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PUSHOVER ANALYSIS OF A SEISMIC STRENGTHENING OF AN EXISTING BUILDING REINFORCED CONCRETE STRUCTURE

Nasser-Eddine ATTARI[1a], Mohamed CHEMROUK[2b] and Sofiane AMZIANE[3c]

[1]EPAU, Ecole Polytechnique d'Architecture et d'Urbanisme, Alger, Algérie.
[2]Université des Sciences et de la Technologie Houari Boumediene, Alger, Algérie.
[3]Université Blaise Pascal, Polytechnique Clermont Ferrand, Département Génie Civil, France.
[a]attari_n@yahoo.fr
[b]Sofiane.Amziane@polytech.univ-bpclermont.fr
[c]mchemrouk@yahoo.fr

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Abstract:
Each operation to strengthen or repair requires special consideration and requires the use of methods, tools and techniques appropriate to the situation and specific problems of each of the constructs. The aim of this paper is to study the pathology of building of reinforced concrete towards the earthquake and the vulnerability assessment using a non-linear Pushover analysis and to develop curves for a medium capacity building in order to estimate the damaged condition of the building.

Introduction:
Recent earthquakes have highlighted the urgency and importance of rehabilitating seismically deficient structures to achieve an acceptable level of performance. This can be achieved, in part, by reducing the load effect input to the existing structures or by improving the strength, stiffness, and ductility of the existing structures.
Seismic deficiencies should first be identified through a seismic evaluation of the structure by using non-linear Pushover analysis (Chopra and Goel 2001)., the seismic performance of a structure can be evaluated and the deficiencies or vulnerable elements can be found. Therefore, the results of these analyses provide critical information for selection of proper rehabilitation techniques (Bai and Hueste 2003).

Description of the Structure:
The structure is characterized by a frame system of reinforced concrete with four levels, 16 columns 30X30 cm2 supporting a floor 16+4 cm, with beams of 12cm wide, the section of longitudinal beams is 30x40cm2, cross beams have a size 30x35.

History of the structure :
The building was constructed at 1979 according to any Seismic Regulations; currently the building is located in an area of level 3, as the Algerian seismic regulations RPA 2003 with an acceleration of 0.25g. It no longer meets current seismic standards in the region.

Materiels :
Reinforced concrete - strength: $f_{ck=}$ 20 Mpa
- modulus of elasticity: 24000 Mpa

Bending reinforcement steel: yield stress f_{yk}= 413 Mpa
Shear reinforcement steel: yield stress f_{ywk}= 413 Mpa

Load: Q= 1,5 KN/M2

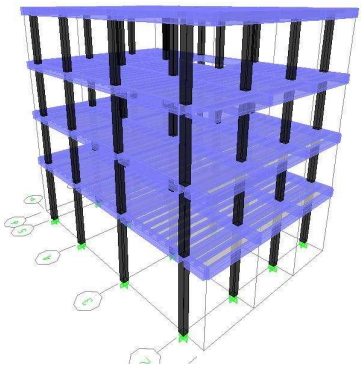


Fig.1

Analysis Software:
All of The analysis structure is performed with the software **ETABS** Nonlinear Version 9.0
Analysis of the Existing Structure:
Modelling of the structure:
The structural elements are beams and columns, which constitute moment resisting frames at both the directions of the building. At the underground levels there are additionally perimeter shear walls. The structural model consists of beam elements for the beams, the columns and the members of the tower and finite elements for the slabs and the walls of the basements.
The analysis of the structure is performed with the software ETABS Nonlinear Version 9.0.
Dynamic elastic analysis:
Under the Linear Dynamic Procedure (LDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly elastic, dynamic analysis.
Linear procedures should not be used for highly irregular buildings (fema 273) .
The spectral analysis of a structure, the Algerian code RPA99 and Eurocode 8 suggest different values of the behaviour factor R, according to the structural type. Considering that the building was constructed without any code the behaviour factor R is taken equal to 2,5. As a confirmation if this value, the Eurocode take R= 2 for this type of structure.

As it concerns the estimation of the seismic action, is being performed according to the Algerian regulation RPA99.The spectral accelerations of the design spectrum are defined by the following expressions:

$$\frac{S_a}{g}=\begin{cases} 1.25A\left(1+\frac{T}{T_1}\left(2.5\eta\frac{Q}{R}-1\right)\right) & 0\le T\le T_1 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right) & T_1\le T\le T_2 \\ 2.5\eta(1.25A)\left(\frac{Q}{R}\right)\left(\frac{T_2}{T}\right)^{2/3} & T_2\le T\le 3.0s \\ 2.5\eta(1.25A)\left(\frac{T_2}{3}\right)^{2/3}\left(\frac{3}{T}\right)^{5/3}\left(\frac{Q}{R}\right) & T>3.0s \end{cases}$$

A : peak ground acceleration ;in our zone A=0,25
η : the damping correction factor,

$$\eta=\sqrt{7/2+\xi}\quad \ge 0.7$$

ξ : viscous damping ratio equal to 5%,
R: the behaviour factor of the structure
T1, T2: the characteristic periods of the spectrum associated to the category of site

Q: Quality factor.

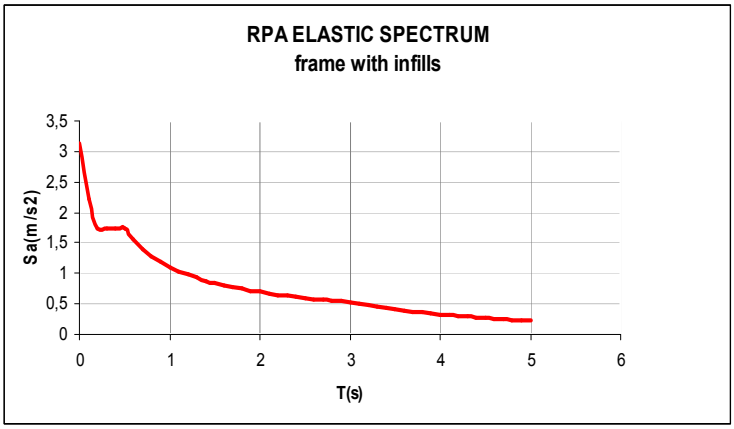


Fig.2 Elastic Spectrum

Verification of Structural Elements:
Two different methodologies are considered for the evaluation of the vulnerability of the structure by means of dynamic elastic analyses. The first method is based on the American Guidelines FEMA 273, with the use of local magnification factors to the members of the building. The second method of evaluation uses a global behaviour factor for the structure. For both methodologies, the knowledge level of the construction of the building is considered (i.e. to what extend investigations for the structural documentation, like material tests etc, was carried out).

Methodology based on global behaviour factor
the behaviour factor q is taken equal to 2,00.
Sd=G+0,3Q+E
The inspection of the elements shall satisfy the following equation: Sd/Rd ≤ 1
Areq/ Aexit ≤ 1
Two different values are considered for the safety factors for the materials of the structure depending on the level of knowledge:
yc=1 ;ys=1

Verification of lateral displacements
Apart from the inspection of beams and columns, the damage limitation of infills and the influence of P – Δ effects are also examined. The damage limitation of infills is fulfilled if:

$\gamma = dr \cdot q/2{,}5.h \leq 0{,}005$

where dr is the interstorey drift, evaluated as the difference of the average lateral displacements at the top and bottom of the storey under consideration. Second – order effects are not considered if the following equation is fulfilled:

$\theta = P_{tot}.dr/v_t.h \leq 0{,}10$

where:
P_{tot} is the gravity load at the storey under consideration for seismic design,
Vt is the seismic storey shear.
h is the interstorey height.

The following Tables (1§2) present the examination of damage limitation of infills and the influence of P – Δ effects. It is observed that at all stories ,the inspections are fulfilled.

Table .1:

Floor	Inter-storey height	Inter-storey Drift		Y%	
		Ex	Ey	Yx	Yy
1	3.8	0.0021	0.002	0.044	0.044
2	3	0.0017	0.0017	0.045	0.045
3	3	0.00148	0.0015	0.040	0.040
4	3	0.001	0.001	0.020	0.02

Table .2:

Floor	Inter-storey height	Pt	Inter-storey Drift		Vt x	V	θx	θy
			Ex	Ey				
1	3.8	3681	0.0021	0.002	541	529	0.003	<
2	3	2740	0.0017	0.0017	459	375	<	<
3	3	1380	0.0014	0.0015	281	275	<	<
4	3	913	0.001	0.001	179	175	0.0017	<

2. Nonlinear Static Analyses:

Static nonlinear analyses:

The model is set up in ETABS 9.0 Version, the program use finite element modelling methods; a Pushover Analyses were carried out.
Static nonlinear analysis does not always guarantee a unique solution, for this reason we are consider many different loading cases.
Some standards such as Eurocode 8, Part 1 (2003) and FEMA 356 (2000) require the calculation of the curve "pushover" for at least two different distributions of force.
For reasons of simplification, a single distribution is considered here. The curve "pushover" represents the relationship between the shear at the base "base shear" and the horizontal displacement of the building.

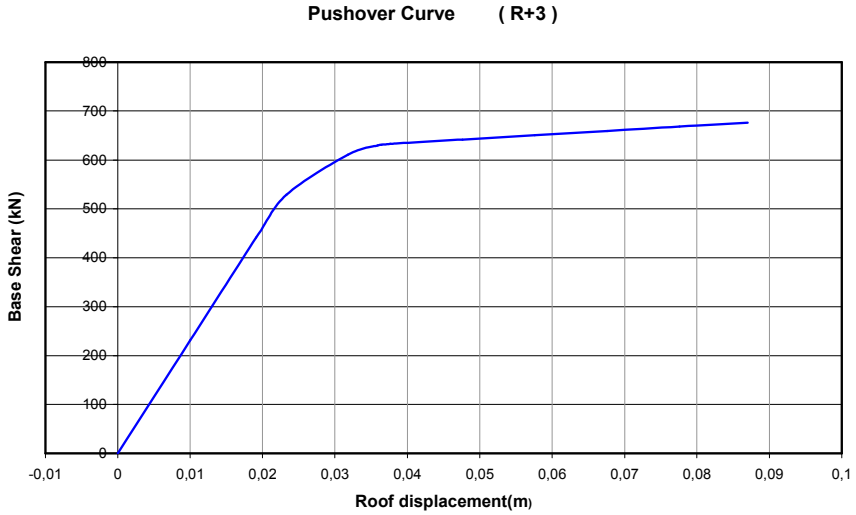


Fig.3: Pushover Curve

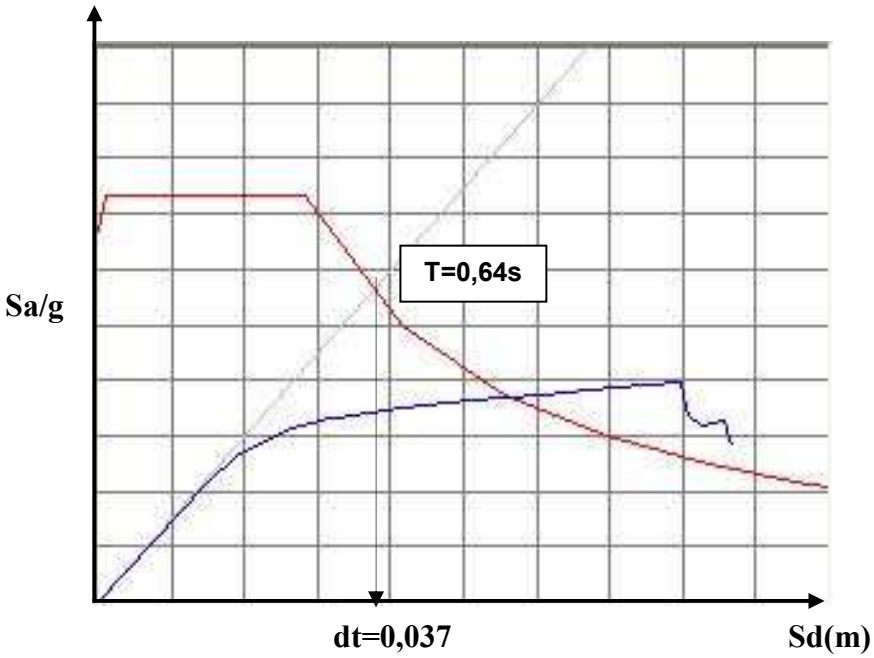


Fig.4: Elastic Spectrum and capacity Curve
Basic Model

Verification of structural elements:

Basic Model

The assessment of the structure is made by comparison of the seismic demands with the capacities for the selected performance level. The actions controlled by deformations depend on the performance level, meaning the level of protection. The following Tables present the maximum allowed plastic rotation angle for beams and columns controlled by flexure, for different performance levels.

Table.3: Modelling parameters and acceptance criteria for reinforced concrete columns Fema 273

Conditions			Modeling parameters			Acceptance criteria				
			Plastic rotation angle (rad)		Residual strength ratio	Plastic rotation angle (rad)				
						Component type				
						Primary			Secondary	
						Performance level				
			a	b	c	IO	LS	CP	LS	CP
Columns controlled by flexure										
$\frac{N}{A_g f_{cd}}$	Transverse Reinforcement	$\frac{V}{b_w d\sqrt{f_{cd}}}$								
≤ 0,1	C	≤ 3	0,020	0,030	0,200	0,005	0,010	0,020	0,015	0,030
≤ 0,1	C	≥ 6	0,015	0,025	0,200	0,005	0,010	0,015	0,010	0,025
≥ 0,4	C	≤ 3	0,015	0,025	0,200	0,000	0,005	0,015	0,010	0,025
≥ 0,4	C	≥ 6	0,010	0,015	0,200	0,000	0,005	0,010	0,010	0,015
≤ 0,1	NC	≤ 3	0,010	0,015	0,200	0,005	0,005	0,010	0,005	0,015
≤ 0,1	NC	≥ 6	0,005	0,005	-	0,005	0,005	0,005	0,005	0,005
≥ 0,4	NC	≤ 3	0,005	0,005	-	0,000	0,000	0,005	0,000	0,005
≥ 0,4	NC	≥ 6	0,000	0,000	-	0,000	0,000	0,000	0,000	0,000

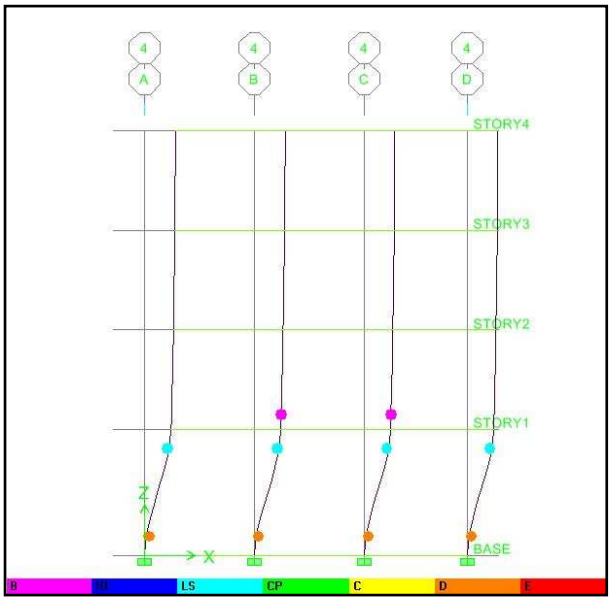


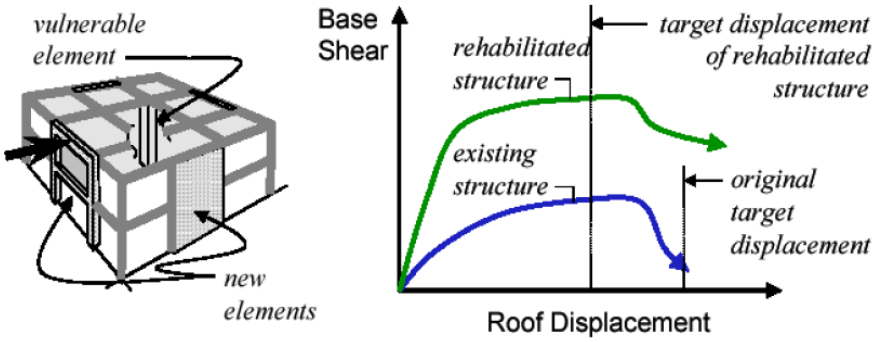
Fig.5. Plastic hinges position

According to Fema and based on the given column properties the plastic rotation capacity at both column ends appropriate for the performance level of collapse prevention (CP) should be taken as 0.005.

Note that, for the performance levels of immediate occupancy (IO) and life safety (LF) a plastic rotation of 0.001 should be taken.
As shown, the plastic rotation demands in columns were **0,017** higher than **0.005**, indicating that the columns experienced severe damage.

According to Paulay & Priestley (1992), such damage buildings can be still accepted, when the loss in force is smaller than 20% of the maximum force.
Rupture of the plastic hinges at flexure in beams ;induced a redistribution of the internal efforts which do not involve the collapse of the structure as long as these kneecaps are still able to transmit shearing action due to the actual weights and the seismic action.
For this example, that is allowed for beams because the hinges break by rupture of the concrete in compression whereas the significant longitudinal reinforcement of the cross-piece remains intact.
On the other hand the rupture of the columns causes a fall force until 356 kN, which corresponds to 60% of the maximum force reached of 584kN. That is would be Not acceptable in base themselves on the criterion of rupture .

Analysis of Upgraded Structure:
Addition of RC Structural Walls:



Adding structural walls is one of the most conventional structure system-level rehabilitation methods to strengthen existing structures.
This approach is effective in controlling global lateral drifts and in reducing damage in frame members. Generally, repair of an existing shear wall or infilling one of the bays in the frame structure is used. In order to save time, cost and ensure the quality, panels can be prefabricated and assembled in-site.
Many researches into structural walls have been conducted, and findings corresponding to detailed rehabilitation have been reported (Bai and Hueste 2003). Addition of new shear walls will normally result in better seismic performance.
However, its side-effect on the existing structure and it's relatively intrusive and disrupt construction style are disadvantages. Previous research shows that the infilling process tends to stiffen the existing structure such that the base shear can increase accordingly. The overturning effects and base shear are concentrated at the stiffer infill locations. Along with the addition of the weight, in turn the gravity load effect, gives more pressure to the existing foundation. Therefore, strengthening of the existing foundation is usually required when this
Technique is used, while commonly upgrading existing foundations under the new walls is expensive.

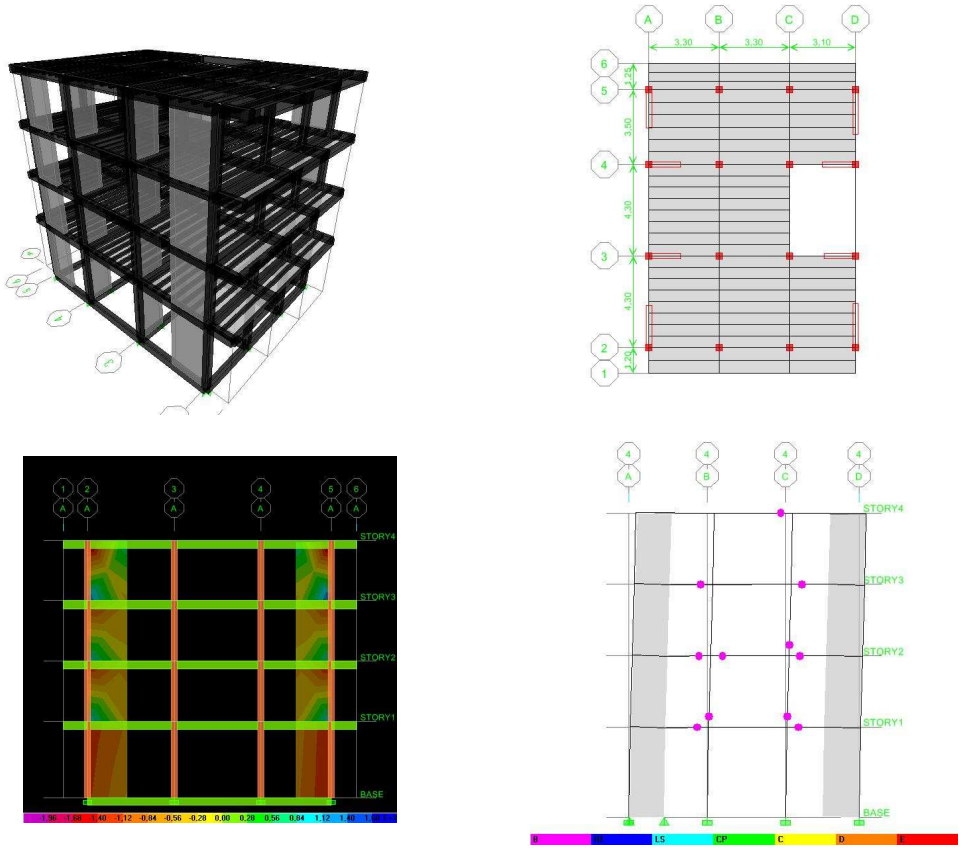


Fig.6: Results of the structure with additional walls

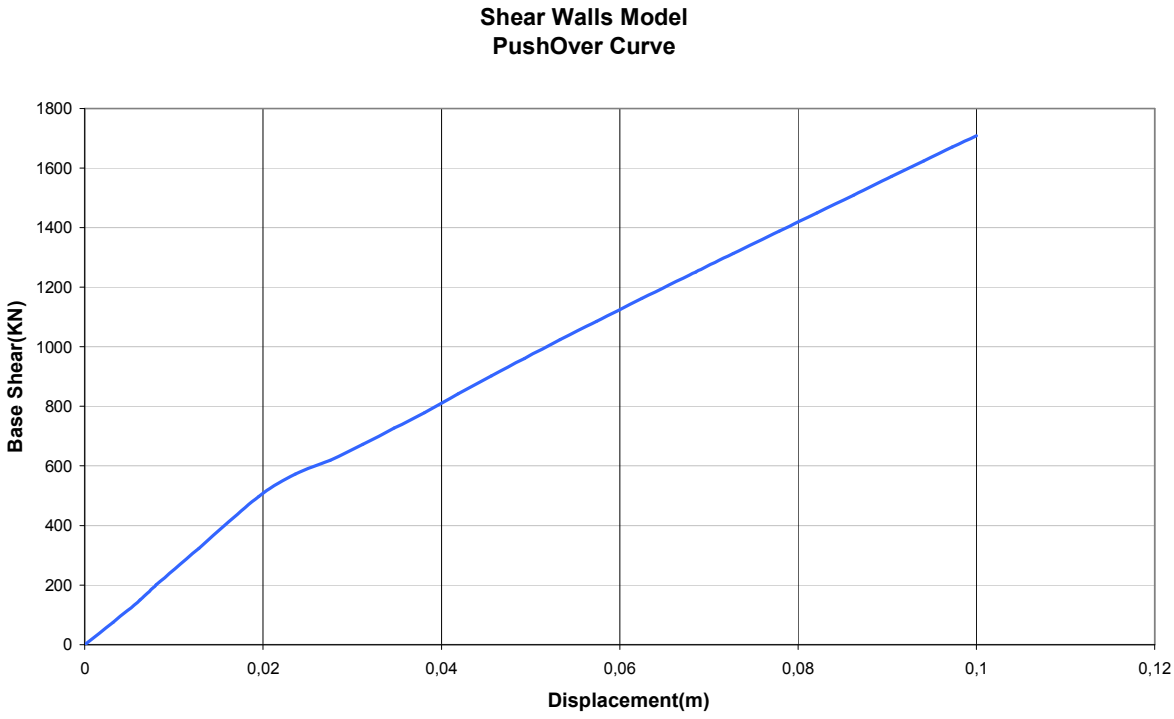


Fig.7

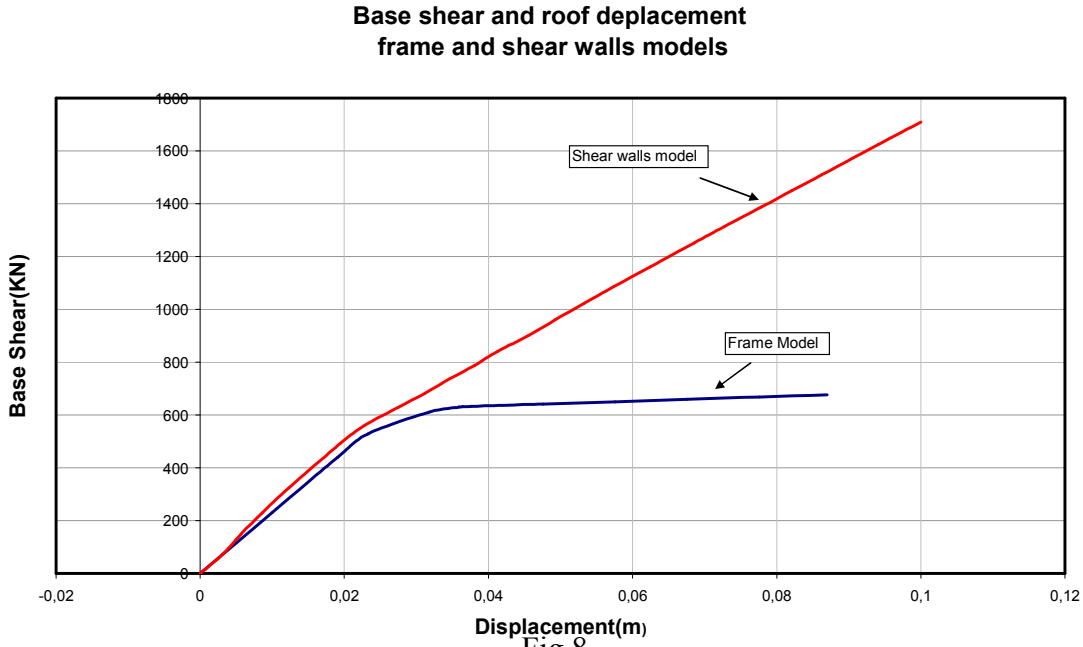


Fig.8

Table.4: Pushover analysis results of the basic and upgraded models

Model type	Base Shear KN	1st Floor dispt cm	4th Floor Dispt cm	Interstorey Drift ratio 1st floor %	Interstorey Drift ratio 1st floor %
Basic Model	584	8,4	9,8	2,2	0,08
Shear Walls Model	1388	0,67	2,95	0,17	0,22

Conclusion:

The method based on the deformations allows to determine the seismic behaviour of the structures of one more realistic manner. For existing buildings, it leads in general to an appreciation of safety parasismic more favourable.

we note that in the basic structure (frames) the columns of the first floor have undergoes a significant displacement and damage endangering the stability of the structure for a coefficient of acceleration 0,25g on the other hand these displacements would be moderate if the acceleration were less, However the upgraded structure exhibited a full elastic behaviour;

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