THREE DIMENSIONAL INELASTIC BUILDING RESPONSE TO SEISMIC LOADING

BY

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SUMMARY

The development of a three-dimensional inelastic time history computer code, in which the inelasticity arises from material yield, has enabled examination to be made of concurrent loading interaction between major and minor axis moments and axial force in ductile moment resisting frame structures. The predicted inelastic response characteristics and corresponding load patterns of a representative six storey reinforced concrete building, subjected to typical earthquake base motions, are compared with the load distrbutions determined on the basis of inelastic planar frame models. The non-linear reponse predicted by the three dimensional analysis is shown to be significantly different from the response based on a planar frame idealisation.

BACKGROUND

In-elastic behaviour is recognised as the primary energy absorbing mechanism in the response of multi-storey reinforced concrete buildings subjected to strong earthquake ground motion. The development of step-by-step integration techniques has enabled inelastic response to be predicted analytically. (1)

In present analyses estimates of the response of three dimensional structures are projected from a study of independent planar frame sub-structures or from a series of them. The planar frame model is useful in predicting the qualitative properties of a structure's behaviour, but may err grossly in providing the quantitative responses which are of particular interest to the designer, for example interstorey drifts and seismic gap displacements. Alternatively three dimensional structures are idealised by a shear-type model analogy consisting of rigid floors interconnected by flexible columns notwithstanding the fact that a shear-type model is in direct conflict with the currently accepted design objective of a "weak girder, strong column" strength heirachy.

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These considerations prompted the development of a computer program which can be used to predict the time-history response of three dimensional frame structures to earthquake ground motion; (2). The program is not restricted to a rectangular grid layout but can model a completely general structure geometry. Yield surface interaction is considered in both beams and columns with a library of surface options available depending on the principal structural action, or actions, of the component elements. For a typical beam element, the yield states may be a function of the principal bending moment only, whereas for an exterior column account can be taken of biaxial bending coupled with axial load.

THE STRUCTURE ANALYSED

The six-storey, two by one bay, structural frame analysed is shown in Fig. 1. The design level of seismic resistance is representative of the intensity specified in current loading codes; (3). The member properties assumed for the analysis are summarised in Table 1. To determine the effective section stiffness parameters, which are representative of a cracked section, a simplified procedure was adopted incorporating an equivalent second moment of area equal to 75% of the gross uncracked value and assuming the width of the slab effective as a T-beam flange to be one eighth of the span length of the beam.

The yield surface parameters applied in the analysis are summarised in Fig. 2. In the post-yield region the beam response is assumed to be perfectly plastic. Since the structure was designed in accordance with the capacity design philosphy with beam hinges forming the essential part of the energy-dissipating mechanism and an overstrength capacity factor applied in the design of the column elements the yield surface choice can be justified. However, since the possibility of hinge formation in the ground floor column zone cannot be discounted, provision for a yield surface for each ground floor column is made as is indicated in Fig. 2.

A lumped mass model was used to represent the mass properties of the structure. Two distributions were assumed in the analyses, the first with the centre of mass coincident with the centre of elastic resistance and the second with a 10% static eccentricity between the centres of mass and eccentricity applied in both the major and minor plan axes at each level.

Viscous damping effects were included in the mathematical model using a Rayleigh distribution: 10% critical damping was assigned at each of the two periods 0.77 seconds and 0.26 seconds respectively.

The dynamic properties of the elastic models are presented as the eigen problem solution for the balanced and eccentric mass distribution assumptions. The properties of the first three modes in each instance are summarised schematically in Fig. 3. In the balanced case, the modes correspond to two distinct lateral displacement modes and a torsional one. For the eccentric model a strong degree of coupling in the displacement components is observed. It is possible to classify the three modes as predominately displacement or torsional configurations the similarity with the related balanced properties being apparent.

The horizontal ground acceleration components recorded in the El Centro, May 18, 1940 earthquake scaled by a factor of 1.5 provided the source of the input earthquake loading for all the comparisons made. In the elastic case the response of the structure was determined for the first five seconds of the earthquake time-history (Fig. 4). In the inelastic case limitations were imposed because of computer resource restrictions. To remain within the bounds of available computer allocation the analyses were confined to approximately 400 seconds of process time. In the case of a yielding structure subjected to concurrent earthquake ground motion this corresponded to approximately 3.2 seconds of earthquake duration. From consideration of the elastic time-history it may be concluded that a relatively small displacement response is excited in the structure during the first second of the earthquake. In order that the structure was subjected to a significant duration of potential non-linear response in the alloted analysis time the earthquake ground motion for the first second of the time-history was skipped in the non-linear studies. Five additional linear increments were introduced at the start of the assumed record to avoid a sudden impulse loading on the structure arising from the displaced origin.

RESPONSE OF THE BALANCE MASS MODEL

The time-history displacement summary of the symmetric building subject to unidirectional earthquake loading is presented in Figs. 4 and 5. The non-linear response (Fig. 5) exhibits the characteristics which have been observed in previous studies based on a planar frame idealisation. Following the onset of yield the effective modal period shows a lenghtening and the structure vibrates about a displaced equilibrium position which is dependent on the time-history of the non-linear deformation. The hinge formation in the beams follows the now familiar pattern in which a band of plastic hinging migrates up the structure. During sequences of strong ground motion the band can extend the full height of the structure.

When the simultaneous occurrence of an orthogonal earthquake component is included the displacement time-history is significantly modified. In Figs. 6(a) and (b) the X and Y coordinate displacements for Level 6 of the building are summarised for concurrent earthquake loading. During the early cycles of yield the pattern of hinging is similar to that observed in the unidirectional earthquake analyses. A band of hinging migrates up the structure in both orthogonal directions. The distribution of yield is generally symmetric at each level in the structure, indicating a lateral displacement dominated response, with little torsional induced effects. Although an asymmetric yield pattern occurs for brief instances its duration is insufficient for a torsional response to develop.

The initial consequence of concurrent loading is to precipitate the onset of first yield and to extend the duration of the yield sequences.

As the duration of earthquake loading progresses, the concurrent earthquake response diverges from the single component response predictions. When the loading approaches its peak intensity, the yield distribution becomes less uniform because of the interaction of the orthogonal displacement components in defining the locus of the yield point on the assumed yield surface, and a torsional vibration pattern emerges. The torsional mode is enhanced by the occurrence of hinges in some of the ground floor columns which effect a sudden significant shift in the instantaneous centre of stiffness at this level which is accompanied by an applied torque about the instantaneous centre of stiffness. It is evident that the torsional mode increasingly dominates the response as the analysis progresses. The duration of earthquake loading considered was insufficient to resolve whether the torque asymptotes to a peak valve or, alternatively, whether it is damped out by the lateral displacement response.

A typical comparison between the results of unidirectional and concurrent earthquake loading is given in Fig. 8. Evidently the non-linear response predicted by a full three-dimensional analysis is significantly different from the response based on a planar frame idealisation since the planar frame model is unable to reproduce all the structural papameters which participate in determining the response of the structure.

RESPONSE OF THE ECCENTRIC MASS MODEL

In Figs. 6(a) and 7(a) the X direction displacement time-histories of the top floor storey building are plotted for the two cases representing the balanced and eccentric mass distributions respectively. A similar comparison for the Y direction components is given in Figs. 6(b) and 7(b). It is seen that the inclusion of 10% static eccentricity induces a torsional deformation in the response prediction in both the elastic and post-elastic phases, with the torsional contribution becoming more dominant in the latter. Before the structure is subjected to significant post-elastic deformation it is possible to relate the response characteristics of the two models. If a torsional deformation is superimposed on the lateral displacements derived from the balanced model, the displacement of the perimeter frames in the eccentric model can be resolved. Following the onset of yield, the eccentricity between the centre of mass and the instantaneous centre of stiffness at each level is no longer constant - the instantaneous torque may be increased or decreased depending on the current distribution of the yielding elements. The superposition comparison is no longer valid.

In the initial elastic section of the response, the elastic displacement of the balanced building serves as a useful prediction of the average response characteristics of the eccentric model. In the post-yield section the torsional contribution is the result of a complex interaction between the centre of mass and the instantaneous centre of stiffness. It is no longer possible to predict the displacement profile on the basis of a projection of displacement time-history produced from an elastic three-dimensional analysis. In a similar manner, the post-elastic response derived from a simplified two-dimensional planar frame idealisation does not model the torsional imbalance observed in the response of three-dimensional structures. A significant torsional deformation can be identified in the response of the initially balanced structure as a consequence of the asymmetric distribution of yield.

CONCLUSION

The analyses of a three-dimensional model enables the determination of characteristics which are not predicted when two-dimensional models are used in determining the earthquake response of frame structure. The displacement time-histories reflect less of the ground motion displacement reversals, the duration of yielding sequences are generally extended and the deformations tend to be increased at lower levels although the displacement components at roof level were found to be essentially of the same magnitude regardless of whether the structure was assumed to remain elastic or to yield, or whether a two or three dimensional model was adopted.

A significant contribution to overall structural response can be attributed to torsional vibration modes. Traditionally the non-linear response of three dimensional frame structures has been estimated on the basis of planar frame idealisations. It is apparent that the response derived on the basis of this planar assumption should be interpreted, in many instances, as a qualitative indication of the hieracy of the yield strengths of the individual members only. A full three dimensional analysis is necessary before an accurate prediction of the time-history displacement response of a structure is possible.

REFERENCES

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DESCRIPTION	AREA (m^2)	I_{yy} (m^4)	I_{zz} (m^4)	J (m^4)	RIGID END BLOCK (m)	F.E.S. (kN)	F.E.M. (kN-m)
Beam							
Levels 1-3 perimeter	0.1050	2.3×10^{-3}	5.9×10^{-3}	1.0×10^{-4}	0.300	25.0	20.0
interior	0.1050	2.3×10^{-3}	5.9×10^{-3}	1.0×10^{-4}	0.300	25.0	40.0
Levels 4-6 perimeter	0.1015	2.3×10^{-3}	5.4×10^{-3}	1.0×10^{-4}	0.275	25.0	20.0
interior	0.1015	2.3×10^{-3}	5.4×10^{-3}	1.0×10^{-4}	0.275	25.0	40.0
Column							
Levels G-3	0.1250	3.91×10^{-3}	3.91×10^{-3}	1.0×10^{-4}	0.250		
Levels 3-6	0.1013	2.56×10^{-3}	2.56×10^{-3}	1.0×10^{-4}	0.225		

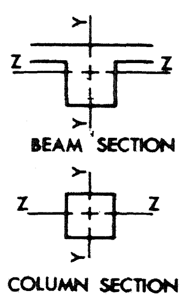


TABLE 1 SUMMARY OF MEMBER SECTION PROPERTIES

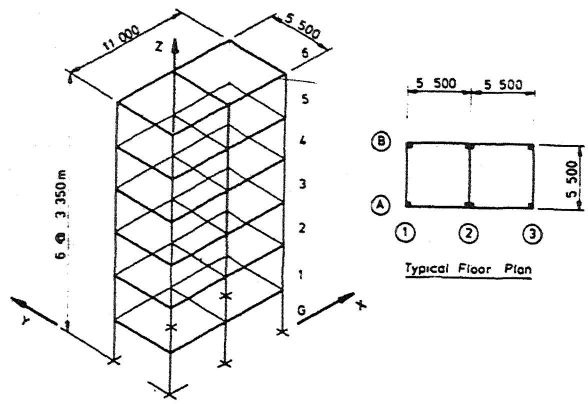
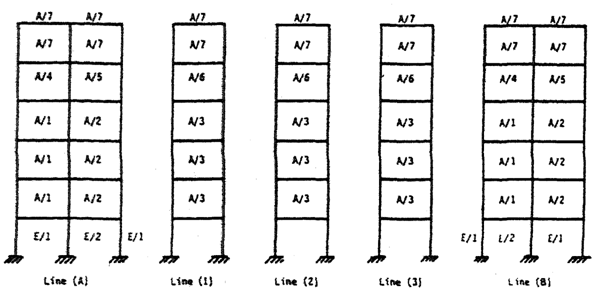
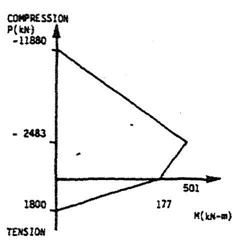


FIG. 1 SIX-STOREY BUILDING FRAME

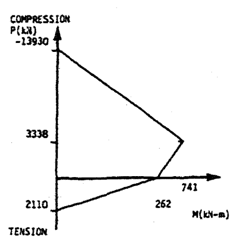


TYPE A YIELD SURFACE SPECIFICATION	HINGE LENGTHS		YIELD MOMENTS			
			END (I)		END (J)	
	END (I) (m)	END (J) (m)	POSITIVE (kN-m)	NEGATIVE (kN-m)	POSITIVE (kN-m)	NEGATIVE (kN-m)
A/1	0.410	0.410	208.0	208.0	190.0	190.0
A/2	0.410	0.410	190.0	190.0	208.0	208.0
A/3	0.410	0.410	208.0	208.0	208.0	208.0
A/4	0.375	0.375	160.0	131.0	115.0	137.0
A/5	0.375	0.375	115.0	137.0	160.0	131.0
A/6	0.375	0.375	160.0	131.0	131.0	160.0
A/7	0.375	0.375	115.0	115.0	115.0	115.0



Hinge Length = 0.350 m

(a) TYPE E/1 YIELD SURFACE (Axi-Symmetric)



Hinge Length = 0.350 m

(b) TYPE E/2 YIELD SURFACE (Axi-Symmetric)

FIG. 2 SUMMARY OF YIELD SURFACE PARAMETERS

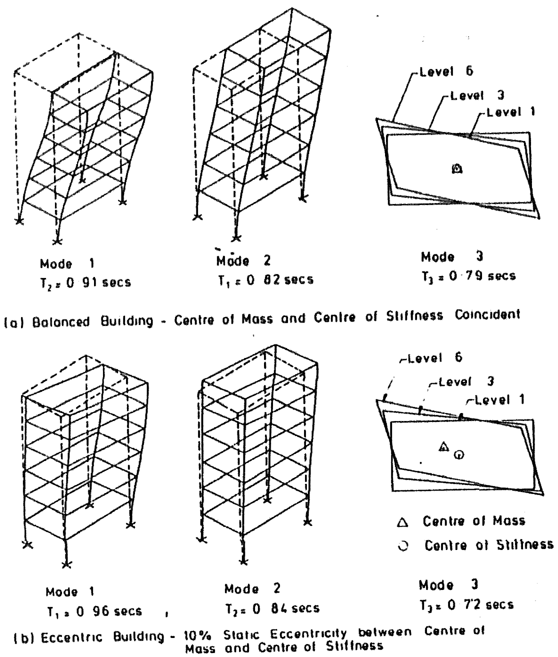


FIG. 3 COMPARISON OF THE MODEL PROPERTIES OF THE BALANCED AND THE ECCENTRIC STRUCTURES

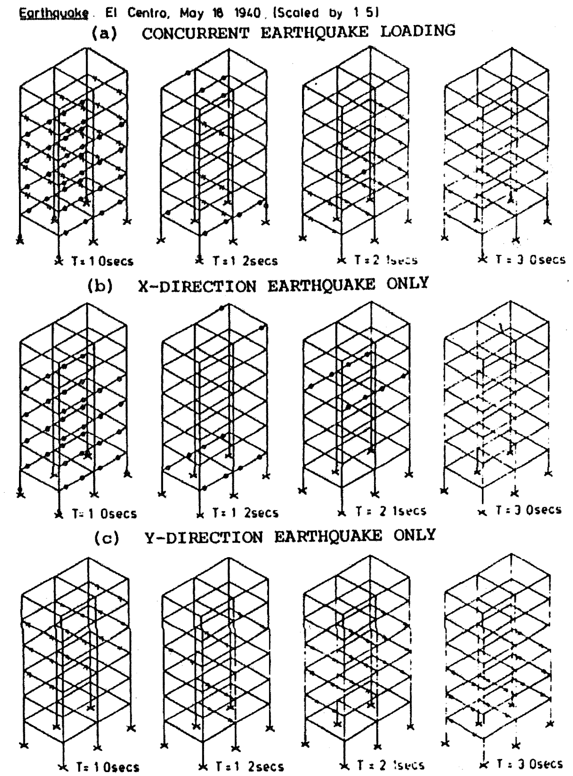
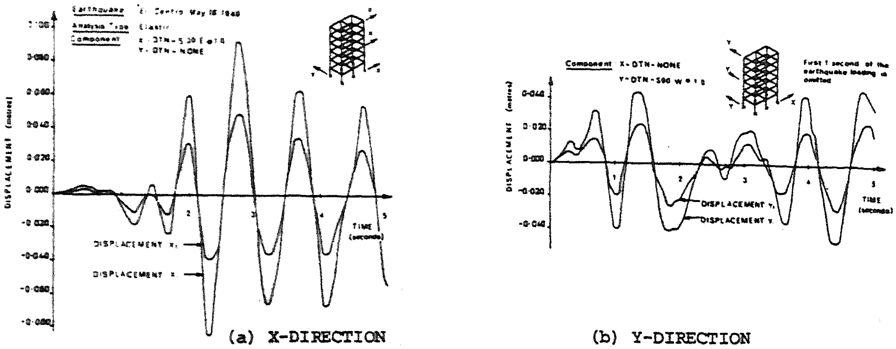
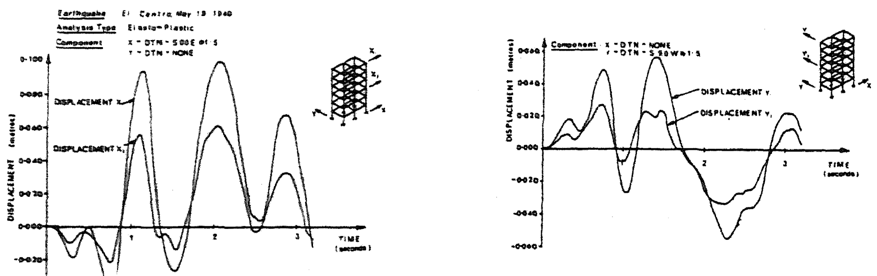


FIG. 8 COMPARISON OF THE YIELD DISTRIBUTION AT SELECTED INTERVALS WITH UNI-DIRECTIONAL AND CONCURRENT EARTHQUAKE LOADING



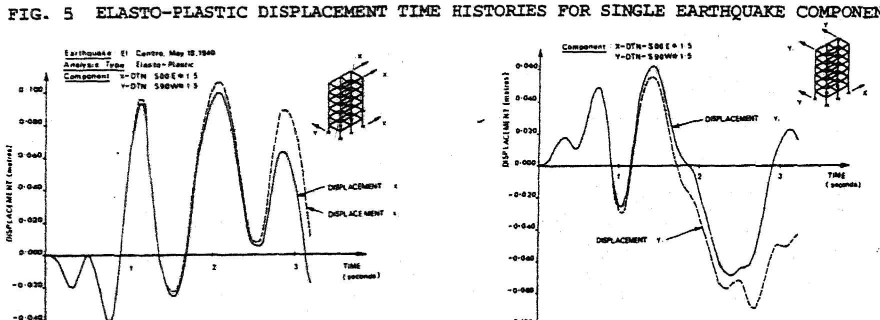
(a) X-DIRECTION (b) Y-DIRECTION

FIG. 4 ELASTIC DISPLACEMENT TIME HISTORIES FOR SINGLE EARTHQUAKE COMPONENTS



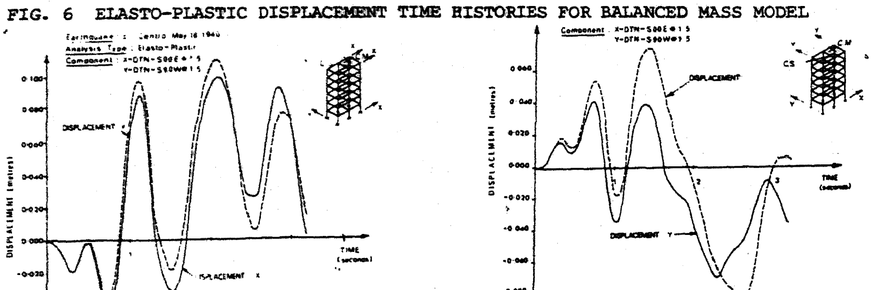
(a) X-DIRECTION (b) Y-DIRECTION

FIG. 5 ELASTO-PLASTIC DISPLACEMENT TIME HISTORIES FOR SINGLE EARTHQUAKE COMPONENTS



(a) X-DIRECTION (b) Y-DIRECTION

FIG. 6 ELASTO-PLASTIC DISPLACEMENT TIME HISTORIES FOR BALANCED MASS MODEL



(a) X-DIRECTION (b) Y-DIRECTION

FIG. 7 ELASTO-PLASTIC DISPLACEMENT TIME HISTORIES FOR ECCENTRIC MASS MODEL