SEISMIC REHABILITATION OF A TEN-STORY CONCRETE FRAME BUILDING USING VISCO-ELASTIC DAMPING ELEMENTS

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Paper No. 511. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

ABSTRACT

This paper discusses the conceptual seismic rehabilitation design of an early 1970s-vintage office building located in Southern California. The building is a ten-story non-ductile concrete moment frame structure located above four levels of subterranean parking, originally designed in 1970, with a toal area of approximately 162,000 square feet. The design of the seismic rehabilitation of this building is of particular interest as it uses the concept of adding visco-elastic damping devices to reduce the amplitudes of building seismic motions.

The design of conceptual seismic rehabilitation measures for this building has shown that non-ductile concrete frame buildings can be effectively upgraded to substantially reduce earthquake risk by utilizing innovative seismic retrofit systems of supplementary bracing and damping elements. Both the cost and the predicted seismic performance of this approach compares very favorably with conventional approaches.

KEY WORDS

Seismic retrofit, visco-elastic damper, concrete frame structure

BUILDING DESCRIPTION

The subject facility is a ten-story office building above grade constructed over a four-level subterranean parking structure. The overall building dimensions are approximately 85 feet by 140 feet, yielding approximately 119,000 square feet of floor area. The building was designed in July of 1971 and constructed was completed shortly thereafter.

The building vertical load-resisting system above grade consists of cast in place one-way reinforced concrete flat slabs spanning over reinforced concrete beam and column framing. The floors below grade are supported by circular concrete columns and perimeter concrete walls. The building foundation system consists of isolated reinforced concrete spread footings supporting columns and similar continous strip footings supporting the perimeter walls.

The building lateral force-resisting system is categorized as an "Ordinary Concrete Moment Resisting Frame System" (in building code terms) due to the nature of the concrete frames which provide support for gravity and lateral loads. The rigid floor and roof diaphragms transfer lateral forces into the concrete frames. At the first floor (plaza level), lateral forces from the frame are transferred through the plaza diaphragm, into the surrounding shear walls/soil bearing system of the subterranean parking garage. Overturning forces from the building frames are resisted by the columns and foundation system of the garage.

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ANALYSIS PROCEDURE

Computer models were developed for the subject building to determine the performance of the existing structure and the same structure implementing two conceptual strengthening schemes, a conventional concrete shear wall installation and an added damping system using visco-elastic damper bracing elements.

Seismic Demand/Capacity Analyses

Seismic forces ("Demands") on the structure were determined using the response spectra provided in the 1991 UBC. The modification factor Z (0.4 for Seismic Zone 4) was applied to the S2 and S3 (standard soil profiles) response spectra to include the effects of regional seismicity. No Rw reduction factor (for structural system type and ductility) was applied so as to simulate unreduced spectra for the site, or "actual" anticipated ground shaking.

The "Capacity" of building structural elements, e.g. columns and girders, is based on nominal ultimate values as prescribed in the UBC. Thus, the calculation of Demand-to-Capacity, D/C, represents on an elemental basis the total imposed elastic load or stress divided by the nominal ultimate capacity for the member or material type as recognized by the code. This calculation only has meaning in the context of comparison with Inelastic Demand Ratios (IDRs), as described below. Once the structure begins to behave inelastically (i.e., concrete cracks and reinforcing steel yields), the demand changes due to shifts in structural period, increased damping, load redistribution, and energy absorption due to damage.

In the evaluation of the exisitng structure, the D/C values were compared with IDRs for the purposes of assessing the required degree of inelastic response to an acceptable value of such response given the material (e.g. concrete or steel), type of member (e.g. beam or column), and type of stress condition (e.g. shear or flexure). This simplified approach is widely accepted[1].

Computer Model

ETABS computer models were developed for the subject building for each of the following three conditions; a) the existing building, b) the strengthened building incorporating the conventional shear wall strengthening concept (Concept 1 below), and c) the strengthened building incorporating the damper bracing concept (Concept 2 below).

The building was modeled as 11 stories (including the mezzanine) with 5 bays of frames in the longitudinal direction and 3 bays of frames in the transverse direction. These bays were framed with a total of 24 column lines. The fixed base of the model was selected as the plaza level as very little movement at this level is expected relative to the surrounding "free field" of seismic ground motions.

The building was modeled assuming a uniform dead load of 125 psf, and code level live load of 50 psf. To account for torsional effects at each level, the mass of the building was displaced 5% of the building dimension perpendicular to the applied earthquake direction. To bound the problem, beams and columns were modeled using both cracked and uncracked section properties. However, the difference in dynamic response between cracked and uncracked sections had little impact on the analysis of seismic demand (and thus D/C results) as the code response spectra remains essentially unchanged over the affected structure periods. Nevertheless, worst case load and displacement values were evaluated for each case.

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[1] Army TM 5-809-10-1 Technical Manual, "Seismic Design Guidelines for Essential Buildings" - also referred to as the Tri-Services manual and NAVFAC P-355.1/.2.

Analysis of Exisitng Structure

Salient results from the ETABS analysis for the existing (unstrengthened) building are given below.

- Primary Structural Periods (among 33 total modes):

1st mode (longitudinal)	2.22 seconds
2nd mode (transverse)	2.11 seconds
3rd mode (torsional)	1.71 seconds

- Effective Base Shear:

Longitudinal direction	6900 kips
Transverse direction	6300 kips

- Top Story Deflections:

Longitudinal direction	25 inches
Transverse direction	22 inches

Results of the D/C analyses for major elements of the building's lateral load carrying system (primarily columns and girders) indicated D/C ratios greatly exceeded guideline limitations for acceptable performance for many critical structural members. In general, where the D/C values greatly exceeded the guideline limitations on IDR, it was concluded that severe structural damage and loss of strength was possible for the MCE event used in the analysis. Results of the investigation indicate that the building could sustain substantial structural damage and long term closure for repairs as a result of a major earthquake. The most significant areas of concern are the non-ductile girders and columns that comprise the building's lateral load carrying frames, where again, D/C ratios greatly exceed guideline limitations for acceptable performance for many critical structural members.

It should be noted that the excessive D/C values were more significant for the beam elements rather than the columns. This is generally indicative of a strong column-weak beam system which is highly desirable and required by current codes for ductile moment frame systems (note, however, that this framing system does not literally meet the full extent of current code requirements for column-beam strength relationships). This characteristic helps assure that as the building undergoes inelastic response and becomes damaged, the damage and loss of strength will first occur in the beams. As such, much energy absorption and favorable shifts in dynamic response of the structure will occur while the building is still stable (because the columns, which support the building, are experiencing less damage and loss of strength).

Though the D/C values for the columns exceed guideline limitations, the aforementioned general strong column-weak beam conditions leads us to conclude that possibility of collapse of the structure in strong ground shaking appears remote.

SEISMIC RETROFIT CONCEPTS

For areas found to contain deficiencies based on the approach and methodology described above, two conceptual strengthening methods for mitigating or substantially reducing the deficiencies were developed. The intent of the strengthening concepts was to provide the structural capacity judged necessary to conform reasonably close to current code requirements for safety and performance of ordinary structures (e.g. offices), and thereby substantially reduce the concern for severe damage, collapse, and long term closure following a major earthquake.

Retrofit Concept No. 1 - New Concrete Shearwalls

Concept No. 1 consisted of adding shearwalls to the exterior of the building at each side from the base of the structure to the roof. This concept also entails strengthening the columns and foundations in the garage. Concept 1 was developed primarily for the purpose of cost comparison with the visco-elastic damper concept below. Refer to Figures 1 and 2 for general sketches of major elements of retrofit Concept No. 1. In general, the proposed retrofits may be described as follows:

1. New exterior shearwalls in selected bays on all four sides of the building. Wall thicknesses vary from 10 inches to 18 inches.

2. Strengthening of the subterranean parking garage columns by jacketing them with a thick layer (on the order of 18 inches) of new concrete and reinforcing steel.

3. Increase the thickness of a major load bearing interior 12 inch thick concrete wall in the parking area to 18 inches thick.

4. Increase the parking garage columns support footing sizes.

5. Provide additional diaphragm boundary elements at the plaza level.

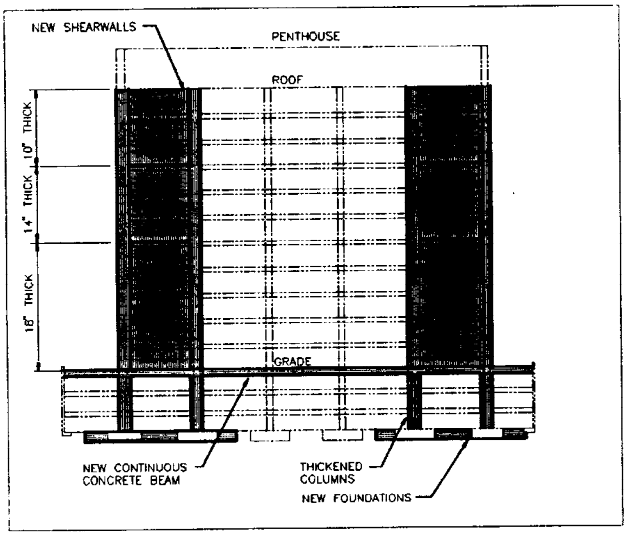

Figure 1

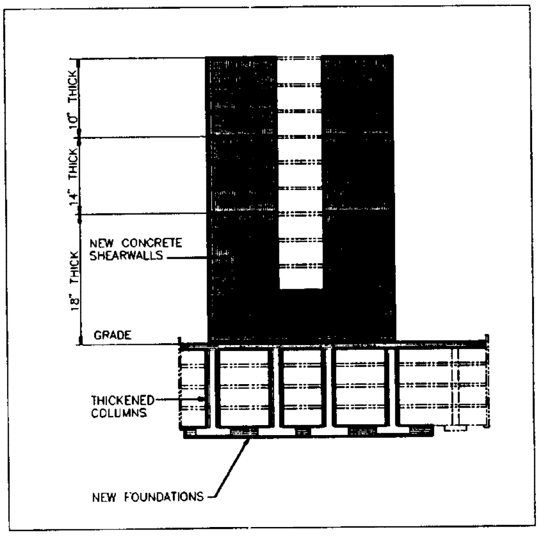

Figure 2

Retrofit Concept No. 2 - Steel Bracing and Dampers

Concept No. 2 consisted of adding steel diagonal bracing in conjunction with viscoelastic dampers from base of the structure to the roof. Some strengthening of columns and foundations at the garage levels may also be required for this concept, but such strengthening should not be as extensive as will be required for the shear walls of Concept No. 1.

Passive viscoelastic dampers have been used for many years for reducing wind displacements in tall buildings, but are a relatively new technology for seismic damage control. The dampers consist of a viscoelastic material bonded to steel plates which are placed within a steel bracing system (e.g. '/' or 'X' bracing). An example of a viscoelastic damper is illustrated in Figure 3.

Damping is achieved by imposing building displacements on the damper material via the braces. Substantial structural system damping can be achieved using this system; for example, a concrete frame might exhibit 5%-10% damping during strong shaking. With the addition of viscoelastic dampers, the structural system can achieve on the order of 20% damping.

Viscoelastic dampers will absorb and dissipate seismic vibrational energy through deformation of the viscoelastic material. Heat will be generated in the viscoelastic material and dissipated through the steel members of the bracing system. These dampers are passive devices and do not require external power for operation. Their function is similar to that of an automobile shock absorber. Viscoelastic dampers have the ability to significantly reduce lateral drifts and absorb the shock that is normally carried by the building framing members alone. The addition of dampers into an existing system will assist the system in maintaining stability and integrity where instability and collapse may otherwise occur, e.g., a nonductile concrete frame.

Because the extent and layout of dampers proposed for the subject building substantially reduce the seismic forces in the structure, strengthening of the subterranean garage will be minimized. To maximize damping efficiency in both torsional and translational modes, dampers were located in perimeter bays as far apart as possible. Other bracing locations may prove to be feasible upon further and more in-depth analysis.

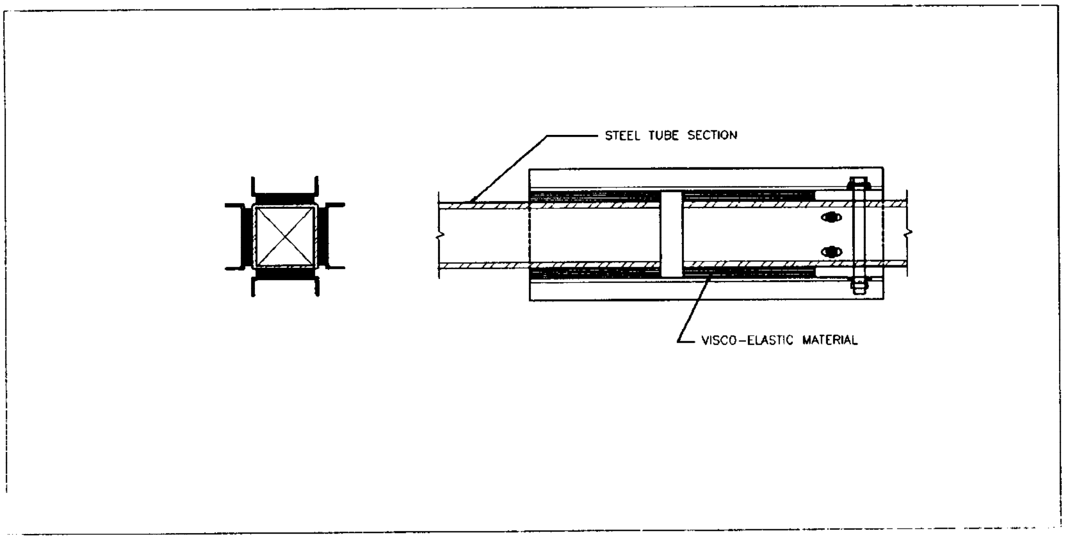

Figure 3

Refer to Figures 4 and 5 for general sketches of major elements of retrofit Concept No. 2. In general, the proposed retrofits may be described as follows:

1. The damper braces would be added to four bays in each direction (two each side) for each floor level above grade. These braces could be installed on the inside of the building within the shadow of the exterior beams and columns.

2. To account for the two-story exterior columns at the ground level, an additional horizontal girder would be installed in each braced bay.

3. The end of each damper brace would be connected at beam-column joints in the existing concrete frame. A steel jacketing attachment with through-bolts at the beam-column joint would be required so that the damper brace could adequately transfer seismic forces to the concrete frame.

4. On the inside of the building, the damper brace could be enclosed by drywall or sheet metal for appearance purposes.

5. Below the ground floor (plaza level), continuous beams may need to be added to transfer loads from the bracing system to the plaza level diaphragm and surrounding parking garage shear walls.

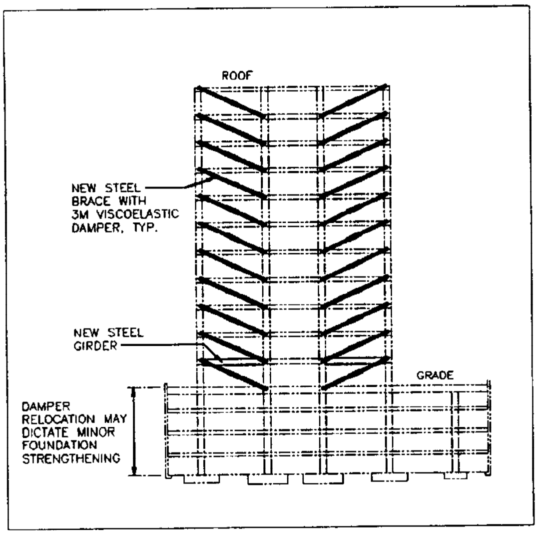

Figure 4

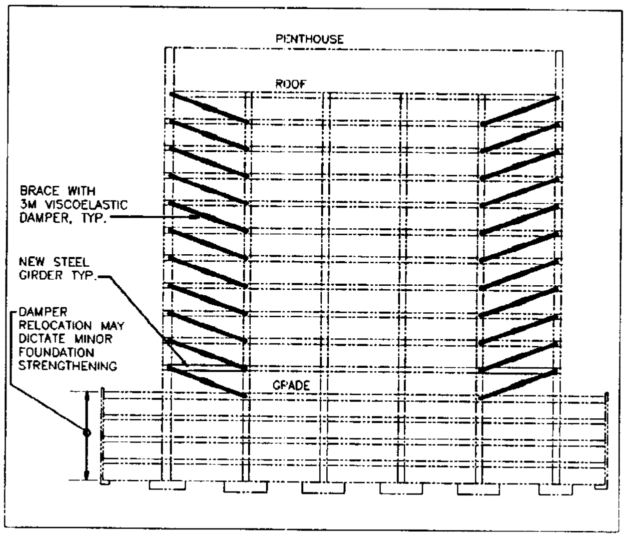

Figure 5

Our preliminary analysis indicates that with the implementation of the above damper braces, most existing structural elements in the concrete frame will no longer be severely overstressed for the load levels investigated. Although the probable D/C ratios of selected members may exceed unity by up to 25%, the expected response would remain largely elastic, significantly reducing building damage levels. Thus, such a response irepresents a great advantage over that of the building strengthened using the shear wall concept and that required by the ordinary occupancy provisions of the Uniform Building Code.

<u>Visco-Elastic Damper Analysis Methods</u>

Visco-elastic dampingr can be characterized by a complex number consisting of loss (k'') and storage (k') stiffness as

$$k = k' + jk'' \qquad (1)$$

where j equals $\sqrt{-1}$. The ratio of loss to storage stiffness is defined as the damper loss factor, $n = k''/k'$. The size of the viscoelastic material slab in the damper can be determined from the damper storage stiffness as

$$k' = \frac{G'A}{h} \qquad (2)$$

where G' is the shear storage modulus, A is the shearing area and h is the thickness of the viscoelastic material.

It has been shown that the brace connecting the damper to the structural members will affect the damper efficiency. For instance, if the ratio of the brace stiffness to the damper storage stiffness is 6 and the damper loss factor is 1.3, the effective loss factor of the damper-brace system will become 0.9, while the effective damper-brace storage stiffness remains about the same as the damper storage stiffness. To what follows, we shall always use the effective damper-brace loss factor and stiffness unless specifically indicated.

The storage stiffness is like the spring that adds stiffness to the structure and the loss stiffness will provide the energy dissipation capacity. The loss stiffness may be related to damping constant, *c*, as

$$c = \frac{k''}{\omega_n} \tag{3}$$

where ω_n is the natural frequency of the structure.

The added damping ζ to a building provided by visoelastic dampers can be approximately calculated as

$$\zeta = \frac{\eta}{2}\left(\frac{k'}{k'+k_s}\right) \tag{4}$$

where k_s is the story stiffness. For instance, if we want to design for $\zeta = 15\%$ with $\eta = 0.9$, then from Eq. 4, the damper stiffness will be

$$k' = \frac{2\zeta}{\eta - 2\zeta} k_s = 0.5 k_s \tag{5}$$

The dampers will add 50% of stiffness to the building while providing 15% of the critical damping.

The above analysis methods would be significantly less valid if the building response became inelastic. More extensive non-linear time history analysis would be required to accurately analyze the inelastic case. However, in cases where the building response remains elastic, the above analysis methods serves as an effective preliminary evaluation tool. Developing codes and guidelines for added-damping systems suggest that for new design structures should remain primarily elastic. For building retrofit, some structural hinging may be permitted but analysis requirements will be extensive.

COST DEVELOPMENT CRITERIA

The opinions of probable cost for the proposed retrofit concepts were developed based on the preliminary strengthening concept sketches contained in this report. Some specific limitations on cost data presented herein are listed below.

- Work area clearing (e.g., relocation of personnel, furniture, or equipment) and business interruption expenses are not included.
- Costs include restoration of architectural finishes (i.e. floor, wall and ceiling treatments) only in the local area affected by the structural modification.
- Costs represent current retrofit experience, i.e. within the last few years. Higher costs may be incurred if the work is significantly delayed.
- All costs for anticipated retrofit work below the plaza level for both Concepts 1 and 2 are highly judgmental as no significant engineering analyses have been performed on the subterranean parking garage. Dynamic seismic analyses and retrofit concept development have been performed on the superstructure and form the basis for preparing an opinion of probable retrofit cost for this portion of the facility.

Our opinion of the probable engineering and construction cost to retrofit the building is roughly $6.6 million for Concept No. 1 and $5.5 million for Concept No. 2. Tables 1 and 2 present a rough breakdown of the components of these costs.

Table 1
Conventional Strengthening Concept Probable Cost Breakdown

Retrofit Measure	Description	Summary of Retrofit Costs % of Total	Total Costs ($)
1	New Shear Walls	39	2.0 million
2	Drag Strut @ 1st Level	8	400,000
3	Jacketing Garage Columns	8	400,000
4	New Footings	45	2.3 million
	SUBTOTAL	**100**	**5.1 million**
	Contingency (20%)		1 million
	E/A Fee (8%)		500,000
	TOTAL		**6.6 million**

Table 2
Viscoelastic Damper Concept Probable Cost Breakdown

Retrofit Measure	Description	Summary of Retrofit Costs % of Total	Total Costs ($)
1	Install 88 Visco-Elastic Dampers	43	1.8 million
2	Install Steel Girder at Plaza Level	5	200,000
3	Install Steel Braces	33	1.4 million
4	New Footings	19	800,000
	SUBTOTAL	**100**	**4.2 million**
	Contingency (20%)		850,000
	E/A Fee (8%)		350,000
	TOTAL		**5.5 million**