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SEISMIC DESIGN AND BEHAVIOR OF A THIRTY-STORY REINFORCED CONCRETE BUILDING

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SUMMARY

This paper describes the seismic design of a 30-story R/C apartment building, and some following analytical and experimental investigations to assess the performance of the structure. Design considerations developed to assure a whole collapse mechanism and the ductility of plastic hinges were presented. Dynamic nonlinear analyses indicated that the maximum inter-story drift would be 1/340 and 1/170 when subjected to the maximum probable and the maximum credible earthquakes, respectively. Tests on columns and beam-column subassemblies demonstrated their ductile behavior. It was assured that the designed structure had sufficient margin of the seismic capacity against the design earthquakes.

BUILDING DESCRIPTION

The building is a 30-story apartment house constructed at Hikarigaoka, Nerima-ku,Tokyo, Japan. The perspective view, typical floor plan and section of the structure are shown in Photo 1, Figs.1 and 2, respectively. The structure consists of moment resisting space frames having 6 spans in both directions, while it has shear walls at the basement story. The structure is supported by cast-in place reinforced concrete piles extending 27m below the ground level into firm gravel layer. All columns and beams are cast-in place reinforced concrete, while semi-precast concrete decks were used to construct composite floor slabs.

DESIGN PHILOSOPHY AND CRITERIA

<u>General Design Philosophy</u> The philosophy of so called 'strong column and weak beam' was adhered. The desirable collapse mechanism of a multi-story framed structure must be a whole collapse mechanism rather than a partial sidesway mechanism (soft story mechanism) so as to provide the structure with enough energy dissipating capability as well as adequate strength and stiffness. Plastic hinges were designed to be formed at only beam ends, bottom of columns in the lowest story and top of columns in the upper-most story. The formation of plastic hinges of exterior columns under axial tension was allowed since such columns would behave in a ductile manner.

<u>Design Consideration for Members</u> The special requirements for the design of members were proposed as follows,in order to assure the ductility of members having potential plastic hinges and to avoid a soft story mechanism. The experimental studies done by the authors (Ref.1,2,3) as well as other researchers

(Ref.4) were reflected to those requirements.

Beams were designed and detailed to possess enough ductility (displacement ductility of more than 4) as well as sufficient shear capacity. The design of a beam for shear was based on

$Qsu > QL+1.1Qmu$ for potential plastic hinge regions (1)

$Qsu > QL+Qmu$ for other regions (2)

where Qsu is ultimate shear strength, QL is shear force due to gravity loads, and Qmu is shear force calculated from the flexural strength at both ends. In a potential hinge region, closed-stirrups are arranged so that each longitudinal reinforcement may be restrained against buckling.

Columns were designed to have sufficient flexural and shear capacities in order to ensure the intended formation of plastic hinges in beams. The design of a column for flexure was based on

$cMmu > 1.3cMu$ (3)

where cMmu is ultimate flexural strength of the column under the given axial force and cMu is column moment at the hinge mechanism of the structure. The design of a column for shear was also based on

$cQsu > 1.3cQu$ (4)

where cQsu is ultimate shear strength and cQu is column shear force at the hinge mechanism of the structure. Columns having potential plastic hinges were designed and detailed also for ductile behavior. Special attention were paid on the level of axial compression and lateral reinforcement to ensure the required ductility. Axial forces attained in columns at the hinge mechanism (Nu) were limited as follows.

$-0.25bDFc < Nu < 0.65bDFc$ for exterior columns (5)

$Nu < 0.4bDFc$ for interior columns (6)

where b,D and Fc are width and depth of the column,and specified compressive strength of concrete. The lateral reinforcement were arranged so as to confine the core concrete and to restrain longitudinal bars against buckling.

Beam-column joints were designed to possess sufficient shear capacity to prevent brittle failure. The development length not less than 20 times the bar diameter of beam reinforcement passing through interior joints was provided to prevent the significant deterioration of energy dissipation capability due to the loss of bond inside the joint panel subjected to cyclic reversed inelastic deformation.

Performance Criteria Two levels of earthquake input, the maximum probable and maximum credible levels with the intensity of 25 and 50cm/s in terms of the maximum ground velocity, respectively, were considered for the dynamic response analysis. It was aimed for the first level input to remain the structure within pre-yielding range, or no occurrence of plastic hinges in any structural members sustaining the inter-story drift less than 0.5%. It was also aimed for the second level earthquake to sustain post-yielding displacement, however, less than 1% in terms of the inter-story drift. For the member level displacement the ductility not more than 4 was allowed. It should be noted that the levels of seismic input and performance criteria mentioned here were determined referring to design practices for tall buildings of other structures of steel or steel encased reinforced concrete in Japan.

DESIGN OF THE STRUCTURE

Design earthquake loads In order to determine design earthquake loads, preliminary nonlinear response analysis was carried out for some recorded earthquake ground motions factored to meet the peak velocity of 25cm/s, idealising the structure as a lumped MDOF system. The estimated fundamental period of the structure was 1.98sec. and 2.02sec. for longitudinal (X) and transverse (Y) direction, respectively. Referring not only to the result of the preliminary response analysis shown in Fig.3 but also to available design recommendations for tall buildings (Ref.5) and design practices, the value of design base shear coefficient was taken as 0.12. The distribution of lateral forces was determined as shown in Fig.3 based on the result of the preliminary analysis. Note that it was aimed to provide the structure with the ultimate lateral load carrying capacity approximately 1.5 times the design lateral forces.

Structural Design The structure was designed based on the A.I.J. Structural Standard (Ref.6) and the special considerations described previously. Typical cross sections of beams and columns at the lower stories of the structure are shown in Fig.4. The structure utilized the concrete of the specified strength ranging from 420kg/cm^2 (41Mpa) at the lower portion to 210kg/cm^2 (21Mpa) at the upper portion of the building. It also utilized steel bars of the specified yielding strength 4000kg/cm^2 (392Mpa) and of the diameters from 41 (D41) to 25mm (D25) as flexural reinforcement. It is noted that welded deformed wire fabrics and flash-welded closed-stirrups were utilized as lateral reinforcement in the columns and beams, respectively(Fig.4). It should be noted that the bars of welded deformed wire fabrics were welded so as to develop in tension 1.35 times the specified yield strength, or the breaking strength of the bar,whichever is smaller,and that the bars of flash-welded stirrups were butt-welded to develop in tension the breaking strength of the bar.

SEISMIC ANALYSIS OF THE STRUCTURE

Static Analysis Nonlinear static frame analysis in member to member level was conducted to investigate the formation of plastic hinges and the inelastic displacement of the designed structure. The results are shown in Fig.5. It was indicated that the hinge mechanism would be formed as designed, that is, whole collapse mechanism. The obtained ultimate capacity of the structure in terms of the base shear coefficient was 0.18, which is 1.5 times the value of the design lateral forces.

Dynamic Response Analyses Idealising the structure as a lumped MDOF system and considering the soil-structure interaction, nonlinear dynamic analysis was carried out to investigate the response of a overall structure to earthquake motions. The shear force vs displacement relationship of each story was determined based on the static inelastic frame analysis, and was idealised as shown in Fig.6. The recorded earthquake ground motions listed in Table 1 were used. The amplitude in each acceleration record was scaled in the manner where the maximum velocity reaches 25 and 50cm/s, corresponding to the maximum probable and the maximum credible levels of earthquake motions, respectively. As shown in Fig.7, the result indicated that the maximum response of the inter-story drift would be 1/337 and 1/172 when subjected to the maximum probable and the maximum credible motions, respectively. It was also indicated that the maximum shear force would not reach the ultimate capacity at any story.

In order to determine the seismic behavior of not only the whole structure but also structural members in detail, the nonlinear dynamic analysis in member to member level was conducted for a frame to represent the whole structure. The recorded ground motions of EL CENTRO 1940 NS with the scaled maximum velocity of 25 and 50cm/s were used as input motions. The obtained maximum inter-story drifts corresponding to the two levels of motions were 1/520 and 1/211, respectively, as shown in Fig.8. No plastic hinge was developed in any beams and columns.

TEST FOR THE STRUCTURE

Simulated Seismic Loading Test of Columns Five 1/2.5-scale column specimens representing lower story columns of a 30-story framed structure, were tested (Ref.1,3). Main objectives of this test were to investigate the ductility and the shear capacity of such columns under high axial compression force corresponding to the upper limit for the design of exterior columns specified previously, and the effect of biaxial loading on the behavior of the columns. As shown in Fig.9, the columns exhibited stable behavior until the displacement of 3% without any drop of strength even under the high axial compression. It was also indicated that the columns had sufficient shear capacity. Only minor effect of loading direction was detected.

Test of Beam-Column Joints Four half scale interior beam-column subassemblies representing a lower part of a 30-story framed structure were tested(Ref.1,3). Deformed bars with the nominal diameter of 16 and 19mm (D16, D19) were provided as main bars in beams, providing the length of 21 and 25 times the bar diameter, respectively, for the development of bars passing through the joint panel. All the specimens exhibited ductile behavior up to the story drift of 8% as shown in Fig.10. The joints had sufficient shear capacity and no significant pinching effect was observed on the hysteresis loops.

CONCLUDING REMARKS

The seismic design of a 30-story R/C building and following analytical and experimental investigations were presented here. Emphases were put on the hinge mechanism, the special provisions for members subjected to flexure, shear and axial force to assure the ductile behavior of the whole structure, and the non-linear static and dynamic analyses.

It was assured by the analytical and experimental investigations that the designed structure would have sufficient margin of the seismic capacity and it would resist severe earthquake motions without any major structural damage.

REFERENCES

1. S.Sugano and T.Nagashima ' Structural Design and Research of Reinforced Concrete Tall Buildings ', Proceeding of the Seminar on Reinforced Concrete High-Rise Buildings in Japan with Special Concern on Aseismic Design of Beam-Column Joints, PMFSL Report No.87-5, the University of Texas at Austin, November 1987.
2. S.Sugano, et al. ' Experimental Studies on Columns and Frames for Tall Reinforced Concrete Buildings ', Takenaka Technical Research Report No.35, May 1986.
3. S.Sugano, et al. ' Experimental Studies on Columns and Frames for High-Rise Reinforced Concrete Buildings, Part 1 to Part 6 ', Annual Conference of Architectural Institute of Japan (A.I.J.), 1985 and 1986.
4. ' Data for Ultimate Strength Design of Reinforced Concrete Structure ', A.I.J., Sept. 1987.
5. ' Technical Guidelines for Tall Buildings ', A.I.J., March 1973.
6. ' A.I.J. Standard for Structural Calculation of Reinforced Concrete Structure-1985 ', A.I.J.
7. S.Sugano, et al. ' Seismic Resistant Design of Tall Reinforced Concrete Buildings ', Takenaka Technical Research Report No.36, Nov. 1986.

Photo 1 A 30-Story R/C Building

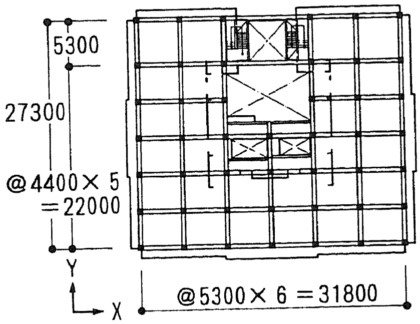


Fig. 1 Typical Floor Plan of The Building

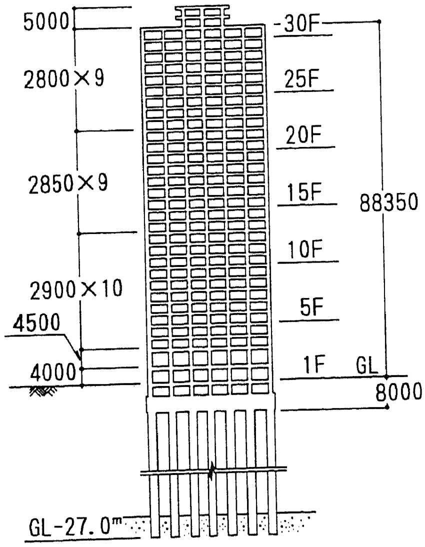


Fig. 2 Section of The Building

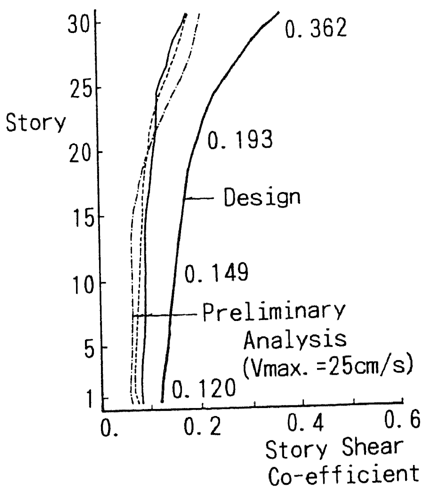


Fig. 3 Design Earthquake Loads

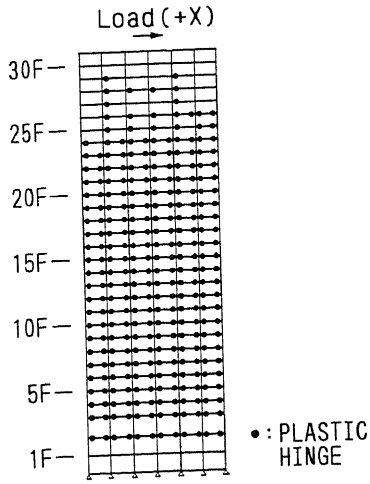


Fig. 5 Development of Hinges Under Static Lateral Loads

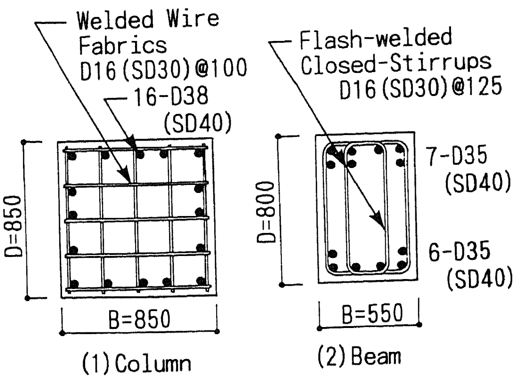
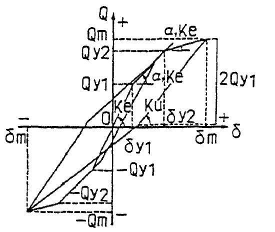


Fig. 4 Typical Cross Sections at Lower Stories

Name of Recorded Earthquake Ground Motion	Scaled Max. Acc. Corresponding to Vmax.=25cm/s	Scaled Max. Acc. Corresponding to Vmax.=50cm/s
EL CENTRO 1940 NS	206	412
TAFT 1952 EW	216	432
TOKYO 101 1956 NS	256	513
SENDAI TH038-1 1987 EW	152	303

Table 1. List of Input Earthquake Ground Motions (unit:cm/sec^2)



$\delta y1 \leq |\delta m| \leq \delta y2$

$: Ku = \frac{1}{2}(Ke + Qm/\delta m)$

$|\delta m| > \delta y2$

$: Ku = \frac{1}{2}(1 + Ke \cdot \delta y2/Qy2) \cdot Qm/\delta m$

Fig.6 Idealized Hysteresis Rule

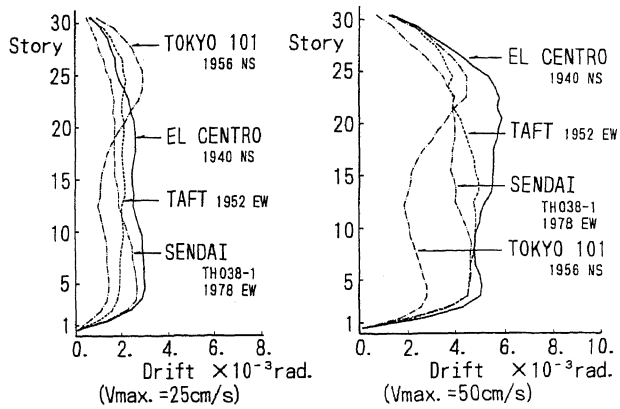


Fig.7 Max. Response of Inter-Story Drift (Lumped MDOF Model)

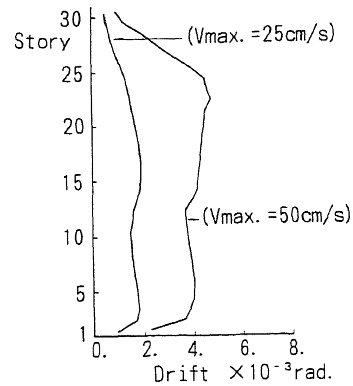


Fig.8 Max. Response of Inter-Story Drift (Frame Model)

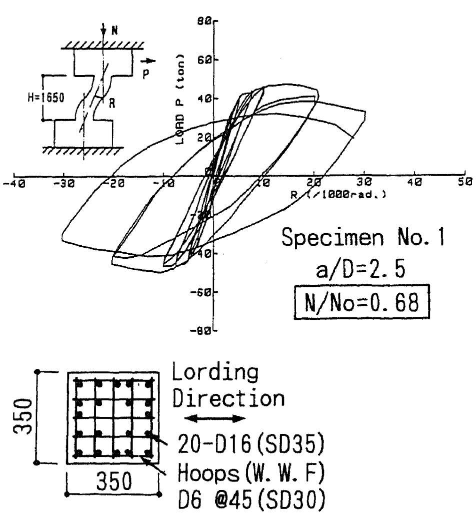


Fig.9 Lateral Load-Displacement Curves of Column Under High Axial Compression

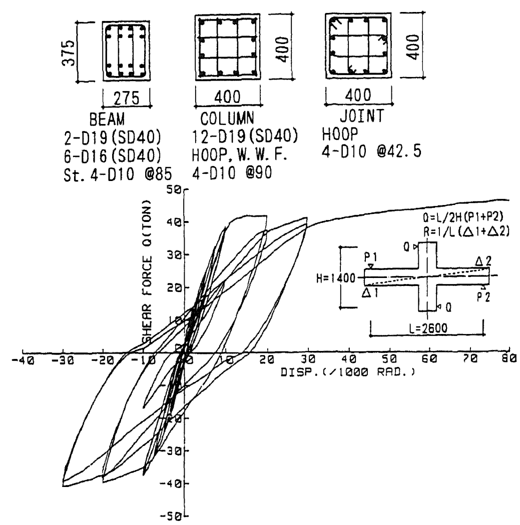


Fig.10 Lateral Load-Displacement Curves of Beam-Column Subassembly