A DESIGN ANALYSIS OF THE IMPERIAL COUNTY SERVICES BUILDING

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SUMMARY

Dynamic response analyses are performed on the Imperial County Services Building which was severely damaged during the Imperial Valley Earthquake of 1979. Using recorded base input and standard analysis programs it is shown that the forces in critical columns exceeded the ultimate capacity and that the result which could be expected was predominantly a brittle compressive failure. It is also shown that lateral forces which developed during the earthquake far exceeded those specified by building code.

INTRODUCTION

On December 15, 1979, the Imperial Valley experienced an earthquake of magnitude 6.4 which was centered in Mexico just south of Bonds Corner, California. The direction of fault propagation was north toward the city of El Centro, California. This resulted in relatively strong ground shaking in the El Centro area for an earthquake of this magnitude. El Centro is primarily a farming community and the commercial area consists of one and two story frame buildings. The only major structure in the area was the six story Imperial County Services Building which was located approximately eighteen miles northwest of the epicenter. The building was designed for the lateral force provisions contained in the 1967 edition of the Uniform Building Code (Ref. 1) and was dedicated in 1969.

Because of the seismic activity in the region, earthquake engineers have been interested in the area ever since the earthquake of 1940 which has been so widely used in earthquake engineering studies and designs. Therefore it is not surprising that this modern structure was instrumented with some sixteen strong motion accelerometers. Furthermore, the building suffered extensive damage during the earthquake. This is a rare combination which is of considerable interest to earthquake engineers.

The purpose of this paper is to investigate the behavior of the Imperial County Services Building using current design analysis procedures and computer programs which are readily available to structural engineers. Hence, only the elastic dynamic response of the structure is considered.

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The intent is to determine if current design analysis procedures can predict the major problems experienced by the County Services Building.

BUILDING CONFIGURATION AND ANALYTICAL MODEL

The structural configuration of the reinforced concrete building has been described in detail by others (Refs. 2 and 3) and will only be summarized here. The six story building rises from a ground floor which is 136 feet 10 inches (41.7m) by 85 feet 4 inches (26m) in plan. The foundation consisted of pile caps resting on Raymond tapered piles which were interconnected by grade beams as shown in Figure 1. The floor plan which is typical of all floors above the ground floor is shown in Figure 2. Lateral resistance at all levels in the longitudinal (E-W) direction is provided by two exterior moment frames on column lines 1 and 4 and two interior moment frames on column lines 2 and 3. Lateral resistance in the transverse (N-S) direction is not continuous. At the ground floor level shown in Figure 1, it is provided by four shear walls located on column lines B, D, E and F. At the second floor level and above, lateral resistance is provided by two large shear walls located at the east end and at the west end of the building as shown in Figure 2. Between the second and third floors these were 7.5 inches (19 cm.) thick and above they were 7 inches (17.8 cm.) thick. This caused the building to appear to be top-heavy and gave the impression of having a soft story at grade level. A longitudinal elevation is shown in Figure 3 and a transverse elevation is shown in Figure 4. It should be noted that the irregularities in stiffness just described would require a dynamic analysis under the recommendations set forth by the Applied Technology Council in their report ATC-3 (Ref. 4). The design concrete strength was 4 ksi. with the following two exceptions: For the columns the design strength was 5 ksi. and for elements below grade it was 3 ksi. All reinforcing steel was specified to be grade 40.

The ETABS (Ref. 5) program was used to model the building because it is readily available in the public domain and is used extensively by structural engineers in California. It has three dimensional modeling capabilities and the loading can be represented as combinations of gravity load, horizontal static load, response spectrum and time history base acceleration. The only limitation is that it is limited to elastic response, however, the consideration of inelastic response was beyond the scope of the present investigation.

In the analytical model, the building is represented by six different frames which are interconnected by the floor diaphragm which is assumed to be rigid in its own plane. The location of these frames is shown in Figure 5. The primary frame includes the beams and columns on column lines 1 and 4 and the shear walls at the east and west ends of the building. In order to facilitate the modeling of the structure, these walls were moved in to column lines A and F as shown in Figure 5. Hence the primary frame is a peripheral frame which has moment frames in the east-west direction and shear walls in the north-south direction. The two interior frames on column lines 2 and 3 are modeled as moment frames which "share" columns with the peripheral frame at locations F2, F3, A2 and A3. It should be noted that at these locations axial compatability is not enforced between these three frames. The remaining three frames consist of the three shear walls between the first and second floor levels on column lines C, D, and E. Prior to subjecting the model to dynamic loading, static load checks were made using the model to insure that equilibrium was satisfied in both directions.

SEISMIC INPUT

The seismic input for this phase of the investigation consisted of two response spectra developed from the strong motion acceleration data recorded on the base slab in the N-S and E-W directions. The recording instruments were located just west of the ground floor shear wall on column line E. These two spectra for five percent damping are shown in Figure 6 and Figure 7. The maximum recorded base acceleration in the N-S direction was .29g and the maximum in the E-W direction was .32g. Also shown on these figures for basis of comparison are the following three design spectra: (a) a Newmark-Hall spectrum (Ref. 6) based on the maximum ground acceleration and five percent damping, (b) a Newmark-Hall spectrum adjusted to reflect a design ductility of four and (c) the seismic coefficient, C, specified in the 1982 edition of the Uniform Building Code. The fundamental period of the County Services Building under ambient vibration was determined by Pardoen (Ref. 7) to be 0.65 seconds in the E-W direction and 0.44 seconds in the N-S direction. Results of the ETABS model gave 0.78 seconds in the E-W direction and 0.30 seconds in the N-S direction. In this period range it can be seen that there is a large difference between the spectral acceleration obtained from the recorded data and that specified by the code coefficient. On the other hand, the elastic design response spectrum (EDRS) suggested by Newmark and Hall gives a close approximation to the actual values. However, it is recognized that this criteria would result in predominantly elastic behavior and may be overly conservative. The Newmark-Hall inelastic design response spectrum (IDRS) with a design ductility of four gives values of spectral acceleration which are intermediate to the other two and may have resulted in a more adequate design.

DYNAMIC RESPONSE ANALYSIS

Both time history analyses and response spectrum analyses were performed on the building during the course of the investigation (Ref. 8). Of particular interest were the forces on the columns on column lines D, E and F since the initial failure and the more significant damage were concentrated in these members. Comparisons between the time history envelope and the response spectrum results indicated that the use of the square root of the sum of the squares modal combination results in a close approximation to the maximum values.

All columns of the ground floor were 24 inches (61 cm.) square. The exterior columns on column lines A and F were reinforced with 10 # 11 bars, whereas the other columns of the ground floor has 8 # 11 bars. The interaction diagram for the exterior columns on line F is shown in Figure 8 along with the calculated column forces due to excitation in the N-S direction. Here the capacity reduction factor, phi, has been set to unity. It can be seen that the developed forces fall well outside the interaction diagram and well above the balance point. This is indicative of an axial compression failure in the concrete which can be sudden and explosive in nature. This agrees well with the observed damage. Eyewitnesses reported that all the columns on line F tended to explode near the base. Measurements made on these columns immediately following the main shock indicated an axial shortening of some 9 inches (23 cm.). Concrete in the lower three feet of the columns was badly shattered and the reinforcing bars were severely buckled.

The interaction diagram for the interior columns on column lines E and F is shown in Figure 9. Here the difference between axial failure and flexural failure becomes readily apparent. The columns on both column lines are overstressed, however, the columns on line F are once again above the balance point indicating a brittle, compressive failure mode. Conversely the columns on column line E are below the balance point indicating a more ductile, flexural failure mode. The columns experienced cracking and spalling of the concrete cover, but no disintegration of the concrete core and no buckling of the reinforcing steel. The envelope of story shears in the two principal directions are shown in Figure 10. In both cases the story shears calculated from the response spectra are compared with those obtained using the 1967 Edition of the Uniform Building Code. In the N-S direction the shear due to the response spectrum is separated into that taken by the columns and that taken by the shear walls. As might be expected, the shear taken by the columns is almost negligible and the combined spectrum shear is almost thirteen times larger than that specified by the code. In the E-W direction, all story shear is resisted by the columns of the moment frames. Here the spectrum shear is eight times greater than the code requirements.

CONCLUSIONS

This study has used standard design analysis techniques to evaluate the behavior of the Imperial County Services Building under the base motions recorded during the damaging earthquake of October, 1979. Recognizing the limitations of using the elastic analysis procedures to evaluate inelastic behavior, the results of this study have shown that these analysis procedures could have properly identified the more serious problem areas associated with the building. Namely, the columns on column line F were severely overloaded and indicated the nature of failure would be predominantly a brittle, compressive mode. The importance of using a dynamic response analysis on structures having stiffness irregularities has also been illustrated. Finally the ability of the seismic coefficient contained in current building codes to produce an adequate level of earthquake resistance should be questioned. This is particularly true in certain frequency ranges such as was the case with the County Services Building. The building had the misfortune to have a natural frequency which placed it in an area of the response spectrum where the difference between the code and the earthquake was maximum. The seismic coefficient should be adjusted so that it is more in line with the shape of the IDRS suggested by Newmark and Hall. This would give a more uniform level of seismic resistance over a larger frequency range.

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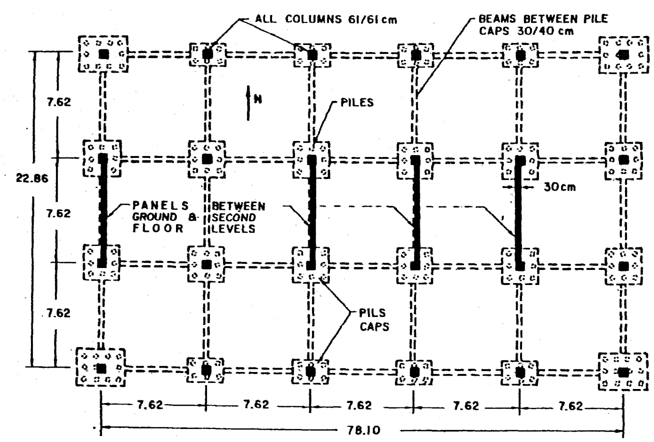

Fig. 1 Foundation and Ground Level Plan

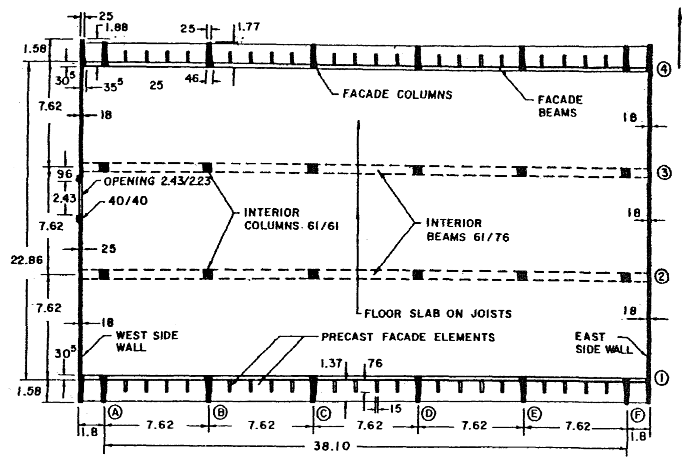

Fig. 2 Typical Floor Plan

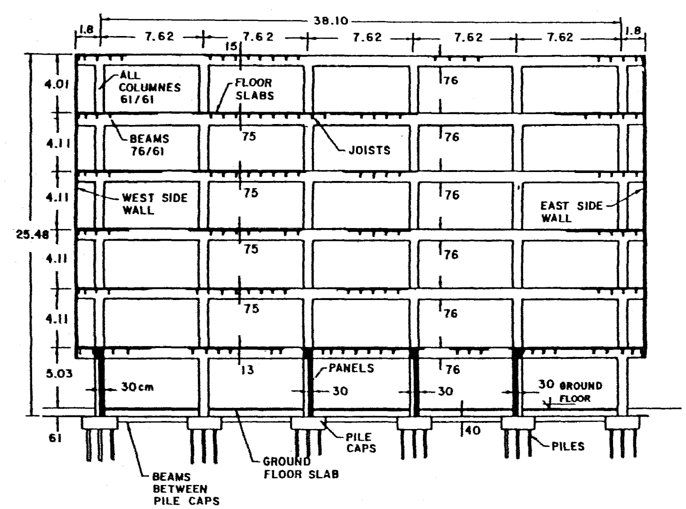

Fig. 3 East-West Longitudinal Elevation

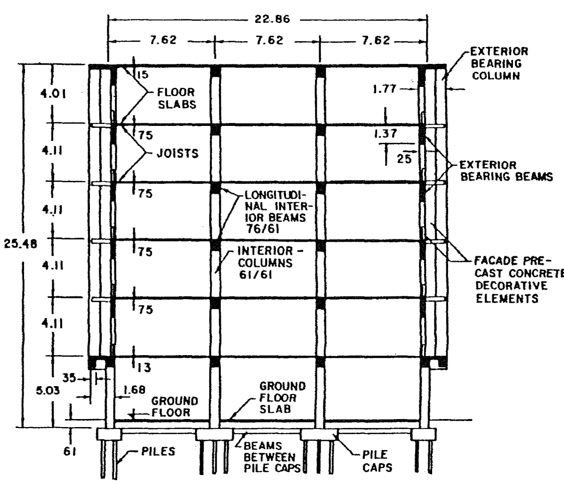

Fig. 4 North-South Transverse Elevation

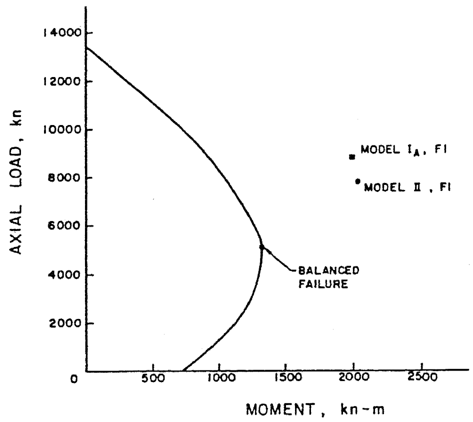

Fig. 8 Interaction Diagram Columns F1 & F4.

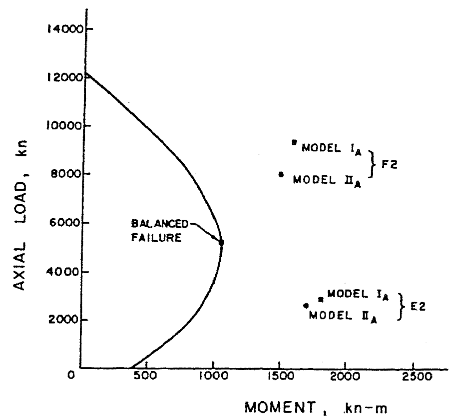

Fig. 9 Interaction Diagram Columns E2 & F2.

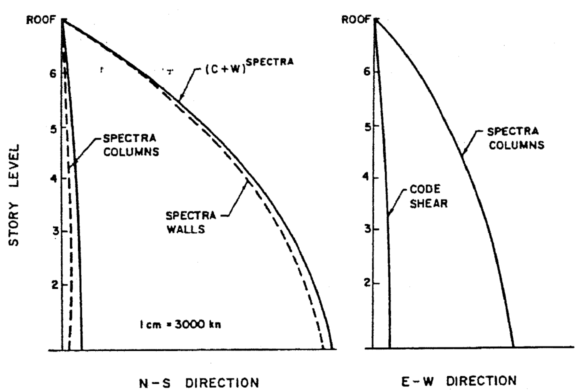

Fig. 10 Envelope of Story Shear Forces

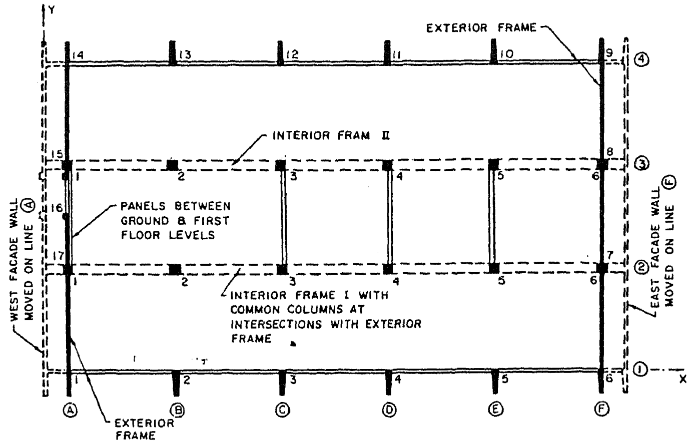

Fig. 5 Frame Layout for Analytical Model

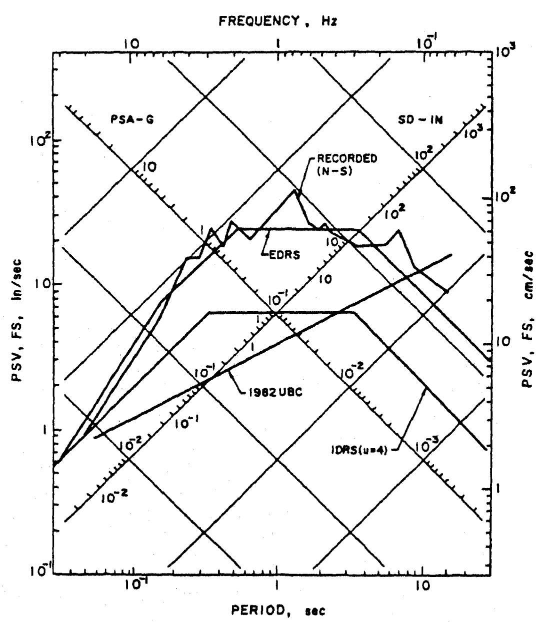

Fig. 6 Response Spectra, North-South

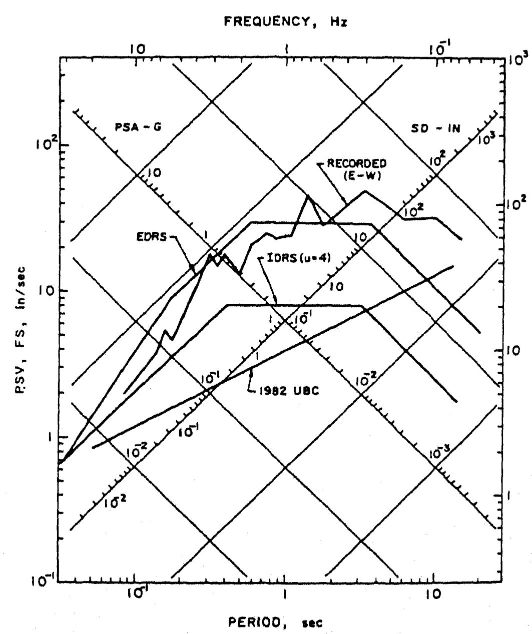

Fig. 7 Response Spectra, East-West