# IMPLICATIONS OF DAMAGES TO THE IMPERIAL COUNTY SERVICES BUILDING FOR EARTHQUAKE-RESISTANT DESIGN

Christos A. Zeris (I)
Rodrigo Altmann (II)
Presenting Author:Christos A. Zeris

## SUMMARY

The Imperial County Services Building located in El Centro, California was severely damaged during the 6.6 Richter magnitude Imperial Valley earthquake of October 15, 1979. Considerable information was recovered from sixteen accelerographs located in and near the building providing a unique opportunity for evaluating the reliability of present methods for predicting the seismic response of this type of buildings as well as for assessing the soundness of modern code seismic provisions. Correlation studies conducted for this purpose are briefly presented and the main results obtained are examined.

## INTRODUCTION

The Imperial County Services Building (ICSB) was a modern six story, reinforced concrete office structure severely damaged during the 1979 Imperial Valley earthquake. The severity of the damage and the availability of reliable acceleration response records (Ref.1) motivated researchers at the University of California, Berkeley to initiate a comprehensive program of field, laboratory experimental and analytical investigations. The main objectives of this paper are to report some of the results of this investigation related to an assessement of (a) the reliability of analytical means for predicting the response of this type of buildings, (b) the suitability of the particular system used in resisting motions similar to those recorded and (c) the adequacy of the seismic provisions used in the design, as well as those of present codes. In order to achieve these objectives different stuctural models are analyzed with a linear analysis program and the results are compared to those obtained from the recorded accelerations.

## DESCRIPTION OF THE BUILDING AND STRUCTURAL RESPONSE

The ICSB was rectangular in plan, having five bays in the East-West (EW) and three bays in the North-South (NS) direction (Fig.1,14). It was designed according to the Uniform Building Code (UBC) 1967. The lateral force resisting system consisted of moment resisting frames in the EW direction and shear walls in the NS direction. A characteristic feature of this structure was that the shear walls provided at the East and West ends (over the full width) were discontinued; lateral resistance in the ground story was provided by four narrower walls (Fig.2, Ref.2). For the design a base shear coefficient (C) of 0.059 was used in conjunction with three K factors: 1.33 for the NS shear wall system, 1.00 for the EW exterior frames (non-ductile) and 0.67 for the EW interior frames (ductile). Other important features were the use of deep

(I) Research Assistant, University of California, Berkeley
(II) Structural Engineer,San Jose,Costa Rica

"nonstructural" reinforced concrete panels above the beams on the N and S facades and the bar offset detailing in the ground story columns (Fig.5). The building was founded on pile groups.

Primary damages to the structure consisted of: (a) spalling of the cover (with buckling of the longitudinal reinforcement also observed) above the base of most ground story columns, accentuated at those in line G. The crushing of these regions led to shortening of these columns by one foot (Fig.4,9). (b) Formation of yield lines along the width of the building at lines F and G (Fig.1) due to the G column shortening. (c) Flexural cracking of the exterior beams mainly in the lower floors. (d) Non-structural damage with separation of the nonstructural elements from the structure especially near lines F and G. (e) Diagonal and horizontal cracking of the walls at ground level, those at frames F and E being more heavily damaged than at B and D (Fig.1).

The building was instrumented with thirteen accelerometers (Fig.2), with an additional triaxial recorder 300' away to monitor the free field response; four instruments located at ground level were used to deduce two translational and a torsional component of input motion. The peak ground acceleration was 0.32g. However one special characteristic of the record was a 0.23g pulse of more than one second duration (Fig.3), influencing significantly the building response.

EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS

Tests performed on steel and concrete samples removed from the building revealed that materials (particularly the concrete) were stronger than nominally specified (Table 1 and Ref.3). Based on the measured material mechanical characteristics, realistic member nonlinear characteristics and capacities were determined. Cracked flexural beam stiffnesses were about 50% of the gross section stiffnesses. However, in the case of the exterior beams, the cracked flexural stiffness was doubled if the contribution of the nonstructural elements was included; consideration also of the slab reinforcement increased significantly the beam flexural capacities (Fig.6). Motivated by the localized damage of the ground columns above the recess (Fig.4,5,9), the effect of detailing on the flexural characteristics of the columns were estimated (Ref.4) - namely the reductions in confinement (Fig.7) and in load bearing capacity of the offset reinforcing bar due to buckling (Fig. 7,8).

Three dimensional models of the ICSB were formulated and analyzed elastically using ETABS (Ref.5), extended so that two translational and one torsional components of earthquake could be prescribed independently. Two additional stories were added to the models to account for the rocking of the building. Beam,column and panel elements were used. An inherent assumption of the program is the use of in plane rigid diaphragms. However analysis (Ref.6) of the building using flexible floors showed that as a result of this simplification the dynamic characteristics of the model significantly differed from those determined by the ambient vibration tests. It is believed that this assumption is acceptable for the small amplitude phase of the motion, for which the elastic analysis is valid.

By comparing (a) the model periods of vibration to the ambient test results before the earthquake (Ref.7) and (b) the predicted and recorded responses, the reliability of the model was evaluated. The sensitivity of the response to the modeling assumptions of using gross versus cracked member properties, true versus nominal strengths of the materials and including the effects of the nonstructural elements was investigated (Table 2,Ref.4). The variability observed is typical of moment resisting frames (Ref.8). From this analysis it was concluded that the most reliable model was the one based on the use of the cracked flexural properties for the elements and included the contribution of the deep nonstructural beams (Model 3, Table 2); the periods for the first three modes correlate well to those obtained by inspection of the records (Ref.1,9) or by frequency domain analysis of the response (Ref.10), but do not correlate well to those obtained from ambient tests; small amplitude ambient tests may overestimate the stiffness of concrete structures relative to their resistance after cracking. The model (No 3, Table 2) was analyzed separately for two different combinations of site recorded ground motions: (a) the two translational and the rotational component applied simultaneously and (b) the EW translational component, to evaluate the significance of the NS and torsional components (Ref.4).

As illustrated in Fig.(10), the correlation of the recorded and the predicted displacements is satisfactory until 6.20 seconds, with minor deviations until the 6.90 sec., when a change in the recorded response is observed; subsequent peaks exceed the elastically predicted ones by a factor of two. By comparing the true member capacities with the predicted demands the sequence of failure is estimated to be: (a) separation of the "nonstructural" elements from the frames around 6.0 seconds and subsequent yielding of the exterior beams in bending at the second and third floor at 6.40 seconds (Fig.10). (b) Probable yielding of the interior second floor beams, between 6.40 and 6.90 seconds. (c) Yielding of the ground level columns at their base (Fig.12) and subsequent large displacements above the second story.

## CONCLUSIONS AND RECOMMENDATIONS

From the results of the studies conducted on the ICSB the following conclusions can be drawn, regarding the building's response during the earthquake and its compliance to codes:

The concrete strengths obtained from the tests were considerably higher than the specified especially for the lower strength concrete. The only exception being the concrete at the ground level columns on line G which was significantly weaker than the other columns' concrete. (Table 1,Ref.3,11).

The design calculations were checked and found in essential agreement with the the UBC 1967 used in design (Ref.3). The detailing of the building was consistent with the system adopted; some additional details were implemented in design to provide limited ductility in the outer frames. Design details not in accordance with present UBC (1982) requirements were: (a) the use of non-uniform tie distribution with spacing larger than code requirements at the ground story columns supporting discontinuous walls, (b) the use of non-ductile perimeter frames in moment-resisting frame systems, (c) the amount of vertical reinforcement used in the end walls above the second story is smaller than the required minimum and (d) reduced ultimate moment capacities were used to estimate maximum shear demands in flexural members.

Although the analysis showed that recorded ground motions exceeded those for which the building was designed to remain elastic, most of the damage was a consequence of poor conceptual design, the main drawbacks being:

(a) The character of the ground motion was such that all ground columns yielded irrespective of applied axial force, in EW response. However, post-yield behavior for the corner columns in particular was adversely affected by the wall discontinuity which introduced high compressive axial forces due to the overturning of the supported walls in the NS direction. This simultaneous action demanded from these columns a ductility they could not provide due to the poor detailing.

(b) Specifying offset reinforcement and a reduced confinement in the columns at ground level (Fig.5), reduced their deformability under maximum axial strength considerably. The observed localized damage at the ground story was further enhanced by: (1) significant strengthening of the second floor beams not accounted in code design, accentuated by the presence of the heavily reinforced slab; (2) the presence of the concrete ground slab between lines 2 and 3 (Fig.1) at ground level restraining the columns above the analytically assumed column bases reducing their effective clear height. On the whole, the high rigidity of the structure above the second story due to the presence of the deep nonstructural girders, and the absence of EW shear walls at ground level tended to concentrate the lateral displacements at the ground story enhancing the behavior of a soft story structure, a system that has proved inadequate in similar near-field earthquakes (Ref.12).

(c) The absence of a shear wall at ground floor in G in the NS direction resulted in the one foot shortening of the East end and the formation of a yield line mechanism at F and G (Fig.1). As determined analytically, the West end columns (B, Fig.1) were equally heavily loaded axially as the columns at G; however, one of the reasons they did not suffer the extensive damage of the latter was because of the presence of the wall in the center span of bay B.

(d) The analysis revealed a prominent contribution of torsion, which due to the structural layout was coupled to the NS translational response. This torsional motion, together with the imposed rotational component of the ground motion induced in the East end ground walls higher shear forces than those at the West end (Fig.12), as well as significant biaxial bending to all four corner columns, reducing even further their resistance.

(e) The presence of the nonstructural elements affected the dynamic characteristics and the performance of the EW system. Analytical studies showed that the lateral stiffness and therefore the periods of the building were very sensitive to the interaction of the frames with these elements. From correlations of displacement and force time histories, the nonstructural elements were believed to significantly contribute up to about 6.0 seconds, when their connections with the structure weakened considerably.

Based on the observed performance of the building, and the analytically predicted response the following are recommended:

1) The use of non-continuous shear walls supported on columns, especially for columns positioned at the edges of the structure should be discouraged. If this particular system is used, well-confined columns especially by closely spaced spiral reinforcement should be employed, to ensure ductile behavior.

2) Stringer detailing requirements and rational design of perimeter members should be implemented, accounting for all factors such as material overstrength, boundary condition restraints and biaxial bending.

3) In column design the effects of all components of the ground motion

acting simultaneously should be taken into account.

4) More severe restrictions on the detailing and careful field inspection during construction of reinforcement offsets and similar details are encouraged.

5) The use of structural systems consisting of frames with different ability of ductile deformation, acting in the same direction of lateral resistance should be discouraged. As was observed in this particular case, since the response of the building required all EW frames to displace equally, the exterior semiductile and stiffer frames had to supply more ductility than the interior ductile frames; this additional ductility they were unable to provide, leading to the system's overall nonductile failure observed.

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| TABLE 1. ICSB Concrete Test Results | | | |
|---|---|---|---|
| Specified Nominal $f'_c$,psi | Location | Measured $f'_c$,psi | |
| | | at 28 Days | after EQ |
| 3000 | Tie Beams, Ground Slab | - | 8820 |
| 4000 | Beams, Joists | 4656 | 5950 |
| 5000 | All columns | 5695 | 7020 |
| | G Line, Ground Level | | 5630 |

| TABLE 2. Sensitivity of Model Periods | | | | | | | |
|---|---|---|---|---|---|---|---|
| Case | Properties | Material | Non Str. | Columns | Periods,sec. | | |
| | | | | | EW | NS | Rot. |
| 1 | Gross Section 100% | Nominal | Yes | Elastic | 0.66 | 0.39 | 0.33 |
| 2 | Gross Section 70% | Nominal | No | Elastic | 0.88 | 0.40 | 0.32 |
| 3 | Gross Section 70% | Nominal | No | Pinned | 0.99 | 0.46 | 0.38 |
| 4 | Cracked Section | True | Yes | Elastic | 0.94 | 0.44 | 0.37 |
| 5 | Cracked Section | True | Yes (Wall) | Elastic | 0.83 | 0.44 | 0.36 |
| 6 | Cracked Section | True | No | Pinned | 1.16 | 0.45 | 0.39 |
| 7 | Cracked Section | True | Yes (Wall) | Pinned | 0.78 | 0.48 | 0.37 |
| 8 | Averaged | Average | Yes (Wall) | Elastic | 0.72 | 0.44 | 0.36 |
| 9 | Ambient(Ref.6)<br>Pre Earthquake<br>Post Earthquake | | | | <br>0.65<br>0.83 | <br>0.45<br>0.52 | <br>0.36<br>- |
| 10 | Observed (Ref.1,9) | | | | 1.00 | 0.50 | - |

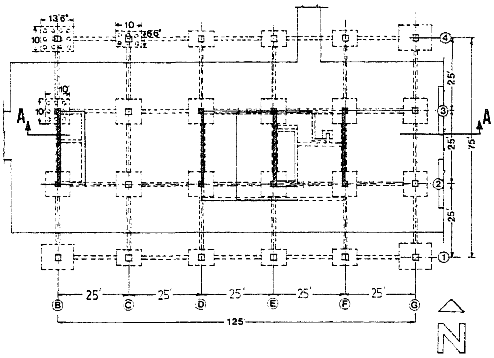



Fig. 1 ICSB Ground Level and Foundation Plan

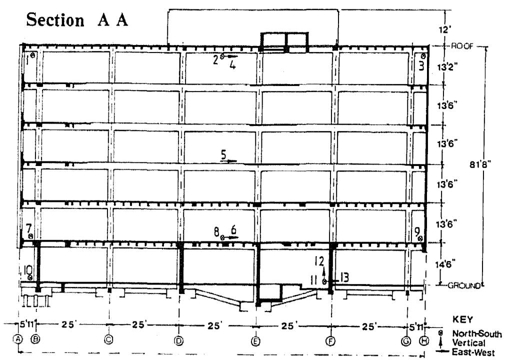



Fig. 2 Elevation and Instrument Location

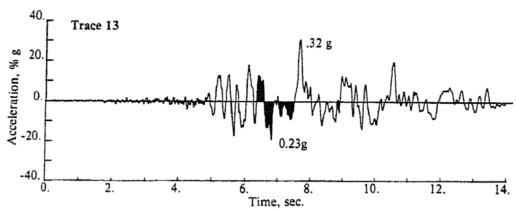



Fig. 3 Ground Motion, EW Direction

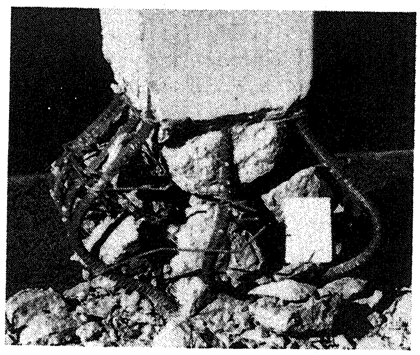


Fig. 4 Detail of Column G4 Failure

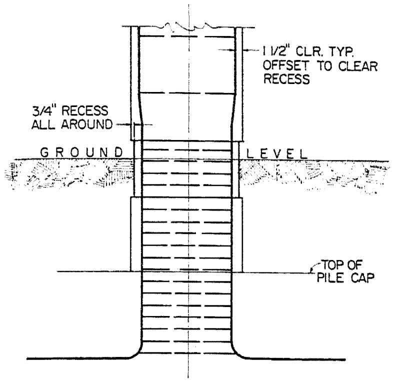



Fig. 5 Typical Ground Corner Column Detail

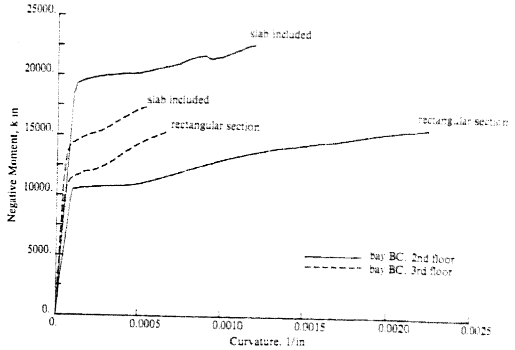



Fig. 6 Typical Exterior Beam Nonlinear Characteristics

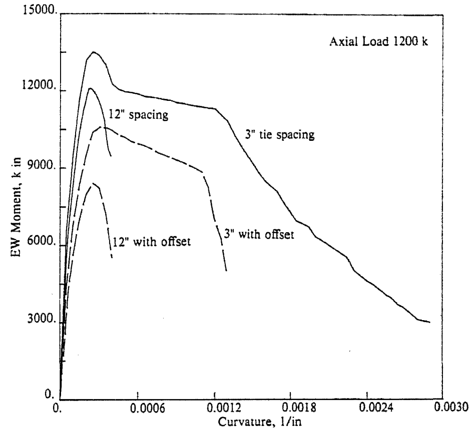



Fig. 7 Influence of Ground Column Detailing

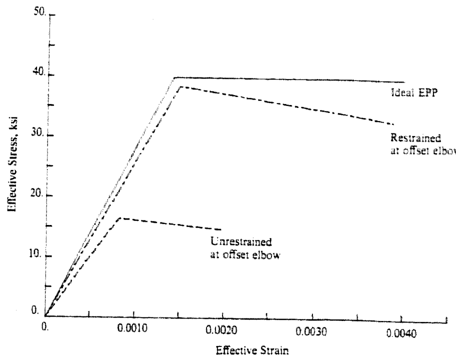



Fig. 8 Offset Bar Behavior

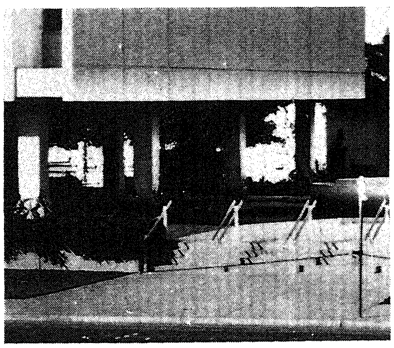


Fig. 9 Damage of G Column Line, Ground Level

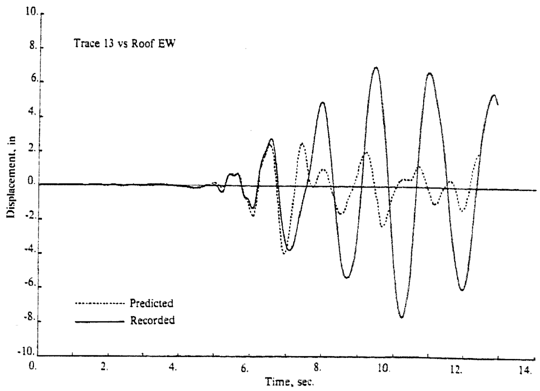
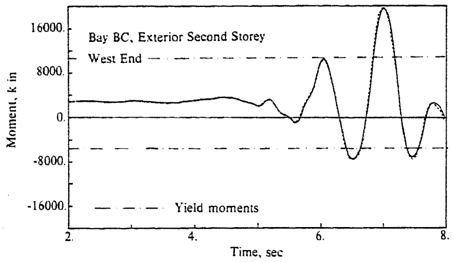



Fig.10 Comparison of Predicted and Recorded Displacements



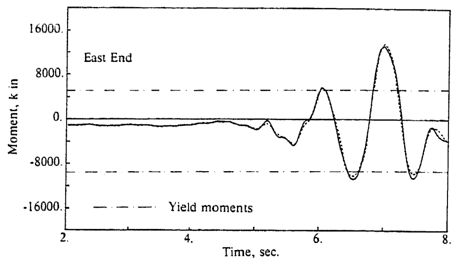
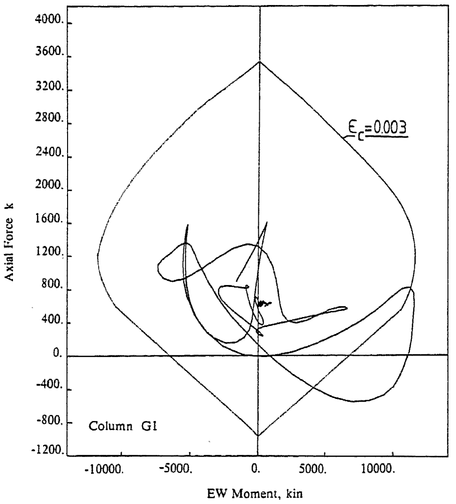




Fig. 11 Typical Beam Moment History



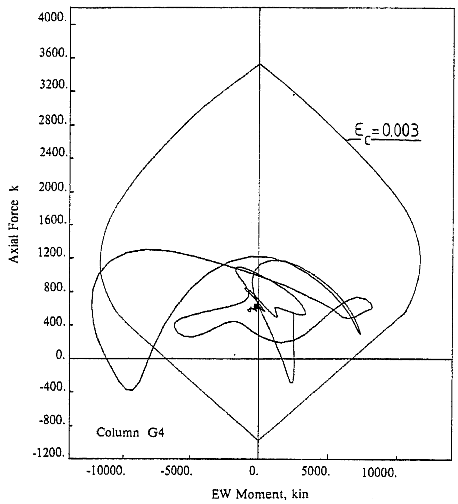




Fig. 12 Predicted Load Paths of Ground Corner Columns

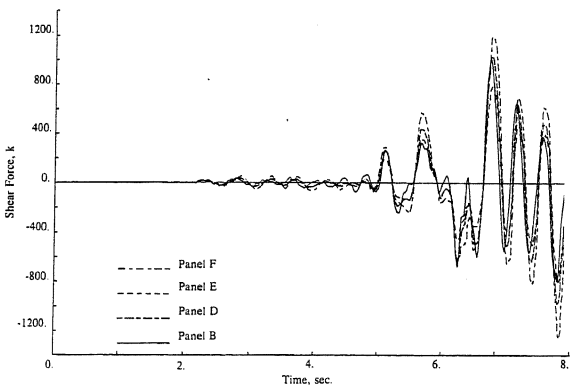



Fig. 13 Ground Walls Shear Force History

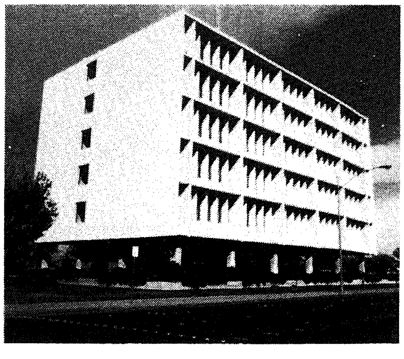


Fig. 14 View of the Imperial County Services Building, from SW