DESIGN AND CONSTRUCTION OF A THIRTY-STORY CONCRETE DUCTILE FRAMED STRUCTURE EMERYVILLE-EAST SAN FRANCISCO BAY

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SUMMARY

A 30-story concrete ductile framed structure, a 55 million dollar project, completed its construction in October of 1983, in the city of Emeryville, California. It is the first construction of this type in the Bay Area.

Code lateral seismic load and A.C.I. 318 were the design base for the minimum requirements. Special site responses of maximum credible and maximum probable earthquake were performed and the members were evaluated and adjusted to satisfy the ductility requirement. It is aimed to design all the elements within their elastic limit when responding to the maximum probable earthquake. The structure will also be designed to resist a maximum credible earthquake without major damage and to exclude collapse of the structure. As a consequence of the response analysis, members were designed to have a relatively higher percentage of reinforcing steel and to be larger in size.

Two level seismic spectrums and the mode superposition method in the evaluation of the responses of the members are outlined in the paper. Their results and design principle will be discussed.

INTRODUCTION

In November, 1981, construction began on a 30-story concrete ductile frame with three identical wings, forming a Y-shaped structure, (Figure 1) and consisting of several hundred condominium units.

Special seismic design provisions, site seismicity and earthquake recurrence evaluations were made and the design response spectra was developed. The design response spectra was also used for final evaluation of the structure and element design.

Two levels of earthquake, maximum probable and maximum credible, were evaluated to determine the seismic response of the structure. Earthquakes of magnitudes 8.25 at San Andreas fault and 7.5 at Hayward fault were selected as bases for the development of the design spectra. Maximum probable earthquake based on 50 years of the structure's life span and 50% probability of exceedance, with damping of 5%, was selected to develop a maximum probable earthquake spectrum.

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The structure utilizes concrete with strength of 6500 psi at the lower portion of the building and yielding strength of 60,000 psi in reinforcing steel. Special fabricated high strength wires were proposed for confinment reinforcement, column ties and beam stirrups. Structural designs were in compliance with the 1979 Uniform Building Code and the special criteria established by the consultant team. The frames will resist a maximum probable earthquake described above within their elastic limits. The beams are designed to resist a maximum credible earthquake with their ductility demand not to exceed four and the columns are designed to resist maximum credible earthquake with their ductility demand not to exceed two.

Story drift and the P-delta effect will be discussed in the paper. A structural model of three-dimensional space frames, with partially linked diaphragms, will be outlined.

LATERAL FORCE DESIGN CRITERIA

A 30-story concrete ductile framed structure was designed based on the 1979 edition of Uniform Building Code and A.C.I. 318 for vertical and lateral loads. Dynamic spectrum response analyses were also performed in evaluating the ductility demands.

In order to develop a design response spectrum for the structure, seismic exposure analyses to estimate the probability of exceedance of response spectral acceleration at the site in a selected time period were made. (Ref. 4)

Active Fault

The site with respect to known or suspected active faults in the bay area is shown in Figure 2. The San Andreas fault zone, which is located 25 KM west of the site, is the predominent active fault in the Bay Area. Its two major branches on the east side of San Francisco Bay are, the Hayward fault, located 6 KM east of site and Calaveras fault, the most active portion which extends to the vicinity of San Ramon, approximately 22 KM east of the site. The San Andreas, Hayward and Calaveras faults have been the source of moderate to strong historical earthquakes.

Maximum Credible Earthquake

A maximum credible earthquake is the largest earthquake that appears capable of occurring on a given fault with our current understanding of the tectonic framework. An estimated maximum credible earthquake is a magnitude of 7-1/2 on the Hayward and Calaveras faults and a magnitude of 8-1/4 on the San Andreas fault.

Analysis Results

Analysis results indicated that earthquake on Hayward fault dominate the contribution of the probability of exceedance of spectal acceleration at short period. However, at long periods, earthquakes on the Hayward and the San Andreas faults contribute approximately equally to the probability of exceedance. The elastic response spectrum based on free field ground motion for various earthquakes, damping ratio, probability of exceedance, and design life of the structures are plotted as shown on Figure 3.

Structural analysis was performed for the following lateral load cases:

1. Code Static Lateral Earthquake Forces

The 1979 edition of the Uniform Building Code of zone 4 with a soil factor of 1.5 and a K factor of 0.67 were used as minimum forces in the formula: V = ZIKCS. The base shear force of this code is basically similar to that specified in the "Recommended Lateral Force Requirements and Commentary" by the Structural Engineers Association of California. The base shears were in the order of 4% of the total weight of the structure. As a minimum requirement, the first mode was assumed in the force distribution in the code static approach.

2. Code Wind Loads

Wind loads of specified pressure tables in the Building Code with a wind map of 20 pounds per square foot were designed. Wind shears at the base level are approximately 36% of those code seismic base shears and the wind overturning moments are approximately 30% of those code seismic overturning moments. As this comparison demonstrates, wind load will not govern the design of the elements.

3. Dynamic Load - Maximum Probable Eartquake

Dynamic spectrum response analysis for a maximum probable earthquake with the damping ratio of 5% and a 50% probability of being in exceedance in 50 years. 80% of the spectral free field response will be used.

4. Dynamic Load - Maximum Credible Earthquake

Dynamic spectrum response analysis for maximum credible earthquake with damping ratio of 10% and a 10% probability of being in exceedance in the period of 100 years. 80% of the spectral free field response will be used.

It is noted that the first mode responses (period of 2.8 seconds for the first mode) for maximum credible and maximum probable earthquake, in terms of shear coefficient, are 0.22 and 0.104 respectively. The code base shear coefficient which is basically a first mode analysis, is 0.04. By considering the modal mass participation factor in the dynamic analysis, the equivalent shear coefficient was in the order of 0.173 and 0.081 respectively for maximum credible and maximum probable earthquake.

RESPONSE ANALYSIS

The three-dimensional framed structures were analyzed using the computer program which is capable of analyzing the diaphragm with the option of skip connectivity at the floor level. This is due to the absence of the floor at the north and south wing on mezzanine level. There is only a mezzanine floor at the west wing which produces a substantially large torque at the level. In order to minimize the twist effect, a shear wall system was introduced at the level below the second floor. The preliminary studies indicated that the walls had a tendency to equalize the stiffness at that level even with the skip floors at north and south wings. The structures were analyzed with and without walls at the lower portion of the building. The story shear, the overturning moments for code forces and the displacement diagram for code and dynamic analysis are shown on Figures 4 and 5.

The seismic story drift index of 1/200 and the P-Delta effects were checked. The P-Delta for each floor and their ultimate effects on the columns were evaluated using the formulae outlined in Applied Technology Council Report A.T.C.-3.(Ref. 1) The formulae is stated briefly as follows:

$$\theta = \frac{P_x \bullet D}{V_x\ H_{sx}}$$

Where V_x is the total story shear; D is the story drift or displacement; H_{sx} is the story height.
When θ = 0.1 neglect P-Δ effect.

The columns for each floor were examined and their indexes were determined using the mode superposition method. The responses of individual modes were then combined using the techniques of the "Complete Quadratic Combination" or simply called the CQC Method. This method has been described in several papers. The primary advantage of the CQC Method is reducing the errors in modal combination and improving the accuracy over the "Square Root of Sum of Squares" method used in building analysis. (Ref. 6)

For a typical force component, f_k: $f_k = \sqrt{\sum_i \sum_j f_{ki}\ \rho_{ij}\ f_{jk}}$

Where: f_{ki} - typical force component which is produced by the modal displacement vector, U_t maximum.

ρ_{ij} - Cross modal coefficients, function of the duration and frequency content of the loading and of the modal frequencies and damping ratios of the structure.

From the diagram in Figure 4, the displacement for the maximum credible earthquake and the code level earthquake were in the order of 3.69 to 1. The maximum probable earthquake is approximately 1.73 to 1.0. The vertical member axial component resulting from the overturning effect in comparison to the code level design was in the order of 1.70 to 1.0 for a maximum probable earthquake. The correspondent beam moments were in the order of 2.1 to 1.0.

Members were sized and reinforced based on the following loading combination factors:

1. For the code level earthquake, the orthogonal independent evaluation of members is:

   U = 1.4 D.L + 1.7 L.L
   U = 1.4 (D.L + L.L + S)
   U = 0.9 D.L - 1.4 S

2. For a maximum probable earthqake response analysis, the orthogonal effect of the earthquake in combination of 100% of the forces for one direction, plus 30% of the forces for the perpendicular direction were used in evaluating the strength.

   U = 1.0 D.L. + 1.0 L.L + 1.0 S
   U = 0.8 D.L. - 1.0 S

Based on above criteria, flexure members of beams and axially loaded vertical supporting members will be subject to ductility examination.

MEMBER DUCTILITY EVALUATIONS-FLEXURE AND AXIAL MEMBERS

In order to predict the behavior of the structural earthquake resisting elements when subject to disturbance of a real earthquake, member ductility was evaluated. The demanding ductility of the element was calculated for both maximum credible and maximum probable earthquakes. The ductility is defined as the ratio of the moment required to the moment of the element when steel yields.

The ductility criterion was set as follows:

1. Under the responses of the maximum probable earthquake, flexure members shall have their ductility factor of unity:

$$\frac{M_{Earthquake}}{M_{Yielding}} \lesssim 1.0$$

2. The axial loaded members shall have their ductility u for the maximum probable earthquake case equal or less than unity:

$$u = \frac{Mox}{Mux}\left(\frac{1-B}{B}\right) + \frac{Moy}{Muy} \doteqdot 1.0$$

Where: Mox, Moy = Maximum summation of the factored moments in x and y direction.

Mux, Muy = The uniaxial ultimate moment capacities in x and y direction based on the axial force defined in the load combinations.

B = A factor as defined in ACI Column design handbook ACI SP-17A (Ref. 7).

3. For the maximum credible earthquake case, the flexure members shall have their ductilities not exceeding 4.0.

4. For the maximum credible earthquake case, the columns shall have their ductilities not exceeding 2.0.

It should be noted that the axial load of the columns under the maximum credible earthquake may not necessarily be the value obtained in the computer modal analysis, since the beams developed their ductility prior to the columns. A thorough check on the beam ductility and a reassessment of the actual axial forces, which is the accumulation of the beam elastic shears, were essential in determining the real ductility value for the columns in this case.

The ductilities of the beams were generally close to unity for the maximum probable earthquake and less than 2.4 for the maximum credible earthquake. This indicates the design is governed essentially by the maximum probable earthquake criteria.

SPECIAL CONSTRUCTION FEATURES

The columns were reinforced with prefabricated high strength steel ties which are machine bent fabric wires (Figure 6). Ties and confinement reinforcements are prebent at the shop and are assembled with vertical column steel forming a steel cage for the column erection operation. Cages will be transported to the site and lifted to their position by a tower crane. The heavy reinforced columns will be mechanically spliced, (Figure 7) in order to avoid lapping and congestion. Number 14 bars, which require mechanical couplers, were used for most of the columns. In order to arrange beam reinforcing in both the x-x and y-y directions, the beam depth was carefully selected. It is generally offset with three inches in depth. Thus, the bottom reinforcings will not conflict with each other.

Column vertical reinforcing steel and beam flexure reinforcing steel are of 60 ksi. The shear reinforcing steel and slabs are also of 60 ksi steel. Confinement reinforcements are 75 ksi steel prefabric steel.

The column sizes were also selected to have each step of three inches or less, in order to avoid sharp vertical bends at the column reinforcement.

High-strength concrete of 6500 psi was planned for the lower portion of the columns which not only reduces the size of the column, but also saves the vertical reinforcing steel. As required in SEAOC recommendation for lateral force design, the beams will develop their plastic moments prior to their connected columns.

For a maximum probable earthquake, the diaphragm stresses were also examined with the loading combination of 100% force in one direction plus 30% in a perpendicular direction. It was found to be adequate with normal reinforcement which had been designed in accordance with the code seismic force level.

A 14 x 14-inch prestressed concrete pile, 60 to 70-feet long, with a flat pile tip was designed to have a capacity of 240 kips for loadings in addition to the prestressing forces. The pile top was reinforced with spiral confinement reinforcing steel. A two-inch diameter hole with a corrogated sheet, for three to five-feet at the top of the pile, was required. It will seat rebar or high strength bars after pile driving and will be grouted and integrated with a pile cap.

CONCLUSION

As the result of the research work on the earthquake engineering, a concrete ductile framed structure can be constructed in a high seismicity zone with adequate structural safety and constructibility. A concrete ductile framed structure can resist a moderate earthquake elastically and can sustain the credible earthquake without major structural damage. Details on element size selection and reinforcing steel arrangements can be developed systematically. Construction work can be planned using skills and techniques such as the fabric steel ties, bar couplers and high strength concrete currently available in the industry.

ACKNOWLEDGEMENT

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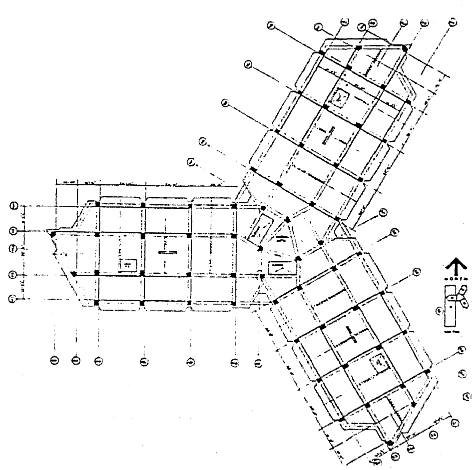

Fig. Ia PLAN

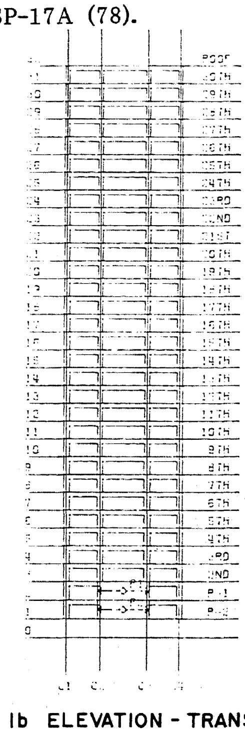

Fig. Ib ELEVATION - TRANSVERSAL FRAME

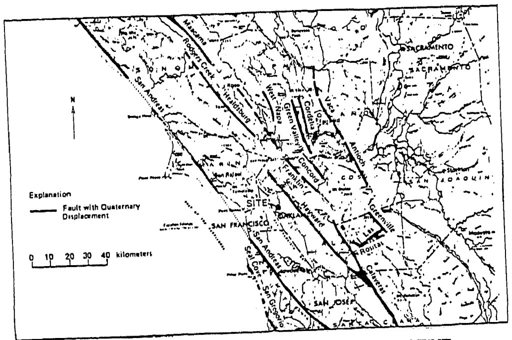

FIG. 2 -SITE LOCATION WITH RESPECT TO ACTIVE OR POTENTIALLY ACTIVE FAULTS

REF.(4)

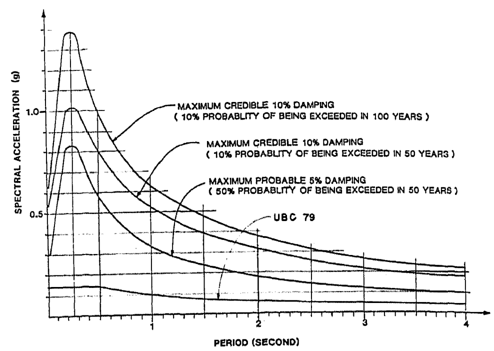

Fig. 3 ACCELERATION RESPONSES SPECTRA

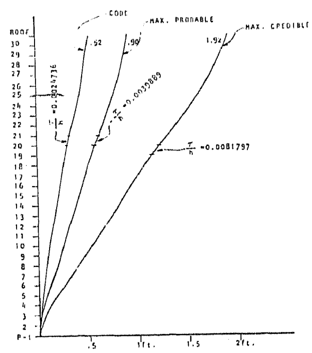

Fig. 4 DISPLACEMENT

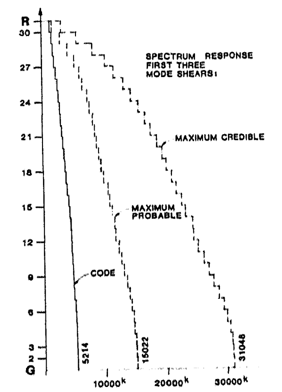

FIG 5 STORY SHEAR

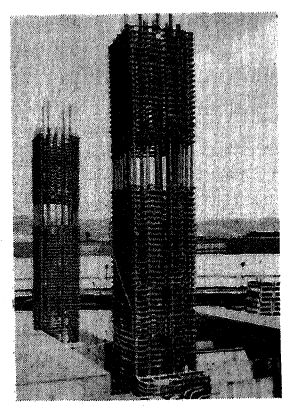

FIG. 6 COLUMN CAGE

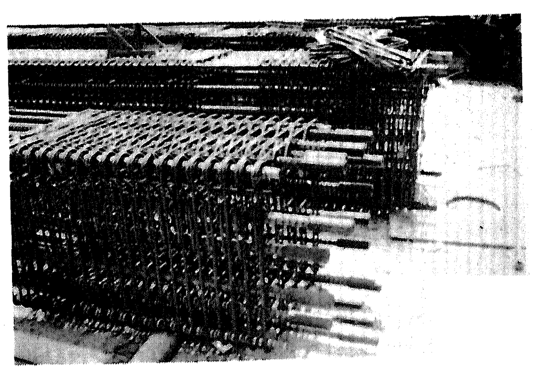

FIG. 7 MECHANICAL SPLICER