DESIGN LESSONS LEARNED FROM THE PERFORMANCE OF INSTRUMENTED HIGH-RISE BUILDINGS IN THE SAN FERNANDO EARTHQUAKE

by

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SYNOPSIS

Eleven high-rise buildings, instrumented with strong-motion accelerographs at the time of the San Fernando Earthquake,were investigated under the co-sponsorship of the Earthquake Engineering Research Institute (EERI) and the National Oceanic and Atmospheric Administration (NOAA). Earthquake records from the buildings were used in conjunction with mathematical models of the structures to establish the dynamic force response. Results of the investigation are presented in the form of recommended earthquake design standards and building code revisions.

INTRODUCTION

At the time of the February 9, 1971 San Fernando Earthquake there were 66 high-rise buildings in metropolitan Los Angeles which were instrumented with strong-motion accelerographs. Fifty-seven of these buildings were within the limits of the City of Los Angeles and had been instrumented under the City Building Code requirements of 1965 which required three strong-motion accelerographs in every new major structure over six stories in height. The instruments were located in the basement, midheight and roof of the buildings. All of these buildings had been designed under modern earthquake resistant codes and constructed under optimum working conditions with continuous inspection. Thus they represented the latest in engineering and construction practice.

Eleven of the more significant buildings, which are listed in Table 1, were investigated by various engineering firms under the supervision of the EERI/NOAA San Fernando Earthquake Investigation Committee.(1) These studies were designed to establish the force and displacement response of the structures utilizing sophisticated computer analysis techniques. Earthquake ground motions recorded at the base of the buildings were used as the forcing functions to the mathematical models of the structures. Results of these analyses were compared with actual building behavior, as recorded in the upper stories of the structures, to establish credibility in the solutions. The analytical response was then compared to predicted performance based on modern design codes.

The investigations revealed areas of weakness in current earthquake design and re-emphasized many of the basic principles underlying

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earthquake engineering practice. This paper will present a composite of the key lessons learned from the investigation and point out code revisions recommended or adopted as a consequence of the study.

GENERAL EARTHQUAKE BEHAVIOR

None of the instrumented buildings collapsed during the earthquake, nor were any of them condemned and demolished. Structural damage in general was very light, as indicated by the repair costs listed in Table 1. Only three of the structures (the two identical Holidays Inns at 13 and 26 miles from the epicenter and the Bank of California at 17 miles from the epicenter) were forced beyond their elastic limits to the point that plastic or non-linear deformations developed in the structural frames at the beam-column connections. These three structures exhibited lengthening of recorded building periods during the earthquake, and visual inspection after the earthquake revealed some signs of cracking and spalling in the highly stressed, flexural zones of the beam elements.

Earthquake damage to non-structural elements, such as cracking of drywall partitions and dislocation of tile facings on walls and ceilings, ranged from light to heavy. For the Holiday Inn on Orion Avenue, 13 miles from the epicenter, earthquake repairs to non-structural elements amounted to 11% of the original construction cost or $145,000, while structural repairs were less than 0.2% of the original construction costs. In general, non-structural damage and repair costs far exceeded the structural damage and associated repair costs (see Table 1).

A rough comparison may be made between the peak story shear response of the structures computed from the recorded earthquake motions and the static shear forces used in the seismic code design [4] of the buildings. An average ratio of story shears computed from earthquake motions to the code design shears provides an index of the relative earthquake force levels developed in the various structures. This ratio serves to normalize the earthquake shear forces to a common base -- the static seismic design shear. It is only a rough indicator, because it neglects the various safety factors assigned to the different construction materials. In addition, it neglects the influence of non-linear structural response upon the force levels developed.

Keeping these limitations in mind, it is interesting to compare the average building shear ratios listed in Table 1 with the estimated costs for earthquake repairs. There appears to be some degree of correlation between total earthquake repair costs expressed as a percent of initial construction cost and the shear ratios. The highest repair costs are associated with the two Holiday Inns, which also have the largest shear ratios. The next largest repair costs are associated with the Bank of California, which has a comparably large shear ratio. All buildings, except the Union Bank Building, have shear ratios greater than one. This means that computed shear forces exceeded design building shears.

The 42-story Union Bank and the 32-story Bunker Hill Tower were designed to higher earthquake force levels than specified by minimum building code standards. These structures were mathematically subjected to the dynamic ground motions recorded at El Centro, California in 1940. All structural elements were designed to resist the computer calculated forces elastically. Thus it is not surprising to find that these two structures have low story shear ratios. In general, structures which have been dynamically analyzed and designed for the potential seismic exposure have a higher margin of earthquake safety than those buildings designed to minimum code standards. [4]

SPECIFIC LESSONS AND RECOMMENDATIONS

DESIGN FORCE LEVELS

The San Fernando Earthquake of February 9, 1971 was a moderate size event of Magnitude 6.6. [1] Past earthquake history for Southern California [5] indicates that an earthquake of this magnitude may be expected once every ten years. Few building owners can tolerate the economic losses suffered by Holiday Inns (i.e., 7 to 11% of the initial construction cost) on a recurring basis.

California earthquake codes in the past have been designed specifically to protect lives, rather than property. The Structural Engineers Association of California (SEAOC) have developed recommended standards for earthquake design (6) which have served as the pattern for seismic design requirements in most of the local, regional and national building codes in the United States. As stated in the SEAOC recommendations -- "The primary function of a building code is to provide minimum standards to assure public safety. Requirements contained in the code are intended to safeguard against major failure and loss of life... More specifically, with regard to earthquakes, structures designed in conformance with the provisions and principles set forth therein should be able to:

1. Resist minor earthquakes without damage.

2. Resist moderate earthquakes without structural damage, but with some non-structural damage.

3. Resist major earthquakes, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as non-structural damage."

From this description of building performance under moderate earthquake conditions, three of the instrumented buildings failed to meet the standard. In two cases, non-structural damage was felt to be disproportionally high.

As a consequence, the Instrumented Buildings Subcommittee recommended that higher seismic design forces be used and that a more rational approach be followed in establishing the magnitude and distribution of the design forces. Specifically, dynamic analysis techniques similar to those used in the post-earthquake investigation of the structures were recommended for the actual building design process. These specific recommendations were developed on the basis of a series of factors uncovered by the Subcommittee's investigation, and not simply the degree of earthquake damage.

DUCTILITY

Current building code practice recognizes the inherent seismic safety of structures which behave in a ductile manner, rather than a brittle manner. Lateral earthquake forces used in the design of frame structures are reduced if the moment-resisting space frame is ductile (i.e., K=0.67 instead of K=1.0 or 1.33).[6] Ductility is the ability of the structure to undergo large inelastic deformations, under cyclic loads well in excess of yield stress levels, without significant loss in load-carrying ability. By building code standards ductility is designed into a moment-resisting space frame if it is "constructed from structural steel (A-7, A-36 or A-441) or from reinforced concrete conforming to special code requirements" of strength, confinement, reinforcing percentages, and cross sectional proportions (see Section 2630 of the SEAOC Code).[6]

Prior to the San Fernando Earthquake, reinforced concrete moment-resisting space frames under 160 feet in height were not required to be designed to ductile standards. The Sheraton Universal Hotel (Table 1) was the only concrete frame among those investigated that was designed to ductile standards. There were five steel frames which also met the code standards for ductility. During the earthquake none of these structures experienced forces large enough to produce inelastic behavior. However, three of the reinforced concrete moment-resisting space frames, constructed without the special ductile design features,experienced large enough forces to perform inelastically. In two of these structures -- the Holiday Inns -- the moment-resisting frames exhibited a degree of ductility with little sign of spalling or reduction in moment capacity at the beam-column joints. However, in the case of the 12-story Bank of California, significant spalling occurred at the spandrel-to-column joints in the exterior moment-resisting frames. The spalling started at the fourth floor level and extended upward. The large portions of concrete which spalled from the lightly reinforced spandrel-to-column joints(see Figure 2) produced a significant reduction in moment-resisting capacity, as well as vertical load-carrying capacity.

The almost brittle behavior of the spandrel-girder-to-column connection above the fourth floor is attributed to a combination of several factors. To facilitate construction, the connection detail was redesigned as shown in Figure 2. Starting with the fourth floor, a construction joint was introduced in the connection between the lightweight concrete floor and spandrel system and the normal weight concrete column. Confinement of the lightweight concrete in the spandrel at the connection was only partially provided by the column ties. As seen from Figure 2, this was the area of major spalling.

The Subcommittee was particularly concerned about the loss of moment-resisting capacity and reduction in lateral load resistance associated with the brittle joint spalling. To protect such structures against possible collapse under earthquake forces in excess of standard design, the Subcommittee recommended that all reinforced concrete moment-resisting space frames designed with a K=0.67 should be designed with ductile characteristics, regardless of height.

The City of Los Angeles in their Interim Seismic Design Code (9) and the SEAOC in their Appendix F (6) have adopted the following standard -- "All concrete space frames required by design to be part of the lateral force-resisting system shall be ductile moment-resisting space frames."

NON-SEISMIC FRAMES

In the Bank of California Building, some of the interior frames were designed to carry only vertical floor loads. These non-seismic frames had no added moment reinforcement at the girder-to-column joints to provide resistance under lateral earthquake loads. The non-seismic frames in most cases were as stiff as the seismically reinforced exterior spandrel frames. Consequently, during the earthquake, these frames carried a significant portion of the total lateral building load until moment resistance in the girder-column joints of the frames -- provided there for vertical load-carrying purposes --was exceeded and the joints cracked. Post-earthquake review of the interior, non-seismic frames revealed serious cracking in the main girders around the supporting columns. Epoxy pressure grouting was used to repair this damage.

To minimize structural damage to non-seismic frames and protect the safety of inhabitants against potential collapse of the vertical load-carrying system during a major earthquake, the Subcommittee recommended that all non-seismic frames should be designed to resist seismic forces in proportion to the relative lateral frame stiffness or rigidity. This procedure should be followed even if other frames or lateral force-resisting elements, such as shear walls or diagonal bracing, were designed to carry 100% of the lateral seismic load.

The City of Los Angeles (9) and the SEAOC (6) in their codes have adopted this recommendation and have gone one step further. The code reads: "All framing elements (beams, columns, etc.) not required by design to be part of the lateral force-resisting system shall be investigated for adequacy for vertical load and induced moment due to four times the distortions resulting from the code required lateral forces." The factor of 4 is intended to account for seismic overloads producing large inelastic deflections of the ductile seismic frames. Under these conditions, the non-seismic frames should not lose their vertical load-carrying capacity.

CORNER COLUMNS

Standard seismic design practice by current building codes (4)(7) considers lateral design forces acting in one principal building direction at a time. Simultaneous earthquake loading from the two principal building directions is neglected, even in the design of corner columns or shear wall flanges at the exterior corner of a building.

Significant damage was observed in the corner columns of several non-instrumented buildings after the earthquake. Accelerograms indicated that peak lateral seismic forces occurred simultaneously in the two principal building directions several times during the earthquake. Under these conditions of simultaneous peak earthquake response, it was calculated that corner columns were the most highly stressed elements in the majority of cases. It was recommended by the Subcommittee that a more realistic approach be followed in evaluating the total earthquake loading on buildings and columns in particular. This recommendation included not only simultaneous lateral earthquake loads, but vertical acceleration effects as well.

Following these recommendations, the City of Los Angeles proposed in their Building Code Amendments (8) the following design standard. "Vertical load-carrying capacity of a structure shall be investigated for the effects of earthquake action in two directions which are perpendicular. The forces imposed shall be not less than 100% of the earthquake effect from one direction, combined with 30% from the other." Furthermore, this proposed building code included a procedural format for dynamic seismic design of the structure under maximum probable earthquake conditions.

HIGHER MODE RESPONSE

A significant result of the post-earthquake analysis was the discovery that a majority of the medium high-rise buildings (12-20 stories) experienced larger force response in the second and third modes of vibration rather than in the first mode, as normally implied in code design. An example is shown in Figure 3 where the peak story shears for the first three modes have been superimposed. In this case, the peak second mode shears exceed the first mode values in the top five stories. Even the envelope of maximum modal shears, with all 8 modes of vibration included, has a distinct second mode contribution in the top six stories, as indicated by the bulge in the envelope plot.

In several of the steel-frame structures, yielding of girders was calculated to have occurred only in the upper stories. This was attributed to the high second and third mode response.

Design overstress in the upper story girders points out a weakness in current code seismic design. Higher mode whip is considered significant only in the design of tall, slender high-rise buildings with height-to-base width ratios in excess of 3. In these structures a concentrated lateral force is added to the normal triangular force distribution, at the top level.

The Subcommittee recommended that current code practice be modified to incorporate higher mode force design in medium-rise structures, as well as high-rise. No code revisions have been made along these lines. However, where dynamic analyses are included in the design process and earthquake motions from nearby faults are considered in the analysis, the intent of this recommendation is generally fulfilled.

OVERTURNING MOMENTS

Seismic design codes have allowed a reduction factor ("J" factor) for design overturning moments. This reduction is based upon the theory that the higher modes of vibration will combine to cancel out part of the fundamental mode effect.[6]

However, the computed response from the 11 instrumented high-rise buildings did not show a cancellation of overturning moments due to higher mode response. Figure 4 presents a plot of peak overturning moments in each of the first three modes, along with the envelope of maximum overturning from all 8 modes. In the upper half of the structure higher modes contribute significantly to the net overturning moment response, while in the lower stories there appears to be little increase or decrease in first-mode response due to higher mode consideration. Similar results were obtained for the other buildings. Overturning moments computed by code procedures with and without reduction (i.e., J=0.58 and J=1.0) are superimposed upon the figure for comparison.

In 1970, shortly before the earthquake, SEAOC recommended that the "J" factor be omitted from the seismic design of new structures until further study could justify its applicability to normal building construction (see Section 2312(h) of Reference 6). The results from the instrumented building investigation provide justification for permanently eliminating the overturning moment reduction factor from the code.

SEISMIC SEPARATIONS

During the February 9, 1971 earthquake, five of the eleven instrumented buildings investigated impacted with adjacent low-rise (i.e., one to four story) peripheral structures. In four cases the structures were seismically separated by a space of two to three inches, as required by minimum code standards. Evidence of impacting was apparent in buckled roof flashing, crushed concrete fire cover, spalled wall tile and sharp discontinuities in the integrated acceleration traces recorded by the strong-motion instruments in the buildings. In all cases the lateral force-resisting system consisted of frames. Three were of steel construction; the other two of concrete.

The Subcommittee recommended that seismic separations be increased. The separation should be based on calculated lateral deflections under maximum possible earthquake forces, rather than upon current code seismic forces. This in essence would more than double the seismic separations currently specified. The City of Los Angeles has doubled the required seismic separation in the Interim Seismic Design Code.[9]

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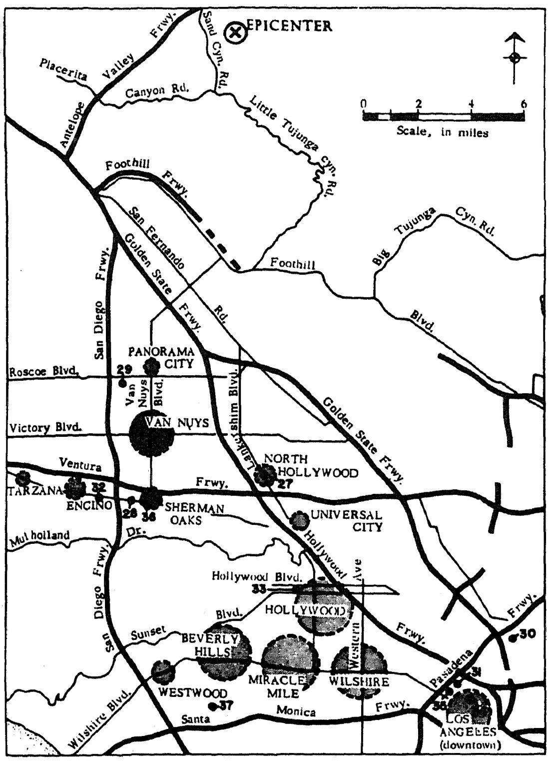

FIGURE 1 LOCATION OF INSTRUMENTED HIGH-RISE BUILDINGS RELATIVE TO EPICENTER OF SAN FERNANDO EARTHQUAKE(3)

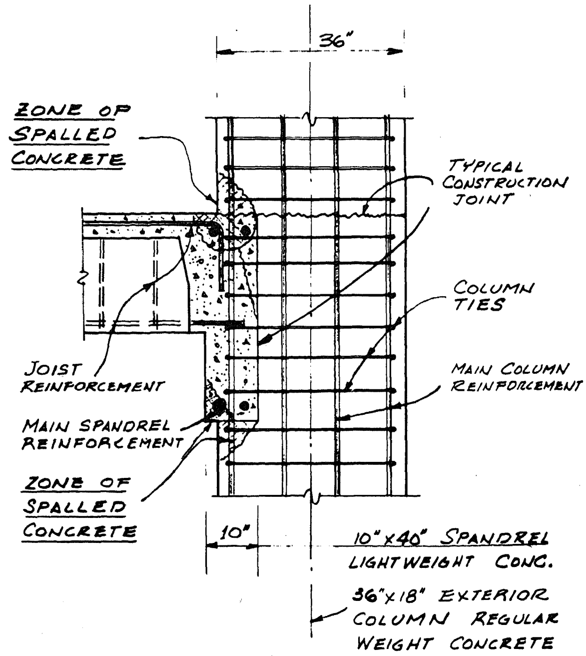

FIGURE 2 TYPICAL CONSTRUCTION DETAIL OF EXTERIOR COLUMN-SPANDREL-JOIST CONNECTION, FROM 4TH FLOOR TO ROOF BANK OF CALIFORNIA (1)

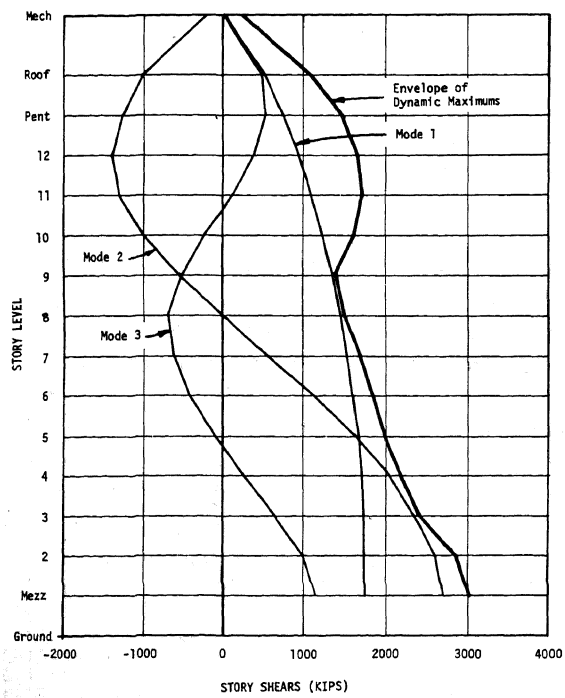

FIGURE 3 MAXIMUM STORY SHEARS NORTH-SOUTH DIRECTION CERTIFIED LIFE BUILDING (1)

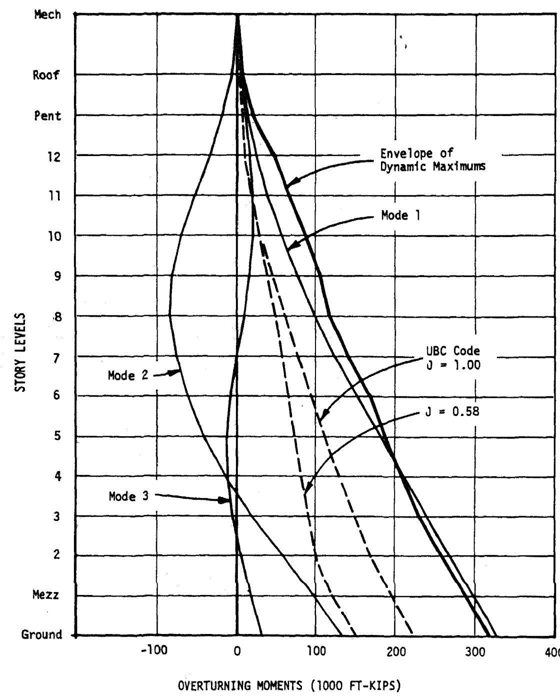

FIGURE 4 MAXIMUM OVERTURNING MOMENTS NORTH-SOUTH DIRECTION CERTIFIED LIFE BUILDING

TABLE 1

PERTINENT INFORMATION ON INSTRUMENTED BUILDINGS INVESTIGATED FOR EERI/NOAA

BLDG. NO.	BUILDING NAME AND ADDRESS	STRUCTURAL CHARACTERISTICS: NO. OF STORIES	CONSTRUCTION MATERIALS	LATERAL FORCE RESISTING SYSTEM	CONSTRUCTION COSTS: INITIAL CONSTRUCTION	EARTHQUAKE REPAIRS: TOTAL	EARTHQUAKE REPAIRS: STRUCTURAL	LATERAL SHEAR RATIO EARTHQUAKE/CODE DESIGN (I): E-W	N-S
27	Sheraton Universal Hotel 3838 Lankershim Blvd. Los Angeles	20	Reinforced Concrete	Ductile Moment Resisting Frame	$ 7,500,000	$ 2,100 (0.03%) (II)	None	1.5	1.0
28	Bank of California 15240 Ventura Blvd. Los Angeles	12	Reinforced Concrete	Moment Resisting Frame	4,000,000	44,000 (1.1%)	$12,000	2.7	3.0
29	Holiday Inn 8244 Orion Avenue Los Angeles	7	Reinforced Concrete	Flat Slab and Perimeter Frame	1,300,000	145,000 (11.1%)	2,000	4.5 (III)	5.0 (III)
30	Holiday Inn 1640 Marengo Street Los Angeles	7	Reinforced Concrete	Flat Slab and Pe imeter Frame	1,300,000	95,000 (7.3%)	2,500	5.0 (III)	4.5 (III)
31	Bunker Hill Towers 800 West First Street Los Angeles	32	Structural Steel	Ductile Frame (Tube System)	7,000,000	Nominal ---	None	2.8 (VI) (1.0) (V)	2.6 (VI) (1.2) (V)
32	KB Valley Center 15910 Ventura Blvd. Los Angeles	17	Structural Steel	Ductile Moment Resisting Frame	4,000,000	3,000 (0.08%)	None	2.2	1.8
33	Muir Medical Center 7080 Hollywood Blvd. Los Angeles	12	Reinforced Concrete	Flat Slab and Perimeter Frame	4,500,000	2,000 (0.04%)	None	2.6	1.6
34	Kajima International 250 East First Street Los Angeles	15	Structural Steel	Ductile Moment Resisting Frame	3,000,000	1,000 (0.03%)	None	2.2	2.2
35	Union Bank 445 So. Figueroa St. Los Angeles	42	Structural Steel	Ductile Frame (Tube System)	30,000,000	50,000 (VI) (0.16%)	None	0.95 (IV)	0.90 (IV)
36	Certified Life 14724 Ventura Blvd. Los Angeles	14	Reinforced Concrete	Shear Wall	3,000,000 (VII)	2,000 (0.07%)	None	1.2	1.4
37	1901 Avenue of The Stars Building Century City Los Angeles	19	Structural Steel	NS-Ductile Frame EW-"X" Braced Frame	Not Reported	14,000 ---	None	Not	Reported

IV Structure was designed to behave elastically under earthquake forces from an El Centro 1940, N-S ground motion. Dynamic analysis of the structure was performed during the design phase to assure adequate strength and proper behavior.

V Ratios in parentheses indicate actual earthquake shears to dynamic design shears under El Centro motions.

VI Rough estimate of damage.

VII Cost of bare architectural shell without tenant improvements.

I Ratio of calculated earthquake shears to equivalent static shears used in the seismic design of the structure. This ratio is only a relative indicator of the force levels producing damage.

II Total Repair Costs as percent of Initial Construction Cost.

III Due to non-linear behavior of the structure, this large a force ratio might not have actually been developed in the structure. Linear elastic analytical methods were used in the dynamic studies.