SEISMIC RESISTANCE OF A 31-STORY SHEAR WALL-FRAME BUILDING USING DYNAMIC INELASTIC RESPONSE HISTORY ANALYSIS

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SUMMARY

Aseismic design of a 31-story reinforced concrete building is carried out on the basis of dynamic inelastic response history analyses under a carefully selected input motion. The design approach makes it possible to: 1) predetermine the sequence of plastification, 2) provide ductility details only where required, and 3) balance the strength and ductility requirements of members. Efficiency, economy, and desired structural performance are achieved as a result.

INTRODUCTION

In the current Code approach to earthquake-resistant design of buildings, Code-specified "equivalent" static loads are applied to the mass locations of a structure, and an elastic analysis is carried out to determine the member forces. These member forces may bear only a nominal resemblance to internal forces that result from an actual inelastic earthquake response of the structure. Also, the distribution and magnitude of inelastic deformations in various structural members cannot be determined through elastic analysis under Code-specified static loads. As a result, ductility has to be supplied throughout the entire structure, although inelasticity may actually occur only in certain levels and locations.

Two-dimensional inelastic dynamic (response history) analysis computer programs, incorporating proper hysteretic characteristics of reinforced concrete and steel members, have recently been developed. With such programs, it is now possible to perform a realistic analysis of the earthquake response of multistory concrete and steel structures at a reasonable cost. Designs based on such analyses make it possible to provide ductility details only where required, and to strike a desirable balance between strength and ductility requirements. A predetermined sequence of plastification can also be designed into a structure - for example, having the beams yield before the columns. It should be mentioned that the ductility discussed in this paper is based on rotations at the ends of individual yielding members. It is not the overall displacement ductility of the entire structure.

The inelastic design approach mentioned above is applied in this paper to a 31-story reinforced concrete frame-shear wall building located in an area of considerable seismicity. The application of the inelastic approach to the analysis and design of the structure results in significant efficiency and economy.

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STRUCTURE, MODELING, AND ANALYSIS

The building and its lateral load resisting system

The building considered is circular in plan, with an area of approximately 9,500 sq ft per floor (Fig. 1a). The building has a total height of about 440 ft over 31 stories (Fig. 1b). It is located in an area with a seismicity equivalent to that of Zone 3 of the Uniform Building Code[1] four-zone seismic risk classification.

The lateral load resisting system in one direction consists of the core walls bending about their major axes; the walls are connected at every floor level to the columns on each side through the main beams. In the orthogonal direction, the two C-segments of the core walls are coupled through two coupling beams at every floor level, and form a box which is connected on both sides to the peripheral columns through the main beams. The orthogonal lateral load resisting system consists of the coupled wall segments bending about their minor axes in interaction with the peripheral columns. The analysis of the building in the coupled direction only is considered in this paper.

Modeling of structure for static and dynamic analyses

The structural model used in analysis has four column lines (Fig. 1b). The two inner lines represent the two C-segments of the central core; the links between them are the lumped coupling beams. The two outer column lines represent the two pairs of columns (each pair lumped into one column) at the two ends. The links between the outer column lines and the inner column lines are the lumped main beams.

In the dynamic analyses, the masses are concentrated at every floor level. Each node has three degrees of freedom - horizontal translation (all the nodes on the same floor undergo the same horizontal translation), vertical translation, and rotation.

Static analyses, periods and mode shapes, dynamic analysis

In accordance with UBC,[1] 20 psf Zone wind forces, as well as Zone 3 equivalent static seismic forces (with K = 1.0), were considered on the building. Elastic static analyses of the building under these forces, in the coupled direction, were carried out. The natural undamped periods and mode shapes of the analytical model of the structure were also determined using a standard computer program.

Dynamic inelastic response history analysis, by the computer program DRAIN-2D[2], was used to determine the amount and distribution of inelastic deformations in the various structural members. Simplified dynamic analysis by modal superposition, used in conjunction with elastic analysis, cannot provide the needed information. DRAIN-2D accounts for inelastic effects by allowing the formation of concentrated "point hinges" at member ends. The moment-rotation characteristics of these hinges are defined in terms of a hysteretic loop with post-yield unloading and reloading stiffnesses gradually decreasing.

Input motion

A geotechnical investigation of the site indicated the design earthquake intensity* to be two-thirds that of the 1940 El Centro N-S record. The following investigation was carried out to select the frequency content of the input motion to be used in dynamic analysis.

Computations showed the building to have initial fundamental periods of 1.995 and 2.614 seconds in the uncoupled and coupled directions, respectively. Four input motions were selected as being potentially critical for structures with such periods - two having their relative velocity response spectra peaking close to the above period values, and two having broad-band spectra ascending beyond the period range of interest. Inelastic dynamic analyses of the structure in the uncoupled direction were carried out under all four input motions normalized to the above intensity. In these analyses, the columns and walls were kept elastic throughout their seismic response. Some of the beams (designed preliminarily by static analysis under UBC[1] Zone 3 seismic forces) yielded somewhat under one or more input motions. The results indicated clearly that the El Centro, 1940, E-W component is the critical input motion for both directions of the structure. The first 10 seconds of this motion, normalized to the intensity given above, is referred to as the design earthquake herein.

Performance criteria

The performance criteria chosen for the structure under consideration are that: 1) the columns and walls must remain elastic throughout their response to the design earthquake; 2) the coupling beams and main beams must remain elastic up to 1.4 times the design wind loads, their inelastic behavior setting in beyond that load; 3) the ductility demands of the beams must be kept below the limit of available ductility; and 4) the nominal shear stress in the beams must not exceed 6 $\sqrt{f'_c}$.

RESULTS OF ANALYTICAL INVESTIGATION

Table 1 shows the extent of the analytical investigation. In addition to the elastic static analyses under Code wind and earthquake forces, three inelastic dynamic analyses under the design earthquake are presented. In all dynamic analyses, the columns and walls were kept elastic throughout their seismic response. 5% of critical damping was assumed. The yield levels for the coupling beams were chosen on the basis of the investigation described below.

Choice of coupling beam strength

Four inelastic dynamic analyses (one of these is Analysis #2, the other three are additional to those listed in Table 1) under the design

* Measured in terms of spectrum intensity or the area under the 5%-damped relative velocity response spectrum between periods of 0.1 and 3.0 sec.

earthquake were carried out with four sets of coupling beam strengths. The strengths ranged from values close to the maximum moments computed for the coupling beams from Analysis #1, gradually decreasing to the values listed under Analysis #2. The yield strengths for the main beams used in these analyses were fixed at values listed under Analysis #2. The coupling beam strengths chosen (Analysis #2) were found to be optimal with respect to shear capacity and ductility requirements in the coupling beams, as well as the axial forces in the coupled wall piers. Some maximum response quantities from Analysis #2 are presented, along with the results of Analysis #1 in Fig. 2. The differences between the two sets of response quantities should be noted. The chosen yield levels for the coupling beams are about one-half (Tiers 1, 2) to one-third (Tiers 3, 4) of the maximum static moments induced in these beams by UBC Zone 3 earthquake forces (Table 1).

Optimization of main beam strength

In Analysis #2, the yield levels of the main beams were chosen close to the elastic moments induced in these beams by UBC Zone 3 equivalent static seismic forces (as computed from Analysis #1). The resulting ductilities of the main beams were only around 2.

In an effort to further optimize the design solution, Analysis #3 was run with the main beam strengths reduced to just over 1.4 times the moments computed for these beams from static analysis under factored Code wind forces (Analysis #0). The results, presented in Fig. 2, show that the corresponding ductility requirements for the main beams in the upper two tiers are excessive.

To remedy the above situation, and to still arrive at an efficient and economical solution, the strengths of the main beams in Tiers 1 and 2 were substantially increased, and Analysis #4 was run. The results (Fig. 2) indicate that the ductility requirements in the main beams are now acceptable (the largest value is less than 8). The ductility requirements in the coupling beams are comparable with those from Analysis #2, and show some improvements over those from Analysis #3. The shear capacity requirements in the coupling beams are the same as those obtained from Analysis #2; the main beams require low shear capacities—below $2\sqrt{f'_c}$. The axial forces in

Table 1: Summary of Analytical Investigation

#	Analysis	Tier	MAIN BEAMS (Lumped)		COUPLING BEAMS (Lumped)	
			Computed Max. Moments, Factored (in.-k)			
0	Elastic Static Analysis Under UBC 20 psf Zone Equivalent Wind Forces	1	4,739		3,782	
		2	8,593		10,406	
		3	12,532		22,996	
		4	17,421		40,369	
1	Elastic Static Analysis Under UBC Zone 3 Equivalent Seismic Forces, with K = 1.0	1	26,460		29,778	
		2	39,775		48,287	
		3	53,778		81,621	
		4	66,900		136,829	
			Chosen Yield Moment (in.-k)	Ductility	Chosen Yield Moment (in.-k)	Ductility
2	Inelastic Dynamic Analysis Under Design Earthquake	1	17,500	1.41	14,300	6.41
		2	37,800	1.53	22,750	10.74
		3	50,400	1.77	31,900	12.32
		4	56,000	2.20	41,650	11.47
3	Inelastic Dynamic Analysis Under Design Earthquake	1	5,600	20.13	14,300	9.25
		2	8,600	12.98	22,750	9.85
		3	12,600	7.90	31,900	9.56
		4	17,500	3.12	41,650	7.64
4	Inelastic Dynamic Analysis Under Design Earthquake	1	12,600	5.67	14,300	6.06
		2	12,600	7.47	22,750	10.02
		3	12,600	7.87	31,900	11.00
		4	17,500	3.80	41,650	8.84

the coupled wall piers are higher than those given by Analysis #2, but are still very much lower than those computed from Analysis #1. The axial forces in the columns are substantially lower than those given by Analysis #2. The other response quantities do not suffer any serious adverse effect in going from Analysis #2 to Analysis #4. It should be noted that the final yield levels chosen for the main beams are 48% (Tier 1), 32% (Tier 2), 23% (Tier 3), and 26% (Tier 4) of the maximum static moments caused in these beams by UBC Zone 3 earthquake forces.

CONCLUSION

This paper presents an approach to the design of earthquake resistant reinforced concrete structures which uses earthquake accelerograms as loading and dynamic inelastic response history analysis to determine member forces and deformations.

An important feature of the design procedure is that it establishes a predetermined sequence of energy dissipating mechanisms by reducing the strength of selected groups of members, and thereby imposes on the structure a desired response. The structure is detailed for ductility only in predetermined hinging regions.

Reducing the main beam and coupling beam strengths below the levels indicated by an elastic analysis, while making sure that they can accommodate the increased ductility demands, results in the following advantages:

- Advances the onset of yielding during an earthquake, thus activating early the utilization of ductility of beams and their energy dissipation.
- Reduces moment input into columns from beams, thus protecting columns from yielding.
- Reduces shear in beams, improving materially their ductility.
- Decreases the congestion of reinforcement at the joints, as well as the shear in the joints.
- Reduces the seismic axial forces (tension and compression) in the columns and walls.

REFERENCES

1. International Conference of Building Officials, "Uniform Building Code", 1979 Edition, Whittier, California.

2. Kanaan, A.E. and Powell, G.H., "A General Purpose Computer Program for Inelastic Dynamic Response of Plane Structures", Report No. EERC 73-22, University of California, Berkeley, August 1975.

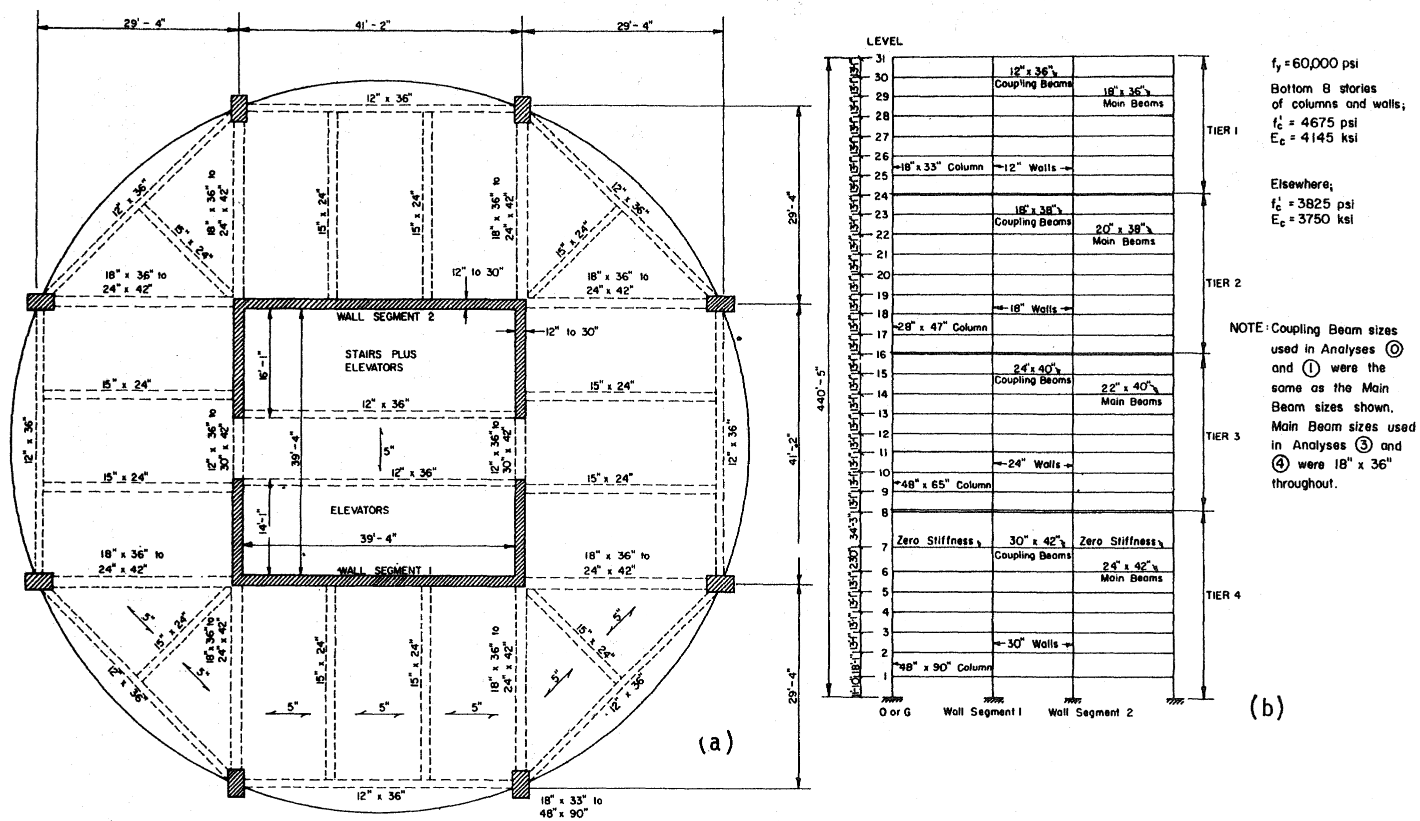

Fig. 1: (a) Floor Plan, (b) Section and Analytical Model of Building Studied

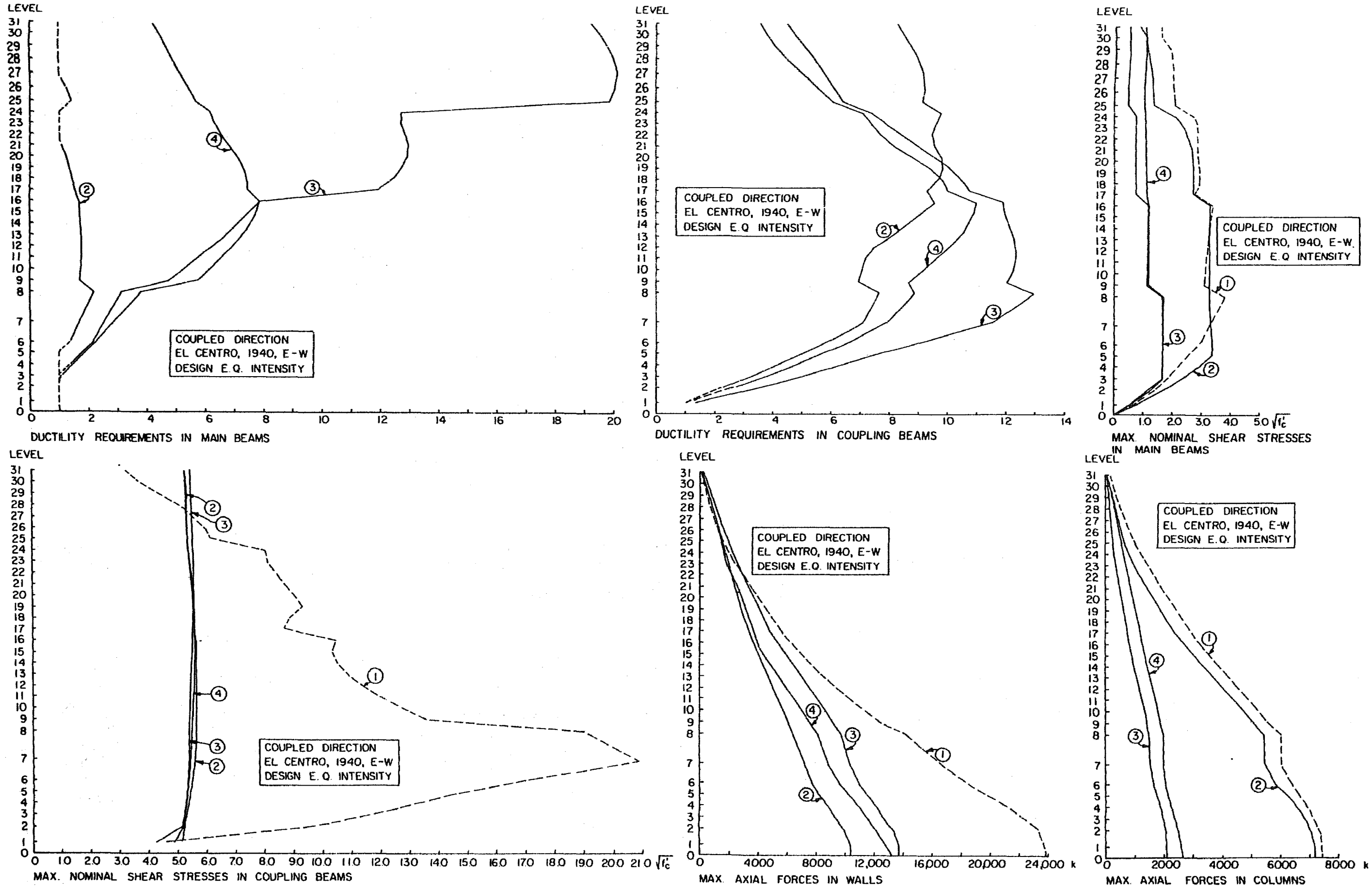

Fig. 2: Selected Maximum Response Quantities from Static and Dynamic Analyses

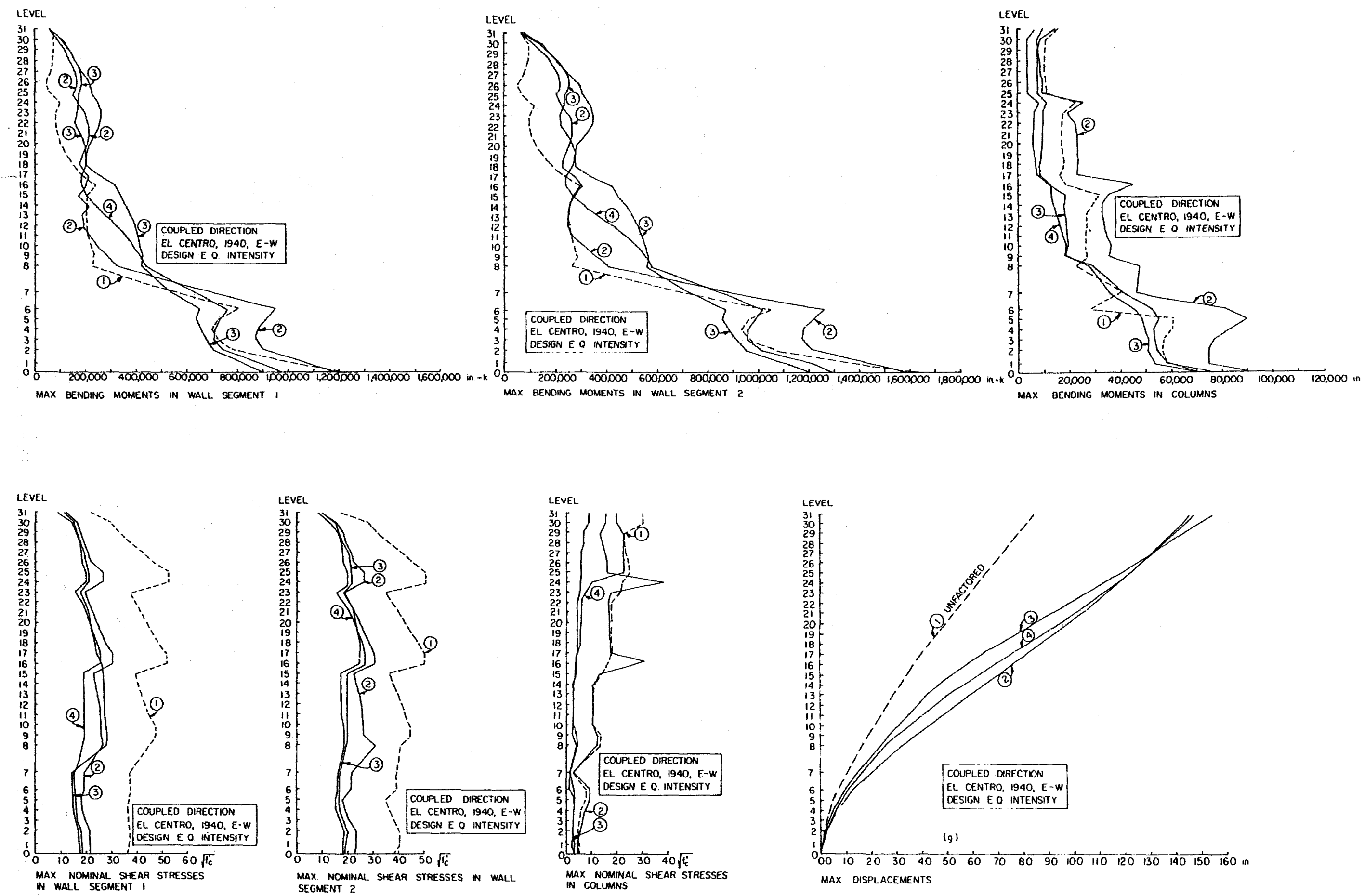

Fig. 2(cont): Selected Maximum Response Quantities from Static and Dynamic Analyses