

# Seismic Rehabilitation of Nonductile Reinforced Concrete Gravity Frame

비연성 철근 콘크리트 중력 프레임에 의한 지진 보강

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## Abstract

This paper represents results of an effort to seismically rehabilitate a 12-story nonductile reinforced concrete frame building. The frame located in the most severe seismic area, zone 4, is assumed to be designed and detailed for gravity load requirements only. Both pushover and nonlinear time-history analyses are carried out to determine strength, deformation capacity and the vulnerability of the building. The analysis indicates a drift concentration at the 1<sup>st</sup> floor level due to inadequate strength and ductility capacity of the ground floor columns. The capacity curve of the structure, when superimposed on the average demand response spectrum for the ensemble of scaled earthquakes indicates that the structure is extremely weak and requires a major retrofit. The retrofit of the building is attempted using viscoelastic (VE) dampers. The dampers at each floor level are sized in order to reduce the elastic story drift ratios to within 1%. It is found that this requires substantially large dampers that are not practically feasible. With practical size dampers, the analyses of the viscoelastically damped building indicates that the damper sizes provided are not sufficient enough to remove the biased response and drift concentration of the building. The results indicate that VE-dampers alone are not sufficient to rehabilitate such a concrete frame. Concrete buildings, in general, being stiffer require larger dampers. The second rehabilitation strategy uses concrete shearwalls. Shearwalls increased stiffness and strength of the building, which resulted in reducing the drift significantly. The effectiveness of VE-dampers in conjunction with stiff shearwalls was also studied. Considering the economy and effectiveness, it is concluded that shearwalls were the most feasible solution for seismic rehabilitation of such buildings.

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## I. Introduction

The problem of nonductile structures became apparent in the aftermath of the 1971 San Fernando earthquake. Prior to that not much was known about the importance of proportioning and detailing in earthquake performance of structures. The early codes used simple formulations for lateral forces as a representation of the seismic effects. The force was specified as fraction of the dead weight of the building along with two requirements, a complete load path to transfer the lateral forces to the ground and the basic strength to enable the building to resist these forces. It was only in the late 1920s that seismic forces were explicitly recognized in the Uniform Building Code (UBC) [1].

By 1958, the seismic provisions were in the appendix of UBC and could be adopted by local jurisdictions at their option, not as a requirement. The first modern seismic code was derived from the recommendations of the SEAOC, as contained in the blue book of 1955. The lateral force was specified in terms of base shear, which was distributed over the height of the building. The ductile systems could be designed for a lower base shear and vice-versa. The building could be designed as an elastic system using allowable stresses, and it was assumed that the reserve strength of materials would ensure reasonable performance. It was not until the 1961 edition that UBC had an extended section on earthquake design and not until 1976 edition that it evolved into today's basic content.

The new form of ductile reinforced concrete frame evolved in the early 1960s. The widespread damage caused by the 1971 San Fernando earthquake emphasized the need for ductility in order to prevent catastrophic failures. The requirements of

seismic codes have changed in the aftermath of every major earthquake since.

The changing seismic requirement of the codes is the source of variability in strength and detailing of structures constructed at different times. As a result of variability in design and detailing, three types of structures currently exist which need attention: the pre-70 structures designed primarily for gravity loads, the pre-70 structures designed for inadequate strength and ductility, and the post-70 structures with inadequate strength and/or ductility. This paper considers the seismic evaluation and retrofit of a concrete moment frame designed for gravity load only (pre-70) located in a severe seismic zone 4 (Uniform Building Code 1994). This frame represents a lower bound on strength and ductility capacity requiring an upper-bound retrofit strategy.

## II. Seismic Analysis

### 1. The Building

Figure 1 shows a typical plan and elevation of a 12-story nonductile beam-column frame used for this investigation. The building is designed and detailed for gravity loads per Uniform Building Code using PCA-Build program [2]. All beams are sized at 20x24 in. (508x610 mm) and all columns at 24x24 in. (610x610 mm). The specified strength of concrete is 4 ksi (27.58 Mpa). The fundamental period of the elastic building in the longitudinal direction using gross-section properties was determined by analyses as 1.9 seconds. The detailing of columns and beams is shown in Fig.2.

### 2. Earthquake Ground Motions

The ground motion records used for the analyses

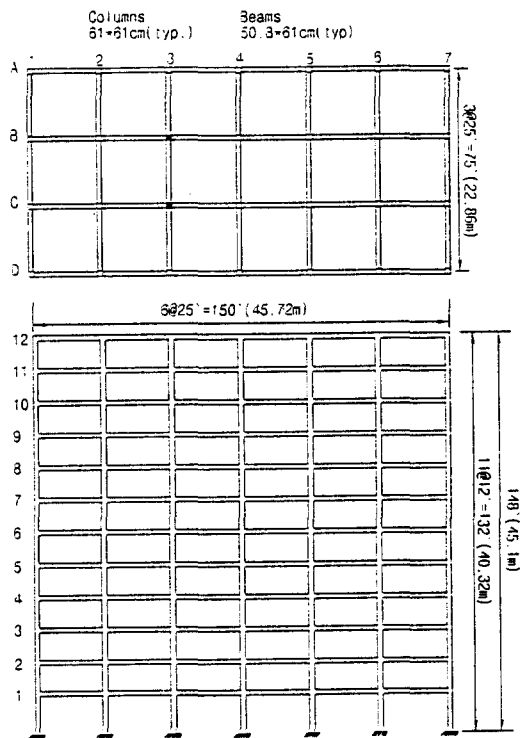


Fig. 1 Plan and elevation of example building

include the El Centro (1940) North South (NS) component, the East West (EW) component of the Tokachioki earthquake at Hanchinohe, Japan (1968) and the Northridge earthquake (1994) as recorded at Sylmar country (EW component) and Newhall station (NS components). The Northridge earthquake records were selected to study the near-field ground excitation effect [Naeim, 1995]. The 2 Northridge records selected were scaled up to a spectral intensity of 1.5 times that of the El Centro (1940) NS component and 1.5 times that of the Tokachioki earthquake.

### 3. Evaluation of Unretrofitted Building

The strength and curvature capacities of the beams and columns (Fig. 2) are determined using the BIAX program [3]. The effect of nonductile detailing such as inadequate anchorage, lap splicing and confinement are accounted for in the moment curvature relationship of the sections. Columns are

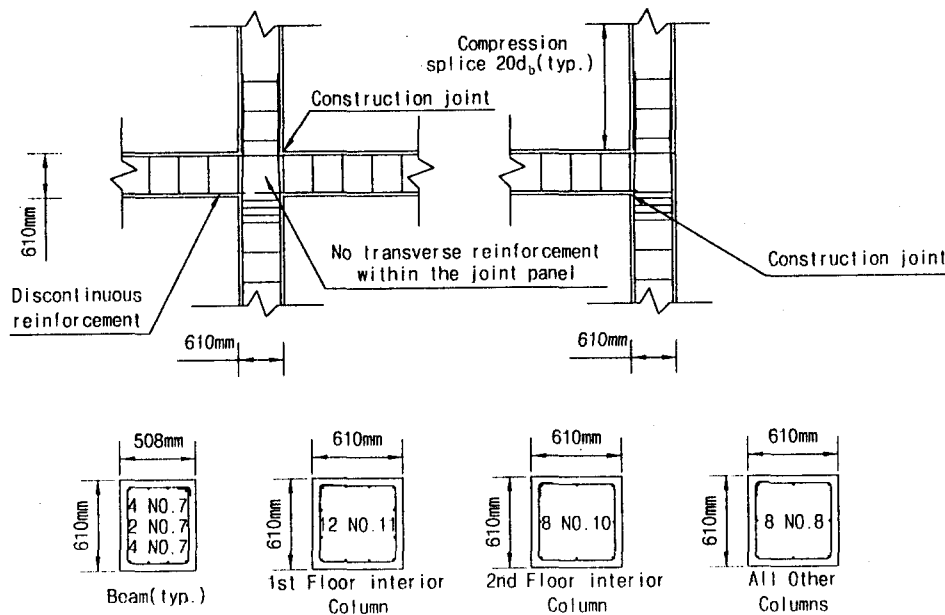


Fig. 2 Detailing of unretrofitted building

assumed to have a typical compression splice of 20 bar diameters near their ends (see Fig. 2). Considering the current code requirements [4], about 30% reduction in the flexural strength of the columns is envisaged as a result of inadequate compression splice. For beams, the effect of inadequate anchorage of the bottom reinforcement in the beam-column joint is taken into account [5] (see Fig. 2).

7. Pushover Analysis.

A monotonously increasing inverted triangular load is applied to the structure using the DRAIN-2DX program [6]. Cracked section properties based on one-half the gross section properties for the beams and the columns are used. The strain-hardening stiffness is assumed to be 5% of the elastic stiffness. The analysis shows a drift concentration at 1st floor level due to inadequate strength of columns at this floor.

4. Capacity vs. Demand

In order to determine the capacity curve, the base shear factors (V/W) and the displacements (Δ) from the pushover curve are transformed into the corresponding spectrum acceleration (Sa) and displacement (Sd), respectively, using the partici-

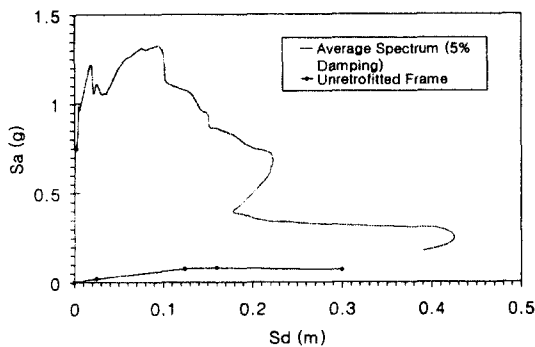


Fig. 3 Capacity and demand comparison of unretrofitted building

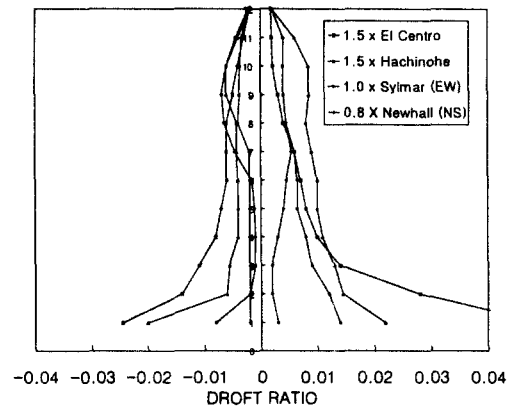


Fig. 4 Drift of elastic building

pation factors and the model analysis procedure, as per ATC-33[7], as shown in Fig. 3. Figure 3 indicates that the structure has substantially inadequate capacity to meet the earthquake demand and may fail catastrophically under an earthquake of this intensity.

4. Nonlinear Analysis

The potential hinges at beam-ends are idealized by using point fiber hinges of the DRAIN-2DX program. The gravity loads due to the tributary dead and live load are input for columns and walls at each floor level in order to simulate the axial load effect on flexural capacity. The analysis assumes rigid floor diaphragms with each node having three degrees of freedom.

The nonlinear analysis of the inelastic building was carried out for the four selected ground motions assuming 5% viscous damping. Figure 4 shows the magnitudes of the drift ratios at different floor levels of the building. The drift concentration at the 1<sup>st</sup> floor level reached as much as 5% due to building. This substantiates the earlier finding of the pushover analysis that the building is likely to experience a catastrophic failure under an earthquake of this intensity.

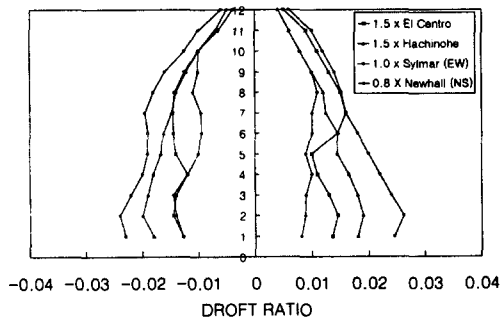


Fig. 5 Drift of elastic building

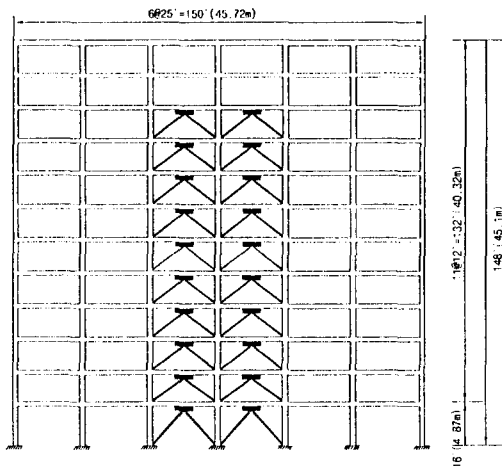


Fig. 6 Retrofit of building using VE-Dampers

4. Retrofit by VE-dampers

The retrofit of the building is based on the drift control criteria [8]. It is assumed that the dampers when installed will tend to promote elastic behavior of the building. Thus dampers were sized based on the average of the elastic drift magnitudes obtained for the four earthquakes, as shown in Fig. 5. Table 1 shows the average elastic drift ratios and the corresponding damping ratios and damper areas required at each floor level. The thickness of the damper was assumed to be 1/2 in. The damping

Table 1 Damper size distribution at different floor levels

Story	Average drift ratio(%)	Damping ratio(%)	Damper area/bay(in <sup>2</sup> )
1	1.8	15	600
2	1.9	15	600
3	1.8	15	600
4	1.6	15	350
5	1.5	10	350
6	1.5	10	350
7	1.5	10	350
8	1.4	6.5	200
9	1.3	6.5	200
10	1.1	-	100
11	0.8	-	-
12	0.5	-	-

was provided at floors where the drift ratios exceed 1%. A maximum damping ratio of 15% was provided at any floor level. This maximum limit of damping ratio was chosen keeping in view the practical size of the damper that can be incorporated. The magnitude of VE-damper stiffness and damping ratio required at each floor level was determined for the drift  $\delta = 1\%$  using a damper loss factor  $d = 1.2$  [7]. Figure 6 shows the damper configuration in the building.

가. Pushover Analysis

Pushover analysis of the retrofitted building was carried out. The VE-damper elements were idealized as elastic spring elements of equivalent

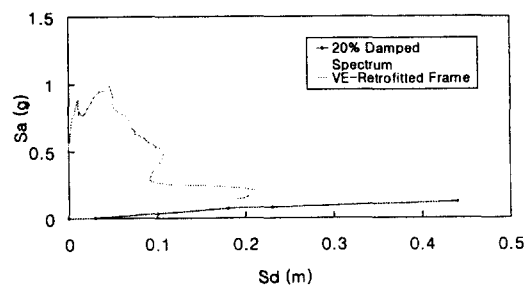


Fig. 7 Capacity and demand comparison of retrofitted building

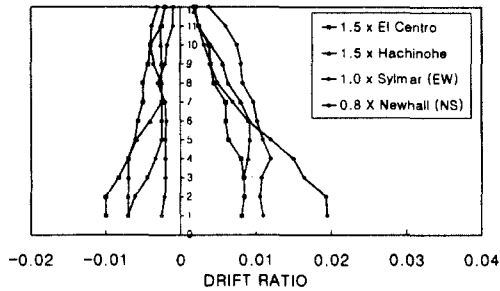


Fig. 8 Drift of VE-damped building

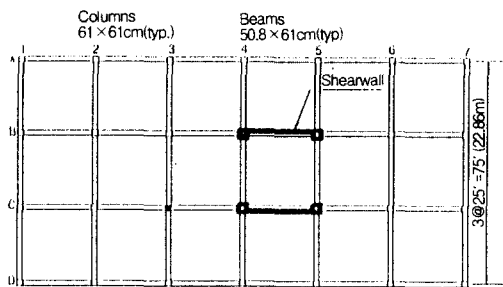


Fig. 9 Retrofit using rc shearwalls

stiffness. The damper stiffness was determined at a frequency equal to the fundamental vibration frequency of the structure. A total equivalent damping of 15% of the critical was anticipated with the above VE-damper configuration. A high damping demand spectrum was obtained from the 5% demand spectrum using different reduction factors for  $S_a$  and  $S_d$  [9]. A brace stiffness of at least 5 times the story stiffness was assumed in obtaining the reduction factors. The capacity curve when compared with the demand indicates that the structure is still unable to meet the strength of the earthquakes (Fig. 7).

#### 4. Nonlinear Analysis

The nonlinear time-history analyses of the inelastic VE-damped building were carried out for the selected four earthquakes. Figure 8 shows that the drift ratios of the building exceed the specified

limit of 1% under certain earthquakes (Fig. 8).

The above analyses results indicated that the retrofit by VE-dampers was inadequate for this building. The VE-damping alone was not sufficient enough to upgrade this non-ductile frame.

#### 5. Retrofit by Shearwalls

Considering the substantial weakness of the frame, a retrofit using additional shearwalls was introduced as shown in Fig. 9. The idea behind this retrofit was to increase the strength of the frame and restrict its drift to reduce the deformation overload on nonductile components. The strength of the wall was obtained for a minimum steel percentage of 8% per ACI 318-95 [4]. The VE-dampers in the adjacent bay were maintained from the previous analysis which resulted in about 7.5% VE-damping for the building.

The capacity of the retrofitted building was compared with the average demand spectrum of the earthquakes for 12.5% damping. Both the strength and deformation demand of the retrofitted structure seemed to match the average demand of the earthquakes. A nonlinear time history analysis of the building shows that the story drift ratios of the building under these earthquakes are well within the limits (Fig. 10).

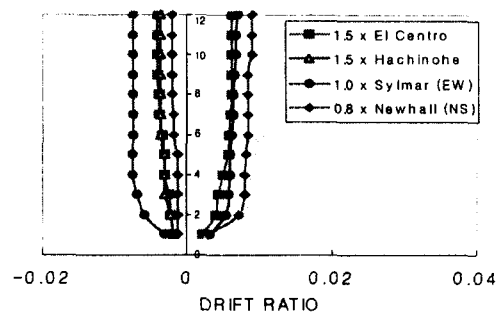


Fig. 10 Drift of building retrofitted with shearwalls and VE-dampers

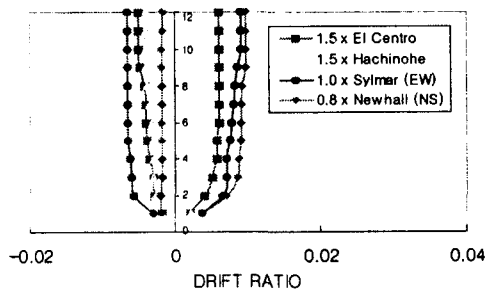


Fig. 11 Drift of building retrofitted with shearwalls only

Note that analysis of the building with added shearwalls and without VE-dampers was also carried out which indicated larger ductility demand on the nonductile components of the building (Fig. 11). A comparison of this analysis with the previous analysis for building with VE-dampers indicated the beneficial effect of providing VE-dampers. It was observed that although VE-dampers did not significantly reduce the displacement of this stiff building with shearwalls, they did, however, provide an energy dissipation mechanism, which helped reduce the ductility demand on the nonductile members of the building. This aspect was previously established elsewhere [10].

## 6. Conclusions

The analysis of the unretrofitted gravity building indicated a drift concentration at the 1<sup>st</sup> floor level due to inadequate strength and ductility capacity of the ground floor columns. The capacity curve of the structure, when superimposed on the average demand response spectrum for the ensemble of scaled earthquakes indicated that the structure was extremely weak and required a major retrofit.

A retrofit of the building using VE-dampers did not seem to be sufficient to rehabilitate this building. It required substantially large dampers

and a large amount of damping which was not practically feasible. With practical size dampers, the analyses indicated that the damper sizes provided were not sufficient enough to remove the biased response and drift concentration of the building. Concrete buildings, in general, being stiffer require stiffer (larger) dampers.

Concrete shearwalls were the most feasible choice considering economy and effectiveness. Shearwalls increased the stiffness and strength and substantially reduce the drift. The effectiveness of VE-dampers in conjunction with stiff shearwalls is also studied. VE-dampers, although not much effective in reducing the drift of the stiff shearwall building, were still effective as an energy dissipation mechanism. They tend to reduce the ductility demand on the nonductile components of the frame.

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