Performance - Based Seismic Strengthening of RC Building Structures

N. B. Makhoul
University of Balamand, Lebanon.

SUMMARY:
The performance - based seismic retrofitting of a specific existing building is carried out. The structure was modeled before and after the strengthening using inelastic dynamic time history analysis, incremental dynamic analysis, and inelastic static adaptive pushover analysis. A solution was proposed to strengthen the structure in order to withstand the level of hazard 10% in 50 years: adding carbon fiber reinforced polymer to columns and adding some shear walls. Results and comparisons are given.

*Keywords: **P**erformance – Based, Retrofitting, Inelastic Analysis.*

1. INTRODUCTION

Recent earthquakes have taught us that designing a seismic resistant new building is quite different from strengthening an existing building. For the latter we need to propose a cost – effective solution aiming to retrofit the structure in order to perform as desired during different loading scenarios. To this purpose, performance – based design procedures are suggested and adopted to prevent implementing the uneconomic solution of applying strictly the current seismic code provisions proposed for new buildings. We note that uneconomical solutions to retrofit buildings are most of the time rejected by the owners, who cannot afford the cost. The limitations of force – based method, based on false assumptions, give rise to more advanced displacement – based method and performance – based method. Since earthquakes are by definition a defined ground motion; displacement or acceleration thus applied to the foundation of the structure. Modern seismic design needs to ensure that the displacement and ductility demands will not exceed a defined limit state given a specific level of ground motion. Nevertheless we note that: «*Force – based design, when combined with capacity design principles and careful detailing, generally produces safe and satisfactory designs*» Priestley, M., Calvi, G. and Kowalsky, M. (2007).

2. PERFORMANCE BASED SEISMIC RETROFIT

The objective of performance – based seismic retrofit, PBSR, is to ensure a specific or several levels of performance when facing a defined or several levels of seismic excitation. In the «Vision 2000» four levels of seismic excitation are considered: Full Operational, Operational, Life Safety, and Near Collapse. It is an improved approach allowing multi – level design objectives. It aims to quantify the demand and capacity parameters for each performance target level.

The performance – based seismic retrofit design uses more effective parameters as displacement, deformation, and energy than the ones followed by force – based which distributes the force in proportion to the assumed stiffness of the members. PBSD considers nonlinear response history of the structure instead of the inaccurate linear procedure used in force – based design, since structures tend to have a nonlinear behavior under seismic load. However this method is more complex and relies on extensive data. Luckily nonlinear static procedures and software have become more robust and accurate to ensure the reliability of the results**.** The FEMA 356 could be used to check the attainment of the performance levels of the building, thus the acceptance criteria.

3. STUDY CASE

The 1965 Van Nuys Hotel building as described before Northridge earthquake 1994, in PEER 2005/11, has 7 floors and an area of 6200m2 in Van Nuys, California. It was build following the Los Angeles 1964 building codes. In plan it is a (19.202m x 45.720m clear span), 3 bays by 8 bays. The long façade has an East – West orientation. The building total height is 26.213m; the first floor is 4.715m height and the 2^{nd} until 7^{th} floor is 2.591m height.

The structure is mainly composed by reinforced moment resisting frames along the external perimeter, but also the interior columns and the two ways slabs participate to the lateral rigidity. The building is founded on 24in. diameter drilled piers in groups of two, three, and four piers per pile cap, and columns centered on the pile cap. The column concrete nominal strength f'c is 34.5MPa for the first floor, 27.6MPa for the second floor and 20.7MPa for the third until the seventh floor. Beams and slab concrete nominal strength f'c is 27.6MPa for the second floor and 20.7MPa from the third to the last roof. Column reinforcement steel is scheduled as A432-62T (Grade 60) for billet bars, f'y is 413 MPa. Beam and slab reinforcement is scheduled as ASTM A15-62T and A305-56T (Grade 40) for intermediate grade, deformed billet bars, f'y is 275.8 MPa. The Van Nuys Hotel structure was studied regarding many aspects in PEER 2005/11, here under this structure was analyzed by different new methodologies in structural analysis that uses other performing software and considers nonlinear analysis.

4. STRUCTURE ANALYSIS

The structure was modeled, as it was before Northridge Earthquake, using ZeusNL Software as a modeling platform. ZeusNL offers the possibility to perform nonlinear structural analysis, and is found to be a best compromise since it is easy to manipulate, and results are obtained in relatively short time. A first mode period of 1.2 seconds was obtained by the eigenvalue analysis of the 3D structural model as seen in Fig.1.

4.1. Foundation modeling and effects of soil foundation structure interaction

As noted in PEER Report 2005/11 , by *B. Kutter, S. Kramer, G. Martin, T. Nagae, T. Hutchinson, J. Stewart*: «*The soil condition at the Van Nuys testbed site is classified as NEHRP Category D. The capacities of the pile foundations well exceed the combined static and dynamic loads applied to the foundations; therefore, for the Van Nuys testbed, static soil foundation structure interaction SFSI (permanent deformations) of the foundations is not considered important*». The SFSI is not important for this structure, and therefore, the models discussed did ignore SFSI and are based on the assumption of fixed base conditions of all columns

4.2. Static pushover analysis

Through nonlinear static pushover analysis, we obtained a roof lateral displacement of central column of internal frame of the East – west façade of 0.026m for a base shear of 177.7KN where we start to have a nonlinear behavior as shown in Fig.2. The East – west façade is considered as X – direction.

4.3. Nonlinear dynamic analysis

The dynamic analysis was elaborated using three ground – motion time history records; Loma Prieta, Northridge, San Fernando. For the three records, the roof lateral displacement of central column of

internal frame of the East – west façade, was used as a reference displacement and compared to the target displacement based on FEMA 356, (the East – west façade is considered as X – direction).
For the 1989 Loma Prieta earthquake ground motion time – history record, (Richter Magnitude 7), we obtained a maximum displacement of 0.22m for a time of 8 seconds. For the 1971 San Fernando earthquake ground motion time – history record, (Richter Magnitude 6.6), we obtained a displacement of 0.82m for a time of 7 seconds. For the 1994 Northridge earthquake ground motion time – history record, (Richter Magnitude 6.7), we obtained a displacement of 0.39m for a time of 7 seconds as shown in Fig. 3. All the three obtained displacements exceeded the target displacement δt of 5.24cm, obtained following FEMA 356 recommendations for the hazard level of 10% in 50 years.

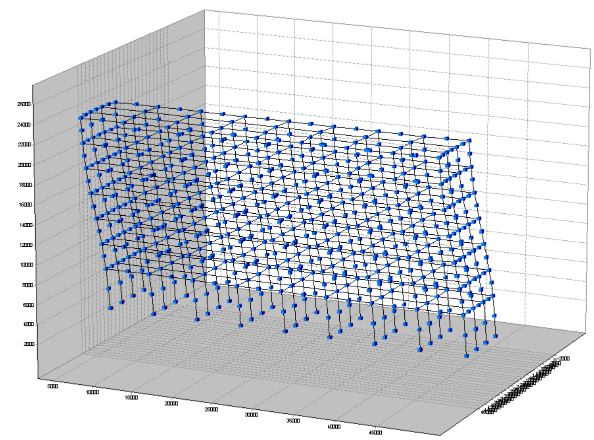

Figure 1. The structure model in 3D deformed for the time of 7.344 seconds, under Northridge time history record

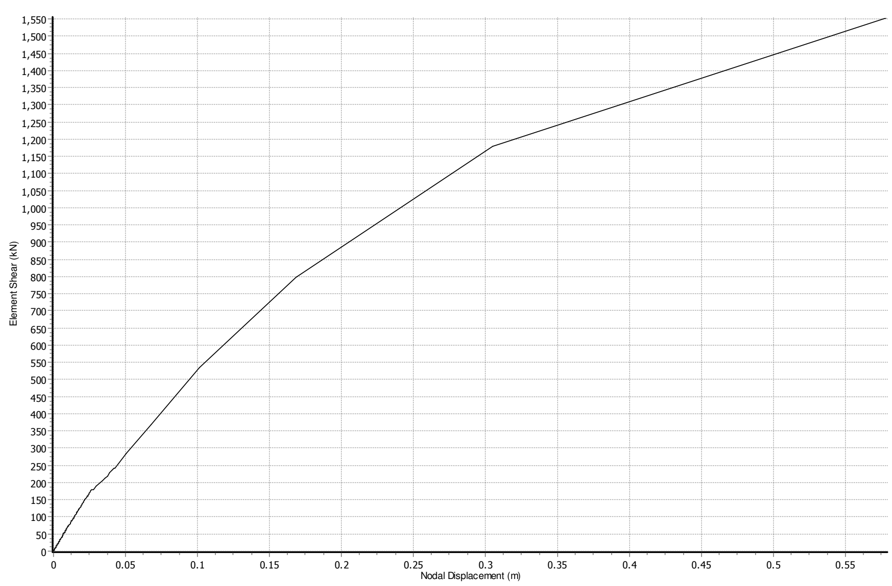

Figure 2. The roof lateral displacement of central column of internal frame of the East – West façade in the direction x, with respect to the element base shear.

4.4. Adaptive pushover analysis

The nonlinear adaptive pushover analysis of the 3D structure modeled through ZeusNL couldn't converge because the matrix of the model had a great dimension, thus the structure was modeled in 2D by adding all four frames as an approximation. We obtained a displacement of 0.016m for a base shear of -44005.9 KN where we start to have a nonlinear behavior. We obtained a displacement of 0.065m for a base shear of 14601.03KN where we start to have a nonlinear behavior. Fig. 4 shows the deformation of the structure for a scaling factor of 92.745.

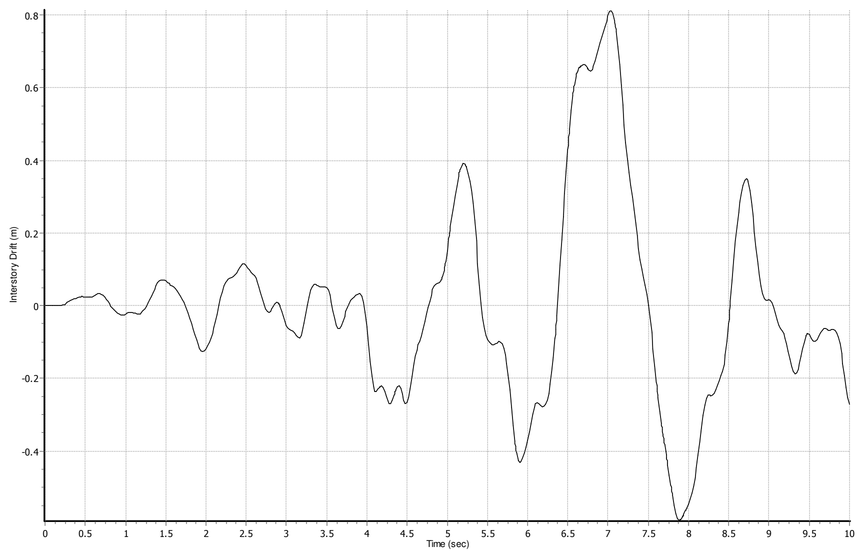

Figure 3. Lateral displacement, or drift, between the roof and base node of central column of internal frame of the East – West façade in the direction x with respect to the time, for San Fernando Earthquake.

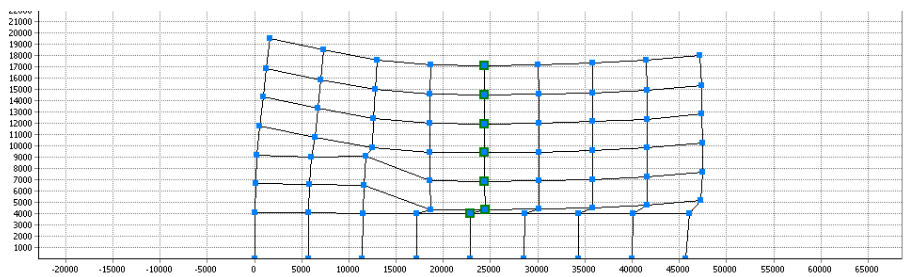

Figure 4. The deformation for a scaling factor of 92.745 under adaptive pushover analysis.

4.5. Incremental dynamic analysis

«The dynamic pushover or Incremental Dynamic Analysis is a special analysis technique where the structural system under consideration is excited by the same ground motion input scaled to different PGA values. For every scaling factor, the maximum response parameters are plotted on a 2D plot just like static pushover curves. The difference with the static pushover is that now each point represents a complete inelastic dynamic analysis» as defined in ZeusNL User Manual**.** The monitor 2 represents the total shear of the first floor function of the drift between the node of the second floor and the first floor (of the middle column of the third frame; internal frame). Similarly are defined the monitors 3, 4, 5, 6, 7, 8.The figure 6.35 represents the maximum drift between 2 successive floors (the column in the middle of the third frame) with respect to the maximum total base shear. We notice that the floors 4 and 5 are the ones that reach the greater drift and shear values for the same scaled factor (which is compatible with the results of PEER 2005/11 report), while the drift at the floor 6, 7 and 8 after reaching a high drift and shear levels, tend to decrease with the increasing total maximum base shear of the lower floor as shown in figure 5.

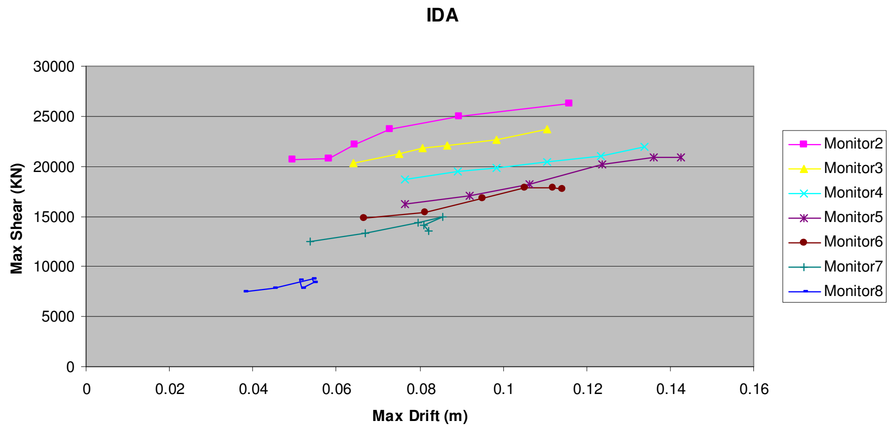

Figure 5. The total shear from columns of the same storey function of the interstory drift between nodes at the top and at the bottom of the same storey (of the middle column of the middle frame; i.e. internal frame) drawn for all stories.

5. COMPARISONS OF RESULTS WITH THE REAL DAMAGES

The results of the IDA show that the maximum drift is reached at the fourth and fifth floor levels. This is probably where the collapse will occur first, which is compatible with the investigations of the real damages the Van Nuys Hotel due to Northridge earthquake, reported in Islam (1996), Li and Jirsa (1998), Browing et al. (2000), Trifunac et al. (1999) and Trifunac and Hao (2001).

6. STRENGTHENING AND RETROFITTING

We propose to strengthen the Hotel for the seismic hazard with a probability of occurrence of 10% in 50 years, which correspond to the hazard level of Northridge Earthquake shown in Fig. 6, the structure behave well for the hazard level of 50% in 50 years. The structure will remain vulnerable for the probability of occurrence of 2% in 50 years, which will probably be ensured by applying fully the code.

The first assumption is to reinforce all the columns using fiber carbon. We used Sika CarboDur M that has E – modulus mean value of 210000N/mm2, a tensile strength mean value of 2400 N/mm2, a mean value of tensile strength at break of 2900 N/mm2, and an elongation at break of 1.2%. The dynamic analysis using Northridge earthquake ground motion record decreased the displacement between the roof and the base from 39.3cm to 34.9cm, which is still far from the target admissible displacement as computed following FEMA 356 that equals 5.24cm. Nevertheless knowing that the structure is weakly confined we kept this solution even though the fiber carbon is expensive. To decrease the displacement, we tried to strengthen by adding shear walls. The second assumption was to add two reinforced concrete walls on the transversal façade of 12.75m length by 0.4m width, and to add eight reinforced concrete walls on the longitudinal façade of 1.86m length by 0.4m width. We obtained a maximum displacement between the roof and the base of 7.5cm which is still greater than 5.24cm target displacement as per FEMA 356. The third assumption was to add two reinforced concrete walls on the transversal façade of 12.75m length by 0.4m width, and to add eight reinforced concrete walls on the longitudinal façade of 3.72m length by 0.4m width. We obtained a maximum displacement between the roof and the base of 9.5cm which is still greater than 5.24cm target displacement as per FEMA 356. The plan of the retrofitted foundation is shown in Fig 9. The forth assumption was to add two reinforced concrete walls on the transversal façade of 12.75m length by 0.4m width, and to add four reinforced concrete walls on the longitudinal façade of 3.52m length by 0.4m width. We obtained a maximum displacement between the roof and the base of 4.9cm which is now less than 5.24cm target displacement as per FEMA 356 as shown in Fig. 7. This last solution is acceptable and is the one chosen to be used for design. All walls used in the three assumptions here above were reinforced with steel area equals 3% of concrete area, as per Eurocode 8.

This last solution offered a great stability for the structure; the obtained displacement being acceptable. It is economically feasible, since we only strengthen some members of the structure, thus the hotel can be functional in a relatively normal way while the retrofitting works are going on; the downtime is relatively small. In addition, the cost is acceptable, since we do not propose to retrofit the whole structure, and the expensive fiber carbon cost is justified by the fact that the concrete is not well confined. This solution is the less disturbing, since we work on a smaller area by only adding four walls.

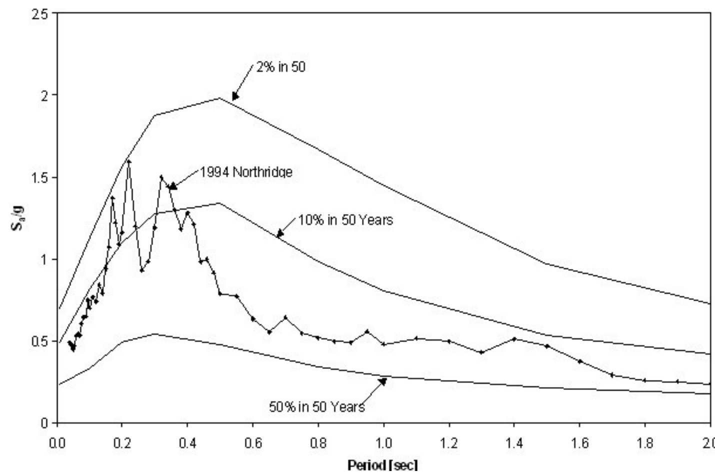

Figure 6. Site-specific response spectra for seismic hazards with a probability of exceedance of 2%, 10%, and 50% in 50 years, along with the 1994 Northridge Earthquake data, PEER 2005/11.

7. COMPARISON OF INELASTIC STATIC PUSHOVER ANALYSIS BEFORE AND AFTER RETROFITTING

Regarding the eigenvalue results, the structure was modeled after retrofitting in 3D through ZeusNL and we obtained the period of the first mode equal to 0.57 seconds.

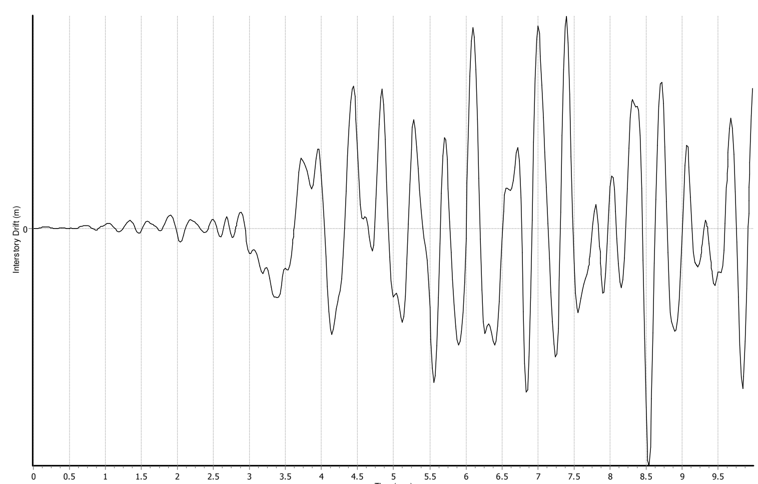

Figure 7. Displacement between the roof and the base nodes in the EAST – WEST direction function of the time.

The period has dropped approximately to half of its initial value. An inelastic static pushover analysis was done using ZBeer of ZeusNL. The structure was modeled before and after the strengthening, in order to compare its behavior. The results of the comparisons were presented in Fig. 8, where the two pushover curves were superposed. The structure after retrofitting has a better behavior: a displacement of 0.2 m after strengthening for example was obtained for a base shear of 140000KN, while for the same displacement before strengthening was obtained for a base shear of 30000KN. The structure after retrofitting is able to withstand for the same displacement a base shear 4 times greater than the one before retrofitting, which can help the structure resist more the seismic loading.

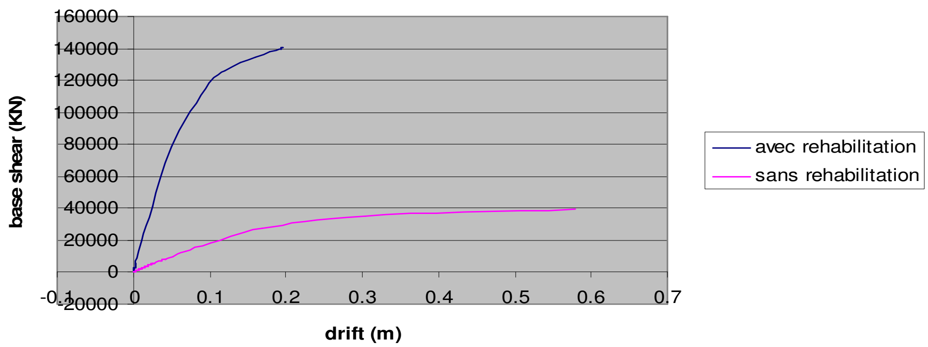

Figure 8. Displacement between the roof and the base nodes in the East – West direction function of the base shear.

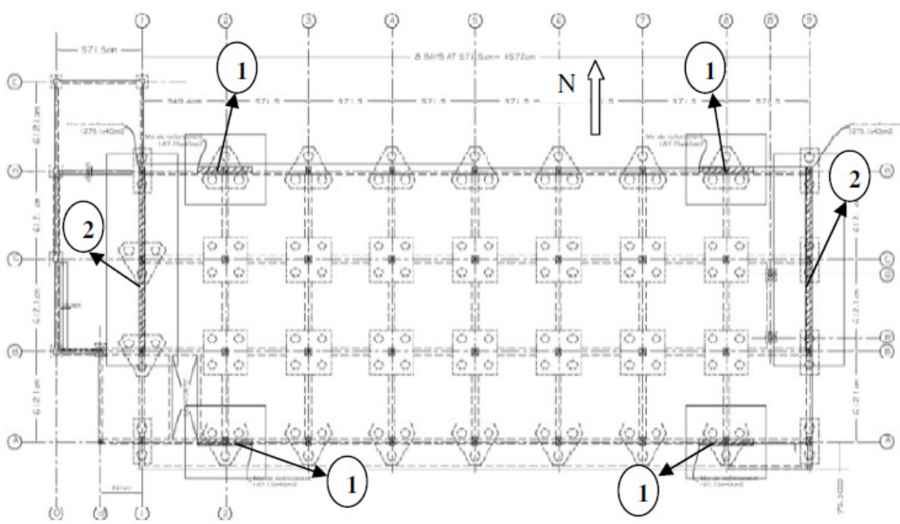

Figure 9. Plan of the foundation after retrofitting; number 1 indicates the walls of 12.75m x 0.4m on the East – West direction and number 2 indicates the walls of 3.52m by 0.4m on the North – South direction.

8. CONCLUSION

A solution was offered to strengthen the structure of the Van Nuys building in California for the hazard level of 10% in 50 years. ZeusNL was used as platform to model the structure; in 3D through inelastic pushover analysis, dynamic time – history analysis, IDA Incremental dynamic analysis, and in 2D through inelastic adaptive pushover analysis. The results were compared to real damages after Northridge to ensure that the model predicted the real behavior of the structure. The structure was analyzed before and after the acceptable retrofitting solution, comparisons and results were offered. The strengthening solution improved the behavior of the structure. It allows withstanding the hazard of

10% in 50 years in California. This study is an application to new methodologies in structural analysis that uses more performing software and considers nonlinear analysis.

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