SEISMIC REHABILITATION OF NONDUCTILE REINFORCED CONCRETE GRAVITY FRAME

Javeed A MUNSHI[1]

SUMMARY

This paper presents results of an effort to seismically rehabilitate a 12-story nonductile reinforced concrete frame building. The frame located in the most severe seismic 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 1st 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 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 increase stiffness and strength of the building reducing the drift significantly. The effectiveness of VE-dampers in conjunction with stiff shearwalls is also studied. Considering the economy and effectiveness, it is concluded that shearwalls are the most feasible solution for the rehabilitation of such buildings

INTRODUCTION

The problem of nonductile construction 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 a 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 UBC [1]. By 1958, the seismic provisions were in the appendix of UBC and could be adopted by local jurisdictions at their option and 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

[1] *Structural Engineer, Portland Cement Association, 5420 Old Orchard Road, Skokie,IL 60077, USA*

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 this 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 with 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 (UBC 1994). This frame represents a lower-bound on strength and ductility capacity requiring an upper-bound retrofit strategy.

EXAMPLE ANALYSIS

The Building

Figure 1 shows the plan and elevation of a 12-story beam-column frame used for this investigation. The building is designed and detailed for gravity loads per UBC 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 as 1.9 seconds. The detailing of columns and beams is shown in Fig. 2.

Earthquake Ground Motions

The ground motion records used for the analyses include the El Centro (1940) NS component, the EW component of the Tokachioki earthquake at Hachinohe, Japan (1968) and the Northridge earthquake (1994) as recorded at Sylmar county (EW component) and Newhall stations (NS components). The Northridge earthquake records were selected to study the near-field ground excitation effect. 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.

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 assumed to have a typical compression splice of 20 bar diameters near their ends (see Fig. 2). Considering the current code requirements (ACI 318-95), 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 [4] (see Fig. 2).

Pushover Analysis. A monotonically increasing inverted triangular load is applied to the structure using the DRAIN-2DX program [5]. 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.

Capacity vs Demand. In order to determine the capacity curve of the structure, 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 participation factors and the modal analysis procedure, as per ATC-33 [6]. The capacity curve is superimposed on the average demand spectrum of the four earthquakes, 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.

Nonlinear Analysis. The potential hinges at beam-ends are idealized by using the 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.

Nonlinear analysis of the inelastic building is 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 1st floor level reaches as much as 5% due to weak columns. 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.

Retrofit by VE-Dampers

The retrofit of the building is based on the drift control criteria [7]. It is assumed that the dampers when installed will tend to promote elastic behavior of the building. Thus dampers are 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 is assumed to be 1/2 in. The damping is provided at floors where the drift ratio exceeds 1%. A maximum damping ratio of 15% is provided at any floor level. This maximum limit of damping ratio is 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 is determined for the drift Δ = 1% using a damper loss factor ηd = 1.2 [7]. Figure 6 shows the damper configuration in the building.

Table 1. Damper Size Distribution at Different Floor Levels

Story	Average Drift Ratio %	Damping Ratio (ξ) %	Damper Area/Bay (in^2)
1	1.8	15	600
2	1.9	15	600
3	1.8	15	600
4	1.6	10	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	-	-

Pushover Analysis. Pushover analysis of the retrofitted building is carried out. The VE-damper elements are idealized as elastic spring elements of equivalent stiffness. The damper stiffness is determined at a frequency equal to the fundamental vibration frequency of the structure. An equivalent damping of 15% of the critical is anticipated with the above VE-damper configuration. A high damping demand spectrum is obtained from the 5% demand spectrum using different reduction factors for Sa and Sd [8]. A brace stiffness of at least 5 times the story stiffness is 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).

Nonlinear Analysis. Nonlinear time-history analyses of the inelastic VE-damped building is also carried out. Figure 8 shows that the drift ratios of the building exceed the specified limit of 1% for some earthquakes ground motions (Fig. 8).

The results of the pushover analysis and nonlinear analysis indicate that the retrofit by VE-dampers is inadequate and that VE-dampers alone cannot be sufficient to upgrade this nonductile frame.

Retrofit by Shearwalls

Considering the substantial weakness of the frame, a retrofit using shearwalls is considered as shown in Fig. 9. The idea is to increase the strength of the frame and restrict its drift to reduce the deformation overload on nonductile components. The strength of the wall is obtained for a minimum steel percentage of 0.8% per ACI 318-95 [10].

Two cases of building with and without VE-dampers are studied. In the first case, the VE-dampers in the bay adjacent to the shearwalls are maintained from the previous analysis which results in about 7% VE-damping for the building. With this arrangement, the strength and deformation capacity of the building seem to be adequate for the demand. 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(a)).

In the second case, building with added shearwalls and no VE-dampers was considered. The drift ratios in this case, though similar, are slightly higher than those for the building with VE-dampers (Fig. 10(b)). The analysis indicated larger ductility demand on the nonductile components of the building as compared to the building with VE-dampers. It is observed that although VE-dampers do not significantly reduce the displacement of this stiff building with shearwalls, they do however, provide an energy dissipation mechanism, which helps reduce the ductility demand on the nonductile members of the building. This aspect was previously established elsewhere [9]

CONCLUSIONS

The analysis of the unretrofitted gravity building indicates a drift concentration at the 1st 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.

A retrofit of the building using viscoelastic (VE-) dampers does not seem to be sufficient to rehabilitate this building. It requires substantially large dampers and a large amount of damping which is not practically feasible. With practical size dampers, the analyses indicate that the damper sizes provided are 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 are the most feasible choice considering economy and effectiveness. Shearwalls increase 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, are still effective as an energy dissipation mechanism. They tend to reduce the ductility demand on the nonductile components of the frame.

REFERENCES

International Conference of Building Officials, Uniform Building Code, CA, 1994.

PCA-Build, Computer Program for Comprehensive Analysis and Design of Reinforced Concrete Building Structures, Portland Cement Association, Skokie, IL (1997).

BIAX: A Computer Program for the Analysis of Reinforced Concrete and Reinforced Masonry Sections, Structural Engineering, Mechanics, and Materials, Report No. CU/CEE-92/4, Department of Civil Engineering, Clarkson University, Potsdam, NY.

FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1997.

Prakash, V., Powell, G.H., and Campbell, S., (1995), “DRAIN-2DX, Static and Dynamic Analysis of Plane Structures, NISEE Earthquake Engineering Research Center, University of California, Berkeley, CA.

ATC-33, Recommended Methodology for Seismic Evaluation of Existing Concrete Buildings, Seismic Safety Commission, Sacramento, CA (1995)

Munshi & Kasai, (1995), “Seismic Retrofit of Moment Resisting Frame with Viscoelastic Dampers”, IABSE Symposium, San Francisco, CA.

Fu, Y., (1995), “Frame Retrofit by Using Viscous and Viscoelastic Dampers”, 11th World Conference on Earthquake Engineering, Acapulco, Mexico.

Munshi, J. A., (1998), "Energy Dissipation of Viscoelastically Damped Reinforced Concrete Systems", 6th US National Conference on Earthquake Engineering, Seattle, WA.

ACI Committee 318, Building Code Requirements for Structural Concrete, ACI 318-95, MI..