

## INELASTIC ANALYSIS OF THE IMPERIAL COUNTY SERVICES BUILDING

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### Summary:

Full scale ambient and forced vibration dynamic measurements of buildings have become a continuing research activity over the last twenty years or so. One building that has been studied in such a way was the Imperial County Services Building which was also instrumented with a number of seismographs. On the 15 October 1979, this building suffered major structural damage in a powerful earthquake.

This paper briefly describes the ICSB and the damage that occurred, before discussing the computer models that were analysed to study the likely inelastic behaviour of the building. Though the analytical time histories do not show very good agreement with the seismograph results, they do show that the transverse component of the earthquake had a significant effect on the behaviour of the building on account of a non-symmetric arrangement of the ground floor shear walls in the transverse direction. Another factor was the placing of a ground level slab above the detailed column hinges.

### INTRODUCTION:

Full scale ambient and forced vibration dynamic measurements of buildings have been a continuing research activity in many parts of the world over the last two decades. In addition, the last decade has seen the emergence of strong motion data recorded by seismographs in a number of buildings caused by significant seismic events.

One such recent research effort was devoted to an in-depth experimental and analytical study of the Imperial County Services Building (ICSB) in El Centro, California (see figures 1-3). This research project was sponsored by the U.S. National Science Foundation under its "Research Initiation in Earthquake Mitigation Program".

The experimental component of the research project was devoted to the measurement of low level structural excitations caused by ambient and forced vibrations. The analytical component was devoted to developing a valid mathematical model to accurately represent the low level forced vibration.

The potential for full scale testing due to strong motion existed since the structure was in a highly seismic area, and for this reason the building had been instrumented under the California Strong Motion Instrumentation Program. On the 15 October 1979 this earthquake potential became a reality when a powerful earthquake jolted Southern California and Northern Mexico causing extensive damage to the Imperial County Services Building. The earthquake, which measured 6.4 on the Richter Scale, was centred on the Imperial Fault near the U.S. - Mexico border. The response of the ICSB to the earthquake stimulated interest within the engineering community because the buildings response represents one of the few responses measured in a building that has received major structural damage. This paper, one of several related to the Imperial County Services Building (2,6,7) is devoted to the presentation of the computer model results for the inelastic analyses.

### BUILDING DESCRIPTION

The Imperial County Services Building (ICSB) served as an office building for Imperial County. It was designed in 1968 (using the 1967 edition of the Uniform Building Code) and was completed in 1971 at a construction cost of \$1.87 million. The building is 41.7 m by 26 m in plan and is founded on a Raymond step-taper concrete pile foundation (see Figure 4).

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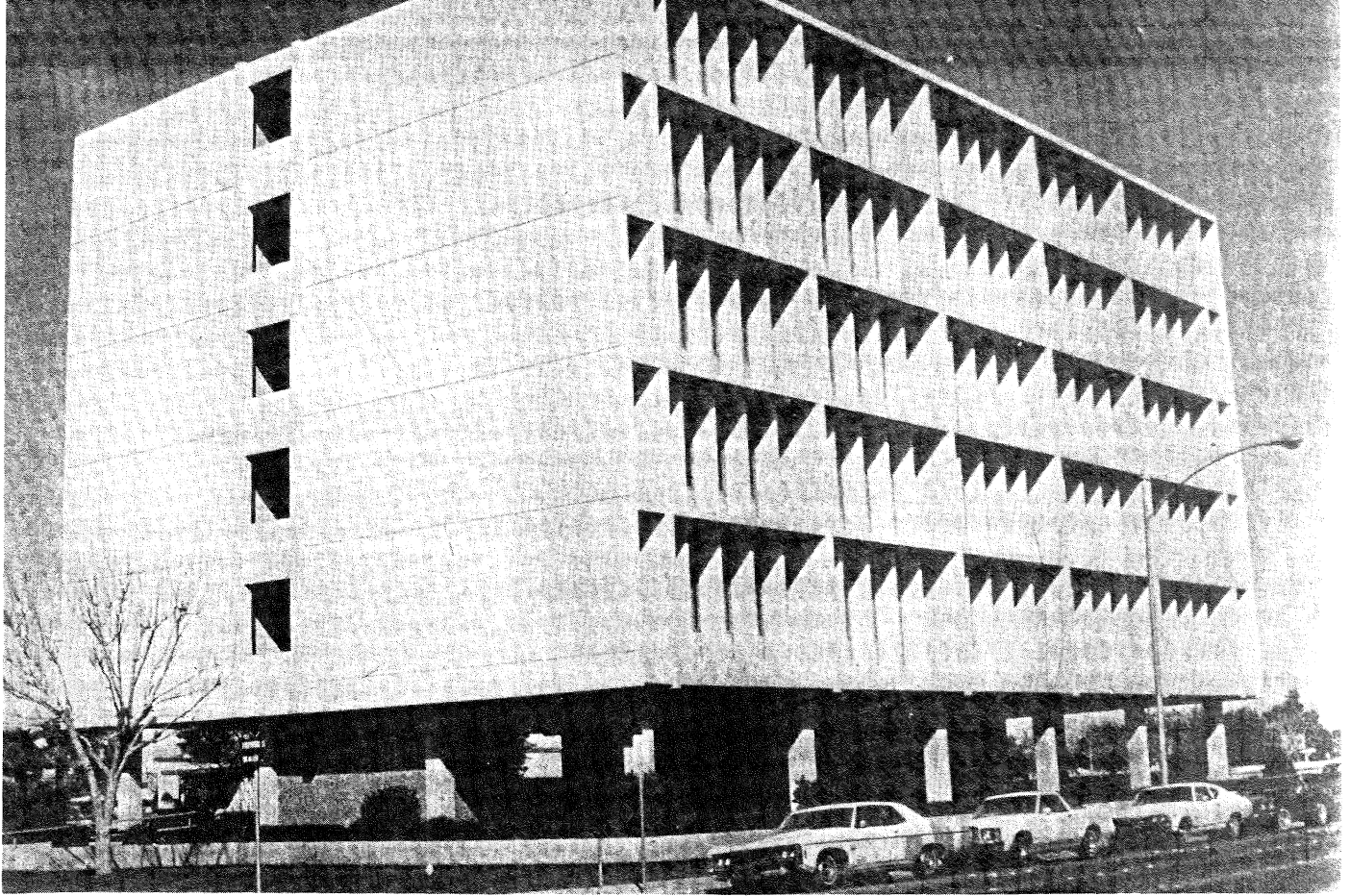


FIGURE 1 : Imperial County Services Building - view looking Northeast

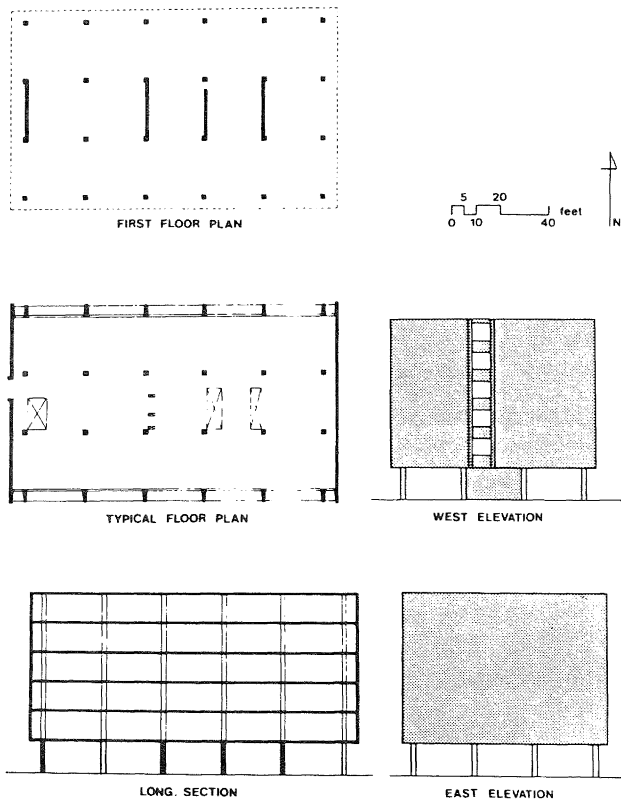


FIGURE 2 : Plan and elevation views of ICSB

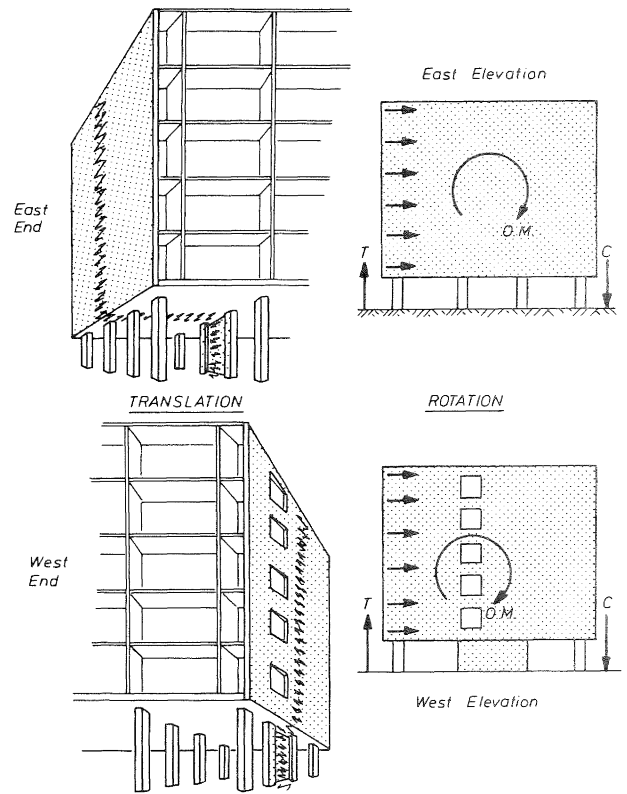


FIGURE 3 : End conditions to resist translation and rotation

The piles are interconnected with reinforced-concrete link beams; they extend 13.7m to 13.3m into the alluvium foundation material composed primarily of sand with interbeds of clay to 13.3m (based on logs from 4 soil borings at the site).

Vertical loads are carried by reinforced-concrete floor slabs (125mm thick at the second floor and 75mm thick at the upper floors) supported by reinforced-concrete 140mm wide by 355mm deep pan joists spanning in the north-south (transverse) direction; the joists are supported by four longitudinal reinforced-concrete frames at 7.6m on centre. The frame columns are typically 610mm square, and the beams vary in size. Beams in the two interior frames are 610mm wide by 760mm deep at all levels; those in the two exterior frames are 610mm by 740mm at the second-floor level, 255mm by 1350mm at the third-floor through sixth-floor levels, and 255 mm by 1270mm at the roof level.

Lateral loads are resisted by the four reinforced-concrete frames in the east-west direction and reinforced-concrete shear walls in the north-south direction. The shear walls are discontinuous at the second-floor level. Below the second floor are three interior and one exterior 1-foot thick shear walls, and above the second floor, shear walls exist only at the east and west ends. Between the second and third floors, the walls are 190mm thick, and above the third floor, they are 178mm thick. According to the design calculations, the design "K" factor [11] was 1.33 for the north-south shear walls, 0.67 for the east-west interior frames, and 1.0 for the east-west interior frames.

#### BUILDING DAMAGE

Failure of the ICSB in the October 1979 earthquake occurred catastrophically in the columns at the east end of the building. From the initial examinations, it appeared that the four free-standing columns at the east end of the building had failed by overturning since the outer pair of columns showed more distress than the inner. At the same time, the end shear wall, the floor diaphragm and the inner shear wall showed no apparent sign of shear distress.

Superficially, the ICSB appears to be a simple, rectangular and symmetrical structure and as such one would expect the four corner columns to react in a symmetrical fashion. However, the floor plans and longitudinal section shown in figure 2 indicate that the ground floor shear walls are not symmetrically distributed. It can be seen that at the west end, the shear wall from roof to first floor is offset some 1½m horizontally at the first floor level before continuing to the foundations (in the centre bay only). At the east end, a similar shear wall stops at the first

floor; shear forces must then be transferred through the first floor diaphragm for some 8-9m to a centre bay shear wall that occurs only at the ground floor.

The situation at the east end represents a classic case of shear wall discontinuity where an abrupt change of strength and stiffness occurs at a point where the shear wall terminates at the first floor. The vertical loads (from the overturning of the shear wall) must be taken down to the ground through the four slightly offset columns while the shear forces from it must be transferred to the smaller shear wall in the adjacent bay in the manner indicated schematically in figure 3. At the west end, the stiff ground level shear wall beneath the upper wall prevents large axial forces from being carried by the columns. While there is a 1½m "kink" in the load path from the upper portion of the west wall to the ground floor portion, compared to the east end it is more nearly a continuous wall.

From figures 2 and 3 it can be seen that in effect, the east end of the building at the ground floor level is more flexible than the west end and it could be expected that in an earthquake the ground level columns at the east end would resist some of the horizontal load from the shear wall above. Coupled with this, it can be seen from figure 4 that the lateral ties to the longitudinal steel were more closely spaced at the bottoms of the columns where plastic hinge formation was most likely to occur. However, a concrete ground slab was poured on the soil layer above the footings at about the same level as the point where the spacing of the lateral ties was reduced. Failure in the east end columns occurred about the slab level and it seems that this failure was caused by the formation of plastic hinges at this level together with the effect of the large cyclic axial force variations in the two outer columns on column line G (in the transverse direction) contributing to their degradation in strength and stiffness.

Further details of the earthquake and its damage are given by the Earthquake Engineering Research Institute Reconnaissance Team and others in References 1, 8 and 12.

#### SCOPE OF ANALYSES

The two dimensional framed structure model of the ICSB in the east-west direction was subjected to the following loading conditions:

- a) The north-south component of the 18 May 1940 Imperial Valley (El Centro) earthquake assumed as input for the east-west direction.
- b) The east-west component of the 15 October 1979 Imperial County earthquake recorded by the free field accelerometer located adjacent to the ICSB.

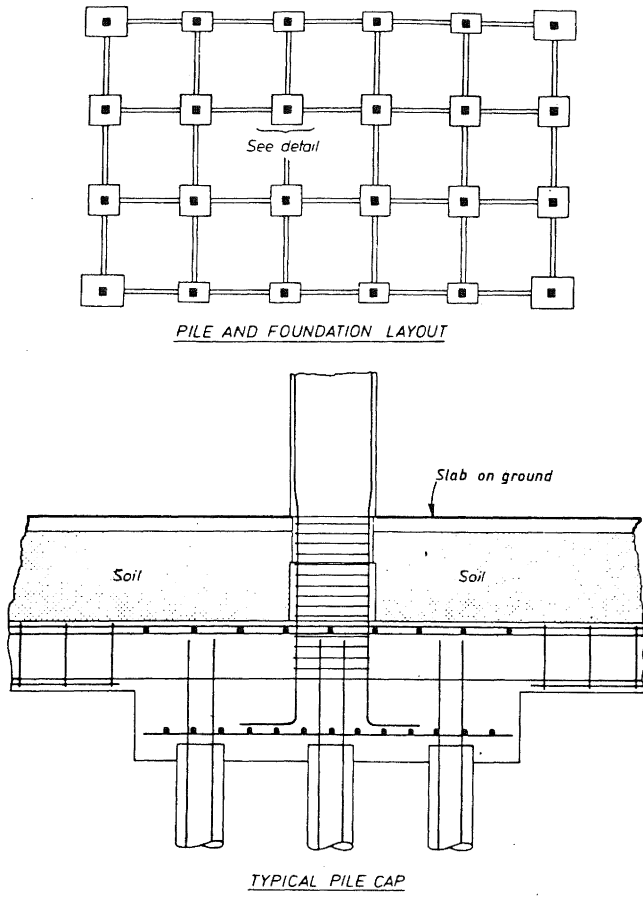


FIGURE 4 : Pile and foundation layout

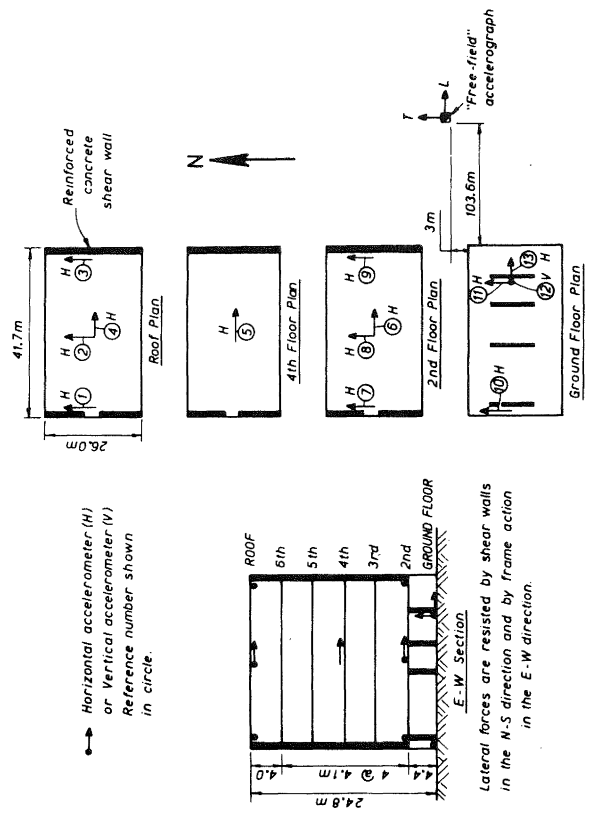


FIGURE 5 : Strong motion instrumentation

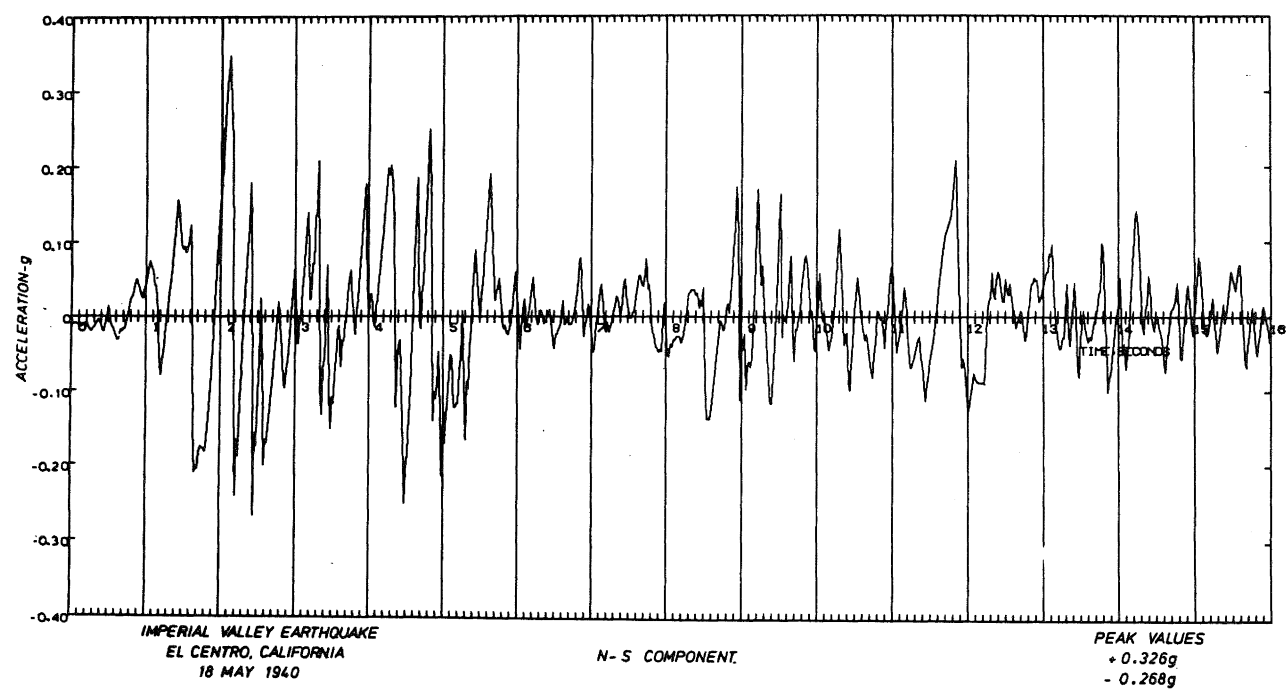


FIGURE 6 : 1940 El Centro earthquake - NS component

IMPERIAL COUNTY SERVICES BUILDING - STRONG - MOTION INSTRUMENTATION

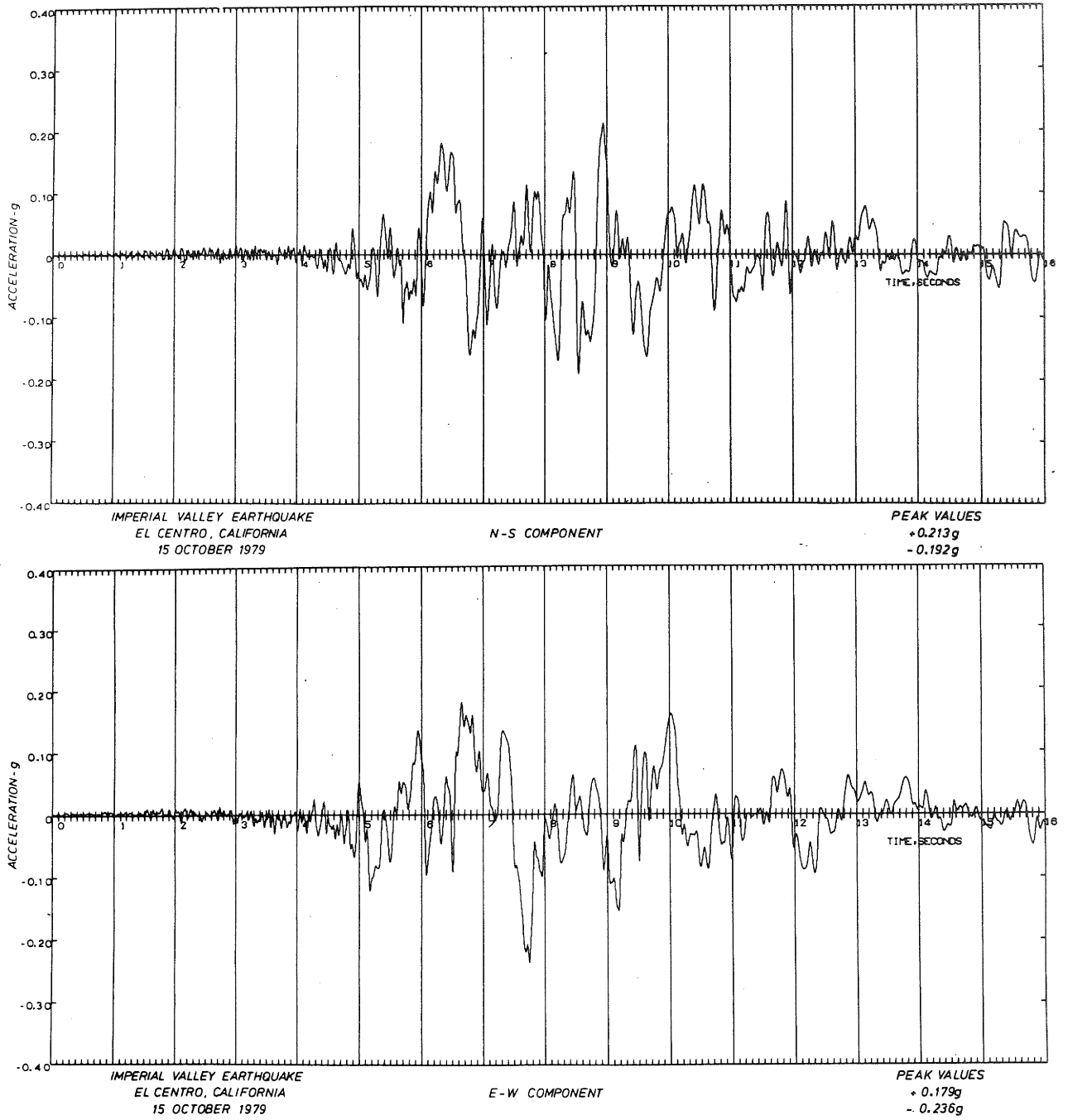


FIGURE 7 : 1979 Imperial Valley earthquake, Top - NS component (free field)  
Bottom - EW component (free field)

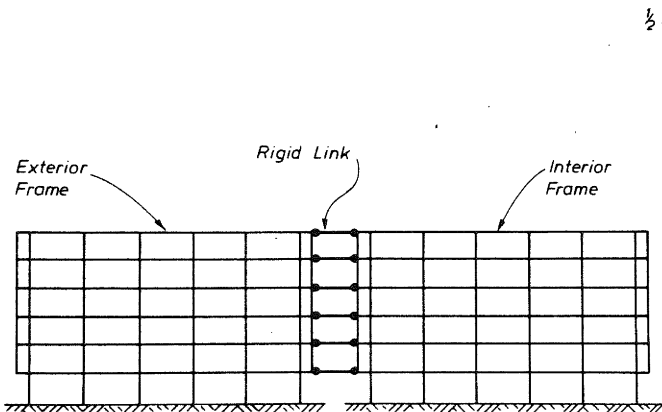


FIGURE 9 : Initial model for inelastic analysis in EW direction

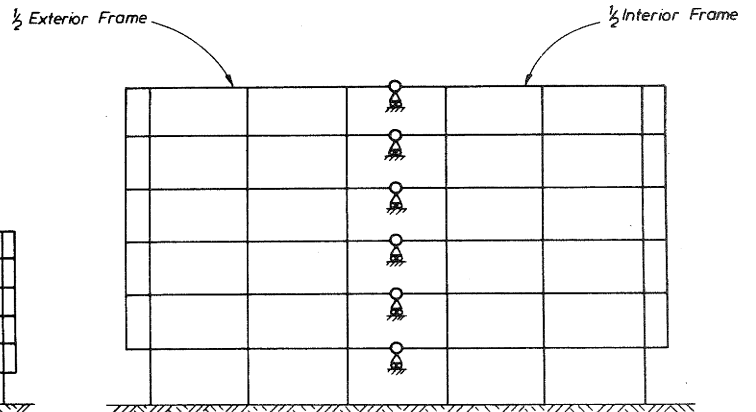


FIGURE 10 : Simplified model used

- c) The east-west component of the 15 October 1979 Imperial County earthquake as recorded by accelerograph No. 13 at the east end of the ground floor of the building (see Figure 5 for location of the instrument).

A two-dimensional model of the east end shear wall and the adjacent frame in the north-south direction was subjected to the following loading condition:

- d) The north-south component of the free field record of the 1979 earthquake.

In all analyses, the vertical load consisting of the dead load of each storey [2] was also applied. In each case, the first 14 seconds of the earth-quake record was used for the time history analyses (see Figures 6 - 8).

#### ASSUMPTIONS OF THE INELASTIC MODEL

These assumptions include:

- a) The cracked concrete cross section is used for beam, column, and shear wall properties.
- b) Section properties are listed in Table 1 and are based solely upon the concrete, i.e. no transformed section properties are used to account for the reinforcing steel.
- c) The mass properties of the beams, columns and walls are as calculated in reference 2.

In addition to the aforementioned stiffness and mass property assumptions, there is still enough latitude in modelling assumptions that one can "fine tune" the analytical model to achieve certain response characteristics. As such the analytical model developed for the analysis was the one which most nearly represented the experimentally derived frequencies. Specifically, the fundamental frequencies predicted by the analytical model as shown for the longitudinal direct direction in figure 10 (using uncracked concrete cross-sectional properties) and those derived experimentally [6] were

<u>Direction</u>	<u>Analytical</u>	<u>Experimental</u>
North-south	2.25 Hz	2.25 Hz
East-west	1.53 Hz	1.54 Hz

The concrete strength of the columns was taken as 34.5 MPa whereas the concrete strength of the beams and shear walls was taken as 27.5 MPa. The basic sectional properties of the beams and columns are given in Table 1.

The second floor slab thickness was 125mm whereas the slab thickness for the remaining floors was 75mm. The roof slab thickness was nominally 75mm except for the Penthouse area which had a thickness of 150mm.

The shear walls have a thickness of 190mm from the second floor to the third floor and a 178mm thickness elsewhere.

The members in the model were such that the stub beams supporting the end walls, the end wall members and all the columns above the ground floor were assumed to remain elastic while the remaining members (that is all the interior beams of both frames and the ground floor columns of both frames) were assumed to have bilinear moment curvature relationships where the post elastic stiffness was 3% of the elastic stiffness. For the beams, the positive and negative yield moments for each end of the members are given in reference 2. The columns used an axial force - moment interaction curve where the actual curves were approximated by a straight line from the axial tension yield point to the pure moment yield point, a central section representing a cubic variation of moment with axial compression and a final straight line segment to the axial compression yield point. The reinforcing pattern for the columns was as follows:

Interior		
frames - Columns B,C,D,F,G	8#	9 bars
- Column E	8#	10 bars
Exterior		
frames - Columns C,D,E,F	8#	11 bars
- Columns B,G	10#	11 bars

#### DESCRIPTION OF THE INELASTIC PLANE FRAME MODEL

Though a three-dimensional structural analysis may be required for a limited number of buildings, such inelastic time-history analyses are, in general, prohibitively expensive. As a result, most inelastic dynamic analyses are limited to equivalent plane or two-dimensional models of the structure.

In the east-west direction of the Imperial County Services Building there are four parallel frames and for the inelastic 2-D dynamic analyses it was proposed to take one interior and one exterior frame and impose a coupling so that the horizontal displacements at corresponding floors in each frame were to be the same. In the real structure, such coupling would be imposed by the floor diaphragm which is relatively stiff in its own plane. This model is shown in Figure 9. The coupling here is achieved by equivalent rigid links and rather than impose common horizontal displacements to all nodes on each floor level, only the ends of the frames were inter-connected so that the bandwidth of the stiffness matrix was minimised. This model has 108 nodes and 176 members. The disadvantage of this particular model is the large number of degrees of freedom and high expected computational cost and it was therefore set aside in favour of a further simplification.

IMPERIAL COUNTY SERVICES BUILDING

BEAM PROPERTIES

Frame	Floor Level	Iyy	Rigid End Zone at Each End		
1 & 4	2	54000	9		
1 & 4	3 - 6	124064	9		
1 & 4	Roof	104167	9		
2 & 3	2 - roof	54000	12		
<u>COLUMN PROPERTIES</u>					
Frame	Storey Level	A	Ixx	Iyy	Top Rigid End Zone
1 & 4	1	576	27648	27648	30
1 & 4	2 - 5	1028	420917	20177	53
1 & 4	6	1028	420917	20177	50
2 & 3	1	576	27648	27648	30
2 & 3	2 - 5	576	27648	27648	53
2 & 3	6	576	27648	27648	50
(units: A = in <sup>2</sup> , Ixx, Iyy = in <sup>4</sup> , Rigid End Zone = in)					
T A B L E 1					

This simplified model is shown in Figure 10 where one half of each of an interior and an exterior frame is coupled at the centre. This model would require two analyses to cover the variations in the column axial forces, depending on whether the excitation was in the west to east direction, or vice versa; however, the low, squat nature of the frame is such that these variations should be small and the assumption of antisymmetry at the centre under lateral loading seemed justifiable. It later transpired that the horizontal roller support at the centre, which could not readily be removed for the initial static analysis and then reimposed for the lateral time-history analyses, led to its removal altogether leaving only the mid-span hinge since the effect of girder shear on column axial loads appears to be minimal. The effect of roller support on the initial gravity load analyses led to peculiar moment patterns in the girders which, in turn, led to bizarre hinge formation patterns during the dynamic analysis.

The analyses were carried out using the two dimensional frame analysis program "RUAUMOKO" which has been developed at the University of Canterbury for the earthquake analysis of planar building frames.<sup>[9]</sup>

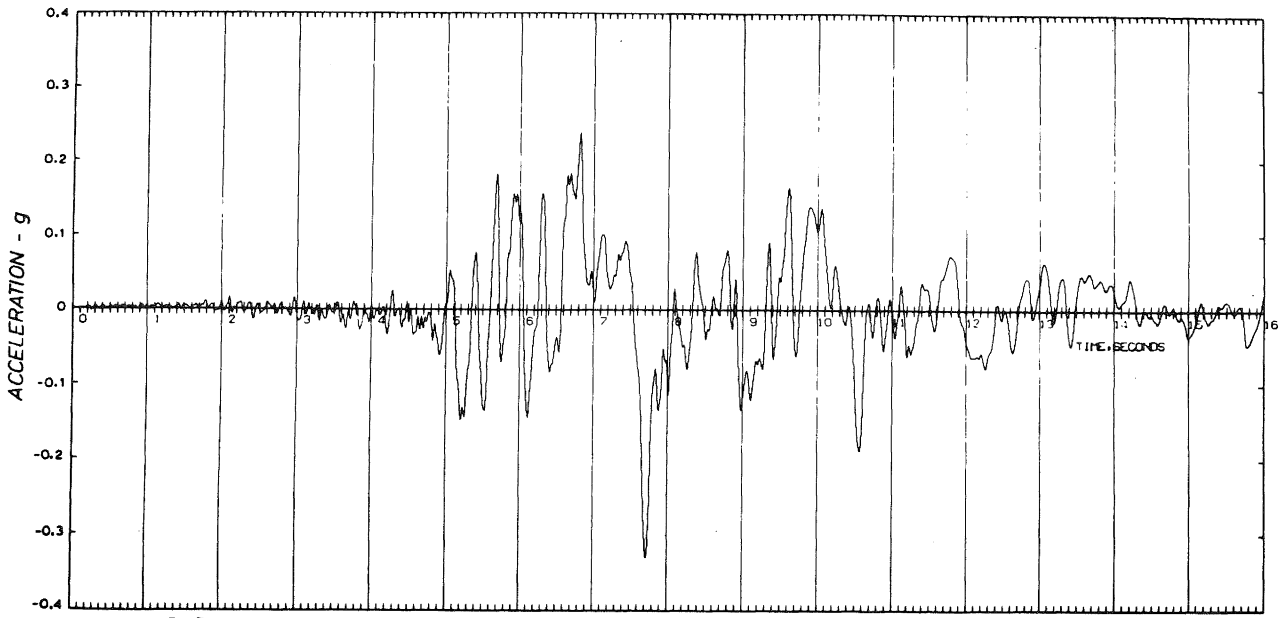
The joint regions of all members were treated as rigid end blocks and the floor displacements determined by the analysis are those at the centroidal axis level of the girders at each floor level.

The end shear wall members were treated as wide columns for the analyses and caused no modelling difficulties.

Each joint of the structure has three degrees of freedom (being the x- and y-displacements and the z- rotation). The mass model may be a consistent mass or a lumped mass model, the latter having been generally used in analyses at the University of Canterbury was used for these analyses also. In the first attempts at analysing this structure, the only masses used were those associated with the horizontal degrees of freedom of each floor and these were taken to be acting at the joint connecting the two half-frames. However, after initial difficulties in obtaining a stable inelastic time-history integration the lumped mass model was modified to associate mass with the vertical and horizontal degrees of freedom of each node together with a representative rotational inertia obtained from equivalent terms from the consistent mass matrix. This was done initially in order to give the joints some enhanced capacity to absorb moment overshoot effects associated with a hinge forming during a given time-step. After a careful study of the apparent instability in the analysis, it was found that the problem was caused by machine accuracy and word length difficulties associated with the use of inches, where the member lengths are of the order of 300 inches, and kips as units. Upon conversion of the dimensions and weights to metres and Kilo-Newtons, the numerical stability problems immediately vanished.

The damping model chosen for the analyses was that of Rayleigh damping where the damping matrix was a linear combination of the mass matrix and the initial elastic stiffness matrix. The fractions of damping prescribed were 8% of

FIGURE 8 : 1979 Imperial Valley earthquake EW component at ground floor



IMPERIAL VALLEY EARTHQUAKE  
EL CENTRO, CALIFORNIA  
15 OCTOBER 1979

ACCELEROGRAPH 13,  
GROUND FLOOR

PEAK VALUES  
+0.238g  
-0.331g

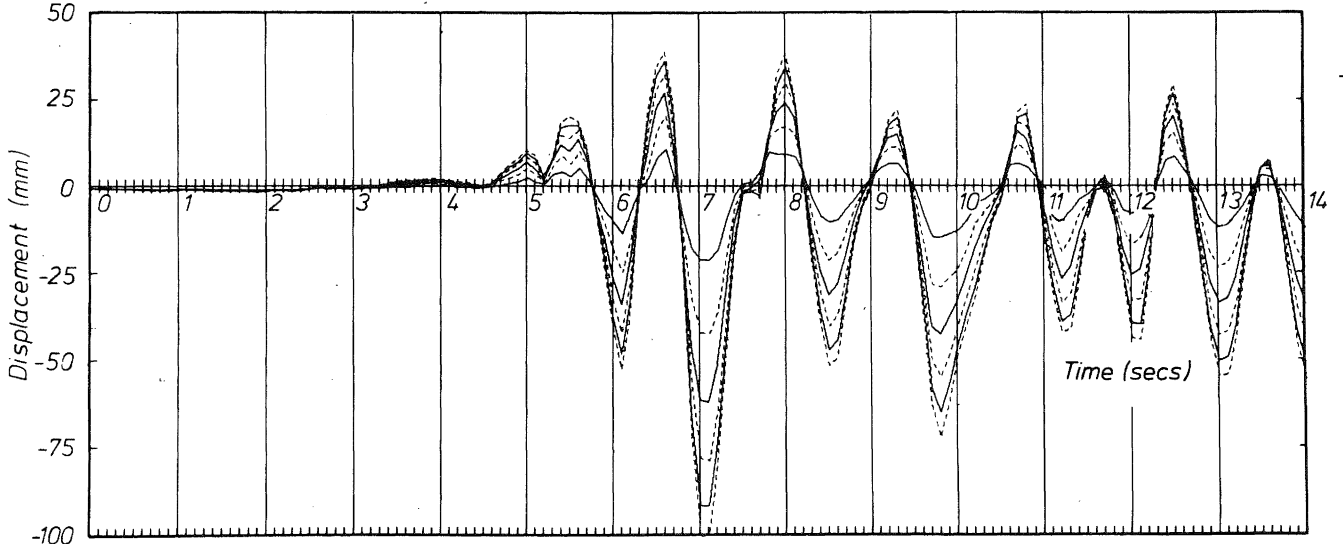
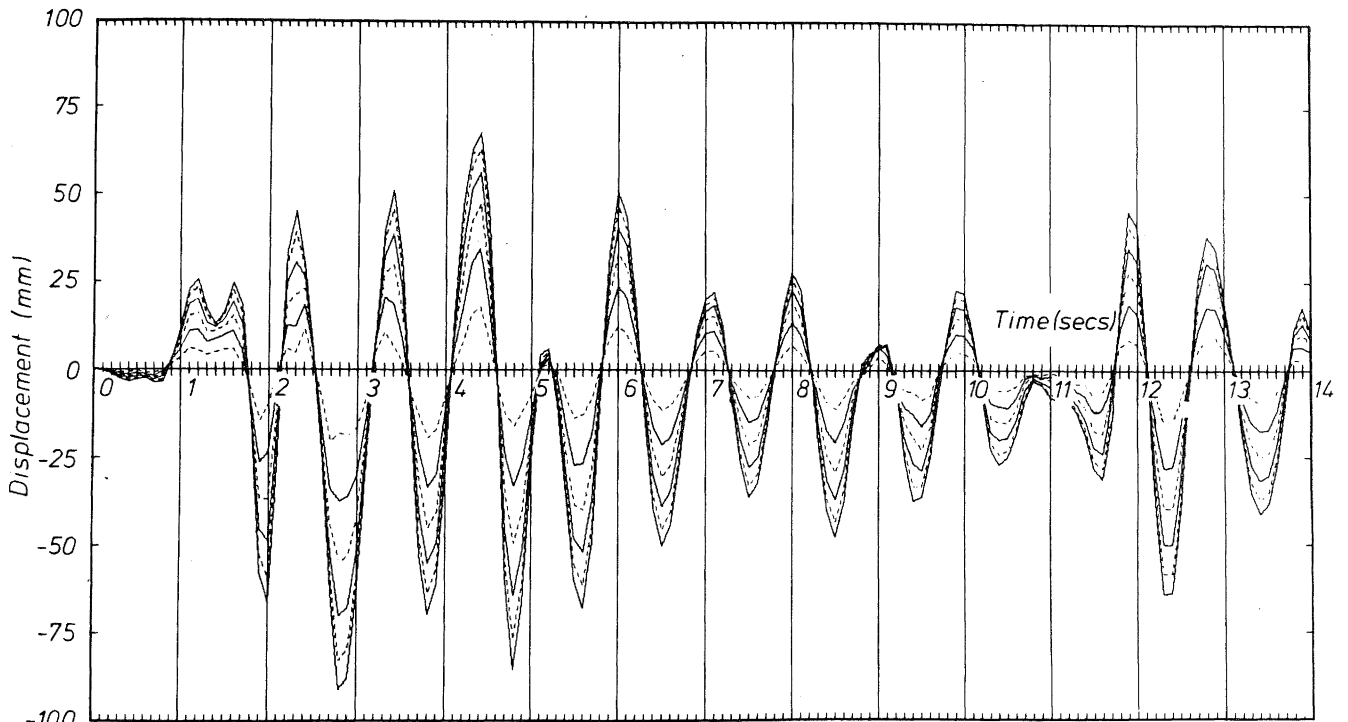


FIGURE 11 : Displacement time-history plots for inelastic analyses using 1940 (top) and 1979 (bottom) EW earthquake records

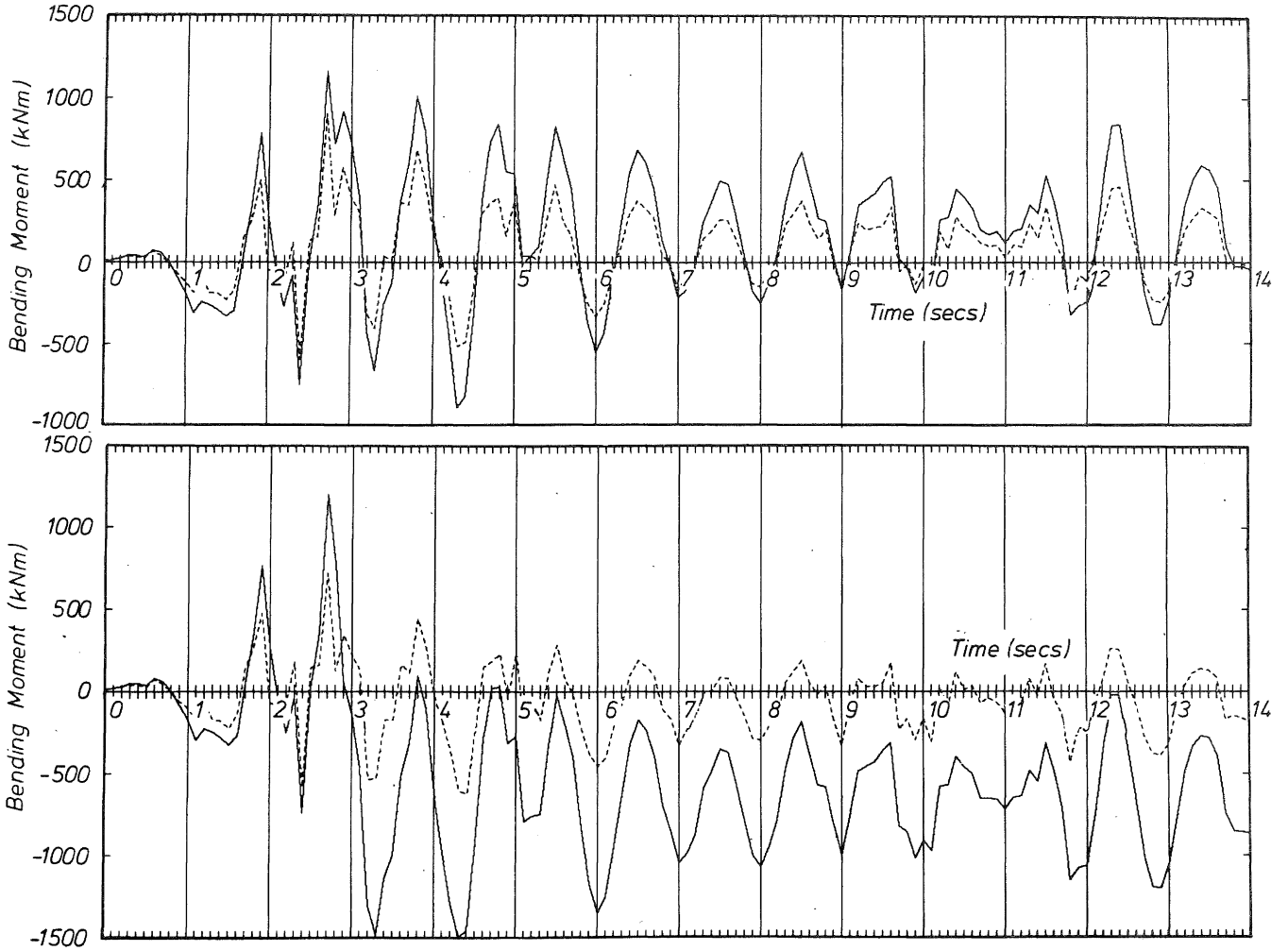


FIGURE 12 : Bending moment time-history plots for outermost column in exterior (top) and interior (bottom) frames under 1940 El Centro record

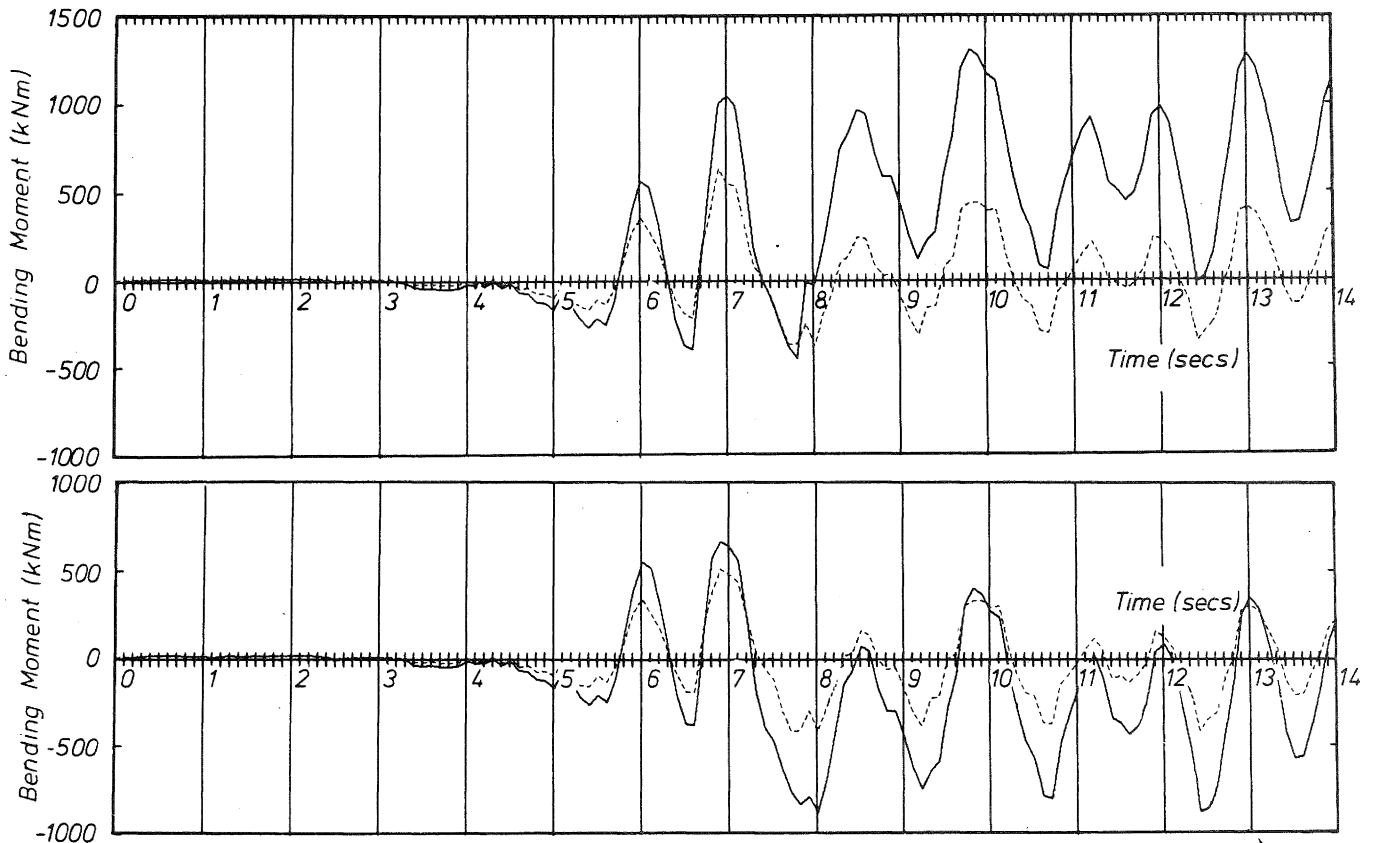


FIGURE 13 : Bending moment time-history plots for outermost column in exterior (top) and interior (bottom) frames under 1979 EW free field record

critical damping in natural modes of free vibration one and six. This ensures sub-critical damping in the higher modes of vibration, with damping in the second and third modes being approximately 7% of critical damping<sup>[3]</sup>.

All the inelastic analyses carried out with the RUAUMOKO program (based on that of reference 9) used a time-step of 0.01 seconds with Newmark constant average acceleration ( $\beta = \frac{1}{4}$ ) method since this had been shown to be satisfactory in many previous analyses carried out<sup>[3,4,9,10]</sup>.

#### RESULTS OF THE INELASTIC EAST-WEST PLANE FRAME ANALYSES

A total of three analyses were carried out, one each for the first 14 seconds of the north-south component (on the east-west frames) of the May 1940 El Centro record, the east-west component of the October 1979 free-field record at the Imperial County Services Building site and the east-west component of the October 1979 earthquake as recorded in the east-west direction at the ground floor of the ICSB by instrument 13 of the accelerograph network in the building (see Figure 5).

Under the 1940 and 1979 free field earthquake records, the deflection at the roof level was found to be 93.0mm and 95.5mm respectively while under the recorded 1979 ground floor motion the roof deflection was 103.4mm. Comparing the time-history plots for the 1979 free field and ground floor records, it was found that the displacement variations were very similar for both records even though the ground floor record had larger peak accelerations than the free field record. Displacement time-history plots from the 1940 and 1979 records are shown in Figure 11.

Time history-plots for the column bending moments at the top (dashed line) and bottom (solid line) in the end columns of the exterior and interior frames are shown in Figures 12 and 13. For both earthquakes, it can be seen that the behaviour of the end columns of the two frames are slightly different. In the case of the exterior frame under the 1940 earthquake, the bending moments at the top and bottom of the columns both alternate in direction in a similar way. For the interior frame, the moment at the bottom of the column maintains the same direction after first few reversals but the top moment continues to alternate in direction. The 1979 earthquake produces moments in the end columns of both frames that fluctuate in direction though the moments at the bottom of the column in the external frame do show some drift. Diagrams showing the plastic hinge formation and floor deflection profiles at various times through the 1979 free field earthquake record are shown in Figure 14.

Under both earthquakes, the member ductility demand in the ground floor

columns of both interior and exterior frames was found to be low being a maximum of 3.1 under the 1940 earthquake record, 1.8 under the 1979 earthquake record, 1.6 under the 1979 ground floor (trace 13) record. The girder ductility demand was a maximum of 4.4 under the 1940 earthquake record and 4.7 and 5.0 under the 1979 free field and ground floor records, respectively.

#### MODELLING THE ICSB FOR THREE DIMENSIONAL EFFECTS

Since the plane frame analysis of the ICSB in the east-west direction was not able to detect the possible large increase in axial force in the corner columns that could come from the overturning moments produced by the end shear walls as a result of the east-west component of the earthquake, an attempt was made to use the ANSR computer program<sup>[5]</sup> to carry out a full three-dimensional analysis.

For the analytical 3-D model it was assumed that the floors were rigid in their own planes, though the horizontal beams and floor slabs were still subjected to bending deformations, thus reducing the total number of degrees of freedom and making the analysis more tractable from the point of view of computer storage and running time. In addition to the previously mentioned assumptions, a further one was that the floor slabs would remain elastic throughout the analysis.

Unfortunately, the particular version of ANSR that was available was essentially version one and as such had several limitations that would prevent accurate comparisons with the plane frame analysis and which would limit the accuracy of the three dimensional analysis. These limitations were

- 1) the beam-column element used did not make provision for rigid end blocks and member loads, and
- 2) the yield moments for bending of the beams were assumed to be equal in the positive and negative directions in the ANSR code whereas the actual beams in the ICSB were reinforced to have considerably different yield moments for positive and negative bending<sup>[2]</sup>.

Prior to carrying out the full three dimensional analysis, it was decided to analyse a plane frame model similar to Figure 11 in order to evaluate the limitations of ANSR code. The yield moments were taken as the average of the positive and negative values and the rigid end blocks were allowed for by basing the elastic stiffnesses on the clear length of the beams. As it turned out, the ANSR analysis was nearly five times slower than the RUAUMOKO analysis and as a result the analysis of the 3-D model was not undertaken since it would be even slower still.

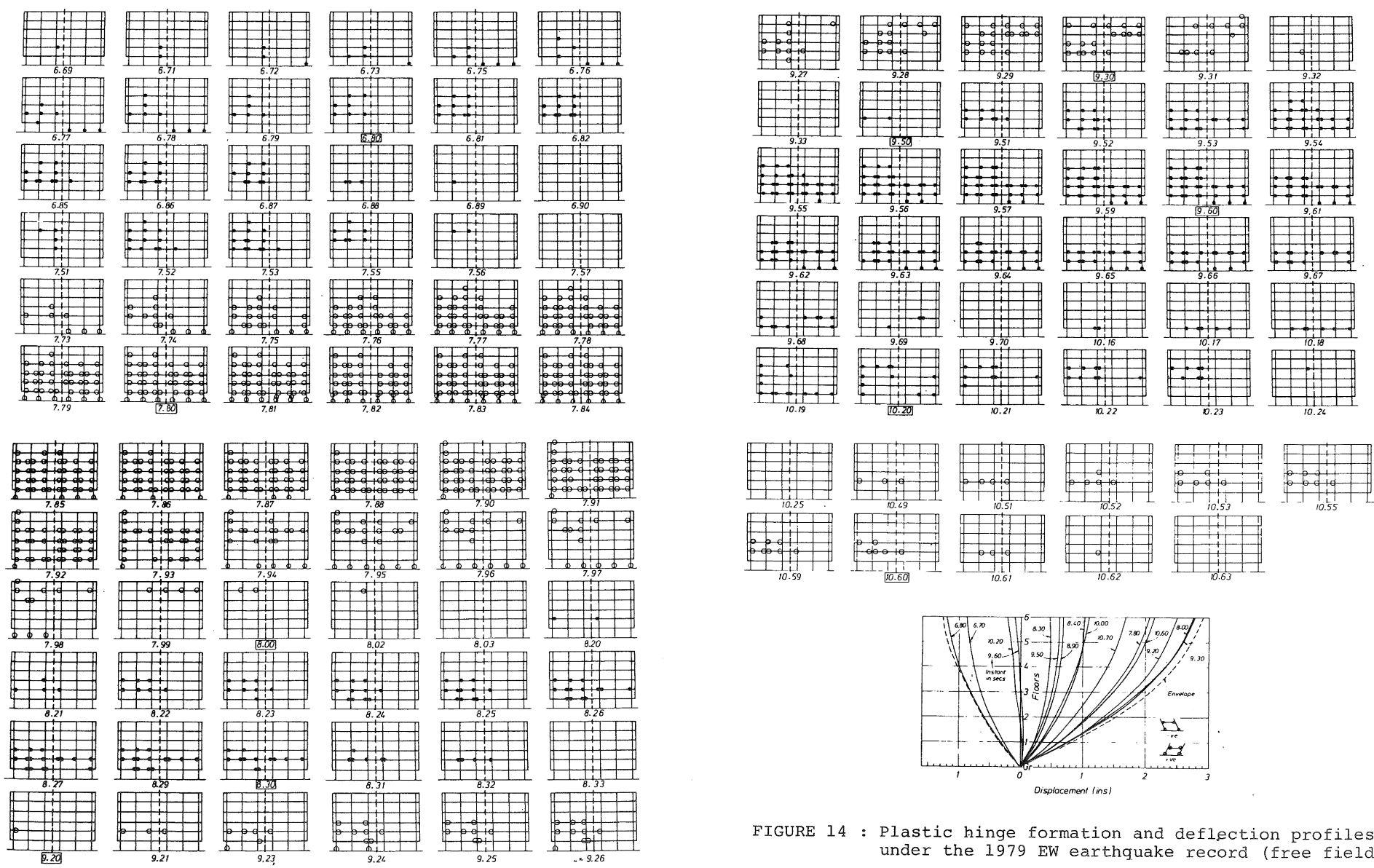


FIGURE 14 : Plastic hinge formation and deflection profiles under the 1999 EW earthquake record (free field)

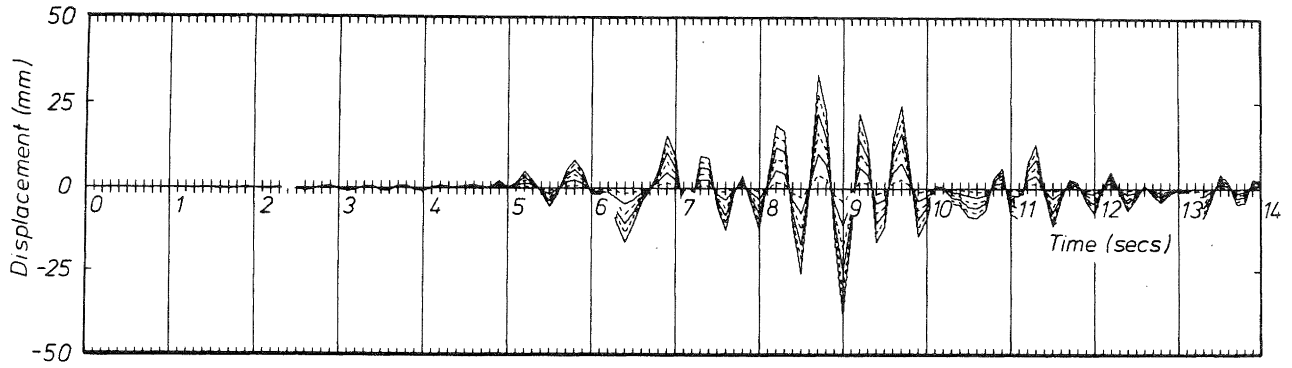


FIGURE 15 : Displacement time history for the transverse (i.e. NS) frame at East end under the 1979 NS "free field" earthquake record

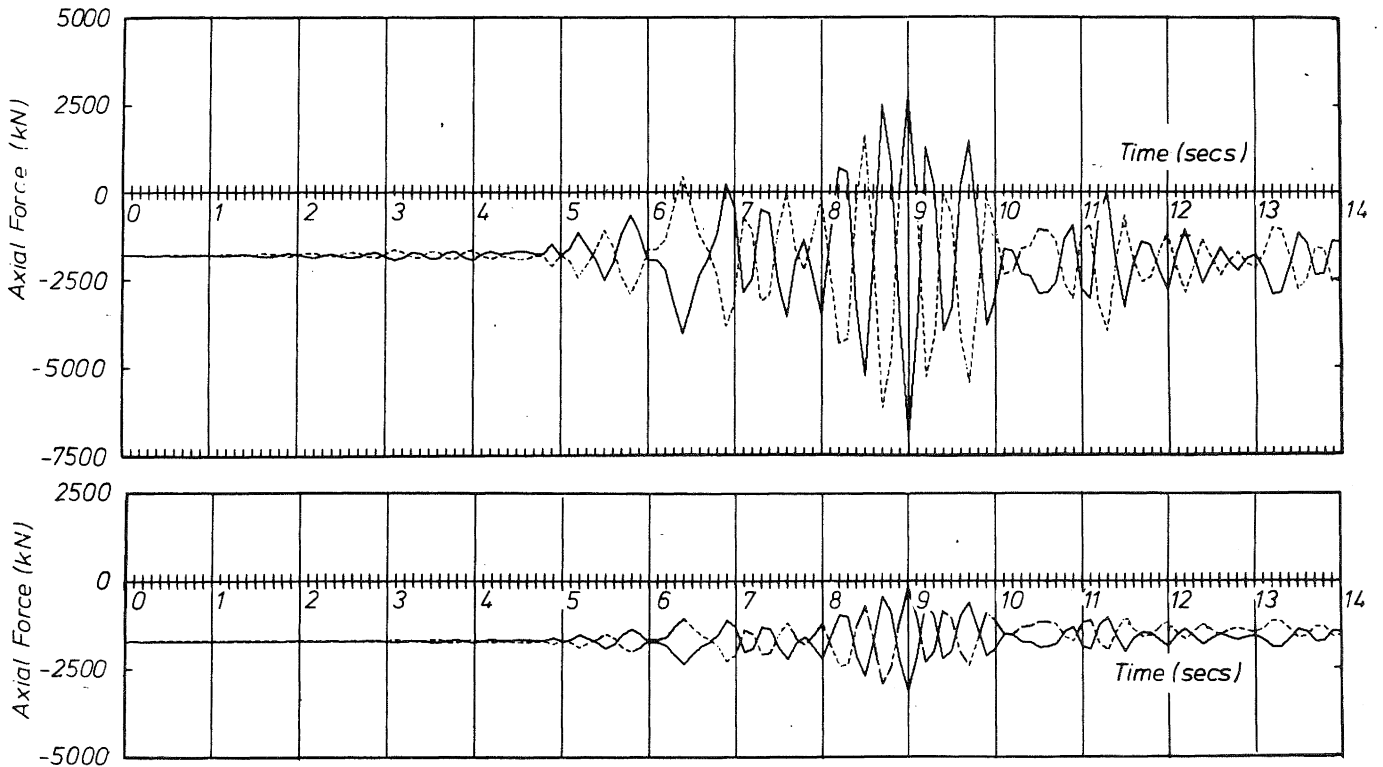


FIGURE 16 : Axial load time-history plots for East end transverse frame under 1979 NS component for outer (bottom) and inner (top) columns

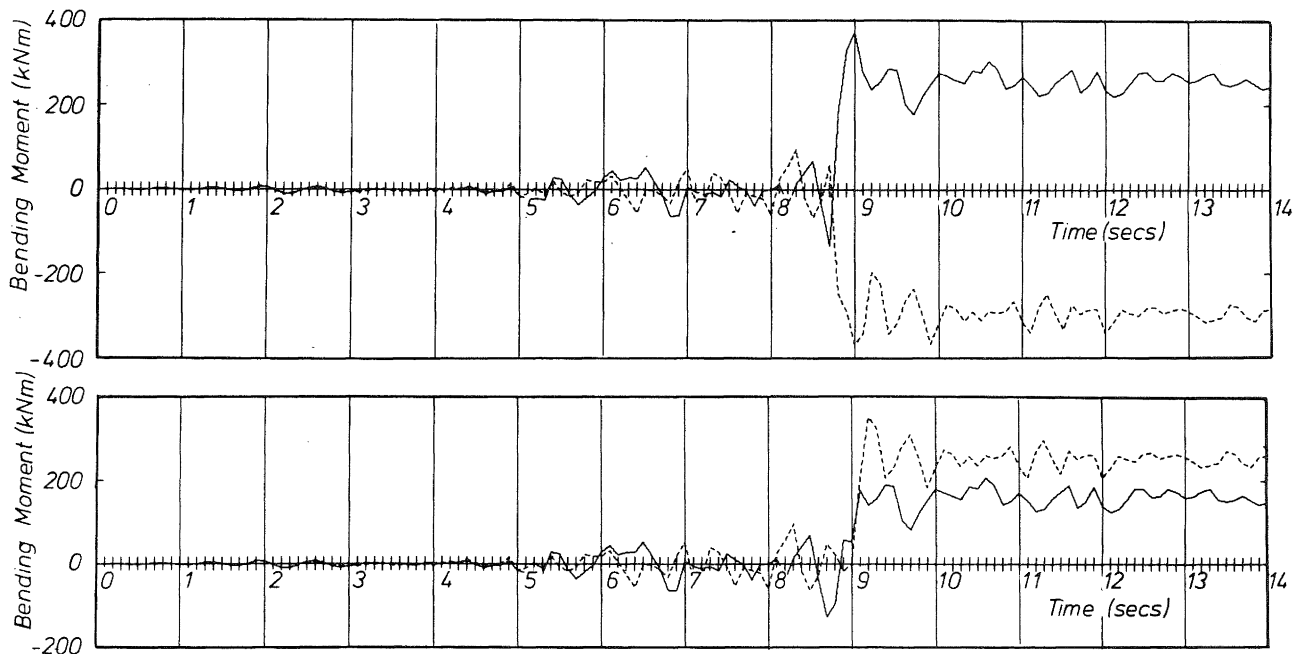


FIGURE 17 : Bending moment time-history plots for East end transverse frame under 1979 NS component for outer (top) and inner (bottom) columns

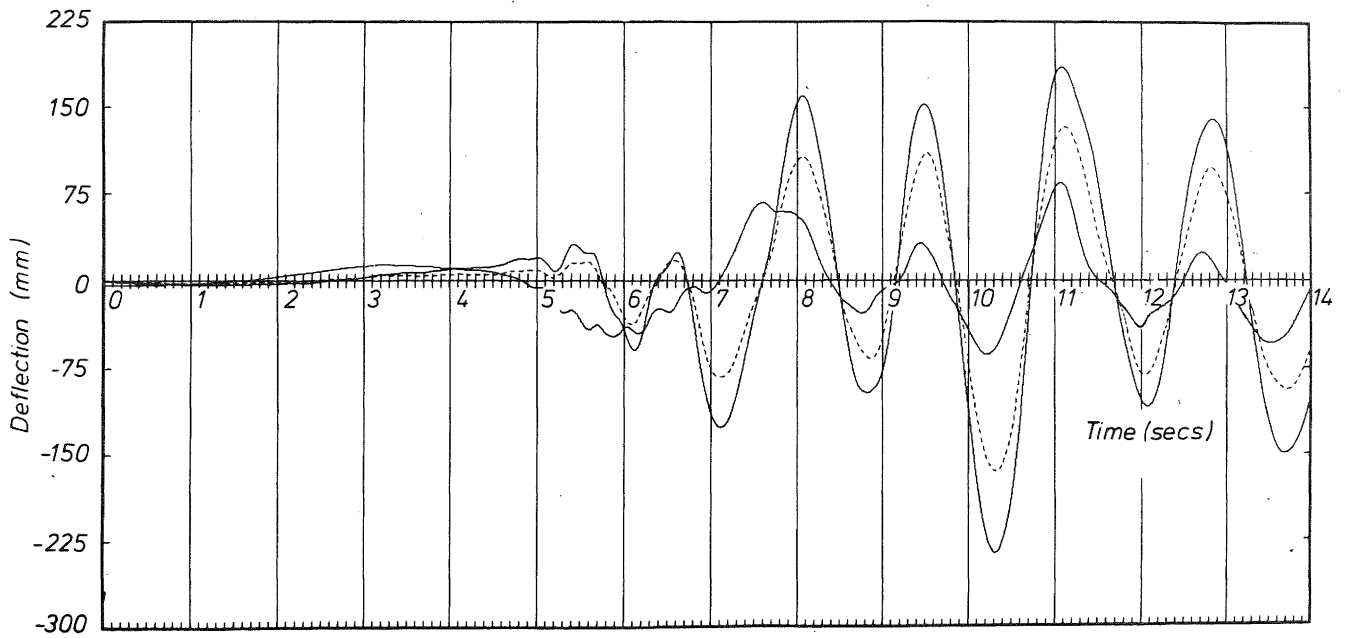


FIGURE 18 : Longitudinal (EW) deflection of ICSB relative to the ground at roof and 3rd and 1st floors

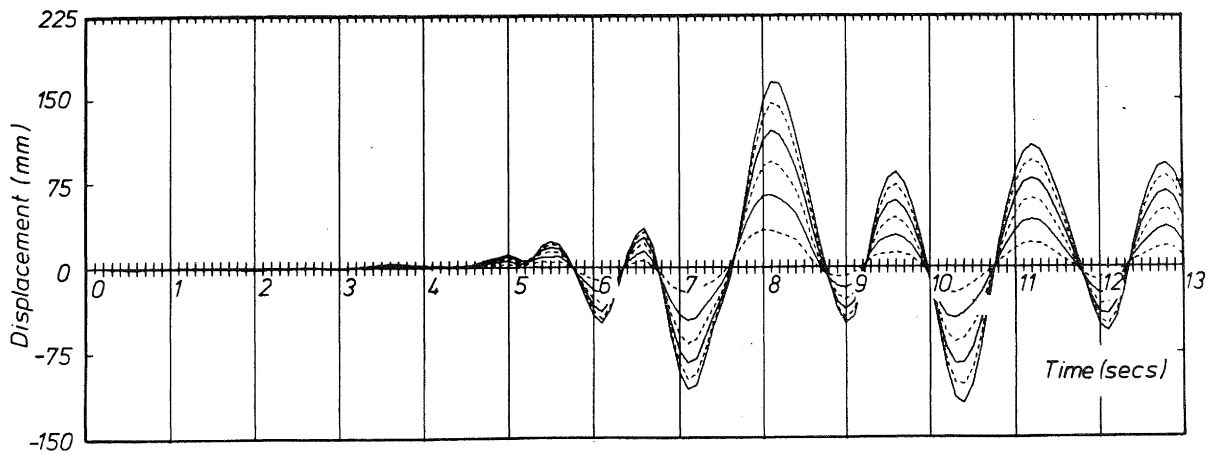


FIGURE 19 : Longitudinal deflection time histories obtained using the 1979 EW record with degrading stiffness and pinching of the concrete hysteresis curves

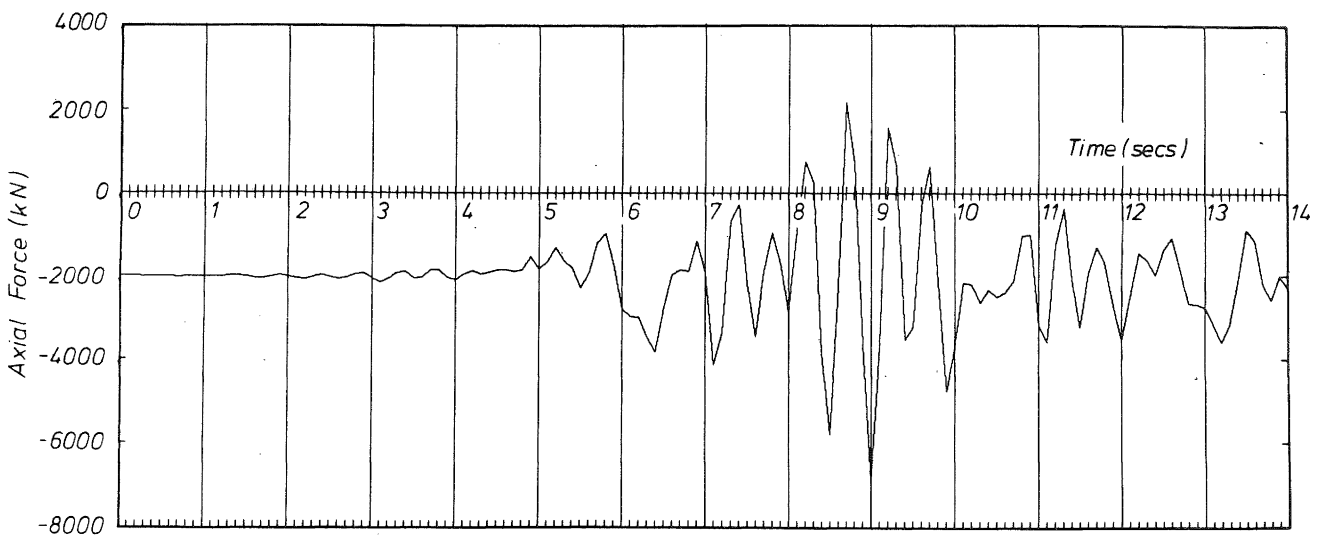


FIGURE 20 : Axial force in a corner column at the East end caused by longitudinal and transverse earthquake components for 1979 record

In order to obtain some understanding of the behaviour of the building in the north-south direction, a plane frame analysis was carried out of the shear wall and frame at the east end of the building (the frame on line G in Figure 9) since the exterior ground floor columns along line G had experienced large axial forces under an elastic earthquake analysis in this direction<sup>[2]</sup>.

In this model, the end wall and the adjacent frame were modelled as a single plane frame. For this, the column members in the frame were given the same properties as before. The beam members were given the properties of a 1.9m strip of floor slab since there were no actual beams in the east-west direction. Since the end shear wall was carried by the frames, it was also necessary to model this wall, especially as regards its stiffness. To do this, the wall was represented by diagonal struts having an axial stiffness that corresponded to the horizontal shear stiffness of the wall panels. Vertical members having the same vertical stiffness of the wall were also used. These vertical and diagonal members were supported on short vertical members whose axial stiffness corresponded to the bending stiffness of the stub beams that supported the wall panels. In order to reduce the number of degrees of freedom in the model, the nodes corresponding to each floor level were slaved together horizontally. All members were considered to remain elastic, except for the ground floor columns which were inelastic and had the same properties as previously detailed. The effect of the nearest ground floor shear wall (on line F in the centre bay) was modelled by a single storey vertical beam member, fixed at the base and hinged at the top, whose bending stiffness matched half the shear stiffness of the shear wall. The resulting frame model of the frame and end wall (assuming uncracked concrete properties) had a fundamental frequency of vibration that compared very well with that measured experimentally, suggesting that the analytical model was a reasonable one for this analysis.

#### RESULTS OF THE INELASTIC NORTH-SOUTH ANALYSES

Under the 1979 north-south free field earthquake record, the maximum displacement at the roof level was found to be 35.6mm, occurring just after the earthquake peak that occurred slightly before 9 seconds (see Figure 15). Plastic hinge formation was very limited and occurred only in the two outer columns during the period from 8.6 to 9.1 seconds. Associated with the yielding at the plastic hinges that occurred at the top and bottom of the outer columns was a very brief period of tensile axial yielding of each column. It can be seen from Figure 16 that as each outer column yields in tension the compression in the column at the other end of the frame increases to about half the compression yield of the column.

Time-history plots for the column bending moments at the top (dashed line) and the bottom (solid line) in the ground floor columns are shown in Figure 17.

#### DISCUSSION

Figure 18 shows the time-history plot for the longitudinal deflection of the ICSB at the roof and floors 4 and 2, as determined from the strong motion accelerographs 4, 5 and 6 (Fig. 5) relative to the ground motion recorded by instrument 13. It can be seen from Figure 11 that these deflections are not very well predicted by the analytical results, though there are similarities up to a time of 8 seconds or so but the large roof displacement (exceeding 230mm or 9 inches) that occurred just after 10 seconds is not shown by the inelastic analysis.

In an endeavour to obtain better agreement with the measured deflections, the analysis of the longitudinal frames was repeated using the 1979 free field earthquake but modelling the inelastic beam and column behaviour by a model that incorporates the pinching that occurs in the hysteresis loops for reinforced concrete members rather than by the simpler, but less accurate, bilinear model. The displacement results obtained using this model are shown in figure 19. Comparing this with figure 18 it can be seen that the large peak displacement that occurs at about 8 seconds is predicted quite accurately but subsequent peaks are not as large and the peak between 10 and 11 seconds is not as well predicted by the computer analysis, though all the peaks in figure 19 do occur at the same time as the corresponding peaks in figure 18.

This discrepancy is probably caused by the fact that the earthquake had components along and across the building of similar magnitude, thus producing significant interaction between the forces in the two directions. Normally, as suggested by building codes, buildings with at least one axis of symmetry can be analysed separately for their seismic behaviour in directions along and across the building. It was mentioned earlier that the arrangement of shear walls and frame at the East end of the building (see Figure 3) is such that the overturning moment from the wall can only be resisted by the development of tension and compression forces in the outer columns of the frame. The variation with time of these axial forces when resisting the transverse component of the 1979 earthquake can be seen in Figure 16 and is quite large during the period from 8 to 10 seconds.

In the analysis of the longitudinal (East-West) frame described earlier, no account was taken of the variation in axial force in the end columns that arises from the overturning moment in the end wall and frame just described. Further, no account was taken of the bending moments that are developed by the transverse forces, especially in the end columns. These transverse moments have large peaks at times through the earth-

quake period and when combined with the longitudinal bending moments will mean that the neutral axes in the columns are continually shifting in position and orientation. The variation in the axial force in a corner column from the simultaneous seismic effects in both longitudinal and transverse directions is illustrated in figure 20.

#### CONCLUSIONS

For the 1979 earthquake, comparisons with the longitudinal deflection time-histories for the roof and floors 4 and 2 show some agreement up to a time of 8 seconds or so but do not show any significant agreement after that time. The recorded ground floor acceleration gave rise to very slightly greater displacements on account of the slightly greater accelerations than those of the "free-field" record but could not produce a better agreement with the measured deflections after 8 seconds of the earthquake.

Analyses of the East end shear wall above the ground floor and the nearby frame showed that the outer columns in the frame were heavily loaded under the North-South component of the 1979 earthquake (as measured by the "free field" accelerographs) and tension yielding could occur momentarily. This variation in axial load would undoubtedly influence the performance of the longitudinal frames in resisting the East-West component of the earthquake.

The inelastic deflections and member ductility demands for both the 1940 and 1979 earthquakes in the East-West frames appear to be small and would not indicate the level of damage observed in the building after the 1979 earthquake.

However, the effect of the North-South component of the earthquake together with the unfortunate placing of the ground level slab above the detailed column hinge reinforcing probably led to the failure of the corner columns.

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