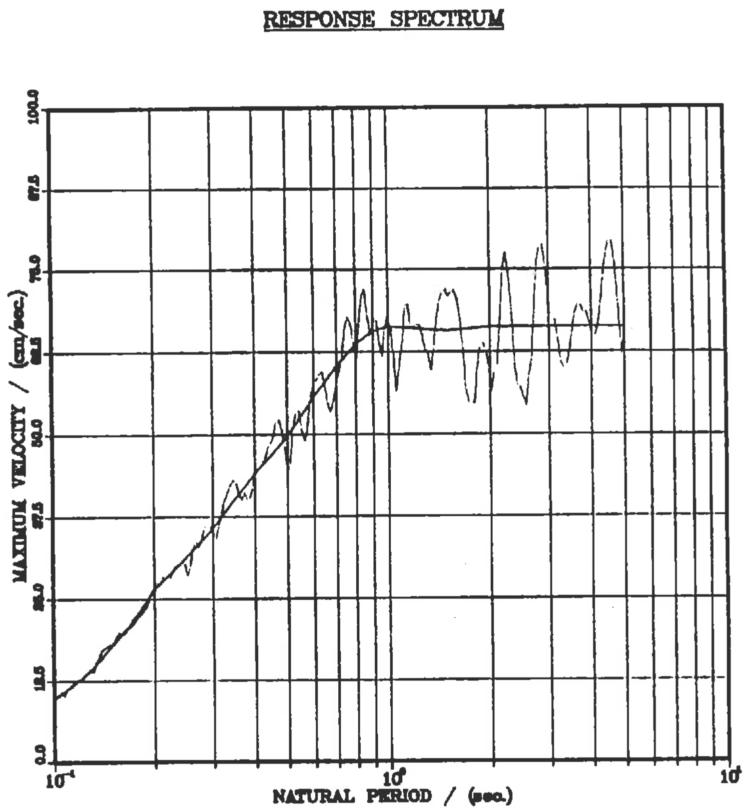


Fig. 2.5 - Velocity Response Spectrum for the Artificial Earthquake

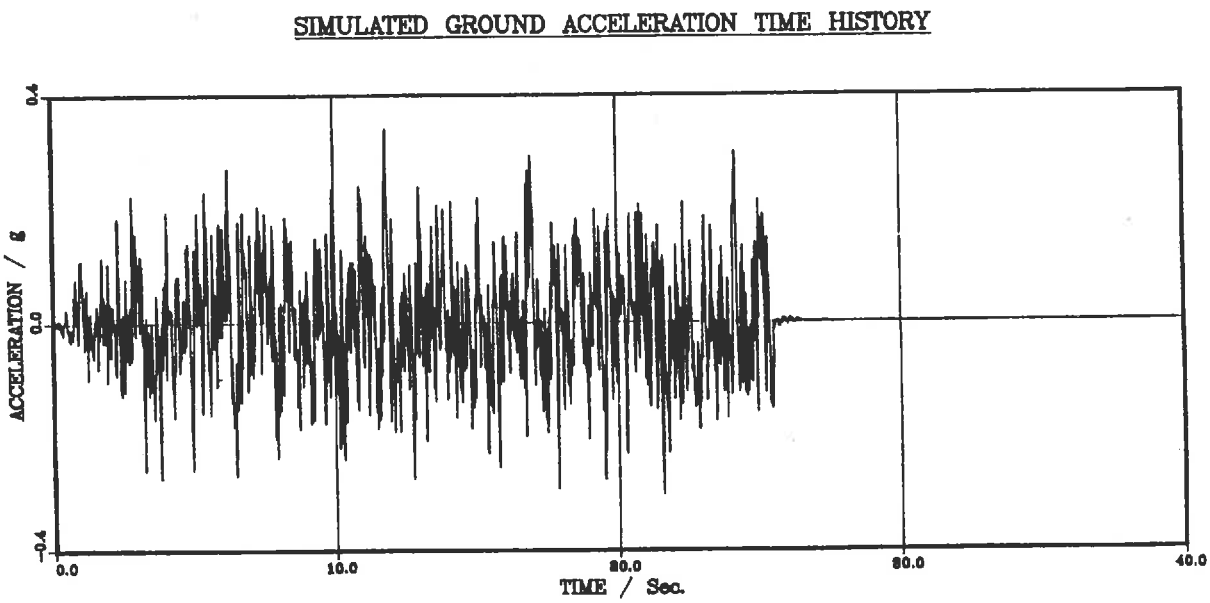


Fig. 2.6 - Ground Acceleration Record for the Artificial Earthquake Motion

changed between individual runs so that the period had values of 0.1, 0.2, 0.5, 0.8, 1.0, 1.25, 1.5, 2.0, 3.0, 4.0, 5.0, 5.4 and 6.0 seconds. For each period the yield strength was varied. By interpolating between the results of the analyses the yield strength corresponding to different displacement ductility demands could be assessed. These results, which are reproduced in Fig. 2.7, give the response spectra for different ductility factors for the artificial earthquake motion. These values can be compared with the corresponding spectra from the draft code[1], which are shown in Fig. 2.8. The close agreement between these two figures indicates that the artificial record reproduced the principal features of the draft code spectra.

In the single degree of freedom analyses the energy flows were examined. The total earthquake energy supplied to the oscillator at any stage is in part stored as elastic and kinetic energies with the remainder being dissipated by yielding, as hysteretic energy and by viscous damping. Some of these results are reproduced in Figs. 2.9 and 2.10. Fig. 2.9 shows that changing the ductility level makes little difference to the total energy dissipated by yielding and viscous damping, and fig 2.10 shows that the energy dissipated by yielding Hysteretic energy) is essentially the same at any given period for ductilities of 4 to 6. The value decreases for ductilities of 2 or less. These findings are in line with previous observations made on single degree of freedom analyses[9.10]. It should be noted that the total energy involved with the artificial record is appreciably greater than the corresponding values obtained for the El Centro N.S. 1940 record[10].

In Fig. 2.11 the response spectral values for ductility factors greater than one from the draft code are compared with the values predicted by the equal displacement concept. It can be seen that for periods greater than 1.2 seconds there is close agreement, and only for periods below about 0.8 of a second does the discrepancy become important.

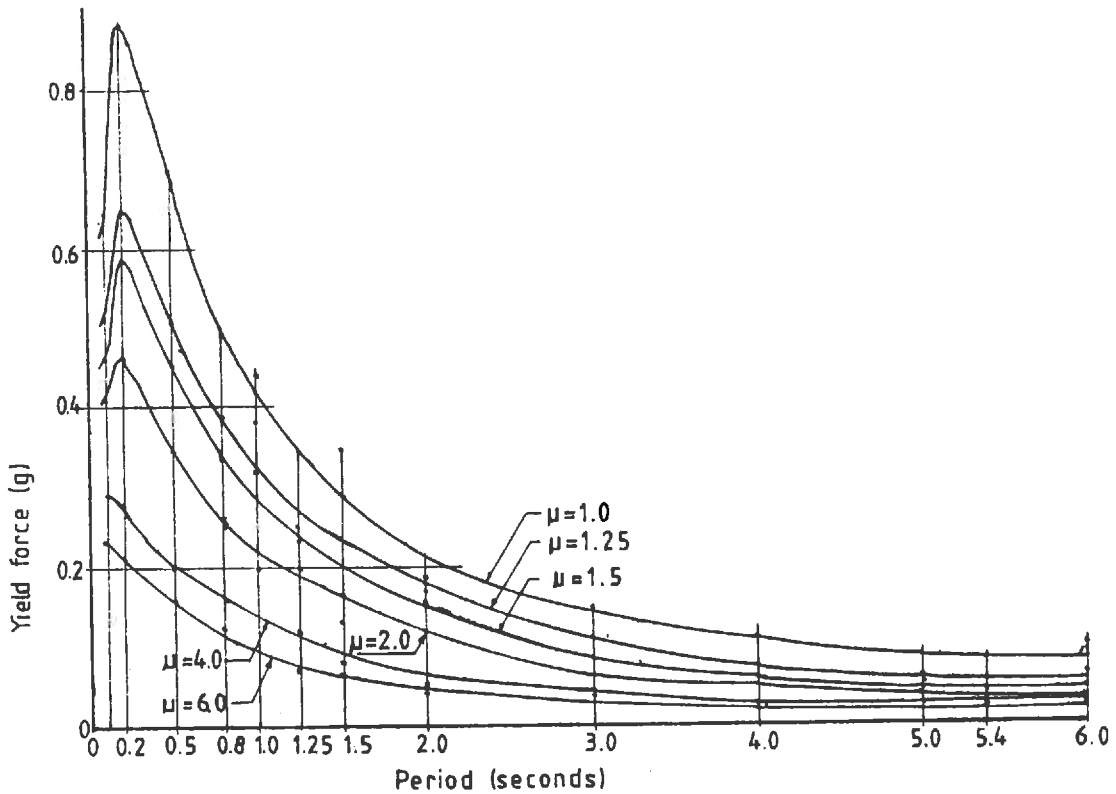


Fig. 2.7 - Response Spectra from Single Degree of Freedom Analyses with Artificial Earthquake Ground Motion

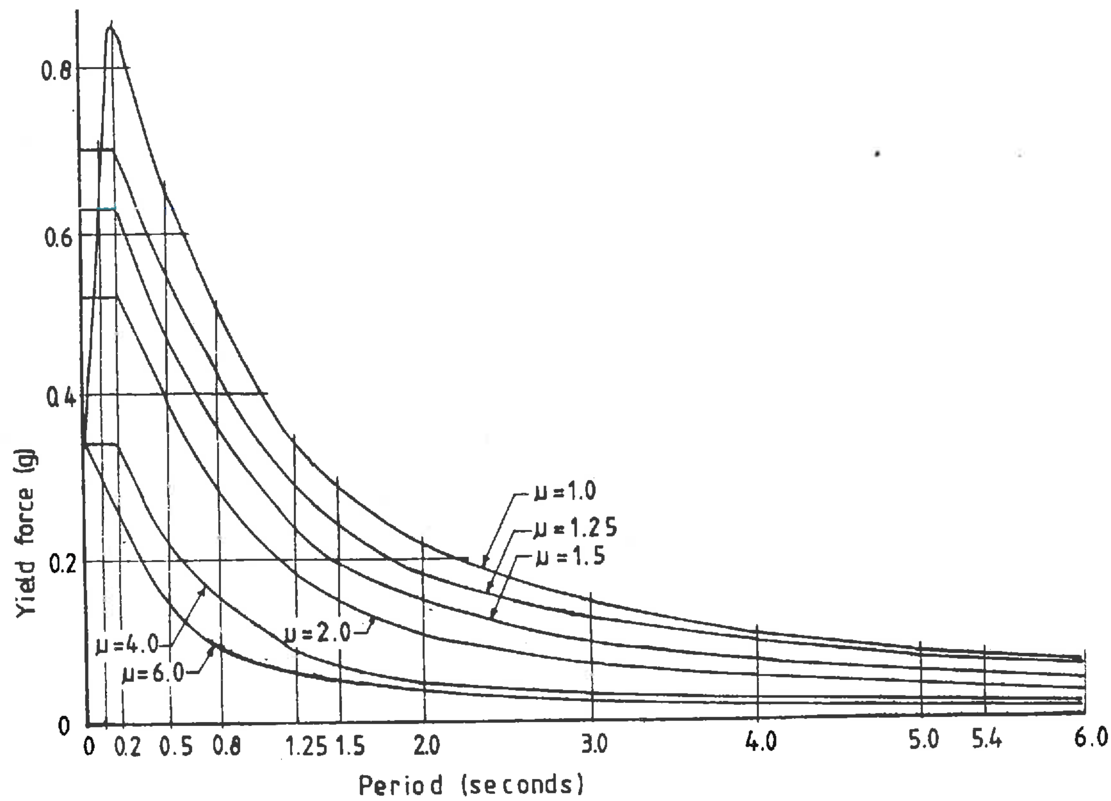


Fig. 2.8 - Response Spectra from Draft Code with Zone Factor of 0.85

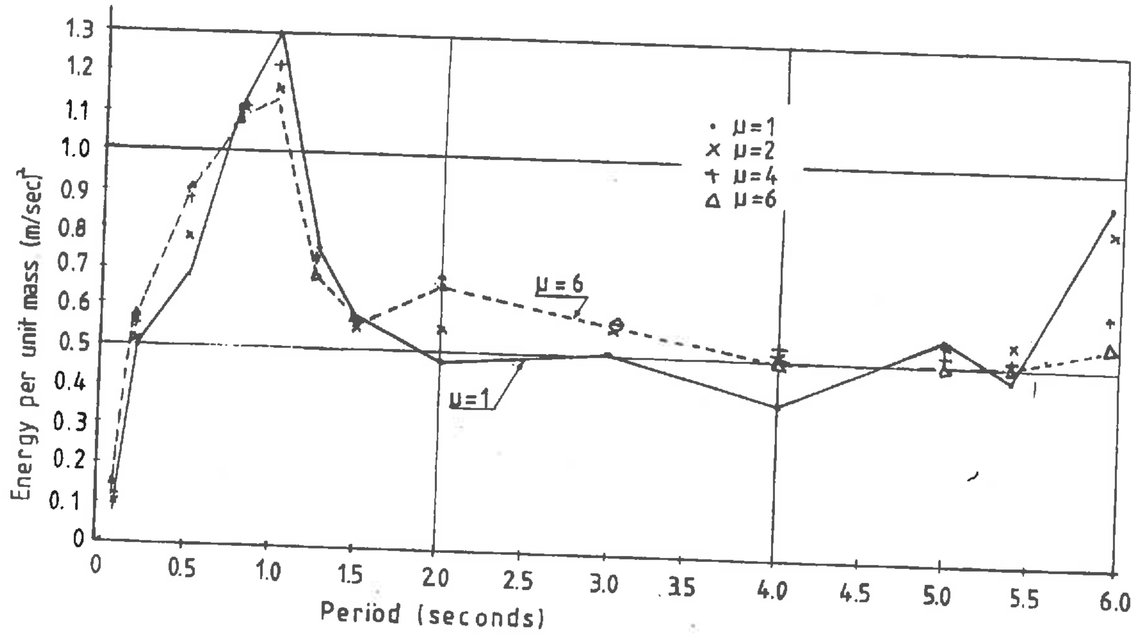


Fig. 2.9 - Earthquake Energy Dissipated in Single Degree of Freedom Analyses for the Artificial Ground Motion

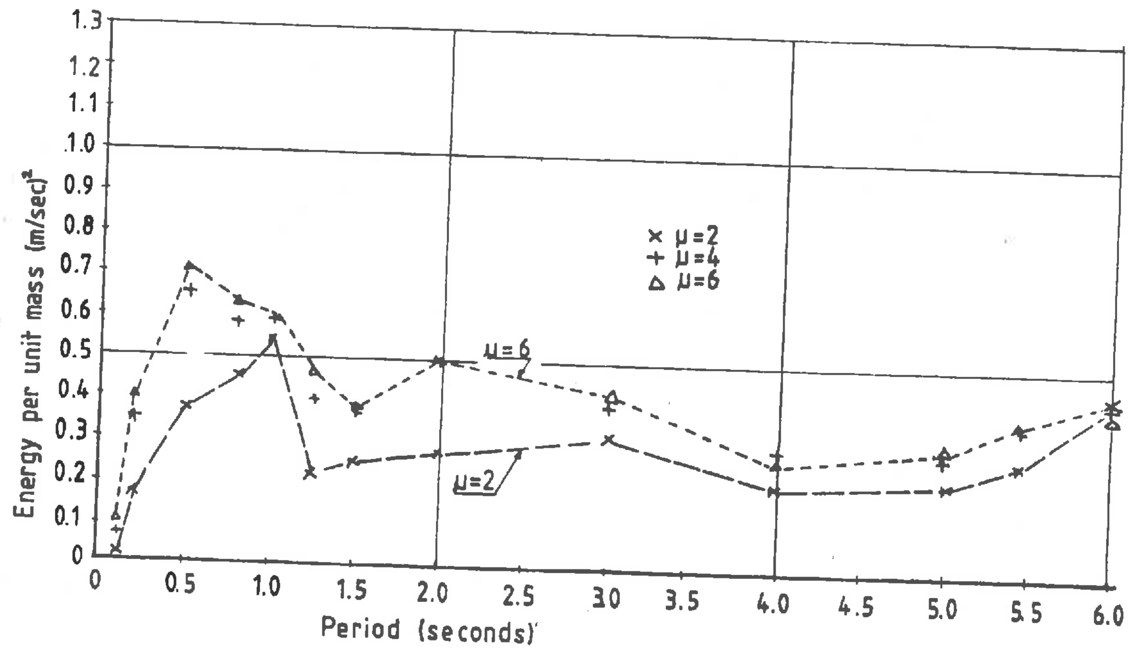


Fig. 2.10 - Hysteretic Energy Dissipated in Single Degree of Freedom Analyses for the Artificial Ground Motion

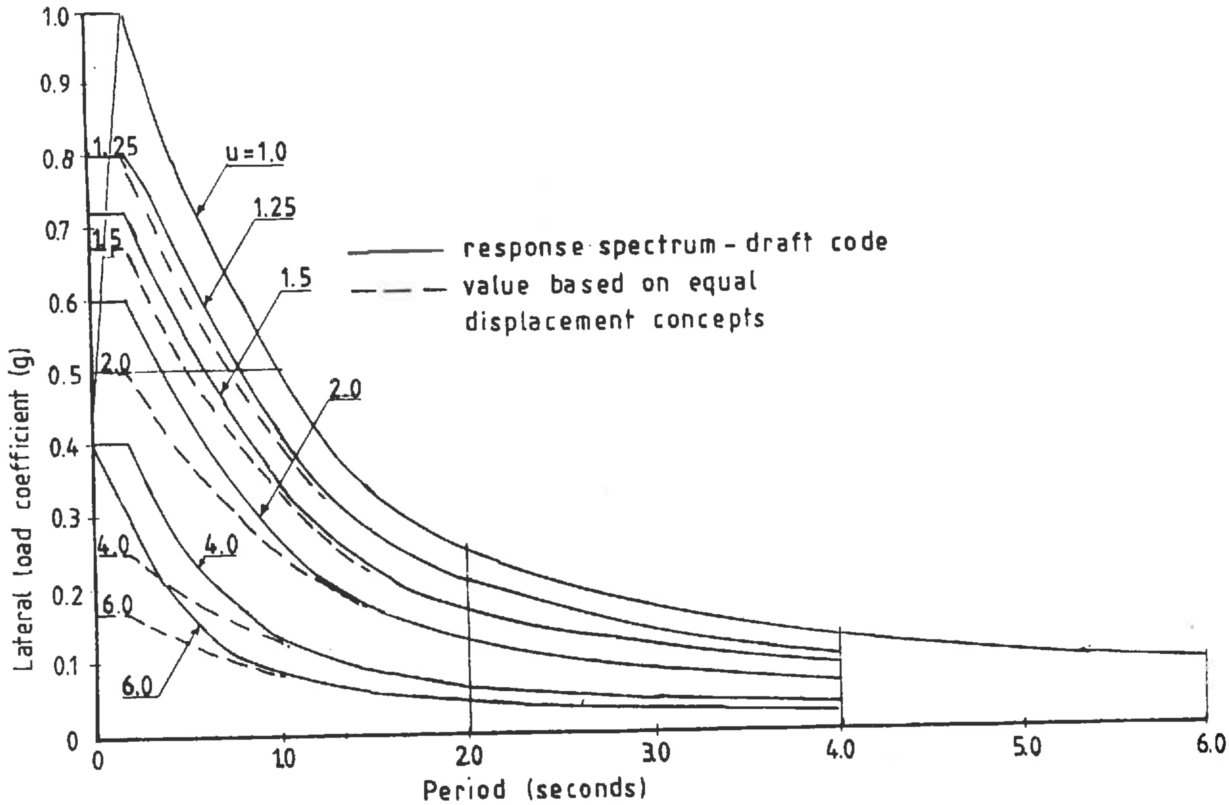


Fig. 2.11 - Response Spectra Curves in DZ 4203 - 86 Compared with Values Predicted by Equal Displacement Concept

Chapter III Results of Equivalent Static and Response Spectral Analyses

3.1 General

In this chapter the results of the equivalent static and modal response spectrum analyses on the regular walls, frames, podium and missing floor structures are compared. The numerical integration time history analyses on some of these structures are reported in chapter 4. In all of the analyses discussed in this chapter linear elastic behaviour was assumed and P-delta effects were neglected.

3.2 Analysis of Regular Walls

Two series of walls were analysed. In both cases the cross-section dimensions were kept constant (see Fig. 2.2) with only the height varying for the 12, 18, 24 and 30 stories. In the first series the moment of inertia of the wall was based on the gross section area from the second floor level up to the top of the structure. For the lower two floors this value was halved (see section 2.2). With the second series the moment of inertia was constant with height and equal to 0.75 times the value based upon the gross section properties. The principal results of the analyses on the walls in these two series are reproduced in Tables 3.1 and 3.2.

From Tables 3.1 and 3.2 it is apparent that there was little difference in the response of the corresponding walls in series one and two, and consequently in the following discussion reference is only made to the second series.

There are a number of marked differences in the structural actions induced in the walls by the equivalent static and modal methods of analysis. One of the most important of these is in the level of base shear. For all but the 12 storey wall the modal base shears are considerably higher than the corresponding equivalent static values, with the discrepancy increasing with the height of the wall (see Table 3.2). The distribution of shear up the

Table 3.1 Principal of Results of Analysis for Regular Walls, - Series I - (I varies)

ITEM		12 storey	18 storey	24 storey	30 storey
		Modal Response Spectrum Analysis			
Periods of vibration (seconds)	T_1	0.96	1.98	3.36	5.10
	T_2	0.15	0.31	0.53	0.80
	T_3	0.06	0.12	0.19	0.29
Base shear (kN)	μ_6 *	785	789	748	667
	μ_1	3 499	3 243	3 254	3 206
ratio μ_1/μ_6					
Base bending moment (kNm)	μ_6	15 574	17 615	19 048	19 430
Deflection at top of wall (mm)	μ_6	24.4	53.1	92.6	140.5
Deflection due to 1st mode contribution only		24.4	53.0	92.3	140.1
Base mode shear as a preparation of total Base shear for μ_6	1 **	0.64	0.47	0.39	0.35
	2	0.71	0.76	0.73	0.63
	3	0.27	0.38	0.45	0.51
		Equivalent Static Analysis			
Base Shear kN	μ_6	740	552	442	362
	μ_1	4 517	3 286	2 663	2 131
Base bending moment (kNm)	μ_6	21 686	22 137	25 432	25 984
Deflection at top of wall (mm)	μ_6	35	77	136	208

* μ_6 - based on response spectrum for structural ductility factor of 6

** mode 1 - 1st etc

Table 3.2 Principal Results of Analysis for Regular Walls, Series II - (Uniform I)

ITEM		12 storey		18 storey		24 storey		30 storey	
		Modal Response Spectrum Analysis							
Periods (sec) and proportion of participating mass.	T_1	0.90	0.65	1.96	0.63	3.42	0.63	5.29	0.63
	T_2	0.16	0.21	0.33	0.20	0.56	0.20	0.86	0.20
	T_3	0.07	0.07	0.13	0.07	0.21	0.07	0.32	0.07
(Mass for modes 1 to 6)			0.99		0.98		0.97		0.96
Base shear (kN)	μ_6	797		791		740		660	
	μ_1	3352		3250		3240		3120	
ratio μ_1/μ_6		4.21		4.11		4.38		4.86	
Base bending moment (kNm)	μ_6	16 188		18 093		19 160		19 199	
Deflection at top of wall (mm) RMSS	μ_6	22.9		52.8		94.5		146.4	
1st Mode Contribution		22.9		52.7		94.2		145.9	
Base mode shear* as a proportion of total base shear	1	0.63		0.45		0.37		0.34	
	2	0.70		0.76		0.71		0.62	
	3	0.35		0.38		0.46		0.53	
		Equivalent Static Analysis							
Base shear (kN)	μ_6	778		562		435		350	
	μ_1	4418		3341		2622		2061	
Base bending moment (kNm)	μ_6	22 836		24 400		25 025		25 115	
Deflection at top of wall (mm)	μ_6	31.7		77.0		143.6		228.0	

* These values are combined by the SRSS method hence the direct sum is greater than 1.

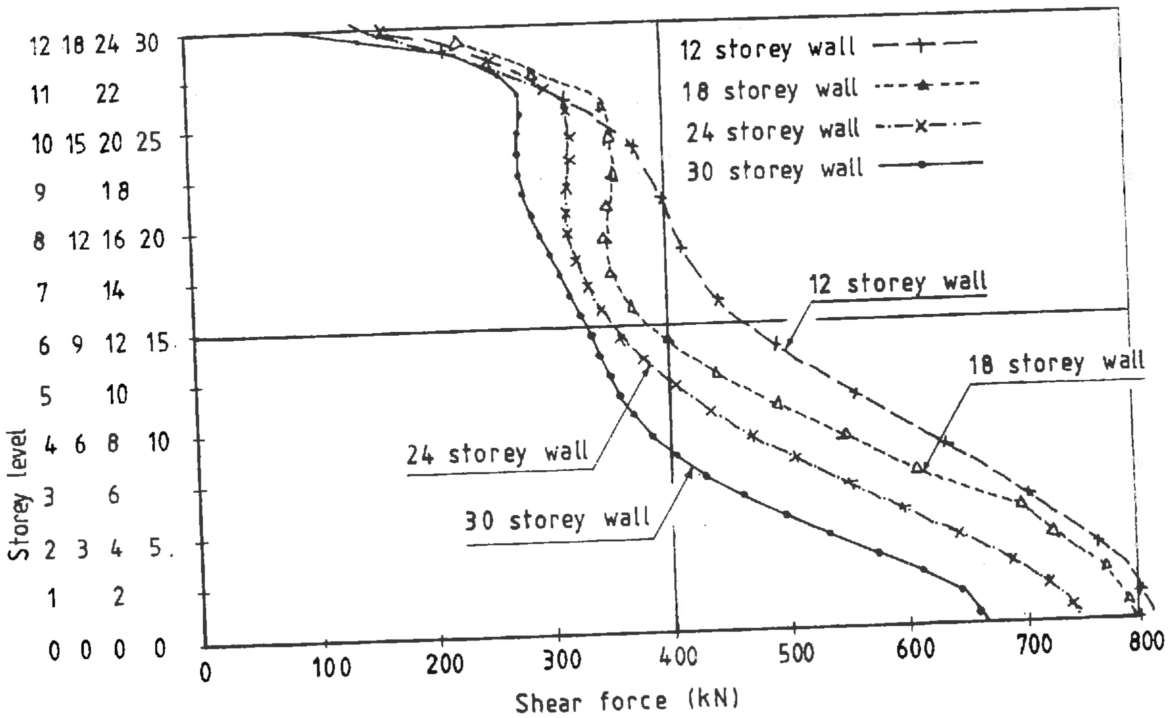


Fig. 3.1 - Response Spectrum Shears in Regular Walls

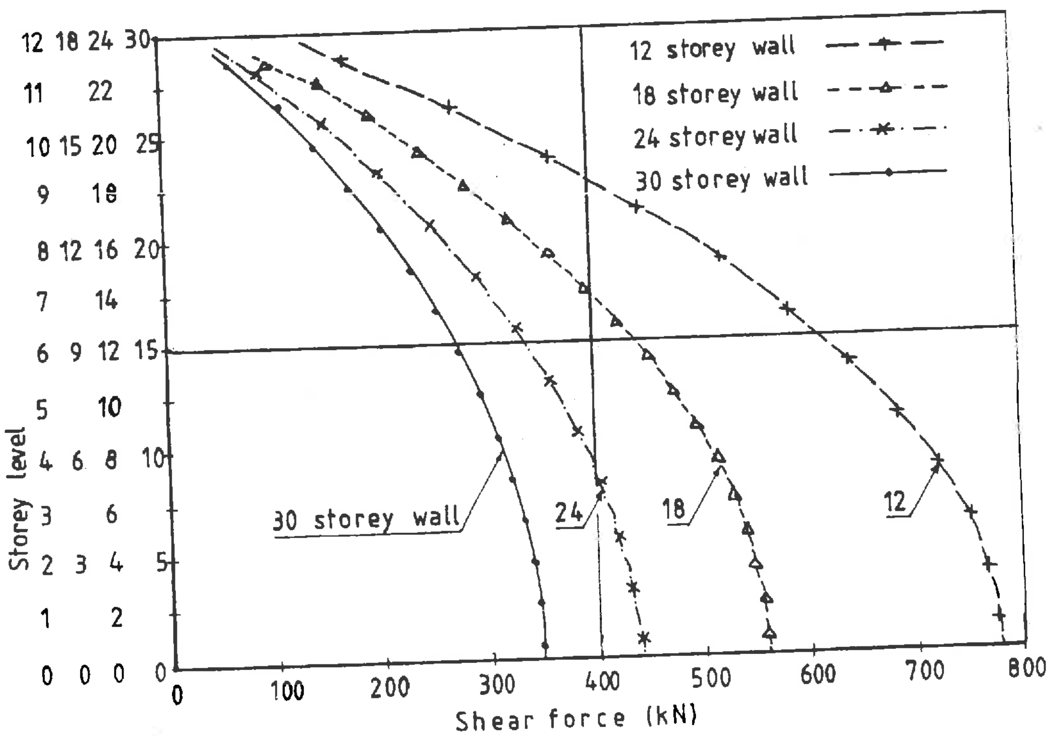


Fig. 3.2 - Equivalent Static Shears in Regular Walls

walls predicted by the two methods shows considerable differences as indicated in Figs. 3.1 and 3.2. This is due to the higher mode (2nd, 3rd and 4th) contributions in the modal response spectrum analyses and it is discussed in a later paragraph.

The bending moments in the walls corresponding to the two methods of analysis are reproduced in Figs. 3.3 and 3.4. For this action the equivalent static method induces base overturning moments which are 30 to 40 percent in excess of the corresponding modal response spectrum values. There are also significant differences in the distribution of bending moments, with the modal response spectrum values being appreciably higher in the upper one third of the 18, 24 and 30 storey walls than the equivalent static values.

The difference in the bending moments corresponding to the two methods of analysis is reflected in the maximum deflections at the top of the wall. The equivalent static values are between 38 and 56 percent higher than the corresponding modal values, with the discrepancy increasing with the height of the building.

To explore the reasons for the different values of shear and bending moment predicted by the two methods of analysis the contributions of the different modes to these actions in the 24 storey wall are plotted in Figs. 3.5 and 3.6. From Fig. 3.5 it can be seen that the bending moments in the lower third of the wall are dominated by the contribution of the first mode, with the second mode making an important contribution for the top half of the wall. With respect to the wall shear forces the second mode makes a major contribution in the lower and upper thirds of the wall, as shown in Fig. 3.6, with the third mode making significant contribution at the mid height. It should also be noted from Table 3.2 that the deflection at the top of the wall is not significantly influenced by the contribution from the higher modes.

The equivalent static method of analysis for a uniform structure gives a conservative estimate of the first mode effects but it does not represent higher mode effects and hence the reason for the discrepancy between the two approaches.

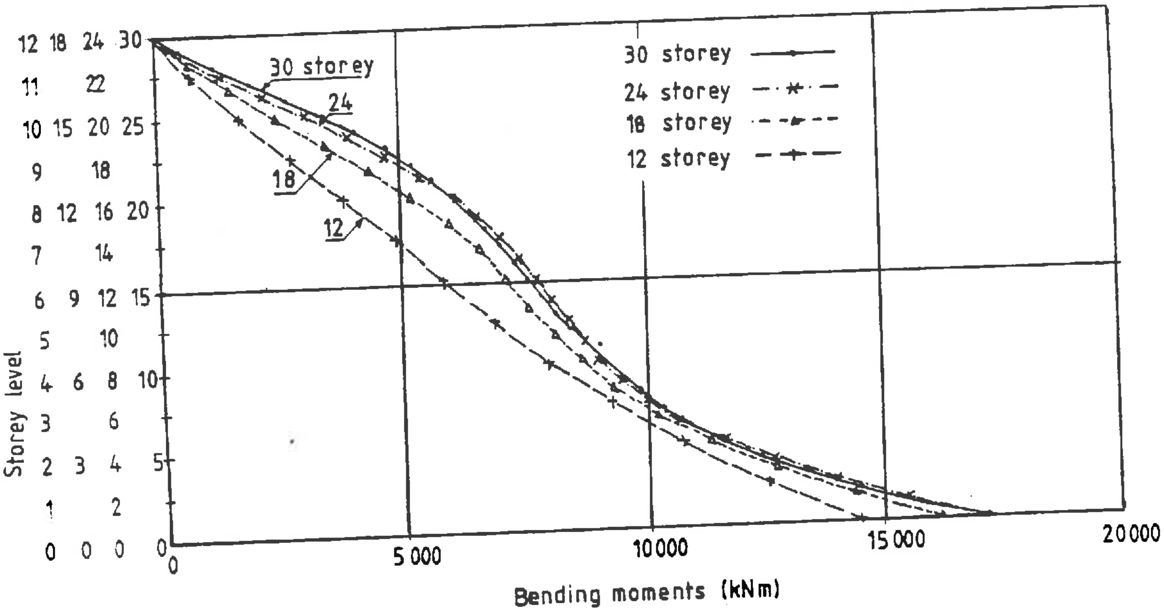


Fig. 3.3 - Bending Moments in Walls with the Response Spectrum Analysis

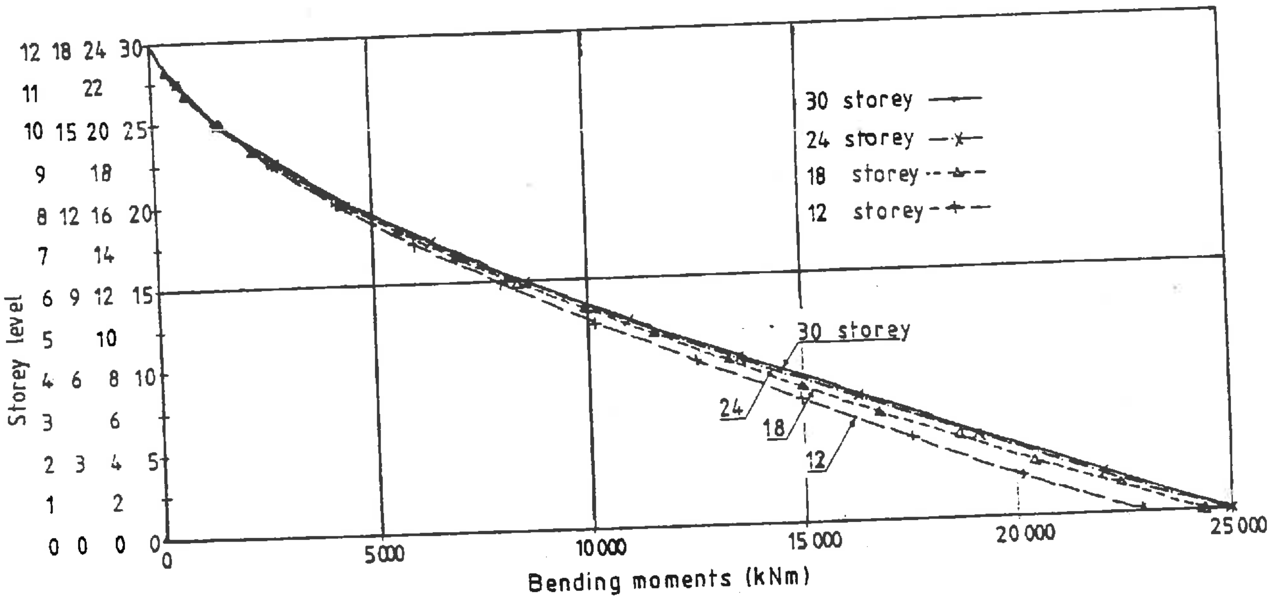


Fig. 3.4 - Bending Moments in Walls with the Equivalent Static Analysis

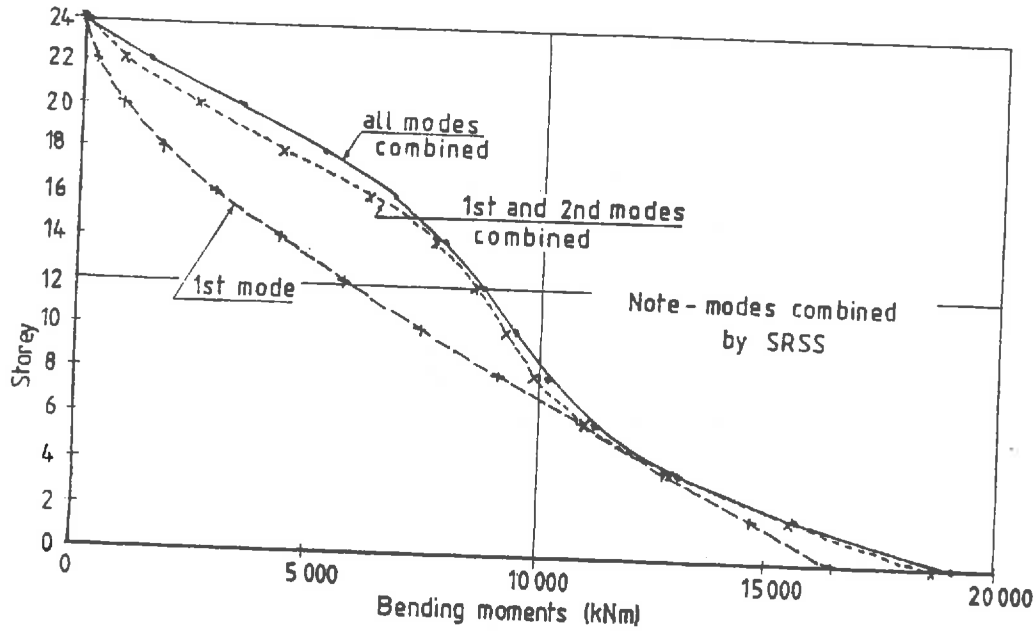


Fig. 3.5 - Bending Moments in 24 Storey Wall found from the Modal Response Spectrum Analysis

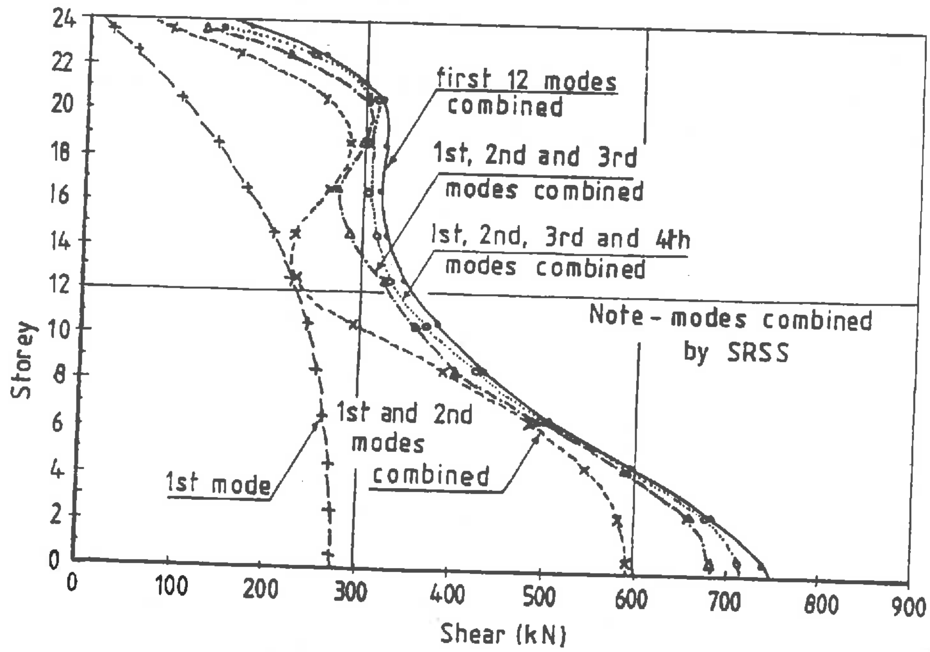


Fig. 3.6 - Shear Forces in 24 Storey Wall found from the Modal Response Spectrum Analysis

It should be noted that the higher modes have short periods, and in this zone of the response spectrum there has been a marked increase in acceleration when compared with the spectrum in the current loadings code NZS 4203 - 84. This change results in a greater contribution arising from the higher modes than was previously the case. Clearly with the draft code spectrum scaling the actions in a wall so that the base shear was equal to 90 percent of the equivalent static value could lead to dangerously low design bending moments in the walls.

The proportion of the weight, "W_e", participating in the response spectrum analysis, in each mode may be found from
from

$$W_e = \frac{\left(\Sigma\, W_x\, \delta_x\right)^2}{\Sigma\left(W_x\, \delta_x^{\,2}\right)} \qquad (3.1)$$

where W_x is the seismic weight of floor x and δ_x is the lateral displacement component of the mode shape of the floor. The participating weights in each mode for all the modes should sum to unity. For this series of walls 96% or more of the participating weight was incorporated in the first three modes. The participating proportion of the weight in each mode is listed in Table 3.2.

3.3 Analysis of Regular Frames

The rectangular frames were intended to provide lateral resistance but not to provide appreciable gravity support to the floors as described in section 2.1. They were sized for 6, 12, 18 and 24 storey frames.

Key values from the equivalent static and modal response spectrum analyses for these frames are summarised in Table 3.3. As in the case of the walls the equivalent static approach yields higher deflections and base overturning moments than the modal method, with the difference in the deflections ranging from 26 percent for the 6 storey frame to 37 percent for the 24 storey frame. The corresponding increase in base overturning moments ranged from 20 to 35 percent.

Table 3.3 Principal Results of Analysis for Regular Frames.

ITEM		6 storey		12 storey		18 storey		24 storey	
		Modal Response Spectrum Analysis							
Periods (sec) and proportion of participating mass	T_1	1.40	0.82	2.52	0.80	2.99	0.77	3.37	0.76
	T_2	0.45	0.10	0.82	0.10	1.02	0.11	1.16	0.12
	T_3	0.25	0.04	0.46	0.04	0.59	0.40	0.67	0.04
(Modes 1 to 6)			1.00		0.96		0.96		0.96
Base shear (kN)	μ_6 *	480		525		649		781	
	μ_1	3 165		3 008		3 815		4 578	
ratio μ_1/μ_6		5.95		5.73		5.88		5.86	
Base bending moment (kNm)	μ_6	6 600		13 130		23 680		37 290	
Deflection at top of frame (mm)	μ_6	32.5		58.8		71.3		84.0	
	μ_1	193		346		432		506	
1st mode contribution to deflection	μ_6	32.3		58.4		70.6		83.1	
+Mode base shear as a proportion of total base shear (μ=6)	1	0.91		0.89		0.87		0.85	
	2	0.34		0.33		0.35		0.38	
	3	0.19		0.22		0.23		0.22	
		Equivalent Static Analysis							
Base shear (kN)	μ_6	532		585		731		879	
	μ_1	3 165		3 447		4 434		5 306	
Base bending moment (kNm)	μ_6	7 920		17 000		31 510		50 310	
Deflection at top of frame (mm)	μ_6	41.1		77.1		96.2		115	

* μ_6 values for response spectrum corresponding to structural ductility factor of 6.

\+ see footnote to Table 3.2

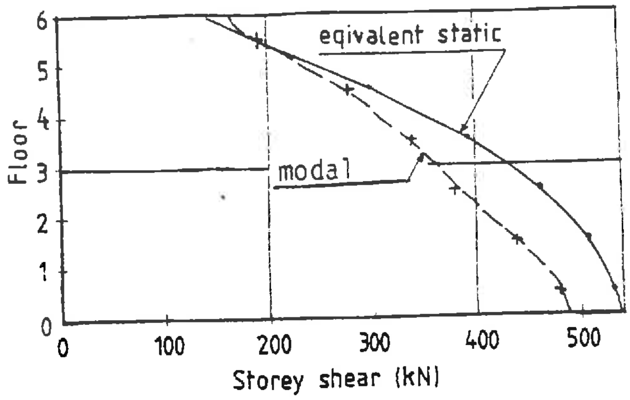


Fig. 3.7 - Storey Shears in 6 Storey Frames

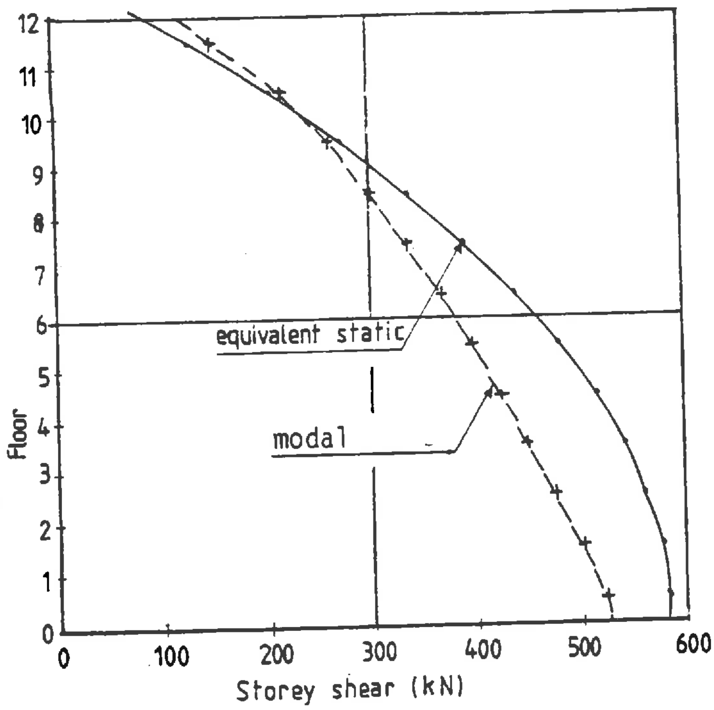


Fig. 3.8 - Storey Shears in 12 Storey Frames

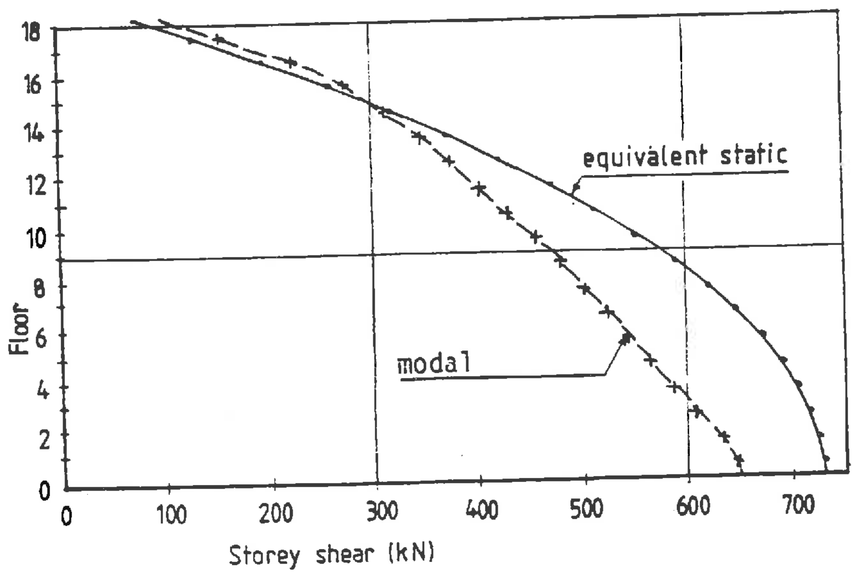


Fig. 3.9 - Storey Shears in 18 Storey Frames

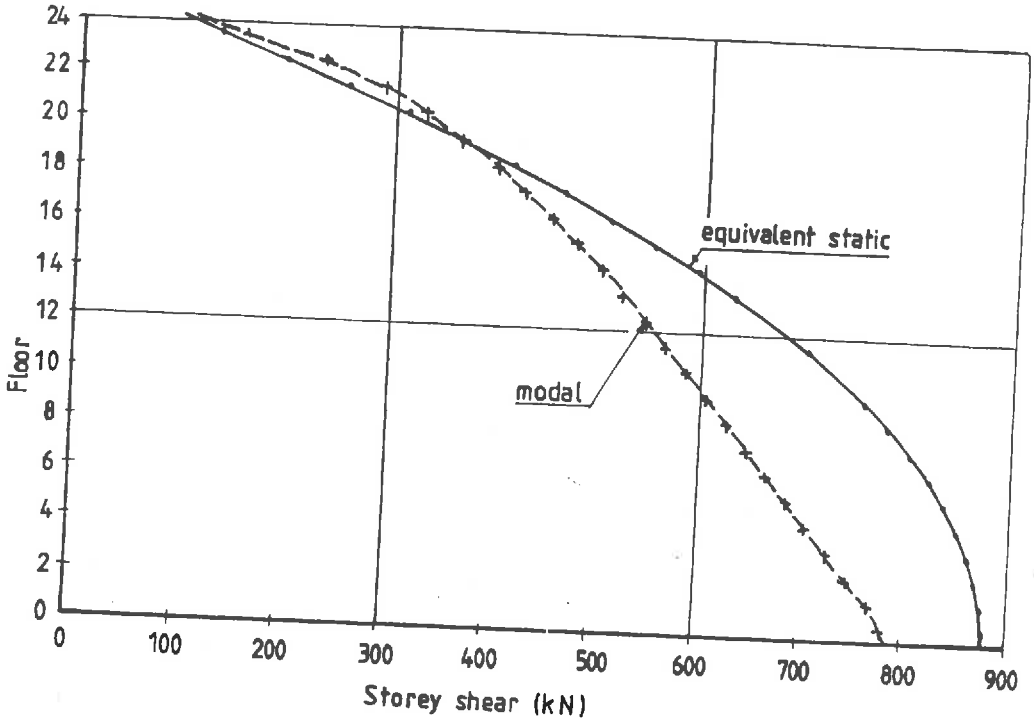


Fig. 3.10 Storey Shears in 24 Storey Frames

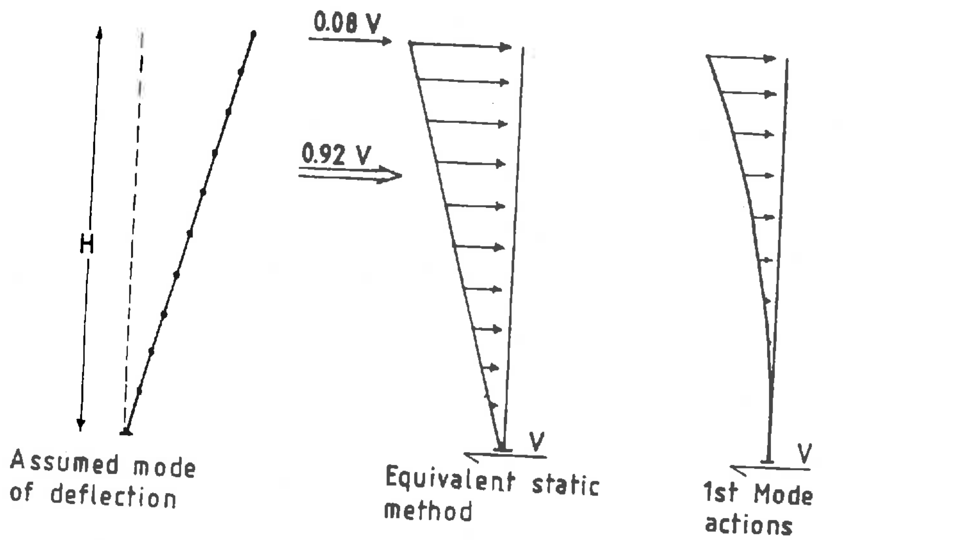


Fig. 3.11 Simplified 1st mode actions compared with Equivalent Static Values

The storey shears resulting from the two methods of analysis are reproduced in Figs. 3.7 to 3.10. It can be seen that the equivalent static approach gives higher shears than the modal response spectrum method except in the top few levels of the 12, 18 and 24 storey frames. The agreement between the two approaches is considerably better than for the walls.

The seismic weight participating in each mode is expressed as a proportion of the total seismic weight is given in Table 3.3. The first three modes account for 92 percent or more of this weight in all cases.

3.4 Comparison of Equivalent Static and Response Spectrum Modal Methods of Analysis for Regular Walls and Frames

In Tables 3.1 and 3.2 for the regular walls and Table 3.3 for the regular frames the deflections at the top of the structures for the modal method are given together with the contribution of the first mode. It is clear that the higher modes made only a minor contribution to the deflection. In comparison the equivalent static approach consistently predicts values in excess of the modal values, and the same trend is apparent in the base overturning moments.

The overestimates of the maximum deflections and base overturning bending moments of the equivalent static method compared with the modal approach can be explained in terms of the deflected shape and mass participating in the first mode. In the modal analyses both the maximum deflection and the base overturning moment depend principally on the first mode contribution. If the form of the deflected profile is assumed to be linear, with rotation occurring at the base, as illustrated in Fig. 3.11, the contributing mass can be found. Such a profile is a reasonable approximation to the first mode shapes of regular walls and frames. It is also assumed that the mass of the structure is uniformly distributed over its height.

In Fig. 3.11 the equivalent static actions, as defined in the draft code[1], are reproduced. For a given fundamental period and structural ductility factor the base shear V, can be determined as

$$V = C_{o,}(T,\mu)\ W_t \tag{3.2}$$

where W_t is the seismic weight of the building and $C_o(T,\mu)$ is the response spectrum value corresponding to the structural ductility factor and the period+. With this distribution of loading the base overturning moment, M_b, is given by -

$$M_b = 0.693\ C_o\ (T,\mu)\ W_t\ H \tag{3.3}$$

where H is the total height of the structure.

These values can be compared with the corresponding modal response spectrum values associated with a first mode response. If it is assumed that the first mode shape is given by the displacements shown in Fig. 3.11, and that the mass is uniformly distributed up the structure then the equivalent seismic weight of the mode is given by a modification of equation 3.1 as

$$W_e = 0.75\ W_t. \tag{3.4}$$

From this value the base shear for this mode can be found from the response spectrum to be

$$V = 0.75\ C_o\ (T,\mu)\ W_t \tag{3.5}$$

and the overturning moment is given by

$$M_b = 0.5\ C_o\ (T,\mu)\ W_t\ H \tag{3.6}$$

For the buildings of a few floors equations 3.4 and 3.5 do not hold as the mass can not be assumed to be uniformly distributed. However, the trends indicated are still valid.

Based on these values the ratio of the equivalent static to the first mode contribution for the base bending moments is 1.39. As deflections are proportional to bending moments the same ratio could be expected to apply to deflections. Table 3.4 shows that this ratio does apply for the uniform mass structures.

\+ For the purpose of this comparative discussion, the zone factor Z (0.85) has not been included

Table 3.4 Base moments and maximum deflections in regular walls and frames

STRUCTURE	Response Spectrum: Resultant values			Response Spectrum: 1st Mode/Total			Equivalent Static Resultant Values			Ratios of Equivalent Static to Modal		
	V_t	M_t	Δ_t	$\frac{V_1}{V_t}$	$\frac{M_1}{M_t}$	$\frac{\Delta_1}{\Delta_t}$	V_s	M_s	Δ_s	$\frac{V_s}{V_t}$	$\frac{M_s}{M_t}$	$\frac{\Delta_s}{\Delta_t}$
WALLS												
12 Storey	797	16 190	22.9	0.63	0.96	1.0	778	22.840	31.7	0.98	1.41	1.38
18 Storey	791	18 090	52.8	0.45	0.90	1.0	562	24 400	77.0	0.71	1.35	1.46
24 Storey	740	19 160	94.5	0.37	0.86	1.0	435	25 030	143.6	0.59	1.31	1.52
30 Storey	660	19 200	146.4	0.33	0.92	1.0	350	25.120	228.0	0.53	1.31	1.56
FRAMES												
6 Storey	480	6 600	32.5	0.91	0.94	0.99	532	7 920	41.1	1.11	1.20	1.26
12 Storey	525	13 130	58.8	0.89	0.98	0.99	585	17 000	77.1	1.11	1.29	1.31
18 Storey	648	23 680	71.3	0.87	0.99	0.99	731	31.510	96.2	1.13	1.33	1.35
24 Storey	781	37 290	84.0	0.85	0.99	0.99	879	50 310	115.0	1.13	1.35	1.37

V = base shear, M = over turning moment, Δ = maximum deflection subscripts, t = combined values,
1 = 1st mode values, s = equivalent values

For regular frame structures a major part of the deflection comes from the storey shears, consequently the 1.39 ratio could not be expected to apply to the same extent. Table 3.4 shows just how closely this approximate ratio does account for the observed variation in the predictions of these two methods of analysis. Also shown in this table are the proportions of the overturning moment and peak deflections contributed by the first mode to the resultant modal action.

3.5 The Irregular Frames

Two different forms of structures were investigated in this group. In the first of these three storey podiums were added to the 12 and 24 storey regular frame structures to give 9 and 21 storey towers above the podiums. The lateral load resistance of the podiums was provided by walls as described in section 2.1. In the second group of irregular frames the bottom two floors of the 12 and 24 storey frames were removed to give 10 and 22 storey structures. In both cases the first floor level became 3 x 3.4m above the base. Two different cases were considered with this missing floor structures. In the first case the columns to the first floor level and the first floor beam were stiffened so the the fundamental period was close to that of the initial regular frame structures, while in the second case the member sizes were unchanged. This alternative gave an impractical soft storey structure, which was seen as an extreme limiting case for the two methods of analysis. The details of the structures are given in section 2.1.

(a) Podium and Tower Structures

The equivalent static method of analysis is not directly applicable to this form of structure due to its vertical irregularity. The base shear contribution of the mass connected with the podium causes conservative forces to be applied to the tower, but unconservative forces on the podium. The ATC-3-1978*[3] recommendations suggested the equivalent static method (ELF equivalent linear force in ATC-3) could be modified and applied to these structures where the effective interstorey lateral stiffness of the podium was five or more times the corresponding stiffness of the tower.
The suggested approach contains three steps.

*and later editions

(i) The tower is subjected to an equivalent static analysis assuming its base is at the top of the podium.

(ii) The podium is considered as a separate structure (without the tower) and analysed by the equivalent static method.

(iii) The final design actions are determined by adding together the results of these two analyses. It should be noted that the periods of vibration for the tower and the podium, which are used in assessing the lateral forces, will be very different for each component.

For both the tower and the podium structures analysed in steps i and ii above the Rayleigh method may be used for calculating the period needed to determine the base shear.

Some of the key results of the modal response spectrum analyses of these two structures are reproduced in Table 3.5 together with the corresponding values from the equivalent static method and the modified equivalent static method based on ATC-3. From the modal response spectrum values it can be seen that two distinct modes do make major contributions to the resultant actions. One of these is the first mode and the second one is a very much higher mode, which represents an effective first mode contribution from the podium acting by itself. These modes are the 7th and 12th for the 12 and 24 storey structures respectively.

The proportion of participating mass in each mode is listed in Table 3.5. It can be seen that to reach an accumulated 90 percent level the 12 storey structure needs 7 modes considered and the 24 structure correspondingly requires 13 modes. This demonstrates the need for calculating the mass participating in each mode. Without these values the very significant contributions of the higher modes could easily be overlooked.

From Table 3.5 it can be seen that the equivalent static method over estimates the actions in the tower but under estimates the values in the podium when compared with the modal analysis. The modified approach based on ATC-3[3] leads to a good estimate of the actions in the tower, but appreciably overestimates the values in the podium.

Table 3.5 Principal Results of Analyses for Podium - Tower Structures

ITEM	12 Storey		24 Storey	
	Modal Response Spectrum Analysis (μ=6)+			
Periods of vibration (seconds) T_1 and proportion of participating T_2 mass of structure - T_3 (mode no. in brackets)	(1) 1.02	0.352	(1) 3.04	0.489
	(2) 0.61	.047	(2) 1.03	0.076
	(3) 0.34	.021	(3) 0.59	0.027
	(4)&(5) -	0.035	(4-10) -	0.088
	(6) 0.12	.063	11) 0.11	0.050
	(7) 0.10	0.374	12) 0.10	0.131
	(8) 0.09	0.041	13) 0.09	0.066
Storey shears - base of tower	572		788	
(μ = 6) - base of podium	4140		2809	
Base shears for different modes - (mode no. in brackets)	(1) 473		(1) 647	
	(2) 200		(2) 301	
	(3) 139		(3) 188	
	(6) 637		11) 328	
	(7) 3 963		12) 2 192	
	(8) 445		13) 1 116	
Deflection at top of frame (mm)	44.8		73.9	
Deflection at top of podium	.07		.05	
	Equivalent Static Methods			
Periods - tower mode	T_t = 1.92		T_t = 3.03	
- podium mode	T_p = 0.10		T_p = 0.10	
	DZ 4203 : 86*			
storey shear - base of tower	1035		1230	
- base of podium	1344		1323	
deflections - top of tower	197		140	
- top of podium	0.3		0.2	
	Equivalent Static - based on ATC-3**			
storey shears - base of tower	577		843	
- base of podium	6 610		6 912	
deflections - top of tower	58		101	
- top of podium	1.1		1.1	

\+ = Analysis mode for structural ductility factor 6.0

* = This method is applied only for scope of comparison

** = Method based on ATC - 3, but using DZ 4203 : 86 specified equivalent static loadings.

Missing Floor Frames

The key results from the modal response spectrum and the equivalent static methods of analysis are reproduced in Table 3.6. Of all the groups of structures considered the agreement between the two methods of analysis is the closest for this group. This agreement is also evident in Figs. 3.12 to 3.13 where the storey shears resulting from the two methods of analysis are compared. Removing the two lower floors results in a large portion of the lateral deflection occurring in the lower two levels in the first mode, and this leads to a very high proportion of the mass participating in this mode. Given this first mode deflected shape and the dominance of this mode it is not surprising that the two approaches are in such close agreement.

3.6 Summary of Significant Differences between Methods of Analysis

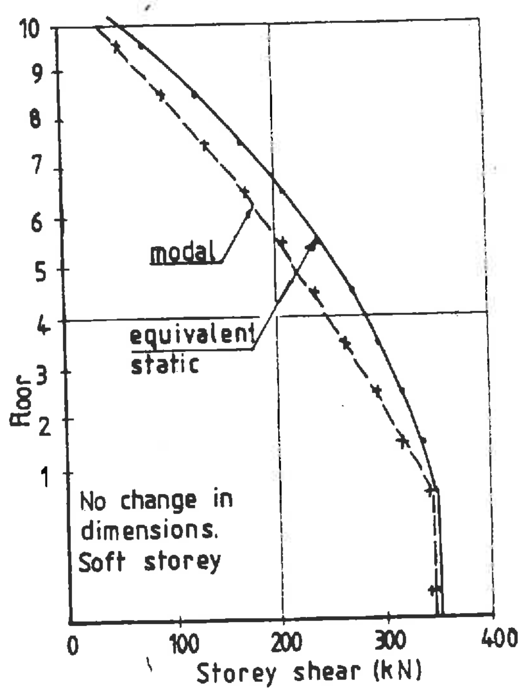
With the podium tower structures the equivalent static method approach is unacceptable and the modified equivalent static method suggested in ATC-3[3] appears to give unduly conservative results for the podium section of the structure. For these structures a modal response spectrum analysis appears to be desirable.

For the uniform wall and frame structures it can be seen that the deflection at the top of the structure is contributed almost entirely by the first mode. As the equivalent participating mass in this mode is about 75 percent of the mass used in calculating the equivalent static response it is not surprising that the equivalent static deflections are always considerably in excess of the modal values. Conservatively the equivalent static value for the structures investigated could be multiplied by 0.85.

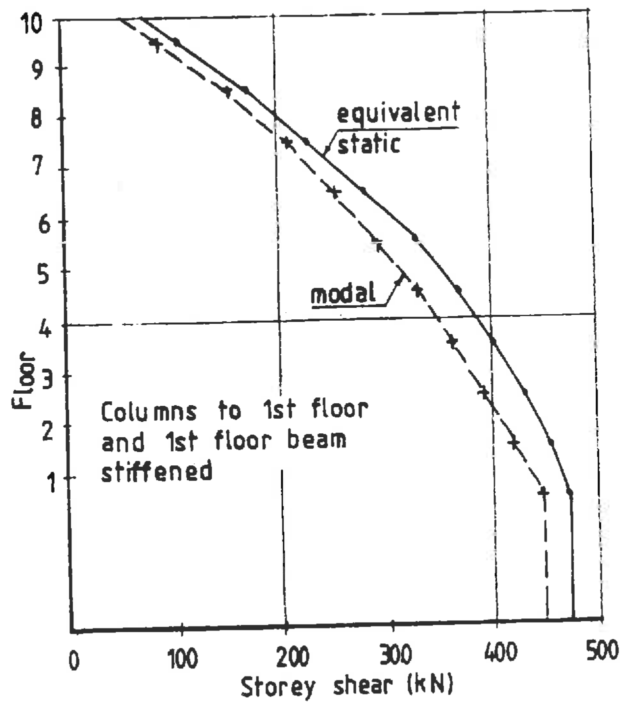
For the wall structures the contribution of the second and higher modes becomes important as far as the wall bending moments and storey shears are concerned for the 18 storey wall. For this structure the modal base shear is 43 percent in excess of the equivalent static value, while the overturning moments are appreciably less than the equivalent static value. As far as the storey shears are concerned the second and higher mode effects are clearly significant for the 18 storey wall but not the 12 storey wall. On this basis it appears that the limit of where the equivalent static method can be applied

Table 3.6 Principal Results of Analysis of Missing Floor Frames

ITEM		10 Storey				22 Storey			
		unstiffened		stiffened		unstiffened		stiffened	
		Modal Response Spectrum Analysis							
Periods of vibration	T_1	3.55	0.976	2.61	0.925	3.81	0.888	3.36	0.821
(seconds) +	T_2	0.93	0.021	0.83	0.063	1.30	0.088	1.16	0.118
proportion of participating mass	T_3	0.47	0.002	0.45	0.009	0.71	0.015	0.67	0.034
Base shear	μ_6	341		446		661		739	
(kN)	μ_1	2058		2637		3941		4420	
Deflection at top of frame (mm)	μ_6	74.1		58.7		91.1		83.1	
		Equivalent Static Analyses							
Base shear	μ_6	348.8		472		709		809	
Deflection at top of frame (mm)	μ_6	81.5		67.5		111.3		105.6	

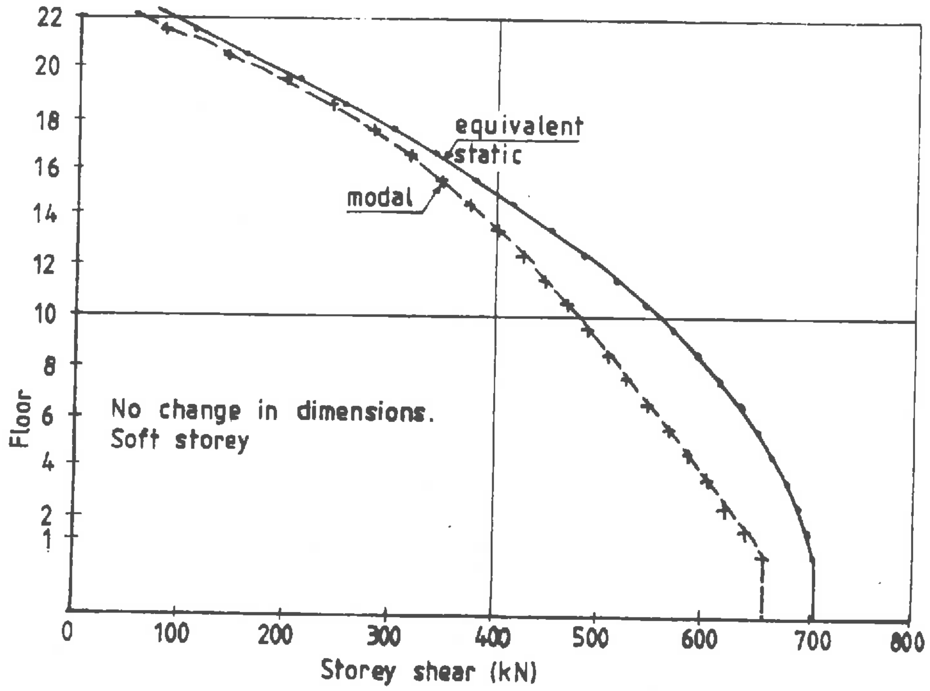


(a) Soft Storey

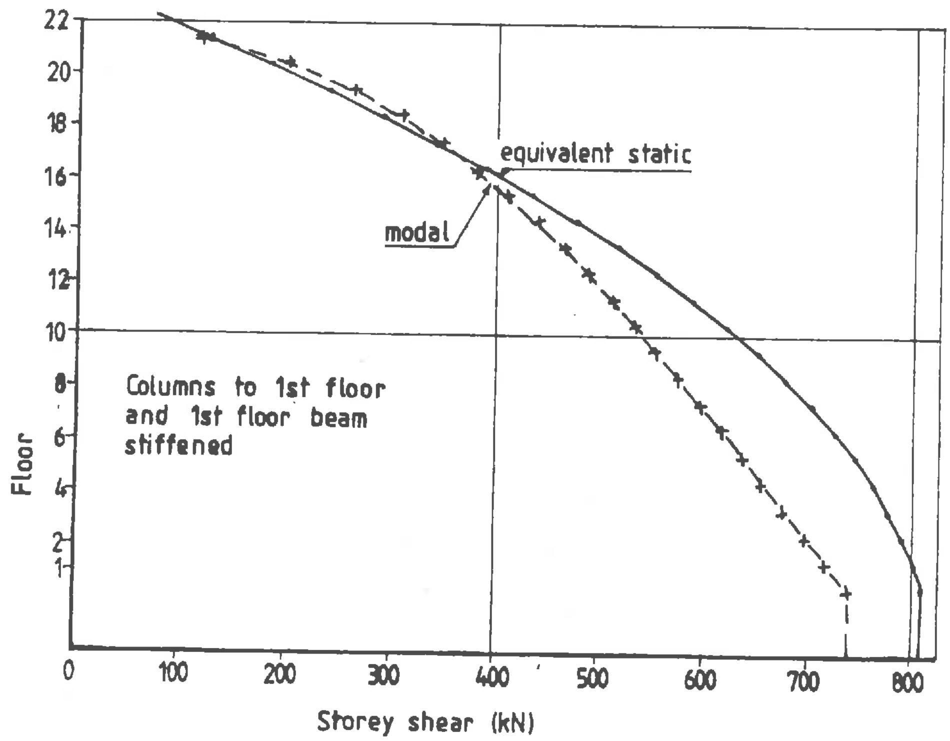


(b) Stiffened Structure

Fig. 3.12 Storey Shears in Missing Floor Frames - 10 Storey Frames



(a) Soft Storey



(b) Stiffened Structure

Fig. 3.13 Storey Shears in Missing Floor Frames - 22 Storey Frame is between these two walls, and a tentative fundamental period of 1.5 seconds is suggested.

For the frame structures the agreement between the two methods of analysis is closer than for the walls, as in the frames the higher mode effects do not appear to be as significant. A suggested limit for the equivalent static method for these structures is suggested as 3 seconds, which in this series would correspond to about a 22 storey structure.

The deflected shape of the uniform walls and frames and the missing floor frames under the equivalent static loading are shown in Fig. 3.14. It can be seen that the uniform walls are closely grouped together. Of all the structures uniform frames form the best fit to the straight line deflected profile, which is the basis of the equivalent static loading equation.

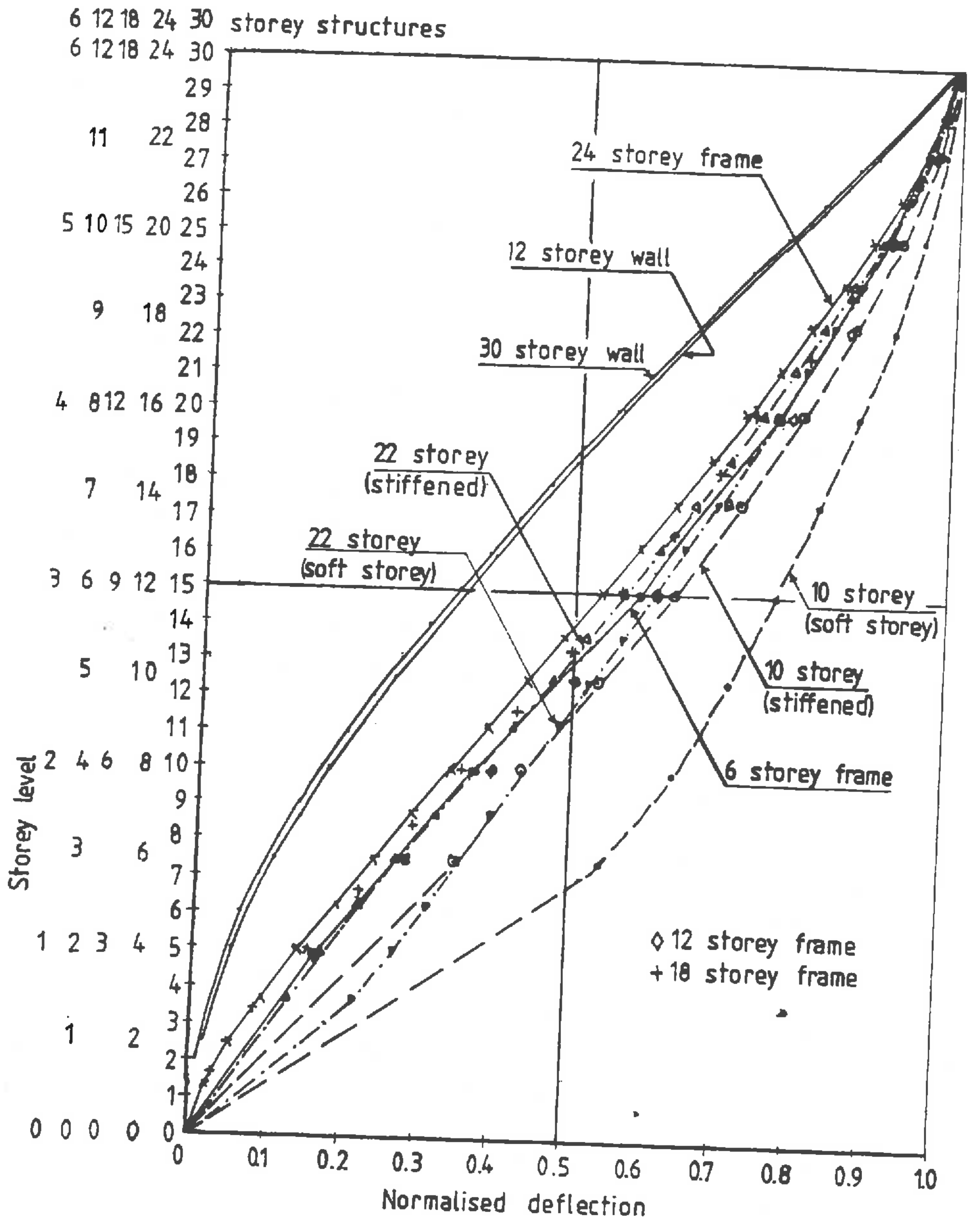


Fig. 3.14 - Deflected Profiles Under Equivalent Static Loads of Regular Frames and Walls and the Missing Floor Frames

Chapter IV - Time History Analyses of Walls and Frames

4.1 General

Four different series of time history analyses were made in this project. In the first of these analyses were made on the wall for the 12 storey building to investigate the length of time step that should be used, the effect of changing the damping characteristics and the response under a different earthquake record. In the second series the performance of the 12, 18, 24 and 30 storey walls were investigated without the inclusion of P-delta effects. This was followed by the third series in which the corresponding analyses were made for the 12 and 24 storey frames. In the fourth series the walls and frames were re-analysed allowing for P-delta effects.

As in the case of the modal response spectrum analyses the seismic mass of each floor was taken as 346.6 tonnes, with half this value being associated with each end frame. As there were two walls used at each end of the structure the seismic lumped mass for each wall was 86.6 tonnes per floor.

In all the analyses strain hardening characteristics were defined for the potential plastic hinge zones to give the structure a nominal displacement strain hardening gradient of 2½ percent. The individual characteristics for each hinge zones were calculated in the following steps.

(i) The displacement occurring at the top of the structure as calculated by the modal analysis for the response spectrum corresponding to a structural ductility factor of 6.

(ii) The appropriate design sway mechanism for the structure as illustrated in Fig. 4.1, was selected.

(iii) Assuming the mechanism in (ii) had developed, the top of the structure was displaced laterally by the deflection found in step (i). For this displacement the rotation induced in each hinge zone was found and the strain hardening characteristics determined so that the bending moments at each hinge location increased by 2½ percent. In these calculations the elastic deformation of the structure was ignored.

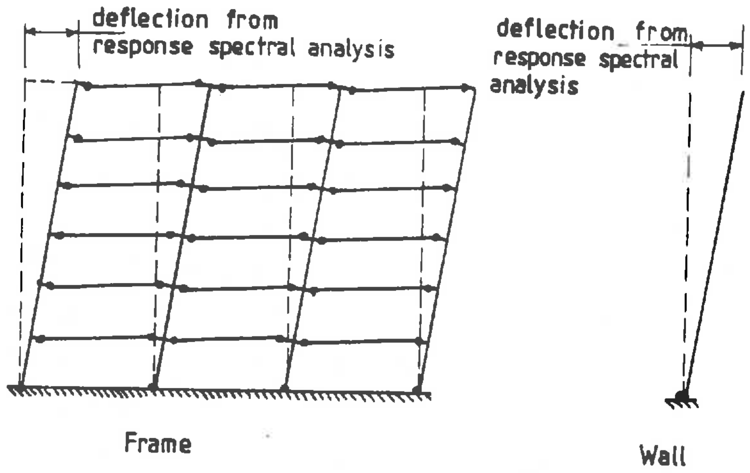


Fig. 4.1 Design Sway Mechanism for Walls and Frames

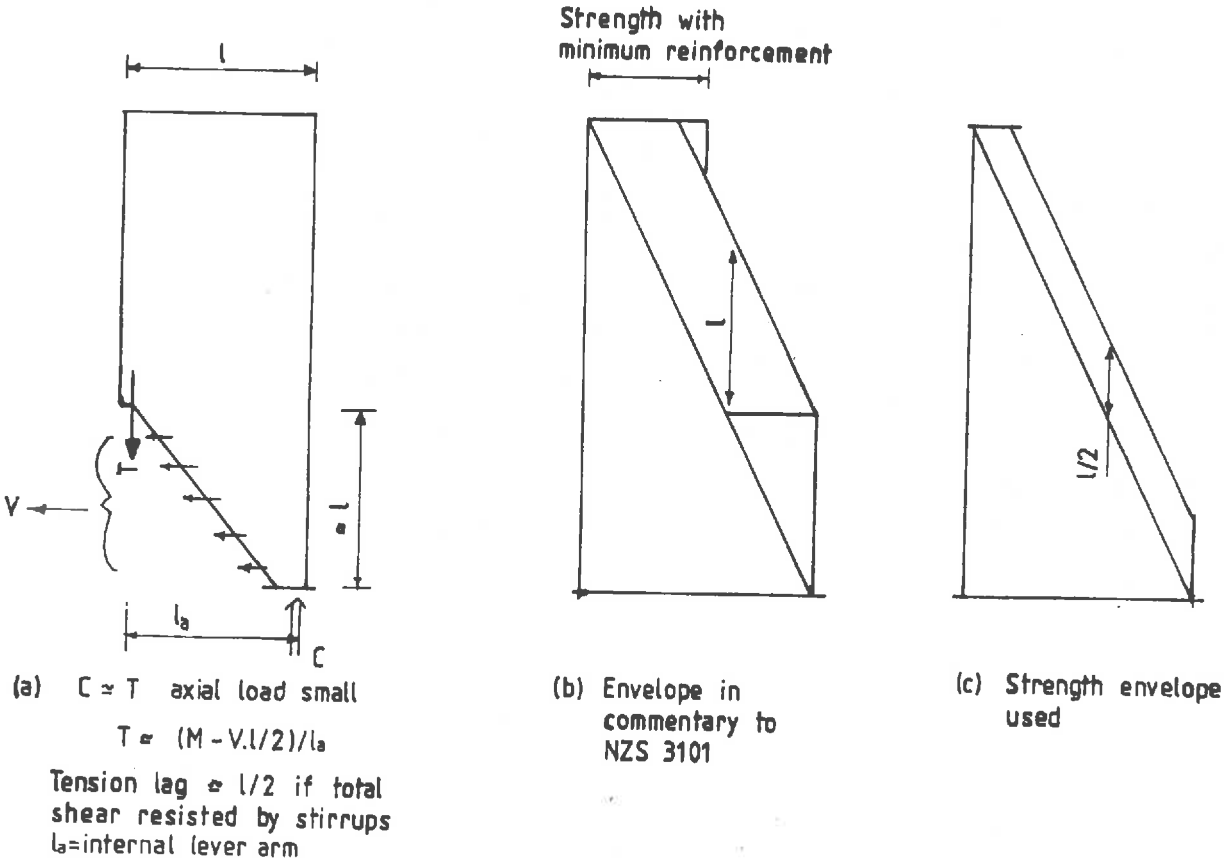


Fig. 4.2 Strength Envelope for Walls

In the first series of walls yielding was assumed to be limited to the base. At all node points above this level the strength limits were set to ensure elastic behaviour. The stiffness of the wall sections followed the assumptions made in the first series of analyses described in section 3.2, that is up to the 2nd floor the moment of inertia was based on half the value calculated from the gross section while above this level it was taken as equal to the gross section value. The first yield moment at the base node was taken as 1/0.9 times the bending moment found from the modal response spectrum analysis. This factor was used as it is the minimum possible value consistent with the strength reduction factor defined in the Concrete Code[5].

In the remaining walls analysed in series II and IV, a uniform moment of inertia was assumed over the height of the wall with a value of three quarters of the gross section value. The strength variation up the wall was based on the suggested design envelope for capacity design on the commentary to the Concrete Code[5]. This was intended to ensure that predominant yielding in a severe earthquake occurs at the base. The yield strength at the base of the wall was taken as 1/0.9 times the bending moment at this location found in the modal response spectrum analysis. In the commentary to the code the suggested envelope is defined by taking a linear variation in the bending moment from the base to the top of the wall, and then offsetting this value up the wall by one wall length, as illustrated in Fig. 4.2(b). This offset is to allow for tension lag and other effects. For walls, which are subjected to low axial load and are provided with sufficient shear reinforcement to carry the total shear force, the tension lag is approximately equal to half the length of the wall, as shown in Fig. 4.2(a). As tension lag effects are not modelled directly in the analysis the strength envelope was based on the linear variation of strength from the base moment to zero at the top of the wall offset up the wall by half a wall height, as shown in Fig. 4.2(c).

4.2 Analyses of 12 Storey Walls - Series I

The corresponding elastic analyses of these walls are described in section 3.2, series I. The earthquake ground motion used in the analyses was the artificial record described in section 2.5, except in run 5 where the El-Centro, 1940, North-South record was used. The variations between the different runs in this series are listed in Table 4.1, and some of the key results are given in Table 4.2.

Halving the time step from 0.005 to 0.0025 appeared to make no significant difference to the results of the analysis. Consequently the 0.005 second time step was used in subsequent analyses.

The bending moment envelopes for the runs are shown in Fig. 4.3 and the corresponding values for the shear force are shown in Fig. 4.4. For the sake comparison the corresponding equivalent static and modal response spectrum analysis values are also shown.

Table 4.1 Time history analyses on 12 storey structural Wall in Series I.

analysis	time step	ground motion		% critical damping in modes			
				1	2	3	4
1	0.005	Artificial	record*	5.0	2.5	4.1	6.5
2	0.0025	"	"	5.0	2.5	4.1	6.5
3	0.005	"	"	5.0	5.0	10.6	17.6
4	0.005	"	"	5.0	10.0	23.0	
5	0.005	El Centre - N.S, 1940		5.0	5.0	10.6	17.6

* artificial record as described in section 2.5
Wall strength and stiffness as described in 4.1 A and 3.1 series A.

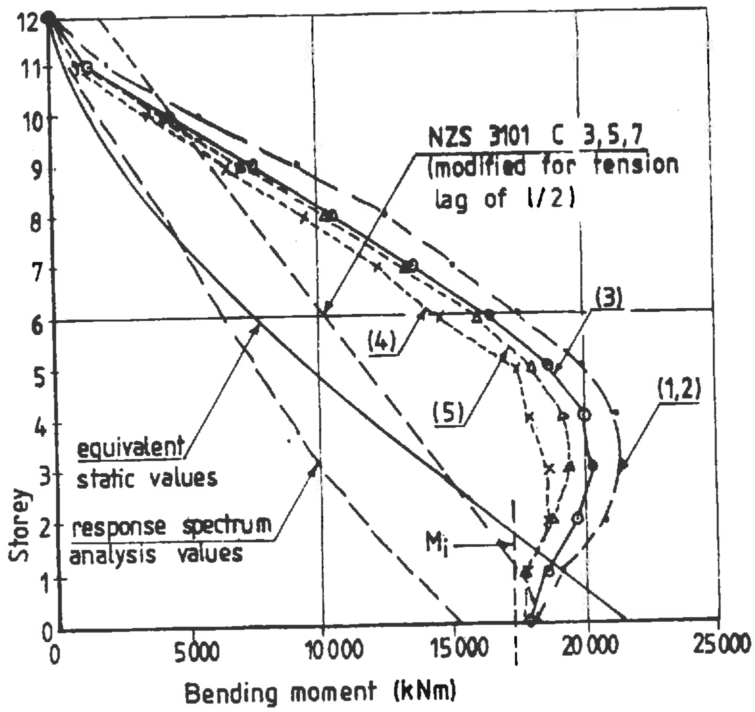


Fig. 4.3 Bending Moment Envelopes for Walls in Series I

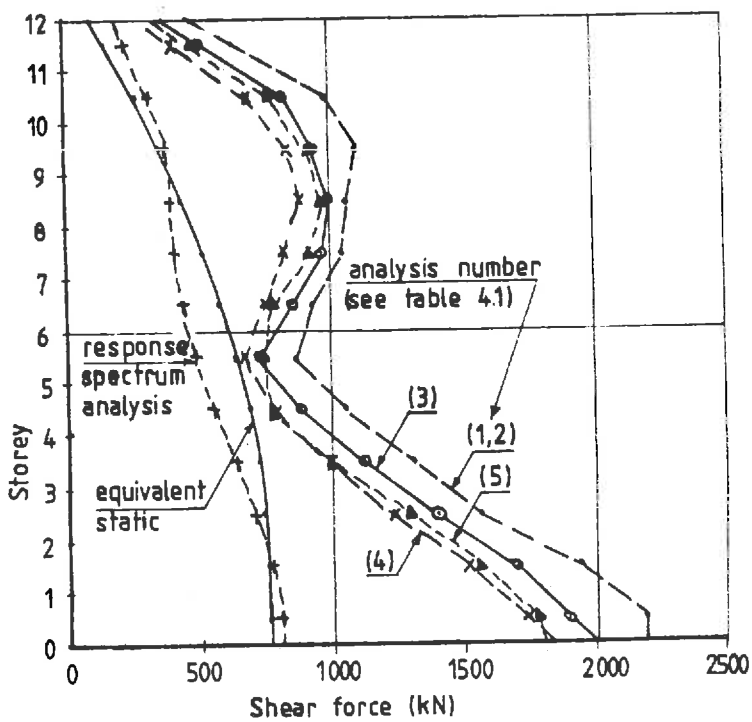


Fig. 4.4 Shear Force Envelopes for Walls in Series I

Table 4.2 Results of Time History Analyses from SERIES I.

Run	Max. deflection mm.	Max. rotation (rad)	Max. base shear kN	Hysteretic energy dissipated (kNm x rad)	Maximum bm. at base kNm.
1	165	0.0034	2,193	451	17 940
2	163	0.0033	2,218	452	17 930
3	158	0.0032	1,905	449	17 904
4	145	0.0028	1,736	444	17 840
5	200	0.0043	1,770	225	18 114

Values from Elastic Analyses

First yield moment at base	=	17 304 kNm
Equivalent static base shear	=	785 kN
Equivalent Static Deflection	=	41.1 mm
Response spectral base shear	=	740 kN
Response spectrum deflection	=	32.5 mm

The form of the bending moment envelope obtained in these analyses was unexpected. The peak moments occurred at or close to the 3rd floor level, which showed that the higher mode effects were important to the structure. With the increased damping used in runs 3 and 4, there was some reduction in both the peak moments and shears sustained in the walls (see Figs. 4.3 and 4.4), but the general form of the envelopes was not changed. Surprisingly increasing the damping did not change the amount of energy dissipated by yielding (hysteretic energy). In all the subsequently analyses in this report the mass and stiffness damping characteristics were selected to give an equivalent 5 percent viscous damping on the first two modes (as in run 3).

When the first runs in this series of analyses were studied it was felt that the unexpected form of the moment envelope might have been the result of some unusual characteristics in the artificial earthquake record that was used. As a check the 5th run on this wall was made using the 1940 El-Centro North-South record. As it can be seen in Figs. 4.3 and 4.4 the results were in many respects similar those obtained with the artificial ground motion. The differences were in the higher displacements and rotations obtained and the lower hysteretic energy dissipated in the El-Centro record.

4.3 Analyses of 12, 18, 24 and 30 Storey Walls without P-delta Effects - Series II

The strength and stiffness characteristics of these walls varied from those in the first series in that the flexural stiffness was taken as uniform over the height and the strength varied as detailed in section 4.1. The slenderness ratio varied from 5.37 for the 12 storey structure to the very slender 13.42 for the 30 storey structure.

Some of the key results from the time history analyses for these walls are summarized in Table 4.3, together with a few of the values from the elastic analyses. The inelastic rotation demands placed on the walls are reproduced in Fig. 4.5. From here it can be seen that yielding occurred in the lower three quarters of the wall. However, away from the base node the inelastic rotation demands were relatively small, with very few of these exceeding a value of 1.5×10^{-3} radian per 3.4m of wall. This rotation approximately equals to that corresponding to an inelastic extension of the reinforcement of two yield strains (f_y = 300 MPa). The maximum inelastic rotation demands were at the base of the walls; the higher walls having proportionally greater values of rotation. In practice the length of the base hinge could be expected to increase with the height of the structure and consequently the section ductility demand would not increase in proportion to the rotation demand indicated in Fig. 4.5.

The accumulated inelastic hinge rotation is defined as the sum of the absolute values of plastic rotations occurring at a node during the passage of the earthquake. These values for the walls are given in Fig. 4.6. They give one measure of the damage sustained at a particular location and they may also be used to assess the energy dissipated by yielding (hysteretic energy). In the 12 storey wall 85 percent of the total hysteretic energy was dissipated at the base hinge. This figure decreased to 55 percent for the 30 storey wall. These results together with the maximum inelastic rotation demands indicate that the design envelope suggested in the draft code is a little on the unconservative side when applied to the very slender walls such as those for the 24 and 30 storey structures. To reduce the inelastic deformation and energy dissipation in the slender walls above the second floor level some increase in strength above that suggested in the concrete code (see Fig. 4.2)

Table 4.3 Principal Results of Time History Analyses on Uniform Walls in SERIES II

ITEM	12 storey	18 storey	24 storey	30 storey
Time History Values				
Max. deflection (mm)	135	297	723	1002
Max. rotation at base (rad)	0.0024	0.0045	0.0045	0.0092
Hysteretic energy dissipated (kNm)	480	620	840	930
Max. base bm. (kNm)	17 990	20 790	21 820	22 220
1st yield bm. (kNm)	17 305	20 100	21 290	21 330
Max. base shear kN	1 690	2 040	1 710	1 610
Elastic analysis values				
Modal response spectrum values				
deflection (mm)	23.1	53	95	146
base bm. (kNm)	16 190	18 090	19 160	19 200
base shear (kN)	797	791	740	660
Equivalent Static Values				
deflection (mm)	31.7	77.0	14.4	228
base bm. (kNm)	22 840	24 400	25 030	25 120
base shear (mm)	778	562	435	350

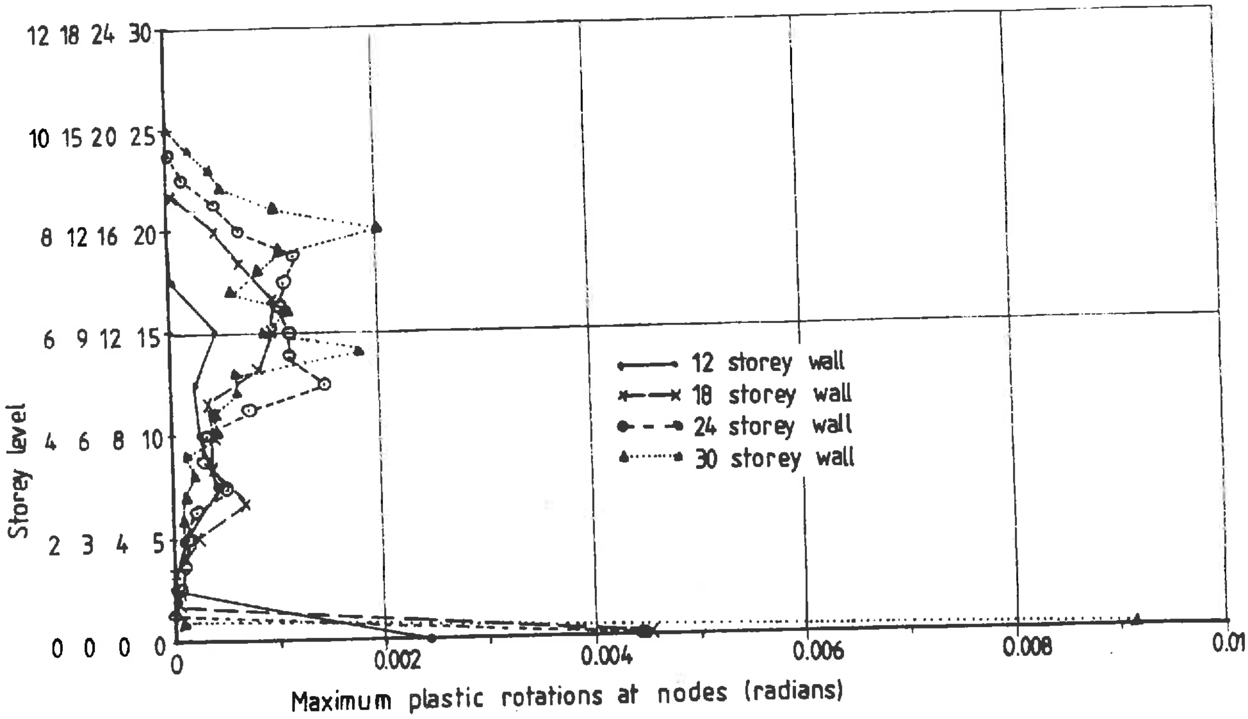


Fig. 4.5 The maximum inelastic rotation demands in the walls - Series II

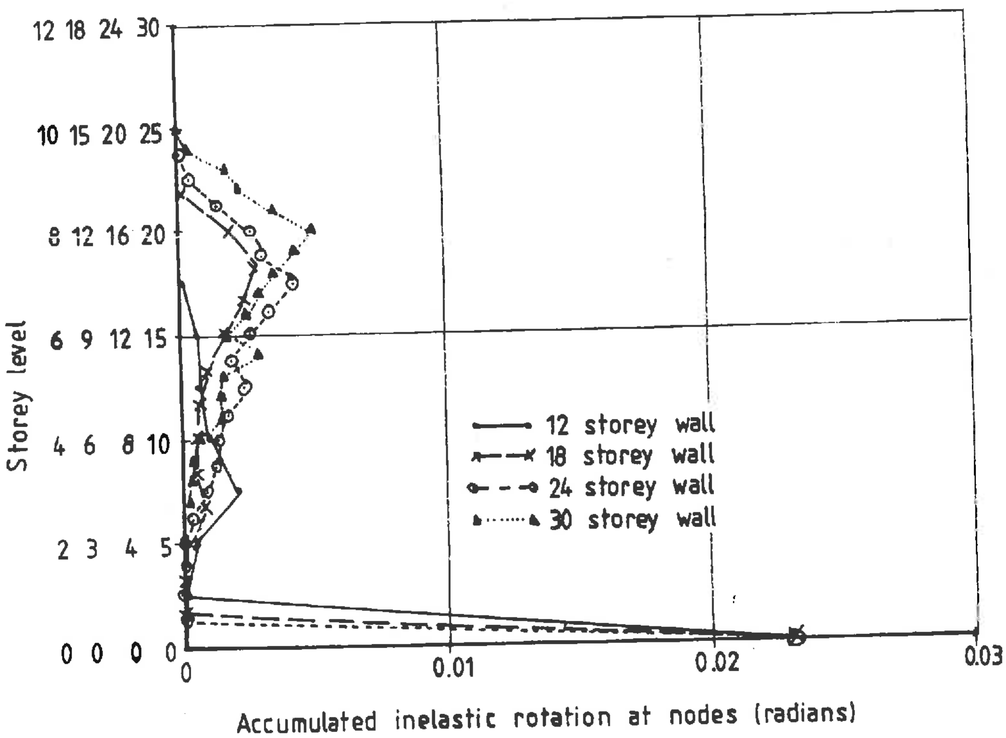


Fig. 4.6 Accumulated inelastic rotation demands in the walls - Series II

appears to be required. This might take the form of off setting the bending moment envelope obtained in a modal or equivalent static method of analysis by half a wall length, to allow for tension lag effects, plus some fraction of the wall height.

The small inelastic rotation demands placed on the lower three quarters of the walls results in the moment envelope closely following the strength envelope up the wall. It is only in the top quarter where the yield levels are not reached. It is interesting to note that for the 30 storey wall the modal response spectrum bending moments in this upper zone exceed the strength envelope values used, but in spite of this yielding does not occur in this location.

The shear force envelopes obtained in the time history analyses of the wall are compared with the equivalent static and modal response spectrum values in Figs. 4.7 to 4.10 for the four walls. For capacity design the equivalent static or modal values should be multiplied by $\phi_o \omega_v$, where ϕ_o makes an allowance for over strength and ω_v is the dynamic magnification factor, which allows for the higher mode actions in the wall. For these analyses ϕ_o is equal to 1/0.9 and ω_v is 1.7 for the 12 storey wall and 1.8 for the higher walls[5]. With this amplification of actions there is still a short fall between the maximum time history shear envelope values and the scaled equivalent static or modal values for the higher walls. However, this short fall is unlikely to have appreciable consequences as the model for the dynamic analyses did not allow for inelastic shear deformation. In practice the high shear stresses would lead to diagonal cracking and associated with this is a marked loss of shear stiffness. In addition some localised yielding of the stirrups could be expected without failure occurring. It is hoped to examine this aspect in a future study. The very jagged form of the shear force envelope indicates the contribution of higher mode actions of short duration.

The deflected shape envelopes for the walls are shown in Fig. 4.11, together with the results of analyses in which P-delta effects were included. These are described in a later section. With the exception of the 18 storey wall the profile lies close to a straight line. In all the analyses at one instant during the record the deflected shape lay very close to the

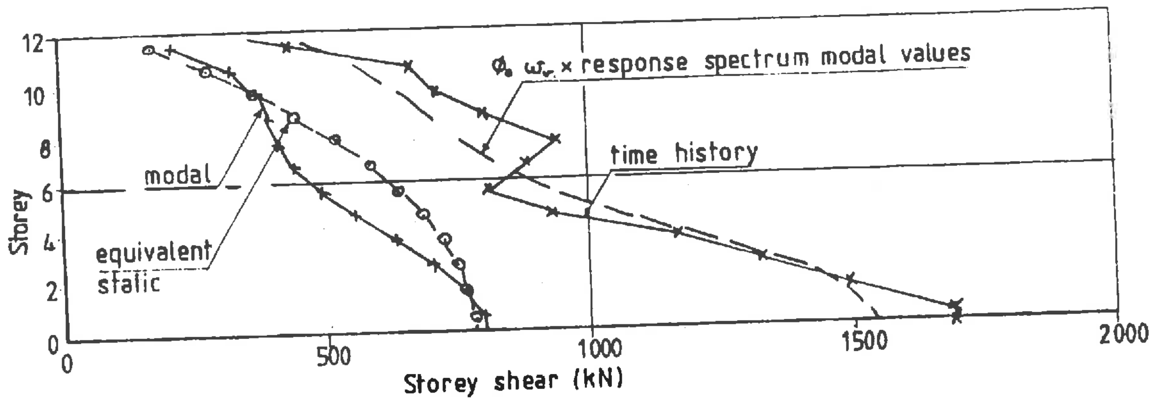


Fig. 4.7 Shear Force Envelope on 12 Storey Wall - Series II

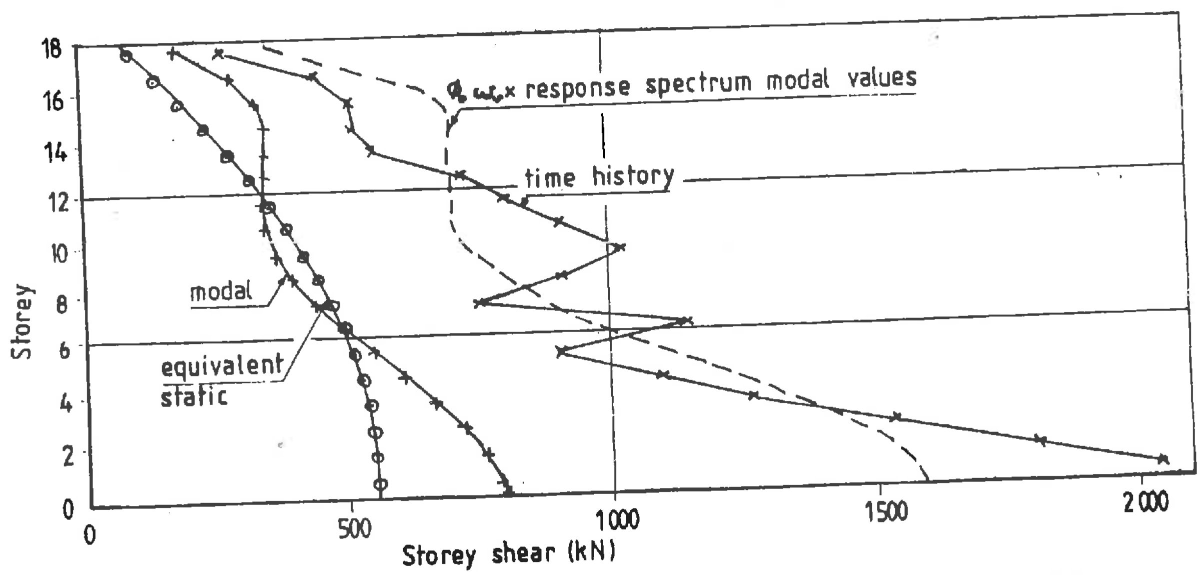


Fig. 4.8 Shear Force Envelope on 18 Storey Wall - Series II

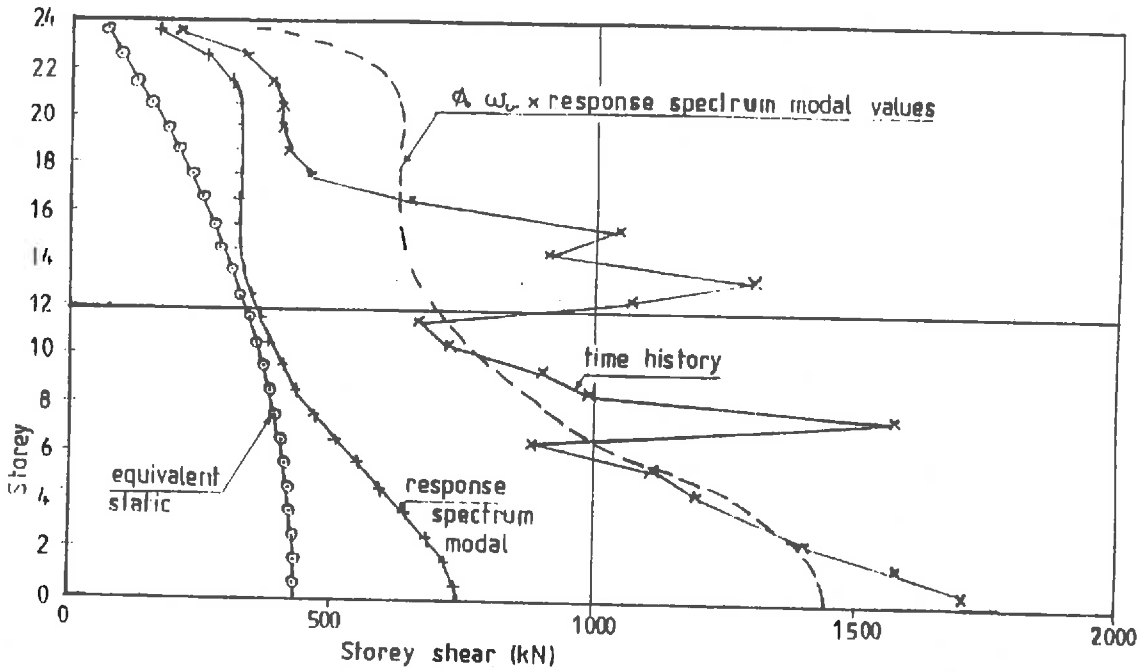


Fig. 4.9 Shear Force Envelope on 24 Storey Wall - Series II

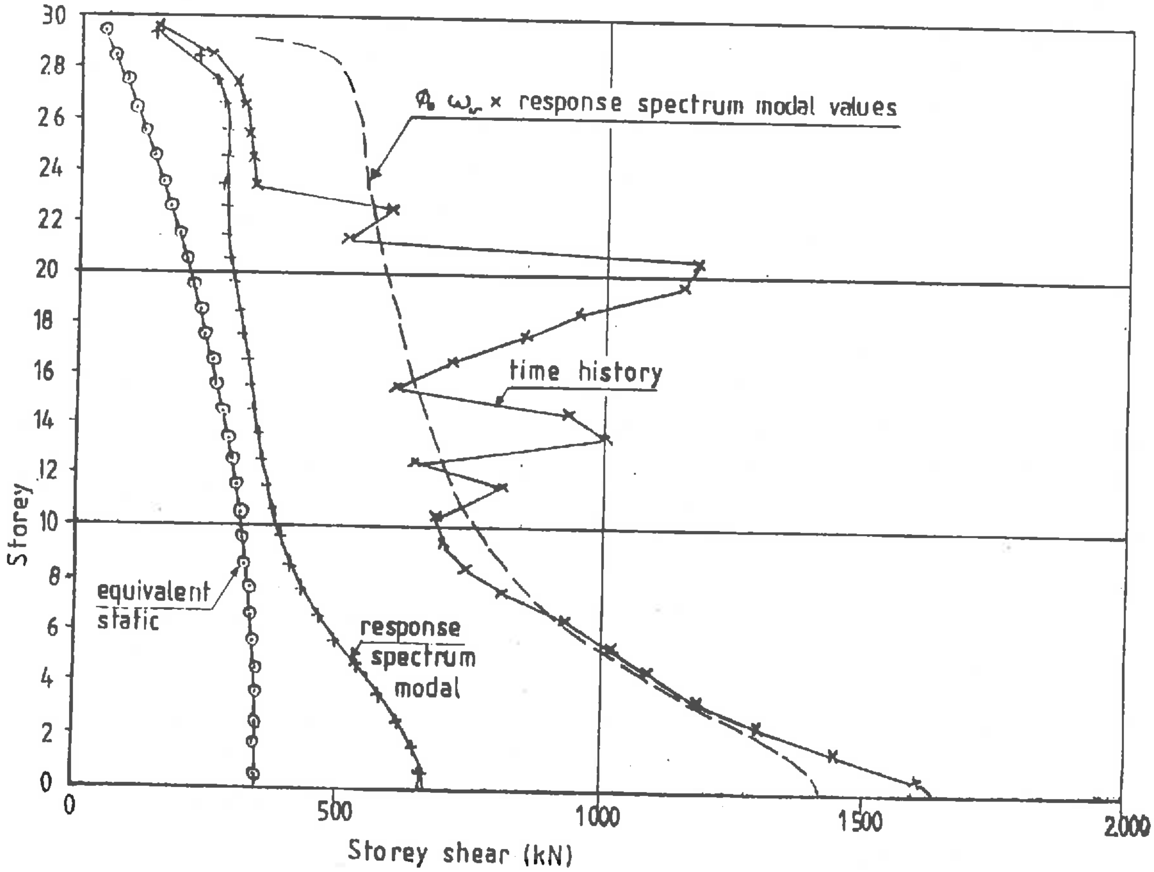


Fig. 4.10 Shear Force Envelope on 30 Storey Wall - Series II

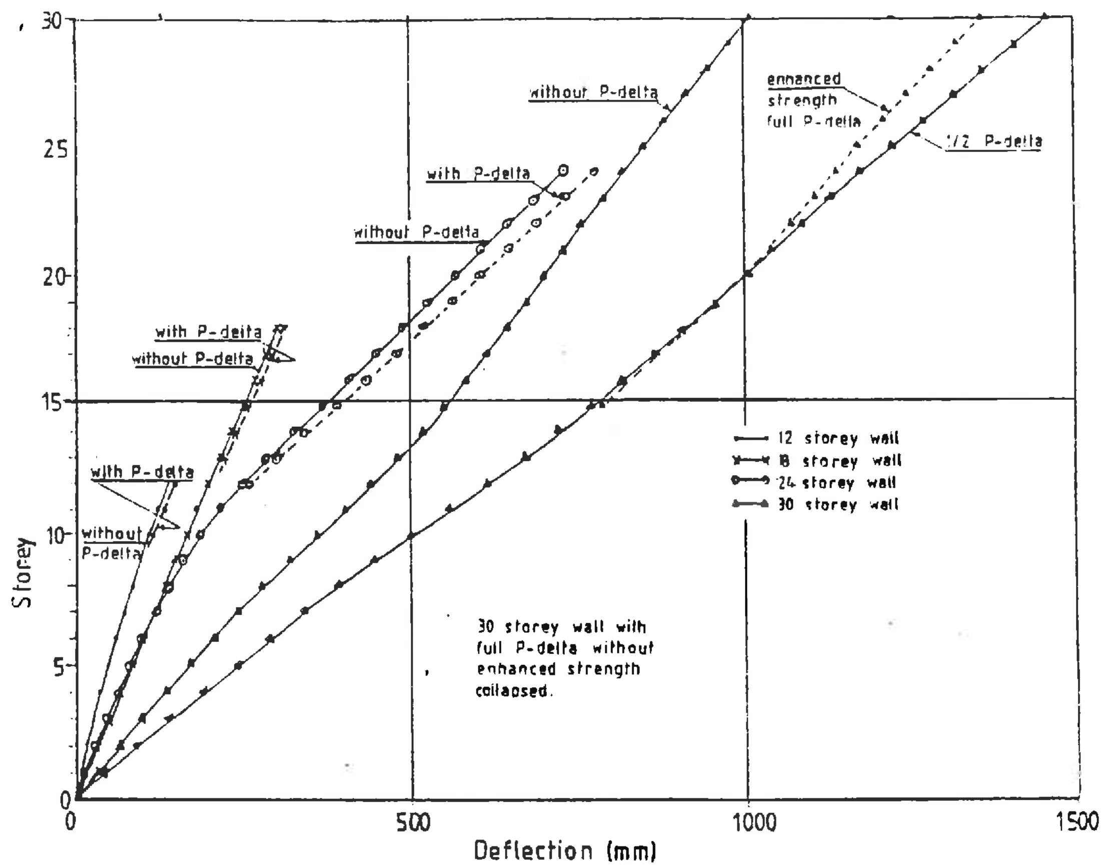


Fig. 4.11 - Deflected Shape Envelopes of Walls in Time History Analyses - Series II and IV

envelope over its full height. It should be noted that the form of the deflected shape envelope is very different from that which would be obtained by scaling up the elastic deflected shapes found in the equivalent static or modal analyses (see Fig. 3.14 on page 43).

In Table 4.4 the peak deflections calculated at the top of the structure from the time history analysis are compared with the structural ductility factor, (6), times the modal analysis deflection and 0.85 times the structural ductility factor times the equivalent static deflection. It can be seen that the values predicted from the modal analyses tend to underestimate the maximum deflection by a few percent while the equivalent static method with the factor of 0.85 suggested in Section 3.6 tends to overestimate these values by a similar margin.

Table 4.4 Ratio of maximum deflection at top of walls in the time history analyses to the values predicted from the equivalent static and modal response spectrum analyses

Method of Analysis	Deflection in time history analysis / predicted deflection: 12 storey	18 storey	24 storey	30 storey	Average
equivalent static	1.02	1.08	0.79	0.87	0.93
response spectrum modal	1.08	0.91	1.02	1.16	1.06

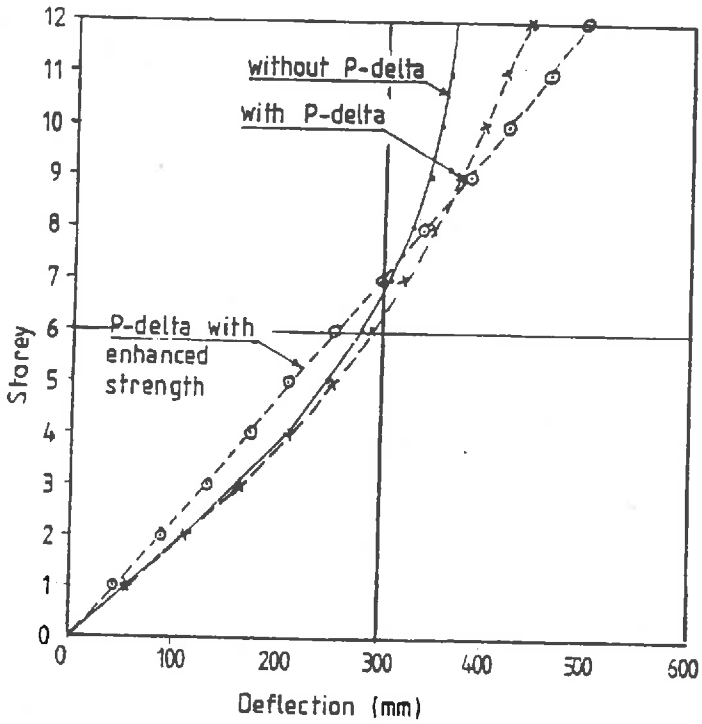


Fig. 4.12 - Deflected Shape Profile for 12 Storey Frames - Series III and IV

4.4 Analyses of 12 and 24 Storey Frames without P-delta Effects - Series III

The 12 and 24 storey frames described in section 3.3 were subjected to time history analyses using the artificial earthquake record (see section 2.7). The yield strengths for the potential plastic hinge zones in the beams at the column faces were found from the modal response spectrum analysis values. At any floor level these values were averaged, to allow for redistribution, and then increased by the factor of 1/0.9. This multiplier allowed for the strength reduction factor. The potential hinge zone moments at the base of the columns were taken as 1.4 times the modal response spectrum values to comply with the minimum requirements suggested for capacity design in the commentary to NZS 3101[5]. At all other locations in the columns yielding was suppressed by setting a high strength limit. As described in section 3.3 there was little gravity loading on the beams in the frame and for the purposes of these analyses gravity loading was ignored.

Table 4.5 Principal Results of Time History Analysis on 12 and 24 storey frames without P-delta effects

Item	12 Storey	24 Storey
Max. deflection (mm)	365	616
Max. inelastic rotation (radians) base of cols. beams (at levels) -1 - 6 -7 - 12 -13 - 18 -19 - 24	0.013 9 0.015 3 0.007 4 - -	0.013 3 0.014 8 0.011 6 0.007 5 0.002 6
Accumulated inelastic hinge rotations base of cols. beams (at levels) -1 - 6 -7 - 12 -3 - 18 -19 - 24	0.037 8 0.049 5 0.031 1 - -	0.035 8 0.040 4 0.028 6 0.016 2 0.010 4
Max. shear base (kN)	963	1 486
	Elastic Analysis Values	
	Modal Response Spectrum	
Deflection "Δ" (mm) Deflection x 6 (mm) base shear (kN)	58.8 353 525	84.0 504 781
	Equivalent Static Analysis	
Deflection "Δ" (mm) Δ x 0.85 x 6 (mm) base shear (kN)	77.1 393 585	115 587 879

Some of the key results from the time history analyses on the 12 and 24 storey frames are reproduced in Table 4.5. Both the inelastic rotation demands and the accumulated inelastic rotations in the beam hinges may be seen to decrease with increasing height in the structure (see also Figs. 4.14 and 4.15). These values are well below the level that can be safely sustained in hinge zones detailed for ductility. The maximum base shear predicted in the analyses of the two frames are of the order obtained by scaling either the equivalent static or response spectrum value by the factors given in Appendix C3A - "A method for the evaluation of column actions in ductile multi-storey frames", contained in the Commentary to the Concrete Code of practice[5].

The peak deflections predicted in the time history analyses, which are given in Table 4.5, can be compared with the values predicted from the modal response spectrum and equivalent static methods. For the 12 storey frame the agreement is close, but with the 24 storey structure the modal response spectrum method under- estimates the maximum deflection by about 20 percent.

The deflected shape envelopes for the frames are shown in Figs. 4.12 and 4.13 (see pages 57 and 60). Included in these figures are curves for analyses in which P-delta effects have been included. These results are considered in a later section. When the P-delta effects are excluded the overall deflected shape envelope lies close to a parabola that has an infinite gradient at the top of the structure. This shape reflects the higher inelastic demands that develop in the lower regions of multi-storey frames. The maximum inelastic rotation demands at the base of the beam hinges are shown in Figs. 4.14 and 4.15. These show that greater ductility is required from the hinge zones located in the lower levels of the building than those higher up.

4.5 Analysis of Walls and Frames with P-delta Effects Included - Series IV

The walls and frames considered in the previous two series were re-analysed only this time P-delta effects were included. For the walls the load per floor inducing the P-delta actions was one quarter of the weight of each floor (850 kN) as there were four floors, while for the frames the corresponding value was half the floor weight (1 700 kN).

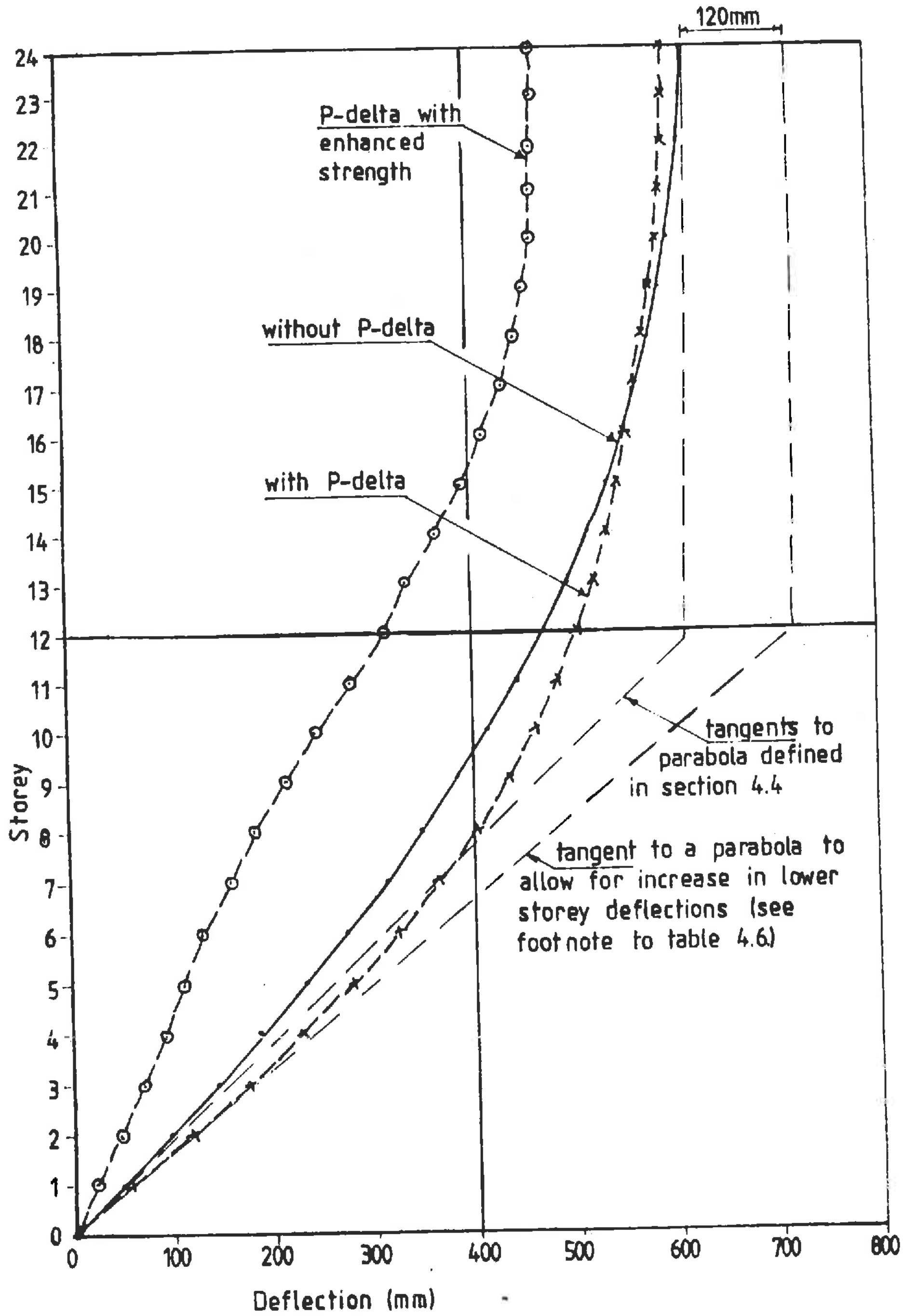


Fig. 4.13 Deflected Shape Profile for 24 Storey Frame - Series II and IV

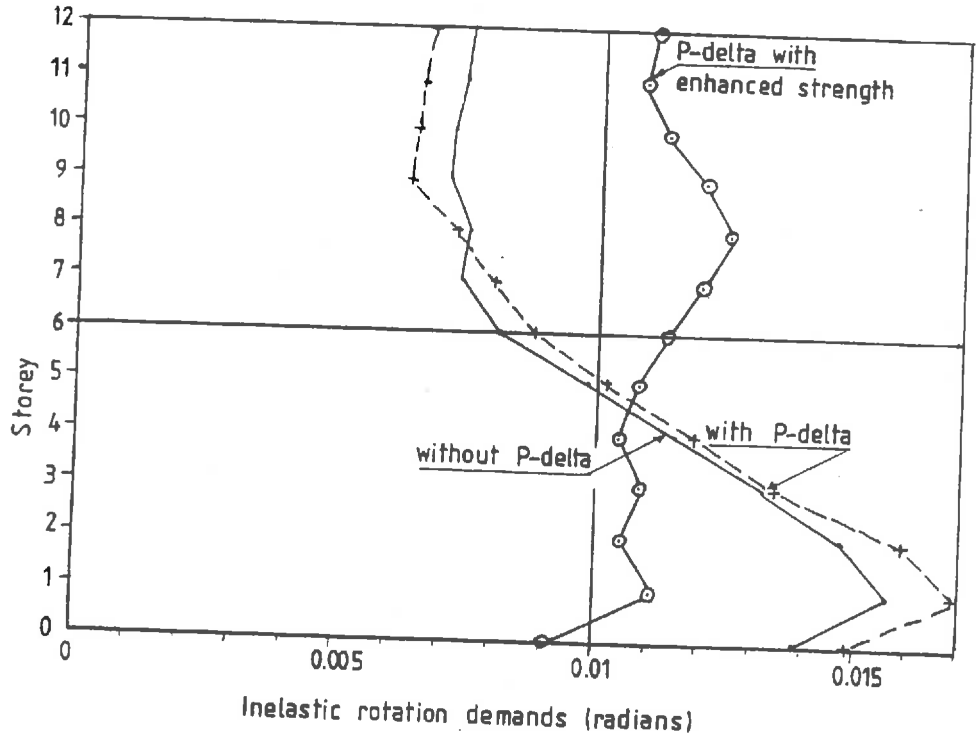


Fig. 4.14 - Inelastic Rotation Demands in 12 Storey Frame - Series III and IV

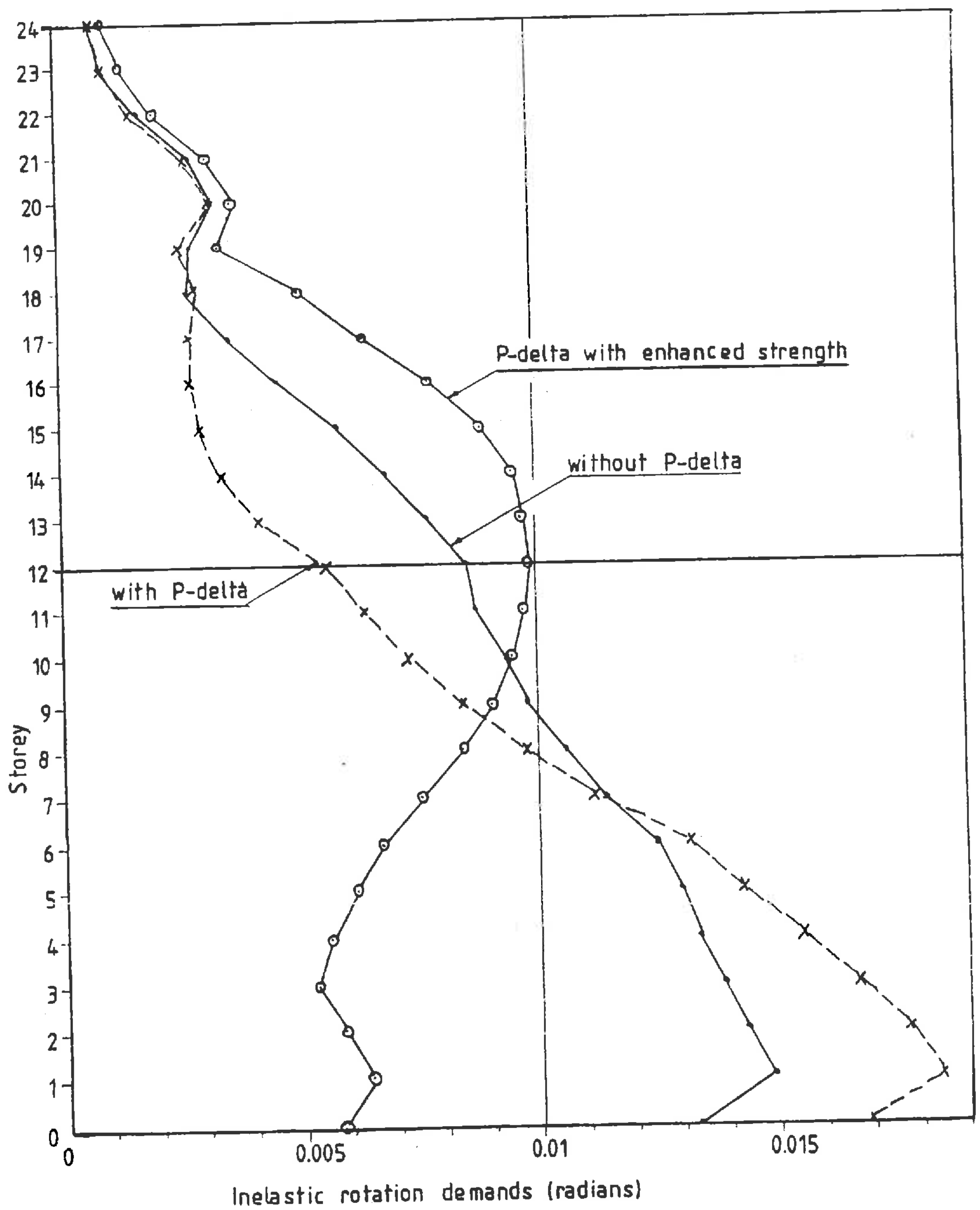


Fig. 4.15 - Inelastic Rotation Demands in 24 Storey Frames
- Series III and IV

There were several different sets of analyses made in this series. In the first of these the flexural strengths of the potential plastic hinge zones was unchanged from the values used in the Series II and III analyses. The inclusion of P-delta effects was found to increase the maximum deflections sustained at the top of the structure for the 12, 18 and 24 storey walls and the 12 storey frame. The 30 storey wall collapsed, while the deflection at the top of the 24 storey frame decreased slightly, but there was a marked increase in the interstorey drifts in the lower levels of the building. Similar changes in deformed shape due to P-delta actions have been noted in frame structures in the literature[11]. The envelope of deflected shapes for these analyses are shown in Figs. 4.11, 4.12 and 4.13.

The 30 storey wall was re-analysed but this time the weight associated with the P-delta effects was halved. As shown on Fig. 4.11 collapse did not occur in this case but there was a 45 percent increase in deflection. This analysis indicated that to prevent the collapse either some increase in strength or stiffness is required.

A number of difficulties occur in trying to establish a rational design approach to allow for P-delta actions. Some of these are briefly outlined below.

(1) Where P-delta actions are significant in the dynamic behaviour it is found that as the earthquake record progresses the structural deflections progressively increase in one direction in a creep like manner. The maximum deflection is generally sustained near the end of the strong ground motion and the magnitude of this deflection clearly depends on both the intensity and duration of the earthquake. Engineers frequently judge the severity of an earthquake by the ductility demand the ground motion places on a single degree of freedom oscillator in which P-delta effects are excluded. This peak demand often occurs near the start of the severe ground motion period of the earthquake. Such assessments tend to overlook the duration factor, which is important for P-delta actions.

(2) The influence of different load deflection response loops for the structural elements needs to be considered. The common bi-linear

response assumed in a large number of inelastic time history analyses overestimates the structural stiffness of members which have been subjected to inelastic displacements. While the effect of stiffness degradation on single degree of freedom oscillator, in which P-delta effects are neglected, does not appear to be significant, there appears to have been little examination of the possible influence of stiffness degradation on P-delta effects.

(3) Many buildings in which P-delta effects are likely to be significant in a severe earthquake have a long period. For these structures there is a marked decrease in the magnitudes of the lateral load seismic design coefficient required by the current loadings code (NZS 4203 - 84)[2] and the draft code (DZ 4203 - 86)[1]. The high value of lateral load coefficient in the current loadings code has undoubtly helped to mask the significance of P-delta effects in multi-storey structures by requiring them to have a strength for earthquake actions which is considerably in excess of that implied by the response spectra of most earthquakes ground motion records used for dynamic analyses in New Zealand. Consequently the proposed change in the design spectra in the draft code will require the designer to consider P-delta actions much more carefully than was previously the case.

(4) The response of the structure to a particular earthquake record can be very sensitive to its dynamic characteristics. A small change in the elastic stiffness can give rise to marked differences in the ductility demand. Including P-delta effects in an analysis results in an effective reduction in the elastic stiffness of a structure. Ideally a large number of analyses are required to predict general trends. However, time and cost constraints tend to prevent this approach from being used.

(5) A large number of analyses of single degree of freedom analysis with and without P-delta actions have been reported in the literature[12], from which both the general trends of including P-delta actions and the wide scatter of results is apparent. However, a number of preliminary analyses indicate that P-delta effects are considerably more severe in single degree of freedom ossilators than in multi-degree of freedom structures.

As a result of the points discussed in the proceeding paragraphs it was felt that a conservative method of assessing P-delta actions for multi-storey buildings was desirable. The analyses reported in reference 12 indicates that the proposals described below are likely to be unconservative for structures, such as soft storey buildings, which behave like a single degree of freedom structure.

For the case of an elastic structure subjected to static lateral loads the P-delta induced actions can be readily calculated. The simple model illustrated in Fig. 4.16 may be used in such an analysis. The seismic weight (lumped mass) at each floor is assumed to be supported by a column that is made up of a series of rigid struts which are pinned at each floor level. Before any lateral load is applied this column is assumed to be straight. As shown in the figure any deflection of the structure due to lateral load causes this pin ended column to apply additional lateral forces (representing P-delta actions) to the structure. Where the building contains rigid diaphrams the lateral loads from the kinking of the column are applied at the centroid of mass at each floor.

In the seismic situation with ductile structures inelastic deformations occur and the lateral forces induced arise from dynamic considerations and not static actions. Hence strictly speaking the model is not appropriate, but for want of a more rational basis it is suggested that it can be used to assess the additional strength required to allow for P-delta actions associated with severe earthquakes as outlined in the following paragraphs.

The first step in the proposed method of allowing for P-delta actions in design is to assess the maximum deflection that is expected to occur at the top of the structure. The deflections found in the dynamic analyses with and without P-delta actions are given in Table 4.6. From these values it was concluded that the deflection at the top of the structure should be made up of two components. The first of these should be the value expected neglecting P-delta actions, Δ_u, which may be taken as the structural ductility factor times the deflection found in the modal response spectrum analysis or 0.85 times the structural ductility factor times the deflections given by the equivalent static method. A second component is to allow for the increase in this deflection due to P-delta effects.

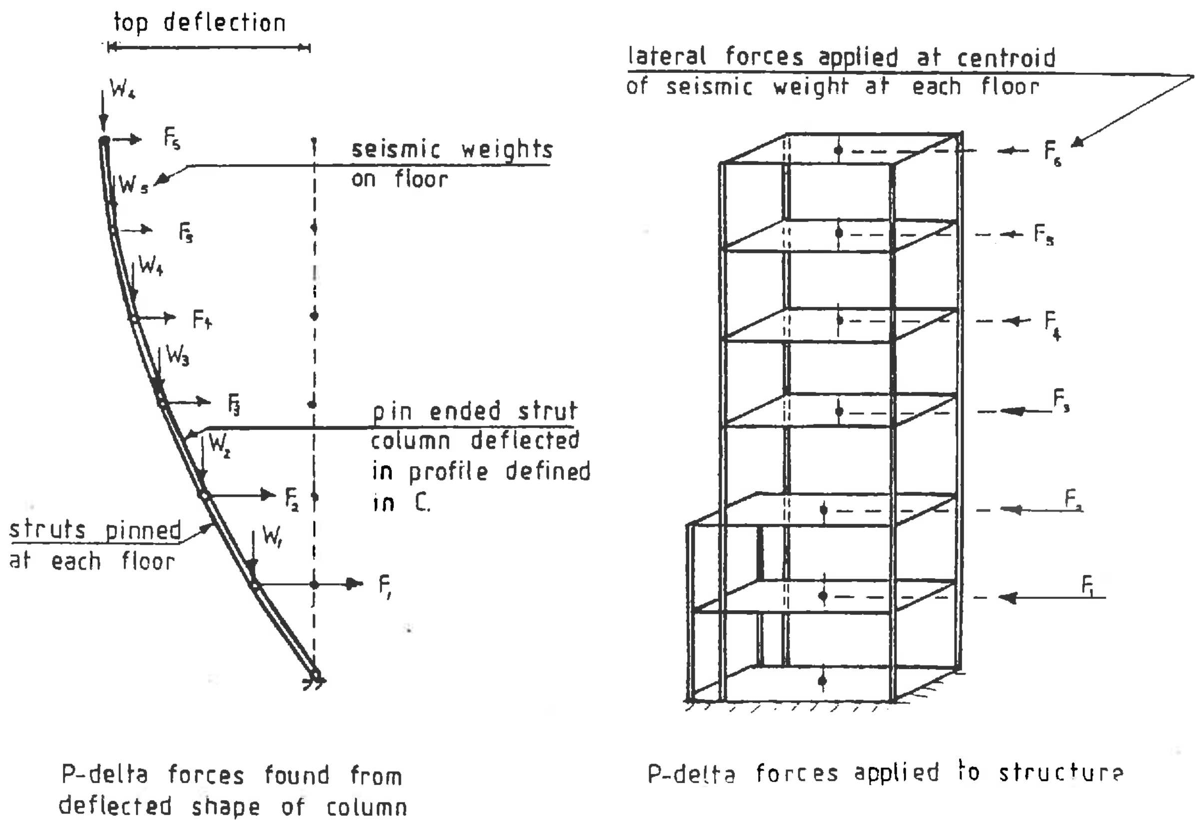


Fig. 4.16 - Model used to Assess P-delta Actions

Table 4.6 Deflections at top of frame and wall structures (mm).

Structure	Response Spectral x μ	Time History deflection			With P-delta and enhanced strength
		Without P-delta	With P-delta	Difference due to P-delta	
Walls					
12 storey	137	135	145	10	
18 storey	317	297	305	8	
24 storey	567	723	773	50	
30 storey	878	1002	{collapse 1453*	∞ 451*	1349
Frames					
12 storey	353	365	490	75	497
24 storey	504	616	599	-17 (120)+	519

* analysis with ½ P-delta actions included

\+ an increase of 120mm at the top without changing the form of the deflection would cover the increased displacements in the lower floors (see Fig. 4.13)

To assess the additional deflection due to P-delta actions the following method is proposed:

(i) Calculate the lateral deflection, Δ_u, at the top of the structure due to seismic actions neglecting P-delta effects as described in the previous paragraph.

(ii) Select an appropriate deflected shape profile for the structure, which allows for the development of inelastic deformations in the potential plastic hinge zones. For uniform wall structures the analyses in Series II indicate a straight line profile is reasonable, while for the uniform frames the parabolic profile mentioned in section 4.4 (see page 59) appears appropriate. It should be noted that a deflected profile based on an elastic deformed shape can seriously underestimate the interstorey deflections in the lower levels of a structure and lead to an underestimate of the P-delta induced structural actions in these zones.

(iii) From the deflection at the top of the structure found in step 1 and the form of deflected profile chosen in step 2, find the deflected shape of the pin ended column in the model illustrated in Fig. 4.16. From this deflected shape find the lateral forces required to stabilise the column. The reverse of these forces are now applied to the structure, and the additional deflection Δ_1, at the top of the structure is found. Assuming static elastic behaviour, this deflection Δ_1 gives rise to a further set of P-delta forces and equilibrium is reached when the total deflection at the top of the structure is

$$\frac{\Delta_u}{\left(1 - \dfrac{\Delta_1}{\Delta_u}\right)} \quad \text{or}$$

Δ_u plus an additional deflection $\Delta_1 / \left(1 - \dfrac{\Delta_1}{\Delta_u}\right)$

However, inelastic deformation can also be expected to occur with the P-delta actions as the earthquake motion occurs, further increasing the additional deflection at the top of the structure. To allow for this a further emperical scale factor is proposed as

$$\left[1 + \frac{(\mu - 1)}{2}\right].$$

The proposed resultant deflection, Δ, at the top of the structure then becomes -

$$\Delta = \Delta_u + \Delta_1 \frac{\left(1 + \frac{\mu - 1}{2}\right)}{\left(1 - \frac{\Delta_1}{\Delta_u}\right)} \tag{4.1}$$

This deflection together with the deflected profile chosen in step 2 is used to determine the structural actions induced by P-delta effects in the structure. In Table 4.7 the increase in deflection predicted by this method is compared with the values found by the time history analyses on structures with and without allowance for P-delta actions.

In determining the required strength of potential plastic hinge zones by adding the P-delta structural action to the value found from the response spectrum or equivalent static method of analysis appeared to be unduly conservative. In this project it was decided to provide a minimum flexural strength at the potential hinge zones corresponding to the greater of the two cases -

(1) the action due to seismic forces, or
(2) 80 percent of the seismic forces found from modal response spectrum analysis plus the P-delta induced action.

It should be noted that gravity loads on the beams were neglected in these analyses.

As a check on this proposed method of allowing for P-delta actions the 30 storey wall and the 12 and 24 storey frames were given the yield strengths specified by the proposed method. Time history analyses were then carried out. Some of the results of these analyses are given in Table 4.6 and in Figs. 4.12, 4.13, 4.14 and 4.15. In all cases the results are labeled as "P-delta with enhanced strength".

Table 4.7 An additional deflection due to P-delta actions

Structure	Predicted deflection (mm)	Time history** deflection (mm)	$\frac{\text{P-delta b.m}}{\text{Eq. b.m.}}$+
Walls			
12 storey	2.8	10	0.05
18 storey	20.7	8	0.14
24 storey	86.5	50	0.31
30 storey	246.8	∞	0.60
½P-delta	123.4	451	0.30
Frames			
12 storey	73.9	75	0.39
24 storey	94.5	-17 (120)*	0.38

* see note at base of Table 4.6

\+ ratio of overturning moments at base of structure induced by P-delta actions and earthquake actions as found in modal analyses.

** Difference in deflection from time history runs with and without P-delta actions.

The performance of the enhanced strength structures appears to be adequate with the maximum inelastic demands on these generally being smaller than the comparable values for the time history analyses made without P-delta actions included. This tends to indicate that the method of combining the P-delta and lateral load seismic actions suggested is on the conservative side for multi-storey structures. Figs. 4.12 to 4.15 show that enhancing the strength to allow for P-delta actions has an appreciable influence on the deflections which develop and on the deflected profile. This is particularly noticeable for the two frame structures, where the previous parabolic profile tends to straighten out. This is no doubt caused by the considerable increase in strength added to the lower part of the building due to the P-delta actions, which tend to make the upper portions of the building more suspectible to inelastic behaviour.

Ideally the proposed design method for P-delta actions should be checked with many more analyses and refined as a result. This step is beyond the scope of this project.

Chapter V - Summary and Conclusion

5.1 General

There is a marked difference in the shape of the design response spectra, or C values, proposed in the draft code to these in the current loadings code (1984). With the proposed changes there is an increase in the "C" values for low periods and a decrease for long periods. There are a number of implications for design if these proposals are accepted.

(i) The design earthquake forces for short period structures ($T \leq 0.8$ seconds) would increase significantly and in the design it would be necessary to establish the period of the structure as the "C" values varies rapidly with period in this range.

(ii) For long period structures ($T \geq 1.5$ seconds) there is a marked decrease in "C" value, which leads to a reduction in the design seismic actions in the structures overall. As a result:

(a) wind loading considerations are likely to become more important than they have in the past in establishing the required strength and stiffness of multi-storey structures,

(b) P-delta actions become relatively more important in ductile structures than was previously the case, and

(c) in modal response spectrum analyses the higher modes (2nd, 3rd, 4th etc) contribute much more to the combined modal actions than was previously the case. This increases the differences in the results obtained from equivalent static and modal methods of analysis.

5.2 Regular Walls and Frames

For the regular walls and frames the deflected shapes and overturning moments are dominated by the first mode contribution in the modal response spectrum analyses. The values are significantly less than those calculated by the equivalent static method. The reason for this discrepancy lies in the smaller participating mass in the first mode response compared with corresponding value inherently assumed in the equivalent static method. Time history analyses were carried out on those structures using an artificial earthquake record, which was developed to give the same response spectra as proposed in the draft code. It was found that the deflections at the top of the multi-storey structures in the time history analyses where P-delta effects were small, could be predicted, either by multiplying the modal response spectrum deflection by the ductility factor, or the equivalent static value by 0.85 times the ductility factor. However, the predicted profile of peak deflections at levels between the base and the top of the structure could not be predicted by scaling the modal or equivalent static values, such an approach would seriously under estimate the deflections at the lower floor levels.

In time history analyses of the walls yielding was found to occur over the lower three quarters of the wall. However, except at the base of the wall, where the majority of the hysteretic energy was dissipated, only low to moderate inelastic rotation demands were placed on the walls. It is suggested that for the slender form of walls considered in this project the design moment envelope suggested for capacity design in the commentary to the loadings code should be revised.

For the regular walls the shear predicted by the modal response spectrum method is greater than that calculated using the equivalent static approach, especially in the upper two thirds of the wall. The reason for this is that the higher mode effects become significant and these are not included in the equivalent static analyses. The modal shears in the walls when factored by the product of the overstrength and dynamic magnification factors (ϕ_o, ω) are of the same order as those observed in the time history analyses. The time history analyses however, show large localised maximum peaks, which the authors believe are caused by abnormalties in the way the structure is modelled. In particular the modelling did not allow for stiffness changes in shear due to diagonal crack formation or the yielding of shear reinforcement.

In the regular frames the modal response spectrum method predicts storey shears which are a little less than those obtained from the equivalent static method. In these structures the first mode contribution tends to dominate the response. The difference between the predictions of these two methods of analyses is of the order of 10 percent.

5.3 P-delta Effects

The inclusion of P-delta actions in the time history analyses lead to a significant increase in the deflections on the structures over 20 storey high. For the 30 storey wall collapse occurred. With the 12 and 24 storey regular frames the increase in deflections occurred primarily in the lower floors where the P-delta actions were at a maximum. This increase in deflection lead to an increase in the inelastic rotations imposed on the hinge zones in these levels. It is suggested that some additional strength should be added to the structure where P-delta actions are significant. A tentative method of assessing the increase in deflection and the increase in strength necessary to compensate for P-delta actions is proposed.

5.4 Irregular Frames

A number of podium with tower and frames with missing floors were analysed by the equivalent static and modal methods. Time and financial considerations did not permit time history analyses to be made on these structures. The analyses indicate that the equivalent static method is not suitable for the podium tower structures. With the modal analyses it was illustrated that it is important to include all significant modes in the modal response, as high contributions can arise at very high modes due to the vibration of the podium virtually on its own.

With the missing floor structures the equivalent static and modal response spectrum methods were in close agreement. However, with such structures caution is required to prevent non-ductile failure modes forming and to allow for the P-delta actions, which appear to be much more severe for this type of structure where the behaviour is similar to that of a single degree of freedom osscilator.

5.5 Further Work Requirements

Further research is required on the following aspects

(i) The effect of replacing the bi-linear behaviour by more realistic models on multi-storey structures with P-delta actions needed to be examined.

(ii) The effect of allowing for the reduction of shear stiffness and yielding of stirrups on the dynamic behaviour of walls should be investigated.

(iii) Analyses are required into the dynamic (time history) behaviour of irregular structures such as podium with tower and missing floor structures.

(iv) Analyses are required in which P-delta effects are examined in structures in which stiffness degradation occurs.

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