INELASTIC BEHAVIOUR OF THREE-DIMENSIONAL STRUCTURES UNDER CONCURRENT SEISMIC EXCITATIONS

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SUMMARY

The paper discusses the earthquakes' phenomenon of generating three concurrent orthogonal excitations. The study considered only the two horizontal earthquake radial and tangential components. The SRSS and λ-percent combination methods were investigated for their reliability in the evaluation of the maximum structural response. The results of an extensive elastic and inelastic numerical time-history study using three-dimensional structures were discussed. These results confirmed that the SRSS and the 30-percent combination methods are inappropriate. The behaviour of the studied inelastic structures was not much different from the elastic structures. The study suggests a 45-percent response combination rule to be used in the elastic and inelastic analyses of structures.

INTRODUCTION

Properties of Seismic Excitations

Earthquakes have three translational components. The translational components are two horizontal and one vertical. The horizontal components are the radial and tangential components as shown in Figure 1. According to Penzien and Watabe (1975) the radial component is following the path from the epicentre to the site of structure and it is the major principal component. The tangential component is orthogonal to the radial and is the intermediate component. The vertical is the minor principal component. At any given time the direction of the horizontal components is instantaneously varying. This direction variation is attributed to (1) the spatial change of the rupture focus on the fault with time and (2) the different soil layers, formation and composition on the path of wave transfer. The different soil formation causes series of reflection and refraction of the seismic shear waves, S_V and S_H, all along their transfer path and hence changing the direction of the horizontal components.

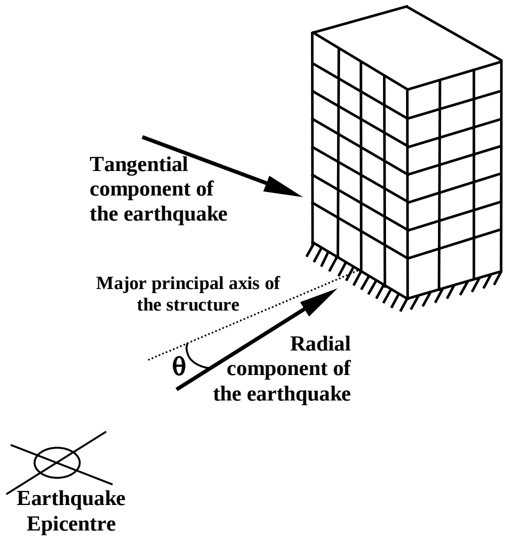

Figure 1: Radial and Tangential Horizontal Components of Earthquakes

Penzien and Watabe (1995) suggested a transformation and time segmentation scheme of the arbitrary earthquake records in order to identify the principal directions of the excitation components. This scheme transforms angularly the records and uses time intervals of few seconds such that the covariance of the excitation components approaches zero and therefore the transformed component records become independent. However, if the time intervals approach zero the fluctuation of the resultant direction becomes very random and its components do not follow the suggested principal directions as indicated in Figure 2.

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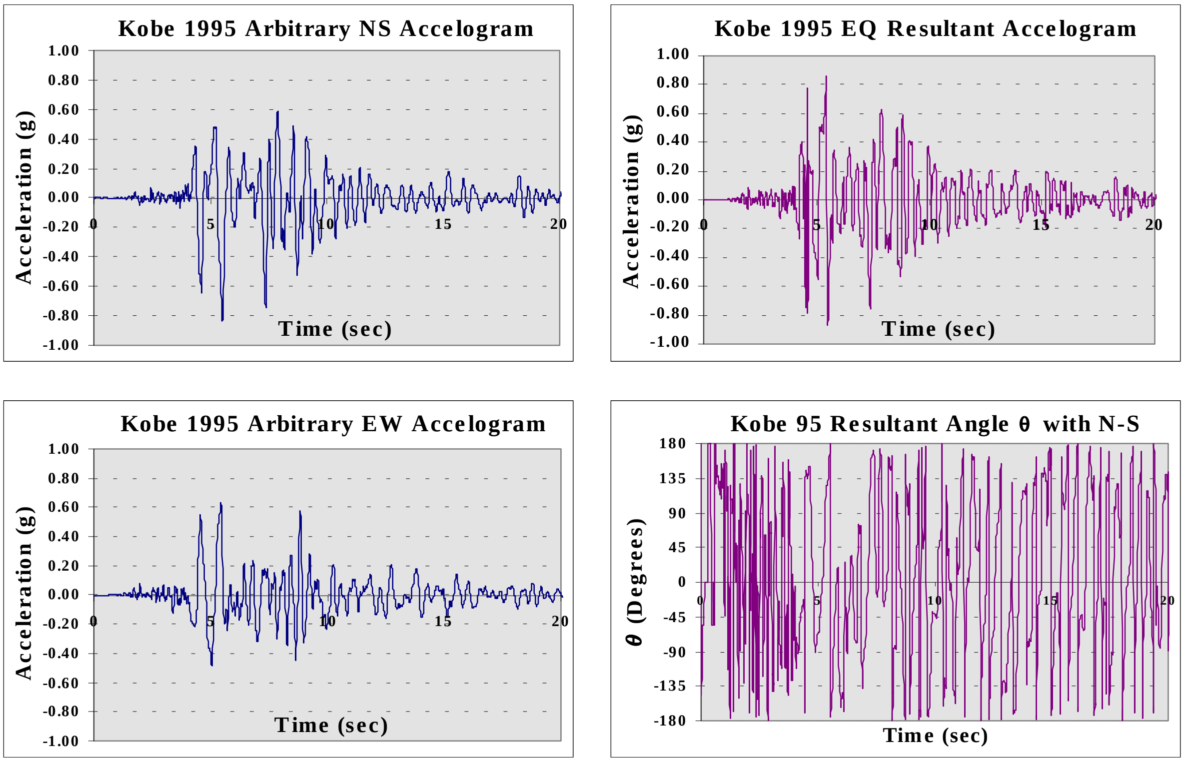


Figure 2: Kobe 1995 Arbitrary Earthquake Horizontal Accelograms, Acceleration Record of the Resultant and the Angle between the Resultant and N-S direction.

Independence of Earthquake Components

The assumption of independence of earthquake excitations is used in the study of seismic multi-component and stochastic responses of structures. The independence assumption was used in the derivation of the SRSS (Clough and Penzien 1993) and CQC (Wilson et al 1981) combination methods. For stochastic seismic response, the earthquake input is formulated as quasi-stationary process of the following form:

$$a_i(t) = \zeta_i(t) \cdot z_i(t) \qquad i=x,y,z \qquad (1)$$

where $a_i(t)$ is the ground acceleration at time t, $z_i(t)$ is a stationary process and $\zeta_i(t)$ is a deterministic random function.

Multi-component Seismic Excitations and Combination Rules

Extensive studies have been carried out on the behaviour of structures under multi-component earthquake excitations. These studies use either elastic modal response analysis (Smeby and Der Kiureghian 1985, Hisada et al 1987, Ger and Cheng 1990, and López and Torres 1997) or have probabilistic bases (Rosenblueth and Contreras 1977) that make them inappropriate for inelastic applications. Contemporary codes and softwares follow these studies and evaluate the structural combined response, R_C, by combining the orthogonal structural responses. These structural responses are evaluated by applying the same code spectrum or equivalent static forces separately to each of the two principal directions of the structure. On using the assumption of independence of earthquake components; the two orthogonal responses of the structure are uncorrelated, thus the combined response is computed either by using the SRSS of the two responses

$$R_C = (R_1^2 + R_2^2)^{1/2} \qquad (2)$$

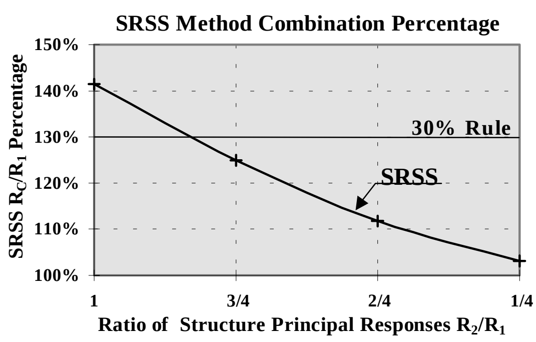


Figure 3. Combination % of SRSS and λ Methods.

or by using the λ-percent rule

$$R_C = R_1 + \lambda R_2 \quad (3)$$

where R_1 is the peak structural response to the seismic excitation along the structure's major principal axis, R_2 is the response in the orthogonal direction to R_1 and always less or equal to R_1, and λ is the orthogonal combination factor. The orthogonal combination factor, λ, generally has a value of 30% as in many codes, such as the AS 1170.4 (1993), ISO 3010:1988 (E), UBC (1997) and Eurocode 8 (1994). However, fewer codes use percentages for λ as high as 40%. Figure 3 shows the values of λ for the SRSS method if formulated as equation (2) and using R_2 as some ratio of R_1.

The objective of this paper is to investigate the behaviour of structures under orthogonal seismic excitations, and hence scrutinise the current combinational methods of bi-directional responses. In addition the study will examine the independence of earthquake horizontal components. The study will be performed using elastic and inelastic time-history analysis.

ORTHOGONAL EFFECTS OF EARTHQUAKES

Orthogonal Excitations

The orthogonal excitations generally generate in the structure members sets of bending moments and shear forces in both directions, along with additional normal forces. In Figure 4, $M_{x,R}$, as an example, is the bending moment generated in the x-axis of the column section due to the radial component of the earthquake. When using the modal response method (Gupta 1990), $M_{x,R}$ is calculated by using the CQC method (Wilson et al 1981) to evaluate it from the response of the contributing modal shapes. M_x is generated from $M_{x,R}$ and $M_{x,T}$ either by using the SRSS (2) or the λ-percent rule (3).

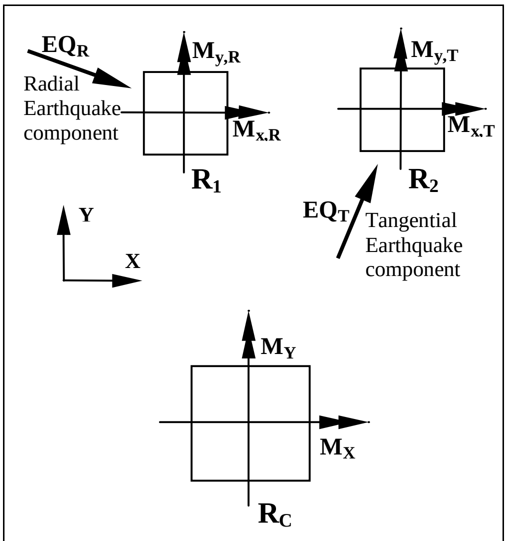


Figure 4. Orthogonally generated Bending Moments in a Column Section.

Orientation of Earthquake Resultants

At any given time, the structure responds to a resultant of the bi-directional horizontal excitation that changes its direction, orientation angle θ, continuously with time as shown in Figure 2. Columns are good example for the study of effects of earthquake resultant orientation. Unless the structure's columns have the same cross section and properties in both directions, they will not share their individual maximum biaxial response at the same θ. Therefore, in the general case one can argue that there is no definable worst orientation of the earthquake excitation input for the whole structure but there are distinct different θ's for certain investigated members.

According to NZS 4203:1992, modal analysis of structures uses a unidirectional spectrum, determined by the code according to the soil type and ductility factor, that is applied to the structure in the direction of the maximum response. In the authors' understanding, this unidirectional spectrum stands for the resultant that the structure responds to. The main problem with this approach is a certain orientation of this resultant unidirectional spectrum will favour the response of some members but not all. Rather than conducting a thorough investigation using many different orientation angles, the method of Wilson et al (1995) can be employed. Wilson's Method is an extension to the study of Wilson and Button (1982). However this method assumes a full 1:1 correlation between the orthogonal seismic inputs and the stability of the orientation angle of the earthquake resultant over a finite period of time.

RESULTS OF THE TIME-HISTORY ANALYSIS

Time-history analyses were carried out using SAP90 (Wilson and Habibullah 1991) on eighteen structures using eleven earthquakes. The 18 structures are 12-, 9- and 6-storey multi-bay 3-dimensional frames and they are rectangular in plan as listed in Table 1. Structural floors act as rigid diaphragms maintaining equal X and Y displacements and Z rotation for all nodes of each storey. They are fully symmetric and involve no eccentricities. The time-history analysis used 11 pairs of horizontal orthogonal accelograms of selected earthquakes. No vertical components were used as the effect of the vertical excitation can be ignored in practice. The selection of this set of earthquakes was based upon their accelograms and acceleration response spectra. Their acceleration spectra well cover the fundamental X and Y periods of the studied structures. These accelograms and acceleration spectra have different patterns that nearly cover the various known ones. The used accelograms are those recorded in the arbitrary orthogonal directions set for accelerometers of the recording station. Later, the accelograms are to be transformed to their principal directions according to Penzien and Watabe (1975). The earthquakes used are as follows:

1. El Centro 1940.
2. El Centro (Imperial County Service Bldg.) 1979.
3. Kobe (Kobe JMA Observatory) 1995.
4. Mexico City (Oficina) 1985.
5. Mexico City (SCT) 1985.
6. San Fernando (Pacoima Dam Station) 1971.
7. Parkfield (Tremblor) 1966.
8. Northridge (Pacoima Dam Station) 1994.
9. Northridge (Sylmar) 1994.
10. Loma Prieta (Corralitos) 1989.
11. Loma Prieta (Treasure Is.) 1989.

For each time-history analysis, one earthquake component record is selected to be applied to the weak direction of the structure (y-axis) and the other component is applied to the strong direction (x-axis). The selected record is the one that provides the maximum acceleration, from its acceleration spectra, for the weak direction period (T_Y). For example, as shown in Figure 5, for the 4x3 12-storey building, Kobe 1995 NS component has larger acceleration at T_X than the EW component, hence the NS component was assigned to the weak y-axis of the structure and the EW component was assigned to the x-axis.

For each analysis, time history of displacements and rotations of ground floor corner column and the top of building were evaluated. As shown in Figure 6, time $t(R_{Y,max})$ is identified for each maximum response of displacement or rotation of the weak direction $R_{Y,max}$. Note that $R_{Y,max}$ is R_1 in equation (2). For each time $t(R_{Y,max})$, the corresponding response of the strong direction, $R_X{:}t(R_{Y,max})$, is found. For each case, $R_X{:}t(R_{Y,max})$ is divided by the maximum response in the strong direction, $R_{X,max}$. This ratio is the percentage activated of the maximum strong-axis response at the time of maximum weak-axis response, i.e., λ.

Several graphs of the values of λ were plotted against the ratio of the structure's fundamental periods (T_Y/T_X) as illustrated in Figures 7 to 12. All plots share the same character of having λ values exceed 30% and approach 100% in some cases. They also have no fixed pattern and generally show a large scatter.

Table 1. The X and Y Fundamental Periods of the Structures analysed using SAP90.

No. of Storeys	X-bays x Y-bays	T_X (sec)	T_Y (sec)
12	8 x 3	1.904	2.107
	8 x 1	1.691	2.078
	4 x 3	1.873	1.944
	4 x1	1.423	1.802
	2 x 1	1.408	1.625
	1 x 1	1.386	1.386
9	8 x 3	1.374	1.510
	8 x 1	1.025	1.327
	4 x 3	1.342	1.389
	4 x1	1.013	1.247
	2 x 1	0.991	1.123
	1 x 1	0.957	0.957
6	8 x 3	0.852	0.929
	8 x 1	0.627	0.782
	4 x 3	0.824	0.850
	4 x1	0.614	0.733
	2 x 1	0.593	0.659
	1 x 1	0.560	0.560

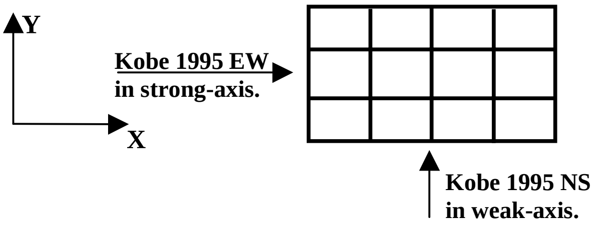


Figure 5: Example of Assignment of EQ Components.

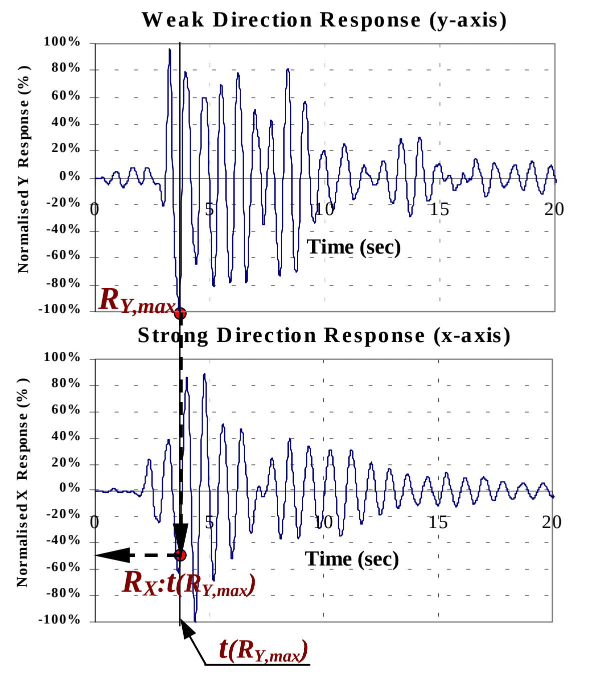
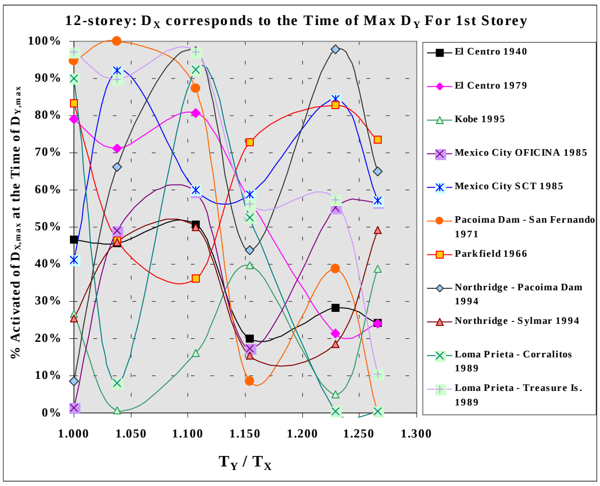


Figure 6: Evaluation of R_X (of strong axis) that corresponds to the Time of Max. R_Y (of weak axis).



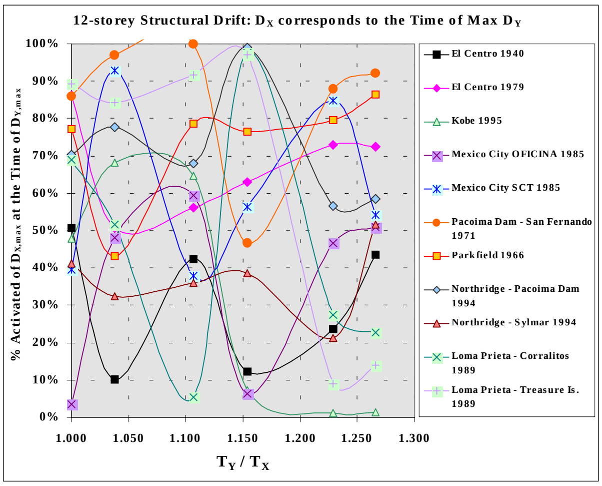
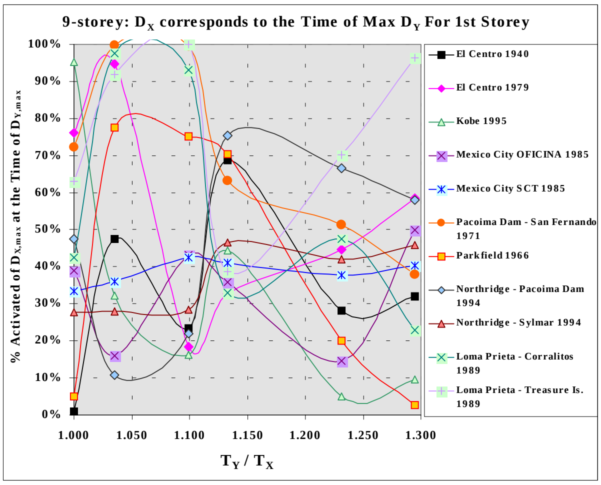


Figure 7. 12-storey λ of Ground Storey Displacements. **Figure 8. 12-storey λ of Top Storey Displacements.**



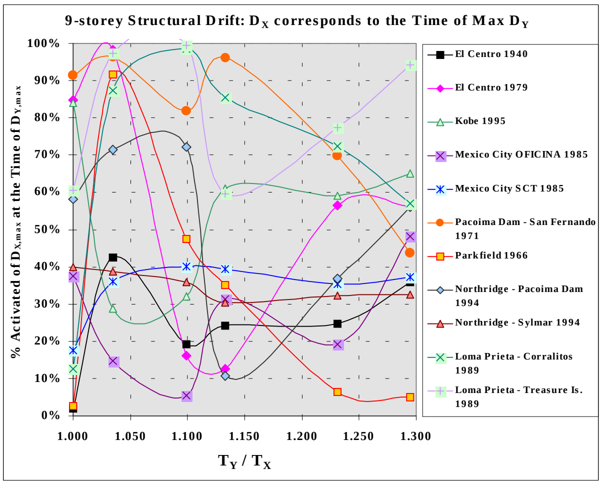
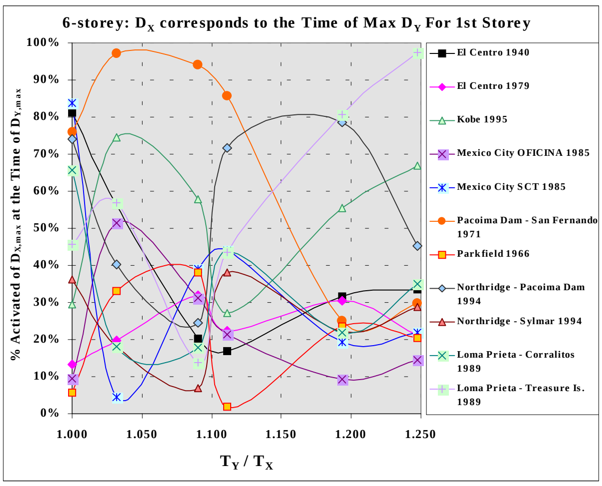


Figure 9. 9-storey λ of Ground Storey Displacements. **Figure 10. 9-storey λ of Top Storey Displacements.**



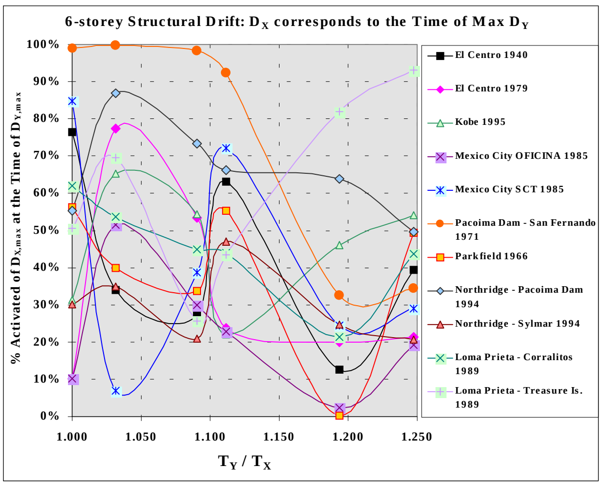


Figure 11. 6-storey λ of Ground Storey Displacements. **Figure 12. 6-storey λ of Top Storey Displacements.**

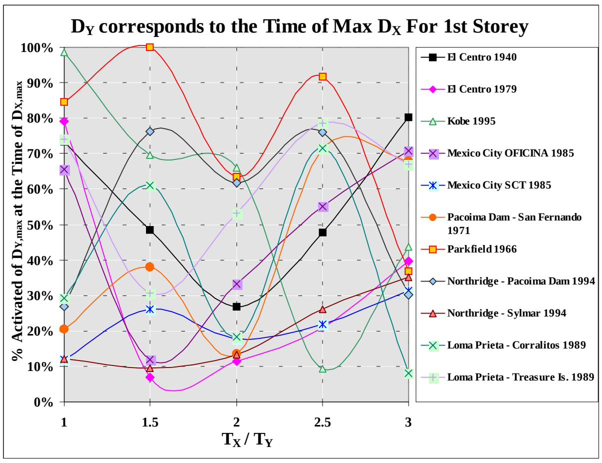
Elastic Study using Earthquake Principal Components

Time-History analyses were performed on the 12-storey structures using the horizontal principal components of the selected earthquake. These principal components were evaluated by angular transformation of the available 2 arbitrary orthogonal records such that their covariance becomes zero. The transformation was made for the full record and not for time segments. This was based on the observation of Penzien and Watabe (1975) of the reasonable stability of average principal directions for the earthquake's full record.

The results of λ came with no difference in character than for the previous elastic analyses using the arbitrary horizontal components of the selected earthquakes. λ values exceeded 30%, approached 100% in some cases, and generally exhibited a large scatter as in the previous analyses. These results show that there are some correlation between the earthquake components. The plots were not provided due to space limitations.

Inelastic/Elastic Study using 3D Ruaumoko

Time-History analyses, using 3D Ruaumoko (Carr 1999), were performed on 12-storey structures using previously selected earthquakes. The structures analysed are 2-X-bay x 2-Y-bay symmetric frame structures but of different column and beam properties. There are 5 structures in total having different fundamental period ratios (T_X/T_Y) ranging from 1 to 3. The time-history analyses were run inelastically and elastically for the sake of behaviour comparison. Inelastic behaviour was allowed in the ground storey columns and all beams. No biaxial strength interaction was considered in this study. Rigid floor diaphragms were used to maintain equal X and Y displacements and Z rotation for all nodes of each storey. Values of λ for displacements of ground and top storeys are illustrated in Figures 13 to 16.



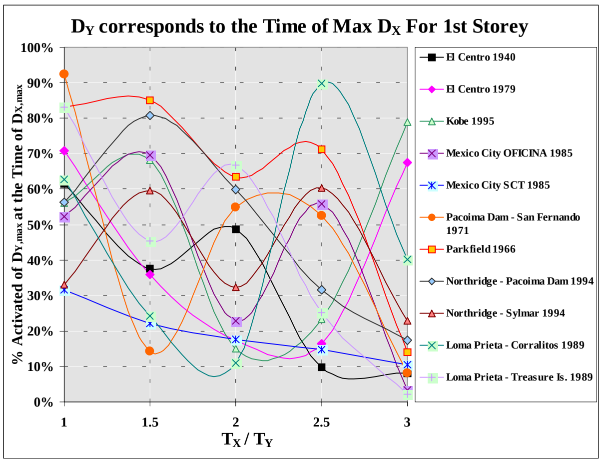
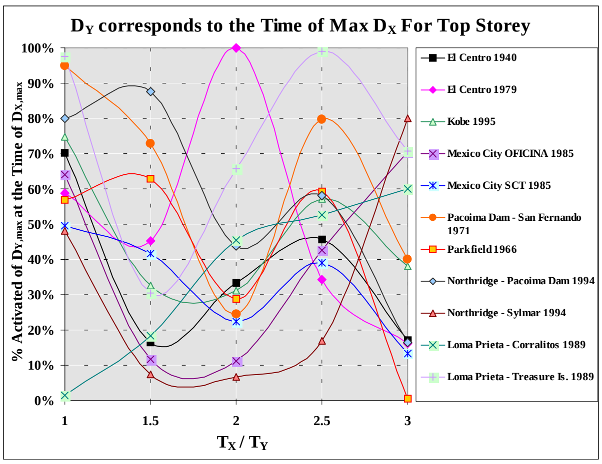


Figure 13. Inelastic λ of Ground Storey Displacements. **Figure 14. Elastic λ of Ground Storey Displacements.**



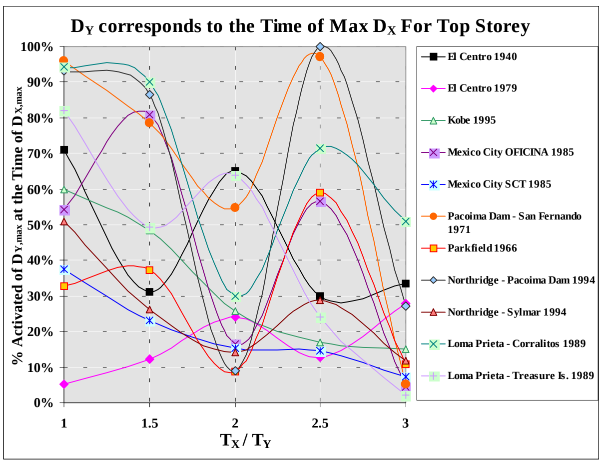


Figure 15. Inelastic λ of Top Storey Displacements. **Figure 16. Elastic λ of Top Storey Displacements.**

Figures 13 to 16 show that λ values exceeded 30%, approached 100% in some cases, and generally exhibited a large scatter as in the previous analyses. The elastic behaviour showed less scatter than the inelastic one. Values of λ in the inelastic structures showed an average of 46% with a maximum of 99.9%, a minimum of 0.5% and average standard deviation of 27%. Values of λ in the elastic structures showed an average of 43% with a maximum of 99%, a minimum of 2% and average standard deviation of 25%.

CONCLUSIONS

1. Structures and its members using the time-history method. The study confirmed that the SRSS and the 30-percent methods are inappropriate. The study suggests a 45-percent rule for biaxial response combination, albeit with a significant high standard deviation of 25%. This is applicable to the biaxial moments and shears of columns and the total structural biaxial drift.

2. The high values of the orthogonal combination factor, λ, evaluated in the study suggests the existence of an inter-correlation between the earthquake components, i.e., they are not independent as suggested by Penzien and Watabe (1975).

RECOMMENDATIONS

1. Each of the principal directions separately and to use the proposed 45-percent rule for evaluating the biaxial moments and shears of the columns.

2. When using programs such as SAP90 or ETABS it is strongly recommended to solve the bi-directional excitation separately to avoid the inappropriate program application of the SRSS rule.

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