

# Lift-up and Base Isolation as a Retrofit Technique for R.C. Existing Building



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### **SUMMARY:**

The L'Aquila earthquake occurred in Italy has highlighted the particular vulnerability of old buildings built in the 70's and 80's. The base isolation system (BIS) has been suggested as an innovative retrofit strategy and adopted for the seismic upgrading of some major buildings. In this paper, a six-storey R.C. building was studied. Different kinds of analysis (spectral, pushover and nonlinear dynamic analysis) were performed by Midas/Gen according to EC8. Results had been compared among the original building and buildings retrofitted by different interventions. Although both retrofitting strategies can reduce the seismic vulnerability of existing building, the comparison pointed out building retrofitted with a "Lift up" technique patented by SOLES Ltd., executed by CONSTA SpA., and presented in the following, exhibited better performance than "column cut" technique. The technique has been successfully applied to the mentioned building under the consultancy of BOLINA Engineering Ltd. and the works are just finished.

*Keywords: Base Isolation, Retrofit Technique, "Lift up" System*

## **1. INTRODUCTION**

A great part of existing R.C. buildings built in the 70's and 80's are damaged to different extents during the 2009 L'Aquila earthquake occurred in Italy. So, seismic assessment and strengthening intervention for these structures become one of the most challenging topics for the structural engineer.

According to the most modern seismic code in Europe (EC8), assessment and retrofitting of buildings are provided in part 3. It is developed to deal with some specific aspects such as knowledge requirements of existing structures; structural modelling; methods of analysis; safety verification and structural intervention. Generally, the conventional elastic analysis methods (Equivalent Static Analysis and Response Spectrum Analysis) cannot capture many important aspects that control the seismic performance of structures in severe earthquakes. So, the use of non-linear analysis methods in seismic design is growing rapidly in popularity, such as Non-linear Static Analysis and Non-linear Dynamic Analysis. Usually, Retrofitting is a need for structures damaged in earthquake. The selection of the type, technique, extent and urgency of the intervention shall be based on the structural information collected during the assessment of the building. Base Isolation, known as seismic base isolation or base isolation system, is one of the most popular means of protecting a structure

against earthquake forces. This technology can be used both for newly structural design and seismic retrofit. Differently from more conventional techniques applied to improve the seismic response of buildings, base isolation protects, together with the structural components, is suitable also for the protection of the non-structural components, plants and contents of buildings which represent the large majority of the total value of the construction. Furthermore, base isolation prevents both tenants and owners from evacuation to allow the safety checks from the Authorities after the earthquake. In the following, a six-storey R.C. building located in via Rauco (L'Aquila) is evaluated using the methods provided in EC8. The outline of analyzed building is shown in Fig. 1.1. The floor plan is about 12.1 m wide and 27.8 m long. The storey height is equal to 2.9 m at all storeys. The columns at the base are rectangular with dimensions 200x600 mm, gradually reduced to 200x500 mm from the second floor to the roof. The beams in the building are rectangular whose sizes vary from 200x200 mm to 600x800 mm. Column longitudinal reinforcement ranges between 1.2% and 3.0%, while transverse 6mm and 8mm diameter ties are used, spaced from 240 mm at the lower storeys to 400 mm at the roof. Beams are lightly reinforced (about 0.6%) with transverse 6mm diameter stirrups spaced from 140 mm to 180mm.

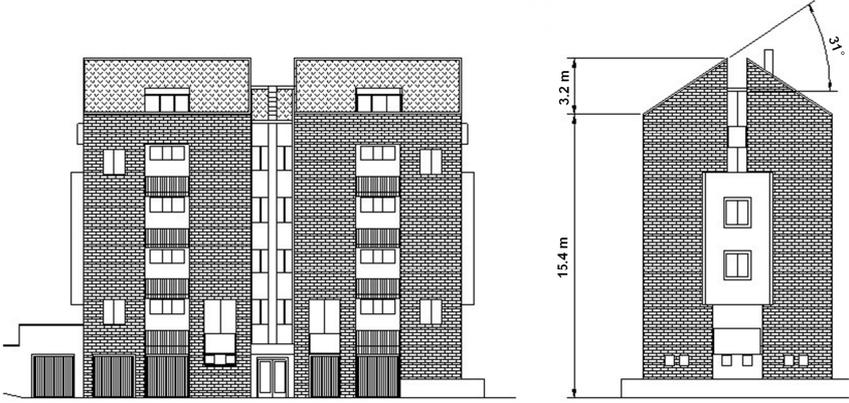


Figure 1.1. Outline of analyzed building

**2. F.E. MODEL OF BUILDING**

**2.1. F.E. model before base isolation intervention**

In this section, structural model before retrofit is simulated by Midas/Gen as real as possible, as shown in Fig. 2.1. The properties of building material and foundation are given by series of in-situ test according to the Technical Standards for Construction. Rigid diaphragm and P-Δ effect are also taken into account. In order to get real answer from numerical model, elements in the bottom of foundation are supported with distributed elastic springs to simulate the interaction between foundation and soil.

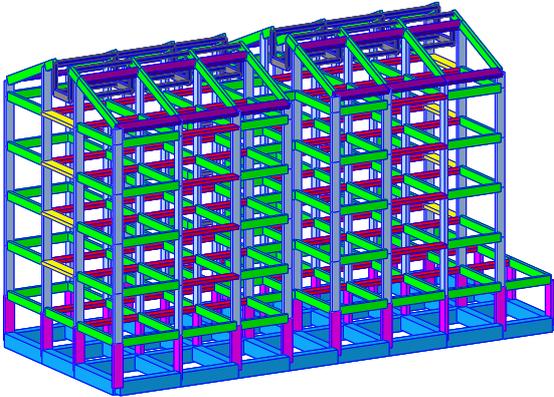
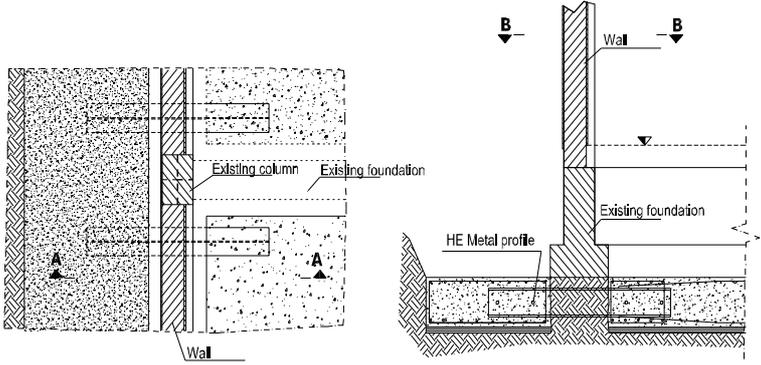


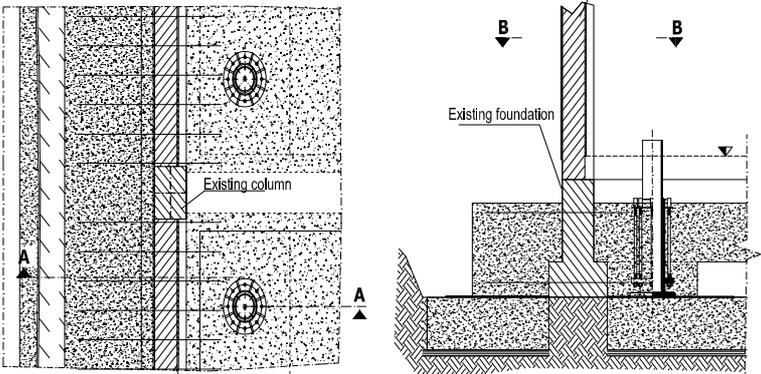
Figure 2.1. F.E. model of building before base isolation intervention

**2.2. Retrofit technology and F.E. model after retrofit**

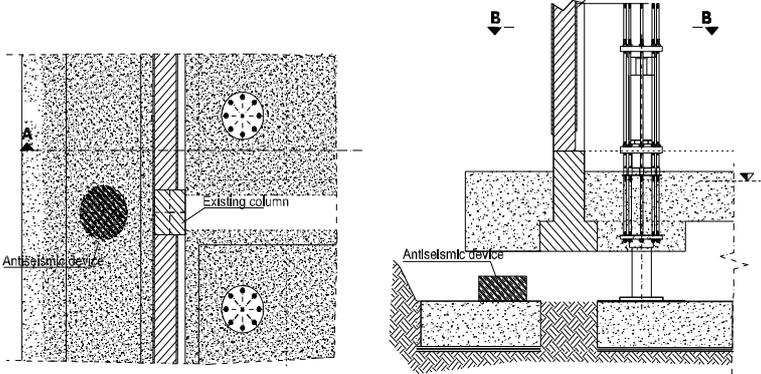
The building was slightly damaged in the L'Aquila earthquake. In order to upgrade the seismic resistance of the structure, two different strategies are used to apply the BIS to existing building. The classical one is based on the “column cut” at the top and it has been applied in several cases. Another innovative system has been developed in Italy (patented by Soles Ltd.) and is based on the “Lift up” of the structures after the separation of the superstructure from the foundations. The system is described in details in below. The construction phases of Soles system in this case can be generally summarized as: 1) realization of a new bottom slab; 2) realization of a new upper slab; 3) lift-up of the whole building and installation of isolators; 4) in-situ casting of the upper slab and removal of lifting devices. Plan and section of construction details in each phase are presented in Fig. 2.2~ Fig. 2.5.



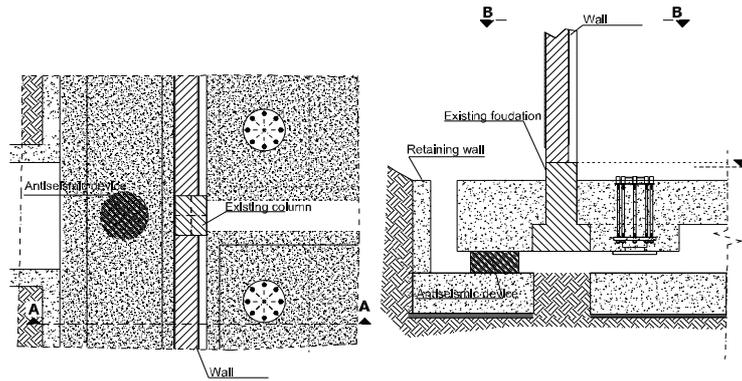
**Figure 2.2.** Realization of a new bottom slab



**Figure 2.3.** Realization of a new upper slab

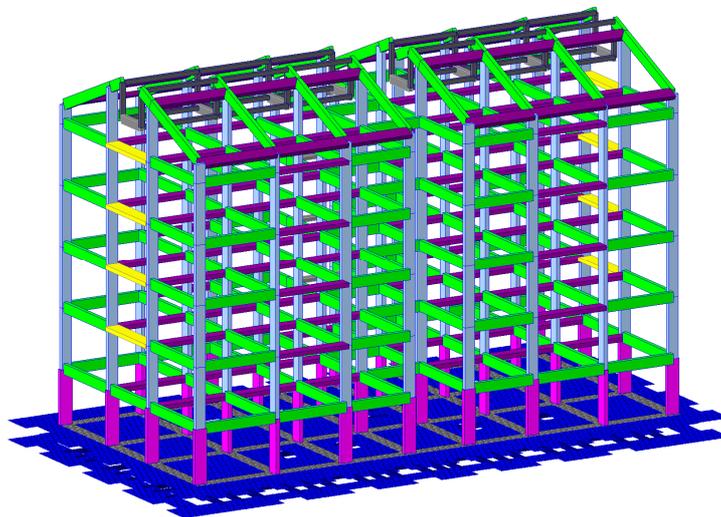


**Figure 2.4.** Lift-up of the whole building and installation of isolators



**Figure 2.5.** Removal of lifting devices

The intervention, as already described previously, realizes a new foundation surface below the existing one. Different parts of the foundation are connected by steel beams (HE 200 B and HE 180 B) so that the foundation can resist seismic action with adequate stiffness. The F.E. model of building retrofitted by “Lift up” is represented in Fig. 2.6.



**Figure 2.6.** F.E. model of building retrofitted by “Lift up”

### 3. DETERMINATION OF SEISMIC ACTION

#### 3.1. Elastic Response Spectrum

The building is located in L’Aquila city, Abruzzo region and seismic actions are determined according to the requirements of NTC 2008. In this case, parameters of elastic response spectrum are: Soil Type C; Category topographic T1 ( $h/H = 0.2$ ); Nominal life of the structure:  $VN = 50$  years; Coefficient of use of building:  $C_u = 1$ .

Three performance levels, referred as Limit States, are considered in EC8-3:2004 for existing structures: Limit State of Damage Limitation (LS of DL); Limit State of Significant Damage (LS of SD); Limit State of Near Collapse (LS of NC). The given structure achieves one of the Limit States (namely, Performance Level) mentioned above when the first of its members or sections achieves the corresponding demand in terms of generalized displacement. The appropriate levels of protection are achieved by selecting, for each of the Limit States, a return period for the seismic action. The elastic acceleration spectrums of various Limit States obtained from NTC 2008 are presented in Fig. 3.1.

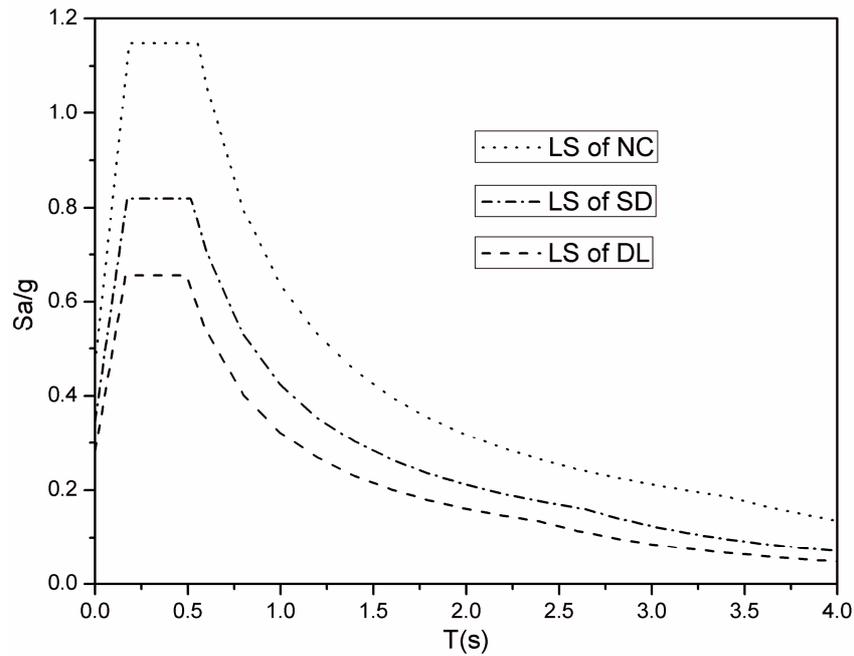


Figure 3.1. Elastic spectrums used in different Limit States

### 3.2. Artificial earthquake records

To give statistically significant results, an ensemble of 10 artificial earthquake records with 20 s duration fully satisfying the EC8 provisions, were used as input of non-linear dynamic analyses. They were created using the program SIMQKE, which generates statistically independent accelerograms based on a user-specified duration envelope and velocity response spectrum. The envelope adopted had a rise time of 2 seconds, a stationary duration of 10 seconds and a decay time of 8 seconds. The elastic response spectrum corresponding to LS of SD was adopted to create these artificial earthquake records. Fig. 3.2 shows the fitness of time histories to target response spectrum (plotted in terms of acceleration).

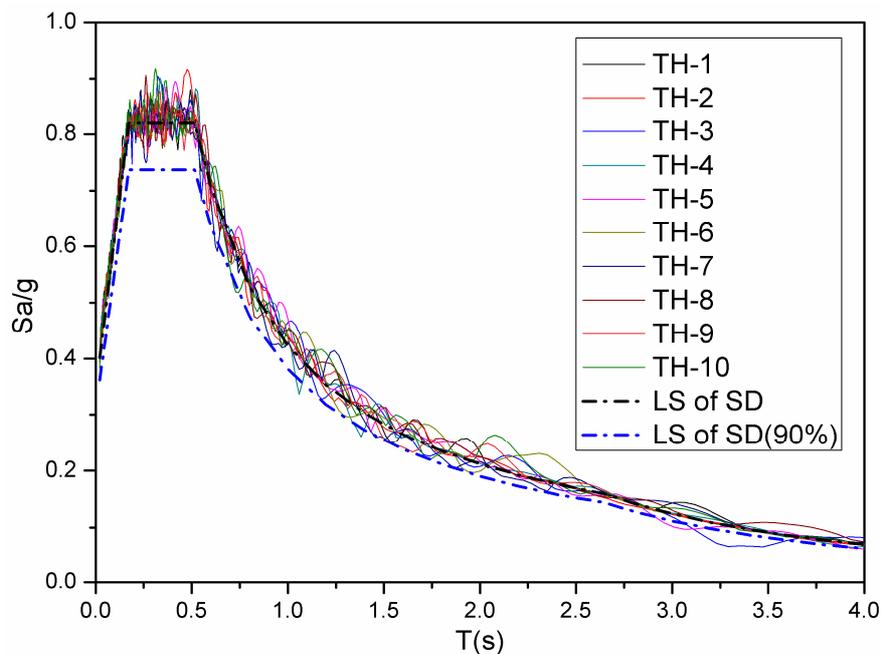


Figure 3.2. Fitness of time histories to target response spectrum

## 4. RESULTS OF ANALYSES

In this section, various analyses were performed by Midas/Gen according to EC8. Response spectral analysis (RSA) was performed based on design spectrum which is defined by introducing a behaviour factor  $q=1.5$  in elastic response spectrum corresponding to Limit State SD. Both uniform (UNI) and modal (MOD) load pattern are used in pushover analyses. Results of time history analysis (THA) are quoted as mean and mean  $\pm$  one standard deviation ( $m \pm sd$ ) for the analyses performed using the different ground motion records. All the results are compared among original building (ORI), building retrofitted by “column cut” (R1) and building retrofitted by “Lift up” (R2) in terms of natural frequency, target displacements, base shear and inter-storey drifts. Wherever possible, data from all analyses are presented in a single table or figure, so that the reader can make comparisons between different retrofitting strategies and analysis methods.

### 4.1. Natural frequency

Natural frequencies of different structures estimated by eigenvalue analysis and pushover analysis are shown Table 4.1.

**Table 4.1.** Comparison of natural frequencies (Hz) between eigenvalue analysis and pushover analysis

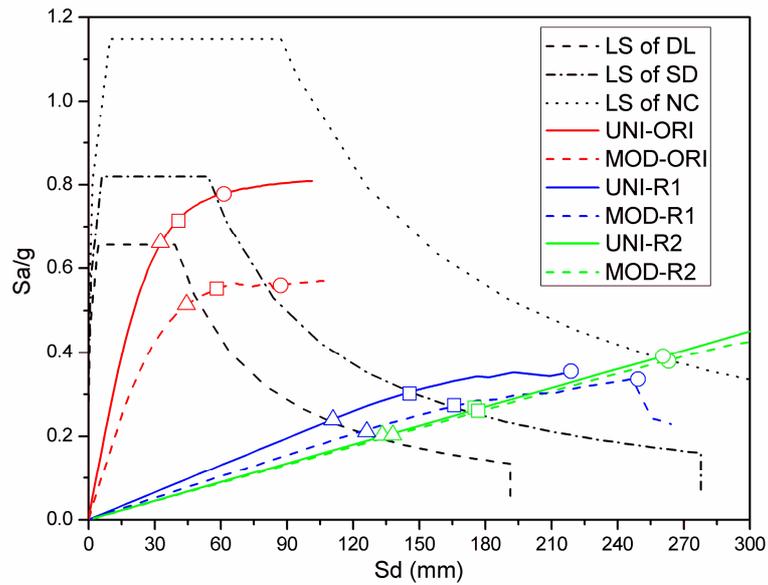
Structure	Eigenvalue analysis	Pushover analysis	
		UNI	MOD
ORI	2.100	1.823	1.36
R1	0.654	0.636	0.558
R2	0.583	0.553	0.545

Compared with the original building, natural frequencies of buildings after retrofitting decrease dramatically which is more favourable for building to resist the seismic action. The first 4 modes in ORI are able to active 75% of the building mass while in the building R1 and R2 almost the total mass is activated with first two modes. Different vibration modes can be found in these buildings. In the original building, the first mode is essentially in bending; the second and third are twist-dominated. But in the buildings after retrofitting, the first two modes are rigid motions.

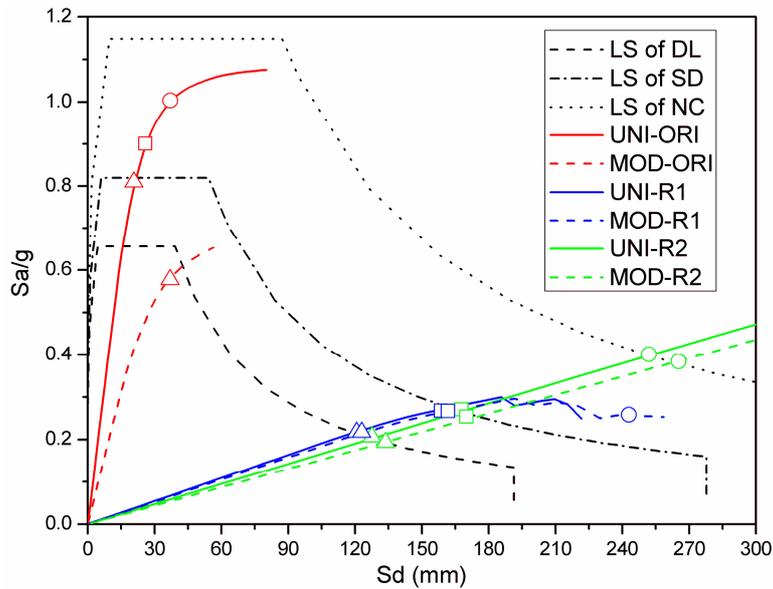
The property of plastic hinges in the EC8, which assumes no post-yield stiffness, provides a very low estimate of the initial stiffness, hence a low estimate of natural frequency compared with that obtained by eigenvalue analyses. It is obvious that uniform load pattern always produces a stiffer response, and so a higher estimate of natural frequency than modal loading pattern.

### 4.2. Target displacement

Comparisons of equivalent SDOF pushover curves are represented in Fig. 4.1 and Fig. 4.2, together with the elastic spectrums corresponding to different Limit States. In the following two figures, symbol  $\triangle$ ,  $\square$  and  $\circ$  mean the target displacement of LS of DL, LS of SD and LS of NC respectively. Legend “UNI-ORI” represents pushover results using uniform load pattern in building ORI and similar meaning is applicable to other legends.



**Figure 4.1.** Pushover curves along the longitudinal direction (SDOF)



**Figure 4.2.** Pushover curves along the transversal direction (SDOF)

In LS of DL and SD, all the structures are able to bear seismic actions except building ORI when modal load pattern is applied along the transversal direction. So it is necessary to retrofit the original building. Building R1 and R2 have good anti-seismic performance because of lower natural frequencies. Soles system installed in R2 makes it intact even under LS of NC.

Table 4.2 and 4.3 show the target displacements of different structures (MDOF) in LS of SD predicted by various analysis methods.

**Table 4.2.** Target displacements (mm) estimated along the longitudinal direction (MDOF)

Structure	UNI	MOD	RSA	THA		
				mean-sd	mean	mean+sd
ORI	54.80	78.13	40.97	47.07	50.94	54.81
R1	165.35	188.41	141.22	152.15	160.25	168.34
R2	186.81	190.10	134.35	182.69	194.15	205.62

**Table 4.3.** Target displacements (mm) estimated along the transversal direction (MDOF)

Structure	UNI	MOD	RSA	THA		
				mean-sd	mean	mean+sd
ORI	41.26	74.99	40.58	42.28	46.24	50.20
R1	165.62	168.75	110.52	149.65	160.02	170.39
R2	193.50	202.07	139.88	181.42	193.11	204.79

It can be found all the pushover methods show reasonable agreement with the time-history results. The relative roof-displacement is larger for the modal load pattern, which can be explained by the fact that the lateral force of the modal load pattern is applied to the structure at relatively higher position than uniform load pattern.

Because the columns are fixed in ORI, it has a small target displacement but severe seismic response. Most of deformation in R1 occurs in the first floor and bearing capacity of the whole building begins to decrease when base column yields, so building R1 has a moderate target displacement. Using “Lift up” technique, R2 moves as a rigid body and it has the largest Target displacement. It is clear that target displacement in RSA is smallest because it is an elastic analysis based on reduced response spectrum.

### 4.3. Base shear

Base shear from various analysis methods are compared in Table 4.4 and 4.5.

**Table 4.4.** Base shear estimated along the longitudinal direction

Structure	UNI	MOD	RSA	THA		
				mean-sd	mean	mean+sd
ORI	2.14 E+06	1.65 E+06	1.66 E+06	1.61 E+06	1.70 E+06	1.79 E+06
R1	1.06 E+06	0.97 E+06	0.66 E+06	1.00 E+06	1.04 E+06	1.09 E+06
R2	1.39 E+06	1.37 E+06	0.88 E+06	1.29 E+06	1.37 E+06	1.46 E+06

**Table 4.5.** Base shear estimated along the transversal direction

Structure	UNI	MOD	RSA	THA		
				mean-sd	mean	mean+sd
ORI	2.46 E+06	1.71 E+06	1.53 E+06	1.80 E+06	1.94 E+06	2.08 E+06
R1	1.07 E+06	1.05 E+06	0.75 E+06	1.01 E+06	1.06 E+06	1.12 E+06
R2	1.43 E+06	1.37 E+06	0.89 E+06	1.30 E+06	1.39 E+06	1.47 E+06

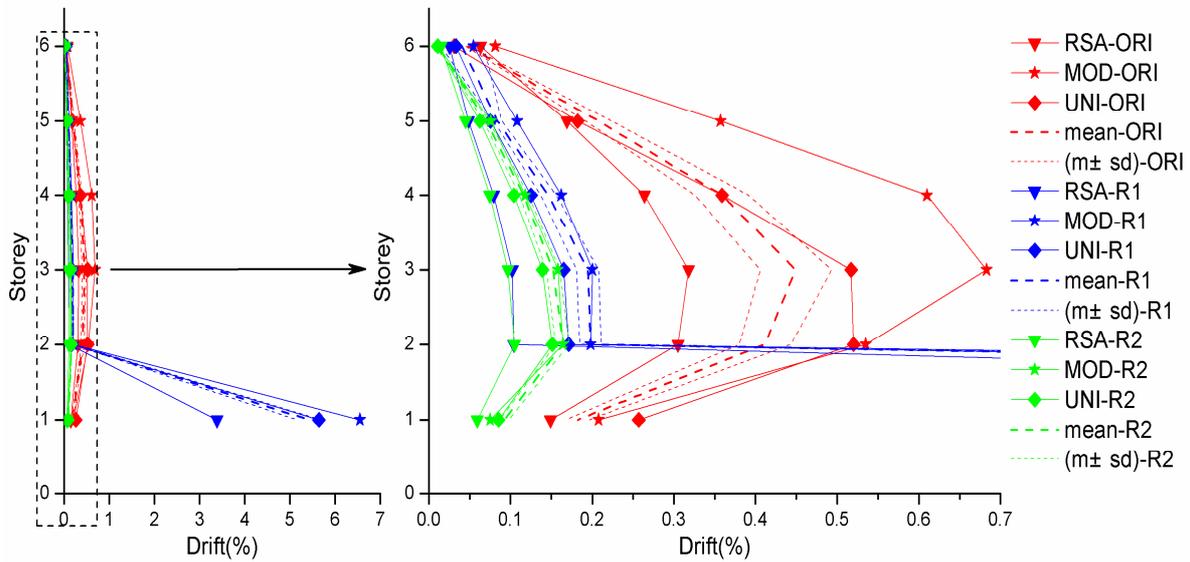
Base shear predicted by pushover analyses are broadly consistent with that estimated by time history analyses. Because of fixed base, ORI has the highest base shear, which means the severest seismic response. Although the same base isolators are used in R1 and R2, a larger target displacement is obtained in R2, hence it has a larger base shear than R1.

As a stiffer response caused by uniform load pattern, it has a higher estimate of base shear than modal loading pattern. What’s more, difference of base shear between uniform and modal load pattern is larger in ORI than that in R1 and R2, which can be explained by the following fact: distributions of lateral force in both load patterns are nearly the same because the dominant modes in R1 and R2 are rigid motions.

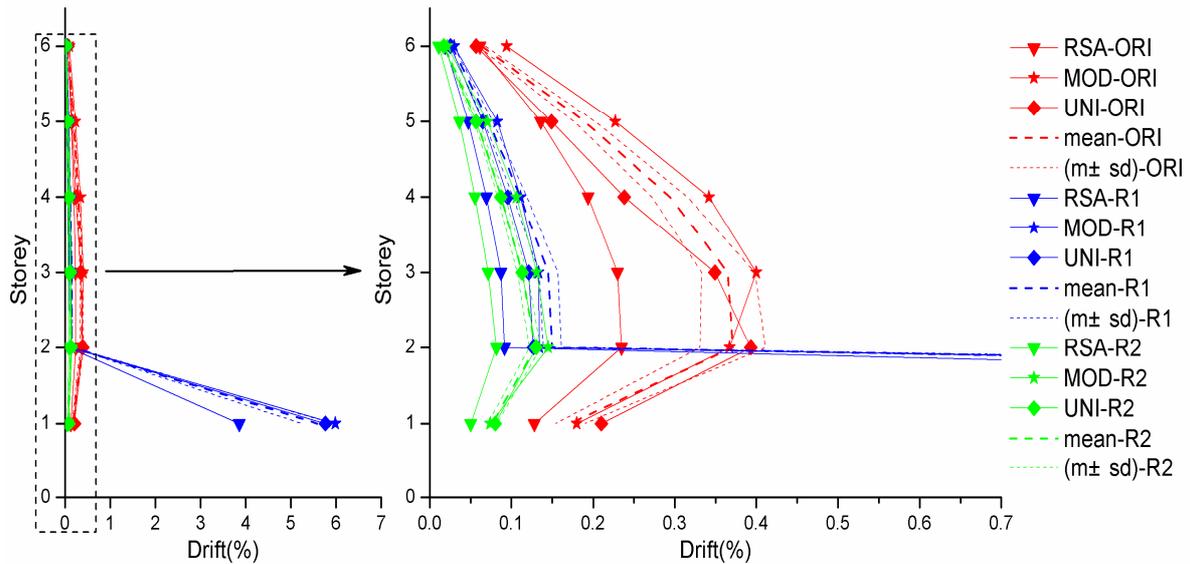
Base shear estimated by RSA based on the reduced response spectrum in ORI has a good agreement with that predicted by pushover analysis and time history analysis, which shows behaviour factor  $q=1.5$  recommended by EC8 is suitable to avoid explicit inelastic structural analysis in the design of the original building. But base shears in R1 and R2 computed by RSA are less than that obtained by pushover analysis and time history analysis. So the design spectrum as defined above is not sufficient for the design of structures with base-isolation or energy-dissipation systems, which is also specified in EC8.

#### 4.4. Inter-storey drifts

Inter-storey drifts estimated by the various methods for different structures are compared in Fig. 4.3 and Fig. 4.4, which are also computed in accordance with LS of SD.



**Figure 4.3.** Inter-storey drifts estimated along the longitudinal direction



**Figure 4.4.** Inter-storey drifts estimated along the transversal direction

It is obvious that inter-storey drifts calculated by pushover analyses have a good agreement with that from time history analyses in both longitudinal and transversal directions. Because of different distribution of lateral force in uniform and modal load pattern, the former has a higher estimate of inter-storey drift than the later in low storeys, while it is opposite in high storeys. Inter-storey drifts estimated by RSA are much less than that computed by pushover and time history analyses.

In ORI, inter-storey drifts are almost three times larger than that in R1 and R2. Because base isolators are installed in the top of base columns, large deformation occurs in the first floor of R1, but inter-storey drifts of the rest storeys are very small. The super-structure of R2 moves like a rigid body, and it has the smallest inter-storey drifts.

#### 4.5. Costs comparison

The technique presented was successfully applied to the mentioned building and the works have just been completed (Fig. 4.5). The associated costs of realization have proved to be convenient, if compared to more traditional techniques



Type of intervention	Average Unit cost in L'Aquila (€/m2)
CFRP Retrofit (Fixed base)	1.013,00
<b>Base Isolation Lift Up</b>	<b>970,00</b>
Base Isolation Top colum cut	961,00
Base Isolation Base colum cut	960,00

**Figure 4.5.** The building after Lift-Up intervention and base isolation and the associated costs compared with other types of techniques

#### 5. CONCLUSION

In the present paper, assessment and different retrofitting strategies of existing building are treated, including “column cut” technique and “Lift up” system such as executed by CONSTA SpA according to the SOLES Ltd. patent on the building presented. Results of pushover analysis show a good agreement with time history analysis in the studied buildings, so it is a good alternative of time-consuming non-linear dynamic analysis. It is also verified that response spectrum method using design spectrum is not sufficient for the design of structures with base-isolation systems. Seismic response decreases dramatically in both retrofitted buildings, while “Lift up” technique is more effective to reduce the seismic vulnerability of existing building. Building retrofitted by “Lift up” technique can be considered, as a matter of fact, as a “new” building and it avoids the strengthening of the super-structure because of the lower natural frequency and smaller inter-storey drifts with associated competitive costs compared to more traditional technologies. From the economic point of view, “column cut” technique and “Lift up” system are equivalent in the presented case. The “column cut” technique generally implies a reduction of the free floor height and strength of structural elements in elevation, so stairs and elevators need special design. But the “Lift up” system has not the disadvantages above and preserves the architectural and functional character of the construction. So, “Lift up” technique is more attractive and should be widely applied in the retrofitting of existing building.

#### REFERENCES

- CEN (2004). Eurocode 8: “Design of Structures for Earthquake Resistance - Part I: General Rules, seismic actions and rules for buildings”, (EN-1998-1). Brussels, May 2004.
- CEN (2004). Eurocode 8: “Design of Structures for Earthquake Resistance - Part III: Assessment and retrofitting of buildings”, (EN-1998-3). Brussels, May 2004.
- Chopra A.K. (1995). Dynamics of Structures-Theory and Application to Earthquake Engineering, Prentice Hall, New Jersey.
- Fajfar P and Gaspersič P. (1996). The N2 method for seismic damage analysis of RC buildings. *Earthquake Engineering and Structural Dynamics* **25**:1,31-46.
- Gasparini DA, Vanmarcke EH.(1970). SIMQKE: A Program for Artificial Motion Generation: User's Manual and Documentation. Department of Civil Engineering, Massachusetts Institute of Technology.
- Midas/Gen. The N2 Pushover Method of Eurocode8-1:2004 and NTC2008, Midas Information Technology Co., Ltd.