RESEARCH REPORT

VTT-R-00988-11

Application of PBD provisions for the r.c. benchmark structure

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Summary

This report is part of the project STEELRETRO, which aim was to develop seismic retrofitting solutions for a reinforced concrete building. This work is based on earlier studies: "*LGS steel shear wall system to retrofit or upgrade r.c. frame structures*" (VTT-R-04770-09) and "*Details of retrofit solution using LGS shear walls and column bracketing*" (VTT-R-00987-11)

The previous reports explored the possibility of using LGS shear walls for the retrofitting of the r.c. benchmark structure used in the STEELRETRO project. The first report was focused on analytical studies of the performance enhancements given by the LGS-SW's. The second report extended the depth of the analytical studies, and added important data on the possible detailing of the LGS-SW solution (particularly the connection details between the SW's and the r.c. frame). The second report also contained cost estimates of the proposed SW solutions.

In this report the LGS-SW solutions performance contribution is discussed in the light of the PBD methodology proposed in WP2 of the STEELRETRO project. The intention here was to (1) apply the WP2 procedure as strictly as possible, (2) to report the performances of the SW retrofit solution in PBD terminology, (3) to discuss the difficulties of using the methodology and (4) to propose solutions for those difficulties to designers attempting to use the PBD recommendations.

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Preface

The report is part of the project STEELRETRO, Work Package 3. The overall aim of the STEELRETRO project is to *“set up steel solutions for the seismic retrofit of existing buildings, furnishing design and construction methodologies, tools for dimensioning of elements and connections as well as for cost estimation”*.

Specifically, Work Package 3 aims at analyzing and designing *“steel solutions to retrofit or upgrade vertical systems of existing reinforced concrete building, in terms of strength or stiffness, by means of steel concentric bracing systems, steel eccentric bracing systems or shear steel/composite walls.”* and *“steel solutions to retrofit or upgrade vertical systems of existing masonry building coupling the existing structure with new a steel structure or with a bracing systems.”*

WP 3 also aims at analysis and design of *“steel solutions to retrofit or upgrade vertical systems of existing reinforced concrete building and masonry buildings, in terms of ductility by the application of dissipative steel systems and in particular by eccentric steel bracings, steel shear panels/walls and BRB (buckling restrained brace) systems.”*

Within WP 3, VTT had the role of analysing ”*possible solutions using light gauge steel shear walls*” for the R.C. frame structures.

This study is continuation of previous studies on seismic retrofitting of reinforced concrete buildings by LGS elements. Numerical analyses (finite element method) are performed using a benchmark building modelled in SeismoStruct. The original geometry and loading of the benchmark building is presented in an earlier report (Fülöp, 2010). Focus of this report is retrofitting by light gauge steel plate shear walls and/or steel jacketing. First, SeismoStruct modelling is validated by comparing analysis models to available literature studies. Second, performance of several retrofitting techniques is tested on the benchmark building. Last, principles of detailing of the proposed retrofit solutions are given and discussed.

Authors

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Abbreviations

SW – shear wall
LGS – light gauge steel
PBD – performance based design
PB – performance based
r.c. – reinforced concrete

1 Introduction

The aim of this document is to summarize and extend the lessons learned from the modeling of the r.c. benchmark building used for the STEELRETRO project [1], with special focus on the LGS based SW retrofit intervention, to improve the earthquake performance of the structure. Very detailed modeling and analysis has already been reported for this building by Fülöp [2] and Mielonen [3]. These modeling results are extensively references here.

The main conclusions of the previous studies [2] [3], was that that study was that the r.c. benchmark is very deficient in terms of earthquake performance targets. Several retrofitting strategies have been tried on the building; the conclusion being that only one of them has real potential to improve the earthquake performance drastically, namely the addition of parallel load bearing structure, which is much stiffer then the r.c. frame itself. It was suggested to use light-gauge steel (LGS) shear walls for the purpose.

The current work extends the previous study in a few crucial areas, which were judged to be the weaknesses of the previously reported study:

- more focus is given to the local phenomena (e.g. strength hierarchy of components and failure modes)
- the shear capacity of members is included in the assessment
- the use of performance based design (PBD) procedure is more strongly emphasized

In particular, WP2 of the STEELRETRO project outlined a PDB procedure to be followed in the evaluation of the performance of the retrofit techniques [4], [5]. The application of this methodology is reported here.

2 Previous modeling and results

Two configurations of the r.c. benchmark structure are studied here. The first one is the original, unretrofited structure. The model for this structure is identical to the ones reported by Fulop [2] with one difference. The initial model was not taking into account the flexibility of the foundations. As a simplification, the structure was considered fixed in the Y direction and pinned in the X direction. In the model analyzed here, the flexibility of the foundations, provided by CERI within the STEELRETRO project, and already reported by Mielonen [3] has been used (Table 1).

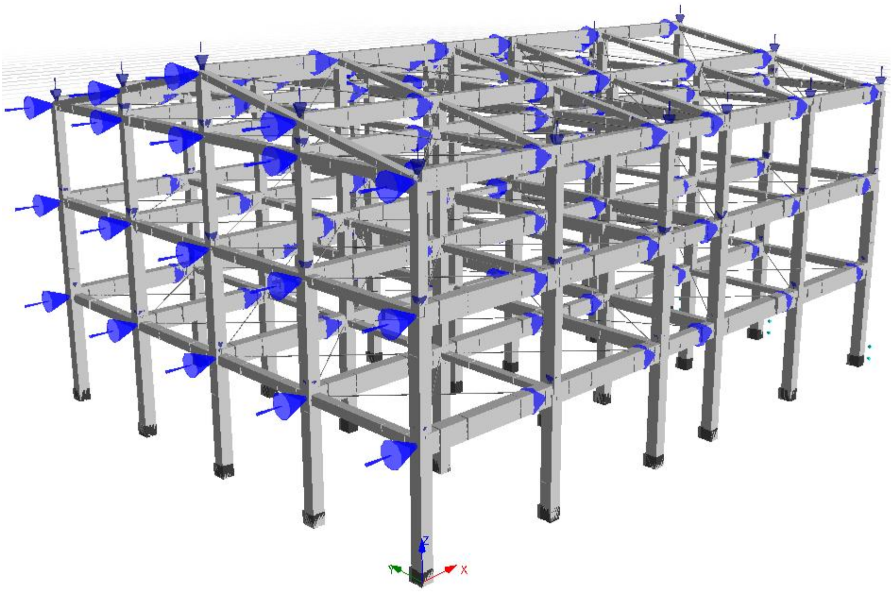
Table 1. Spring properties for foundations.

Spring type	K_x (kN/m)	K_y (kN/m)	K_z (kN/m)	$K_{\theta x}$ (kNm/rad)	$K_{\theta y}$ (kNm/rad)	$K_{\theta z}$ (kNm/rad)
T11	2740	9570	217330	49900	950	2400
T21	36350	11840	393460	89631	7920	27030
T13	34540	47730	478340	128830	61730	27100

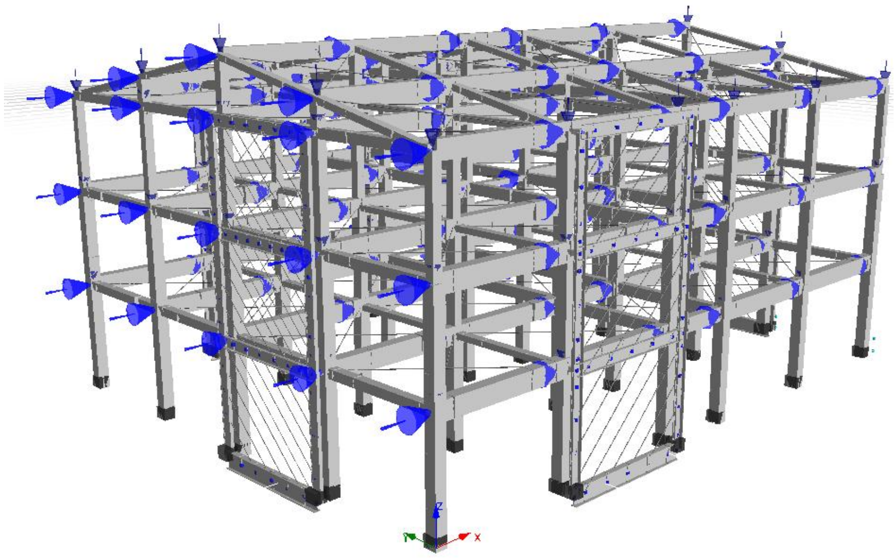
The use of these spring constants resulted in a slight change of the behavior of the structure. Firstly, under pushover loads there is a sliding between the “ground” and the “foundations” due to the inserted flexibility, resulting in a small decrease of the stiffness of the building. Secondly, in the Y direction, the fixed foundations became flexible with the stiffness constant given in Table 1 ($K_{\theta x}$=49900kNm/rad), resulting in a reduction of the bending moment at the base of the columns in Y direction pushover. And thirdly, the pinned bases of the frames in the X direction, became partially fixed with the stiffness from Table 1 ($K_{\theta y}$=950kNm/rad). This results in the attracting of some bending to the base of the columns in X direction pushover and the increase of the base shear. The rotational stiffness is not very large, and plastic hinges are not forming at the base of columns, an the failure mode remained the soft storey mechanism reported by Fulop [2], but at a slightly increased force level.

The LGS-SW retrofitted structure corresponds to the model described by Mielonen [3], and it was using the UPE boundary frame for the SW, and the SW was considered perforated using a perforation ratio of 0.71, which corresponds to an equivalent reduction of the SW thickness by 50.3%. The pushover curves for the retrofitted structure are identical to the ones reported by Mielonen [3] for this configuration.

The general view of the two models in SEISMOSTRUCT is presented in Figure 1, with X direction pushover loads emphasized.



a)



b)

Figure 1.The SEISMOSTRUCT model of the two analysed configurations.

3 Detailed analysis of the modeling results

In the previous study it has been shown [2], using a global model of the structure, that major deficiencies of the structure exist from the point of view of the required earthquake performance. In this paragraph, these problems are highlighted and further explored.

It has to be mentioned that the modeling of the structure was done using the characteristic properties of the concrete and of the reinforcing steel. These were according to the definition of the structure [1]: R_{ck}=20MPa for the concrete and f_y=230MPa for the reinforcing bars.

While the pushover models were using characteristic material properties, some of the below analytical checks will be done using the design properties of the materials. Therefore, below the values of $R_{ck,\ Rd}=R_{ck}/\gamma_C=20/1.5 = 13.33$MPa and $f_{y,Rd}=f_y/\gamma_S=230/1.15=200$MPa are used. When other properties of the concrete and steel are required, they are assimilated from properties of concrete with f_{ck}=20MPa (see Table 3.1 in prEN 1992 [6]).

3.1 Strong beam weak column in the X direction of the frame

One of the identified deficiencies of the structure is that it forms a ground floor soft-storey failure mechanism under X direction pushover [2]. It has been suggested that this happens because in this direction there is a weak column, strong beam configuration; which is unacceptable in case of earthquake design [7]. This aspect of the X direction behavior is further studied below.

In Figure 2 a cutout of the X axis middle frame is given. In the columns of this frame the values of axial forces are the highest. Large values of the axial force also increase the bending capacity of the columns; hence the largest bending capacities of the columns can be found in this frame.

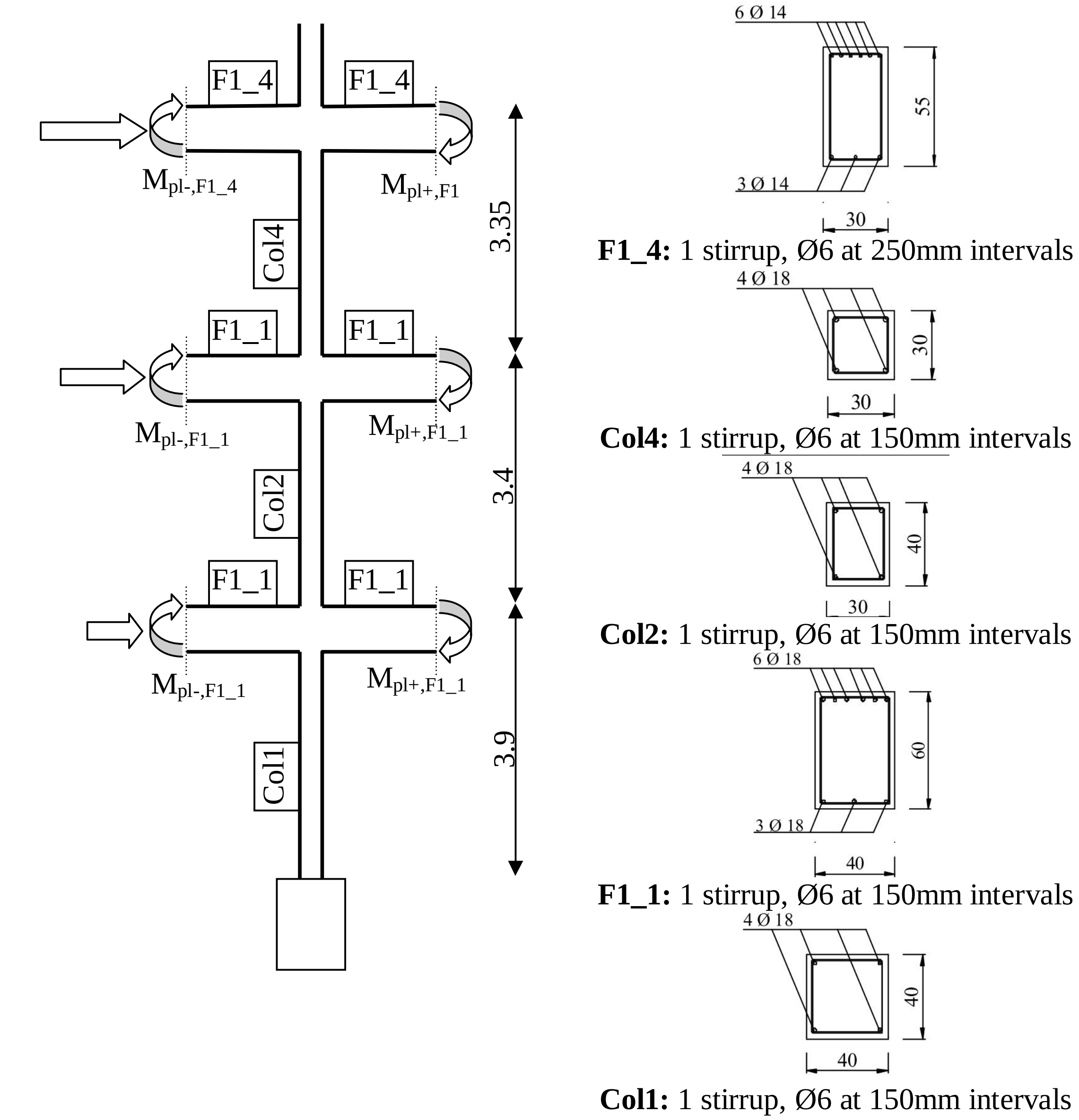


Figure 2. Column extracted from X direction frame (Axes 2-C)

The axial forces in the columns were determined from vertical forces acting in the earthquake combination and they are: N_{Col1}=596.8kN, N_{Col2}=398.4kN, N_{Col4}=194.5kN (see Table 2 for details on other columns).

Table 2. Typology and loads on the columns of the rc structure

a) Ground floor: (1) type of column and (2) axial force in earthquake combination (kN)

			X					
	Axes		**1**	**2**	**3**	**4**	**5**	**6**
		(m)	**0**	**5**	**10**	**13**	**18**	**23**
Y	**A**	**0**	Col1	Col1	Col1	Col1	Col1	Col1
	B	**4.5**	Col1	Col1	Col1	Col1	Col1	Col1
	C	**9**	Col1	Col1	Col1	Col1	Col1	Col1
	D	**13.5**	Col1	Col1	Col1	Col1	Col1	Col1
	E	**18**	Col1	Col1	Col1	Col1	Col1	Col1

			X					
	Axes		**1**	**2**	**3**	**4**	**5**	**6**
		(m)	**0**	**5**	**10**	**13**	**18**	**23**
Y	**A**	**0**	291.3	461.6	356.5	356.5	461.6	291.3
	B	**4.5**	418.0	597.0	467.2	467.2	597.0	418.0
	C	**9**	424.9	***596.8***	459.8	459.8	596.7	424.9
	D	**13.5**	417.8	598.8	459.7	459.7	598.8	417.7
	E	**18**	291.3	461.7	356.4	356.4	461.7	291.3

b) 1st floor columns: (1) type of column and (2) axial force in earthquake combination (kN)

			X					
	Axes		**1**	**2**	**3**	**4**	**5**	**6**
		(m)	**0**	**5**	**10**	**13**	**18**	**23**
Y	**A**	**0**	Col1	Col2	Col1	Col1	Col2	Col1
	B	**4.5**	Col1	Col2	Col1	Col1	Col2	Col1
	C	**9**	Col1	Col2	Col1	Col1	Col2	Col1
	D	**13.5**	Col1	Col2	Col1	Col1	Col2	Col1
	E	**18**	Col1	Col2	Col1	Col1	Col2	Col1

			X					
	Axes		**1**	**2**	**3**	**4**	**5**	**6**
		(m)	**0**	**5**	**10**	**13**	**18**	**23**
Y	**A**	**0**	181.1	295.7	224.9	224.6	297.2	179.8
	B	**4.5**	272.9	397.9	309.5	309.5	398.0	272.9
	C	**9**	274.4	***398.4***	297.1	297.1	398.4	274.3
	D	**13.5**	272.6	399.8	305.0	305.0	399.8	272.6
	E	**18**	179.8	297.2	224.6	224.6	297.2	179.8

c) 2nd floor columns: (1) type of column and (2) axial force in earthquake combination (kN)

			X					
	Axes		**1**	**2**	**3**	**4**	**5**	**6**
		(m)	**0**	**5**	**10**	**13**	**18**	**23**
Y	**A**	**0**	Col3	Col4	Col3	Col3	Col4	Col3
	B	**4.5**	Col3	Col4	Col3	Col3	Col4	Col3
	C	**9**	Col3	Col4	Col3	Col3	Col4	Col3
	D	**13.5**	Col3	Col4	Col3	Col3	Col4	Col3
	E	**18**	Col3	Col4	Col3	Col3	Col4	Col3

			X					
	Axes		**1**	**2**	**3**	**4**	**5**	**6**
		(m)	**0**	**5**	**10**	**13**	**18**	**23**
Y	**A**	**0**	74.9	130.6	96.4	96.1	131.7	74.0
	B	**4.5**	132.7	195.5	153.7	153.7	195.6	132.7
	C	**9**	134.8	***194.5***	146.5	146.5	194.5	134.8
	D	**13.5**	132.7	195.6	153.7	153.7	195.6	132.7
	E	**18**	73.9	131.7	96.1	96.1	131.7	73.9

3.1.1 Bending capacity of members (Example ground floor Col1)

In the first step Col1 was analyzed using the nominal characteristics of the material: fy=230 MPa, and Rc=20 MPa. The maximum bending capacity is calculated using the bellow formula:

$$M_{cap} = b \cdot x \cdot R_c \cdot \left(h_0 - \frac{1}{2}x\right) + A_a{}' \cdot R_a \cdot h_a - N \cdot \left(\frac{1}{2}h - a\right) \qquad 1$$

Where: M_{cap} - bending capacity of the cross-section
b - width of the cross-section
x - depth of the concrete in compression
R_c - compressive resistance of the concrete
h_0 - distance from the tension reinforcement to the compressed side of the cross-section

A_a’ - reinforcement area in the compressed region of the cross-section
R_a - tensile capacity of the reinforcing steel

h_a - lever/distance between the compressed and tension reinforcement
N - axial force
h - height of the cross-section
a - concrete cover up to the centre of the longitudinal reinforcement

As shown in Figure 2, the parameters for Col1 from the ground floor of Ax 2-C are b=400mm, R_c=20N/mm^2,.h_0=375mm, A_a'=509mm^2, R_a=f_y=230N/mm^2, h_a=350mm, N=596.8kN, h=400mm, a=25mm. The compressed part of the concrete is calculated from the force equilibrium condition as:

$$x = \frac{(A_a - A_a')\cdot R_a + N}{b \cdot R_c} \qquad 2$$

Where: A_a – reinforcement area in the tension side of the section

In this case A_a=A_a'=509mm^2, and hence x=74.6mm. Therefore, the moment capacity is $M_{characteristic,cap}$=138kNm, when using the nominal values of the material properties; while it is $M_{design,cap}$=121kNm when applying the safety factors.

If the above procedure is applied with different axial force values, the M-N interaction curve of the Col1 section can be drawn. The M-N interaction curve for Col1 is presented in Figure 3 for nominal material properties. The interaction curve was calculated with: (1) the software application INCA [9] – blue line, (2) the analytical procedure presented above and based on Cadar et al. [8]– cyan line, (3)and using a simplifies beam of SEISMOSTRUCT [10]. It can be seen that all methods are in good agreement with each other. The above particular values i.e. N=596.8kN, M_{cap}=138kNm also fit well in the graph, showing that both the analytical method and the SEISMOSTRUCT model is predicting the bending accurately.

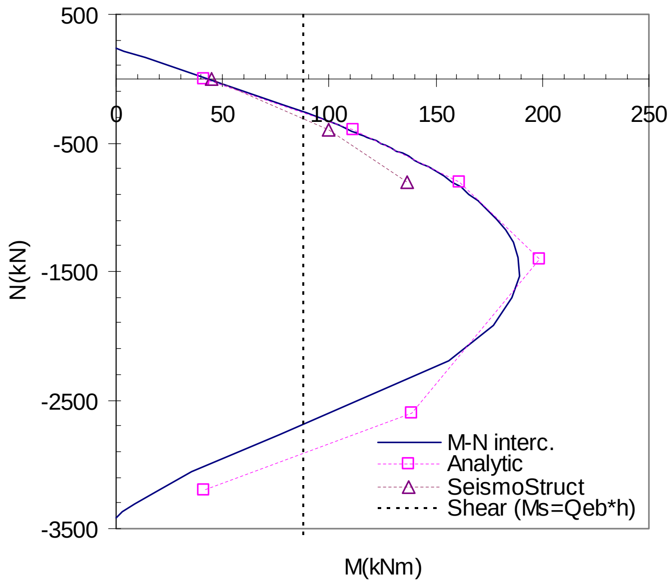


Figure 3. M-N interaction curve of Col1 cross-section using nominal strength for concrete and steel

To investigate the strong beam weak column problem in the X direction, three types of nodes have been extracted from the X direction middle frame (Figure 4), and the moment equilibrium has been written, when plastic mechanism is formed under lateral loads. The design values of the material properties were used in the checks below.

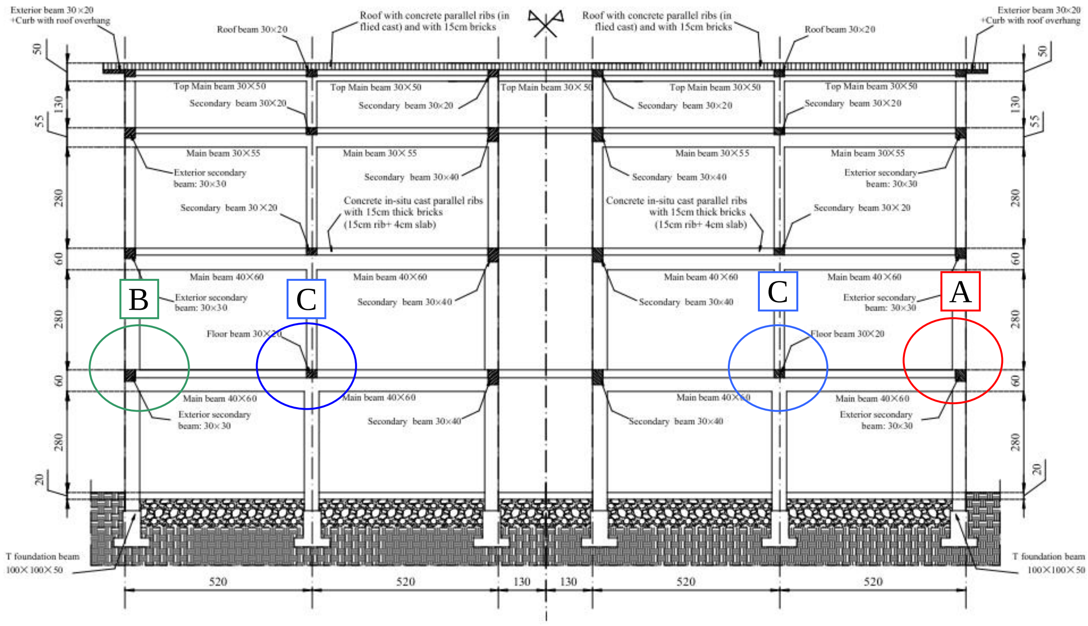


Figure 4. Ground floor nodes extracted from X direction frame

Lateral nodes in the X direction (Type A and B in Figure 4) has the shape presented in Figure 5.a and b. In the columns there is axial force present (Table 2); N_{Col1}=424.9kN and N_{Col1}=274.4kN. In this case $M_{pl,\ Col1}$=103.69kNm and $M_{pl,\ Col1}$=83.44kNm. The bending capacity of the beams, considering zero axial force is: $M_{pl+,F1\_1}$=167.95kNm and $M_{pl-,F1\_1}$=83.97kNm. In positive bending (Type A): ΣM_{Col}=187.13kNm < ΣM_{Beam}=1.3·167.95kNm, violating the provisions of §4.4.2.3 of EN1998 [7]; $M_{Rc}>1.3M_{Rb}$. In negative bending (Type B): ΣM_{Col}=187.13kNm > ΣM_{Beam}=1.3·83.97kNm so the node fulfills the strong column weak beam requirements of EN 1998.

The middle first floor node in the X direction (Type C) there is the arrangement from Figure 5.c. In the columns there is also axial force present (Table 2); N_{Col1}=596.8kN and N_{Col2}=398.4kN. Applying the analytical procedure, the plastic moments can be calculated as $M_{pl,\ Col1}$=121.61kNm and $M_{pl,\ Col2}$=70.33kNm. The bending capacity of the beams, considering zero axial force in the beam is: $M_{pl+,F1\_1}$=167.95kNm and $M_{pl-,F1\_1}$=83.97kNm. Therefore in this node we have ΣM_{Col}=191.94kNm< ΣM_{Beam}=251.92kNm, violating the provisos of §4.4.2.3 of EN1998 [7]; $M_{Rc}>1.3M_{Rb}$.

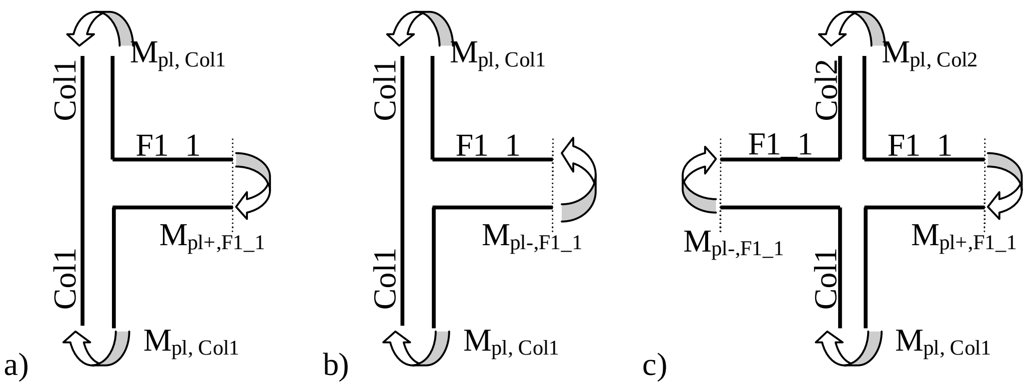


Figure 5. Strong beam weak column behavior on first floor X direction

The two other nodes from the ground floor can be assimilated to Type A and Type B, with the observation that an extra, weak beam deteriorate their performance. In the X direction we have a weak column strong beam configuration.

It can be concluded that, with the exception of Type B node in negative bending, all nodes from the first floor will develop plastic hinge in the columns rather then, as desired, in the beam. This will push the structure to develop, the disadvantageous, soft storey mechanism on the ground floor. This behavior has been observed in the global models.

3.2 Shear of the r.c. members

The occurrence of shear failure in r.c. columns during earthquakes is a well known problem with older structures. This happens mainly because: (1) in the past the importance of the shear reinforcement in the plastic hinge region of the members was not well understood, and most members were under-reinforced in shear and (2) quite often partitions, not taken into account in the structural analysis, are creating short columns where the shear force strongly increases compared to the value considered by the designer who was analyzing the member as if is in bending over the full length. Two cases of such shear failure are presented in Figure 6.

a) b)

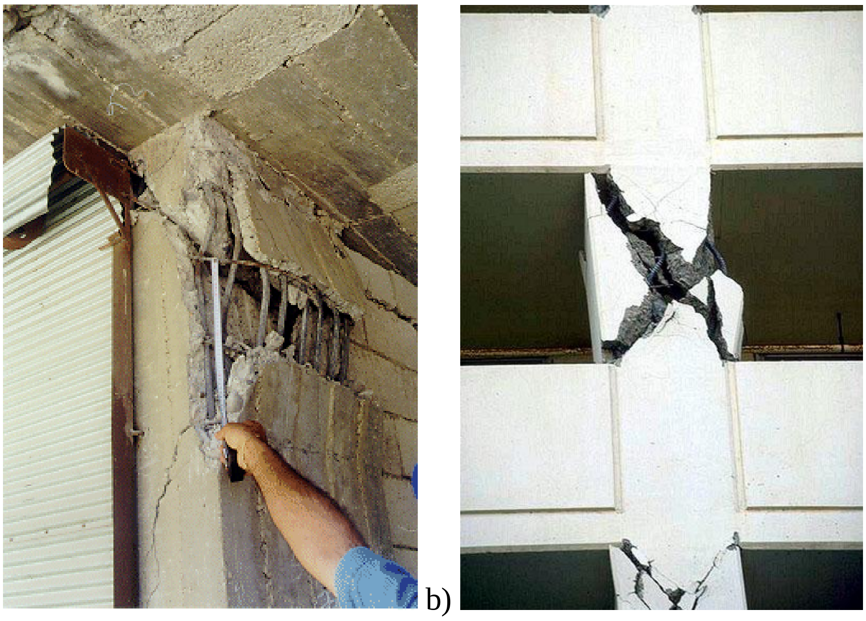
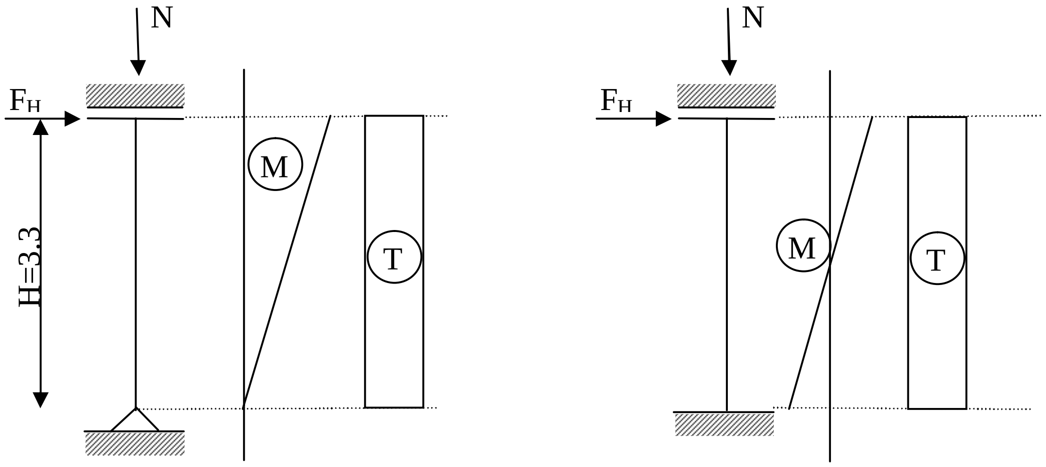


Figure 6. Shear failure of columns due to (a) inadequate spacing of shear reinforcement, (b) short-column effect

The effect of shear versus bending was studied here on two simplified models of the ground floor columns of the structure, the ones most prone to shear failure (Figure 7). The first one represents the typical base column, when the structure is loaded in direction X, the second a ground floor column when the structure is loaded in the Y direction. The flexibilities of the foundations were disregarded in this simplified analysis.



(a) ground floor column *(b) intermediate storey column*

Figure 7. Column model of X direction frame (Axes 2-C)

Using the simplified models from Figure 7, the shear force demand can be evaluated when the columns create plastic hinges at the ends. With the first model we have: $M_{max}=F_h \cdot H$, $T=F_h$, while with the second $M_{max}= F_h \cdot H/2$. In the pinned base case (direction X loading), with Col1 we have T_{x_Col1}=41.8kN shear force present in the column, when the plastic hinge forms. In the fixed base case (Y direction) T_{y_Col1}=83.63kN.

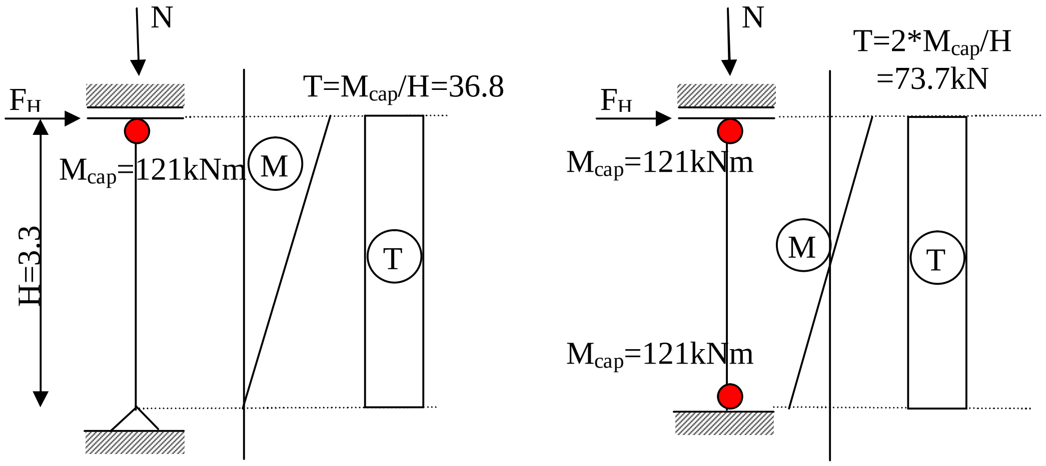


Figure 8. Model of base floor column Col1 with plastic hinges

The shear force capacity of the cross-section Col1 must also be calculated. This is not straightforward because the shear capacity greatly decreases with the rotation of the plastic hinges during cyclic loading. Therefore different analytical models predict substantially different values for the shear strength.

The shear capacity of Col1 was evaluated using an analytical model suggested in by Cadar [8], Eurocode 1992-1-1 [6], FEMA 356 [11], FEMA 273 [12] and AIC 318 [13].

3.2.1 Shear strength prediction by Cadar [8]

The analytical model is based on the same mechanical model to predict the shear capacity as the one in EN 1992-1-1[6], with the following presumptions:

- the concrete does not contribute to the shear capacity
- rotation equilibrium is expressed about point O (Figure 9)

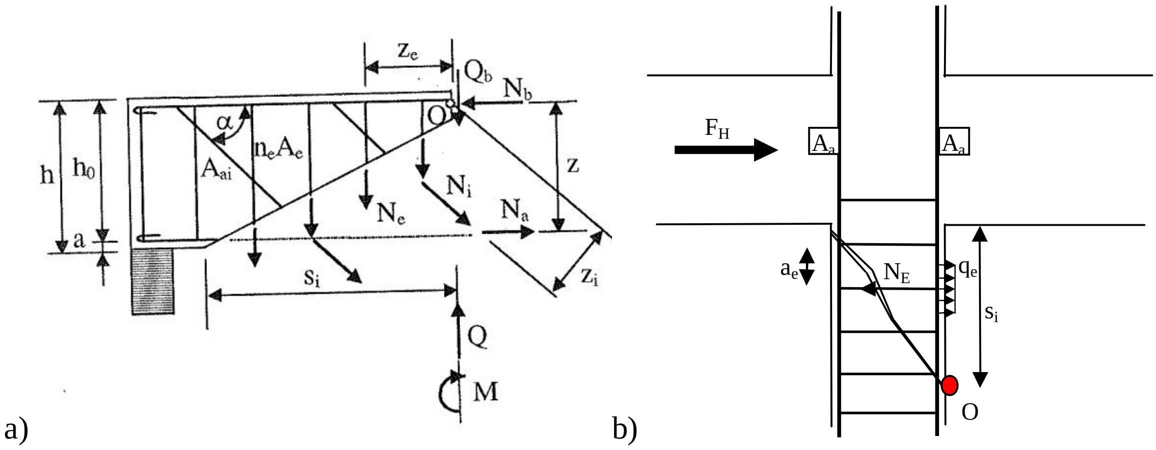


Figure 9. Mechanical model for the calculation of shear strength (a) generic, (b) for Col1

The axial capacity N_E of the transverse reinforcements can be calculated as:

$$N_E = n_e \cdot A_e \cdot m_{at} \cdot R_a \quad (3)$$

Where: n_e – the number of bars in one stirrup

A_e - the cross-section area of a stirrup bar (Ø6 in this case)

m_{at} – reduction coefficient taking into account that not all stirrups within the length of the crack are fully effective; only the ones where the crack opening is large enough can be considered fully (recommended m_{at}=0.8)

R_a – the strength of the reinforcement (equal to the yield stress for calculations using nominal properties)

In case of Col1, we have: n_e=2, A_e=28mm^2, m_{at}=0.8, R_a=230N/mm^2, hence N_E=10.4kN. Therefore, the equivalent distributed load q_e=N_E/a_e=69N/mm.

The calculation of the length of the crack can be done by:

$$s_i = \sqrt{\frac{b \cdot h_0^{\,2} \cdot \sqrt{p}}{q_e} \cdot m_t \cdot R_t} \quad (4)$$

Where: si – the length of the crack

p – the reinforcing ratio on the tension side p=A_a/(b*h)

m_t – coefficient affecting the ability of the concrete to resist shear due to compression (=1)

R_t – tension strength of the concrete (taken R_t=$f_{ctk,0.005}$=1.8N/mm^2 from Table 3.1 of [6])

It results that in case of Col1 the length of the crack is s_i=307mm. This is within the limits $0.5 \cdot h_0 < s_i < 2.5 \cdot h_0$, prescribed by the same reference [8]. The shear force resisted by the cross-section of Col1, including concrete and stirrups is:

$$Q_{eb} = 2 \cdot \sqrt{b \cdot h_0^{\,2} \cdot m_t \cdot R_t \cdot q_e \cdot \sqrt{p}} - n_e \cdot A_e \cdot m_{at} \cdot R_t \quad (5)$$

Where: Qeb – the shear capacity

Using the values: b=400mm, h_0=375mm, m_t=1, q_e=69N/mm^2, p=0.0042, n_e=2, A_e=28mm^2, m_{at}=0.8, R_t=1.8N/mm^2; the shear capacity is Q_{eb}=32.2kN. The full calculation is summarized in

Table 3. Shear strength of Col1 according to Cadar et al.

		Nominal values	Design values
	γ_s		1.15
	γ_c		1.5
Longitudinal reinforcements	fy (N/mm2)	230	200
	Bars	4	
	Bars in tension	2	
	Diameter (mm)	20	
	Area (mm2)	314.2	
	Reinforcing ratio ρ (%)	0.42%	
Shear reinforcement	Nr. of bars	2	
	Diameter (mm)	6	
	Spacing (mm)	150	
	Area (Av)	56.55	
	Reinforcing ratio ρw (%)	0.09%	
Concrete properties		Rtk	Rt*
	Tensile resistence of concrete (Cadar et al.)	1.65	1.1
		fctk,0.05	fctd
	Tensile resistence of concrete (EN 1992-1-1)	1.8	1.2
	mt	1	1
	mat	0.8	0.8
	Ne (N)	10405.0	9047.8
	qe (N/mm)	69.4	60.3
	si (mm)	307.4	269.1
	Qb (N)	21320	16233
	Qeb (N)	**32236**	**23418**

3.2.2 Shear strength according Eurocode 1992-1-1 [6]

EN 1998 recommends in §5.4.3.1.1(1) the calculation of the shear strength according to EN 1992-1-1:2004. EN 1992-1-1 provides formulas to calculate the shear strength of the member in chapter 6.2. If there is need for shear reinforcement (which it is in this case), it is recommended to calculate the shear capacity of the member also only based on the capacity of the reinforcements with:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \qquad (6)$$

Where: A_{sw} – the shear reinforcement area
s – the distance between shear reinforcements
z – the distance between the longitudinal reinforcements
f_{ywd} – the yiels stress of the shear reinforcement
cot(θ) – can be chosen between 1 and 2.5 and it depends on the angle of the shear cracks. In the case of the lightly reinforced column Col1 this angle should be about 45º.

It is worth noting that the mechanical model behind this formulation is identical to the Cadar [8] model.

The predicted shear strength for Col1 is presented in Table 4. The predicted shear capacity is close to the previous one, 30348N using nominal properties and 26386N using design properties.

Table 4. Shear capacity of Col1 according to EN 1992-1-1

	Nominal values		Design Values
A_{sw}(mm2)		56.55	
s (mm)		150	
z (mm)		350	
fywd (N/mm2)	230		200
cotθ		1	
θ (deg)		45.00	
$V_{Rd,s}$ (N)	**30348**		**26389**
$V_{Rd,s,min}$ (N)	30348		26389
$V_{Rd,s,max}$ (N)	75869		65973

3.2.3 Shear capacity according to FEMA 273 [12], AIC 318-2002 and FEMA 356 [11]

According to FEMA 273 [12] the shear capacity can be evaluated using the formula:

$$V_c = 3.5\lambda\left(k + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}\, b_w d \tag{7}$$

Where k – is 1 for regions of low ductility and 0 for regions of high ductility
λ – is 0.75 for lightweight aggregates and 1 for normal aggregates
Nu – is the axial force
f'c – is the compressive strength of the concrete
Ag – the cross section area of the element
bw – the width of the column
d – the effective height of the column (equivalent to h_0 from the previous methods)

The formulation in AIC 318-2002 is based on the summation of two terms, one corresponding to the shear capacity of the concrete and the second to the shear capacity of the shear reinforcements:

$$V_n = V_c + V_s \tag{8}$$

Where:

$$V_c = 0.166\left(1 \frac{P}{13.8A_g}\right)\sqrt{f'_c}\, bd \tag{9}$$

and:

$$V_s = \frac{A_u f_y d}{s} \tag{10}$$

It can be observed that the expression of V_s is essentially the formula given in EN 1992-1-1 [6]. Therefore, it can be expected that AIC 318-2002 will predict larger capacity then EN 1992-1-1.

The more recent FEMA 356 [11] formulation presented below:

$$V_c = \lambda \cdot k\left(\frac{6\cdot\sqrt{f'_c}}{\frac{M}{V\cdot d}}\cdot\sqrt{1+\frac{N_u}{6\cdot\sqrt{f'_c}\cdot A_g}}\right)\cdot 0.8\cdot b_w \cdot h \tag{11}$$

Where:

k = 0.7 corresponding to regions with high ductility demand
λ = 1 for concrete with normal aggregates
f'c = 2900.75psi (20N/mm^2)
M/V = 2…3
d = 0.8*h=14.8in (375mm) the effective depth of the cross-section (CS)
h = 15.75in (400mm) the depth of the cross-section (in the direction of the shear)
b_w = the width of the CS's web
A_g = 248 in^2 (16000mm^2) the gross area of the cross-section
N_u = 134885 pound force (600 000N) axial force

The dimensions of the cross-section for Col1 and the predicted shear capacity values, using the above three formulations, are summarized in Table 5 and Table 6.

Table 5. Dimensions of the cross-section of Col1n (SI and US units)

		SI units	**US units**	
	h	400	15.74803	mm, inch
	bw	400	15.74803	mm, inch
Concrete dimensions & properties	fc	20	2900.755	N/mm2, psi
	Nu	600000	134885.4	N, pound
	Cover	25	0.984252	mm, inch
	Diameter	6	0.23622	mm, inch
	Nr. of bars	2	2	
	Area (Av)	56.55	0.087651	mm2, inch2
Shear reinforcement	Spacing(s)	150	5.905512	mm, inch
	ρw	0.000942478	0.000942	
		0.09%	0.09%	
	fy	230	33358.68	N/mm2, psi
	Neutral axis depth	75		mm, inch

Conversion: 1 N/mm² = 145,07 PSI 1 inch = 25,4 mm
1 PSI = 0,00689 N/mm² 1 mm = 0,03937 inch
1N = 0.224808943 pound force, 1 pound force = 4.44822162N

Table 6. Predicted shear capacity (FEMA 356, AIC 318-2002, FEMA 273)

	FEMA 356 - 2000	**AIC 318-2002**		**FEMA 273 - 1997**	
	US units	SI units	US units	SI units	US units
k	0.7			0	0
λ	1			1	1
h	15.74803148	400	15.74803	400	15.74803
bw	15.74803148	400	15.74803	400	15.74803
d	14.76377951	375	14.76378	375	14.76378
Ag	248.0004955	160000	248.0005	160000	248.0005
Nu	134885.3658	600000	134885.4	600000	134885.4
M/(V*d)	3				
	with nominal material properties				
fc	2900.754755	20	2900.755	20	2900.755
Vc (psi)	24504		31855		11919
Vc (N)		141616		52864	
Vs (psi)			7310		7310
Vs (N)		32515		32515	

Vn (N)	**109001**	**174132**	**85379**	**85533**

As it can be observed from the previous result, the predicted shear capacity values have a large scatter. This has to do with: (1) the way the resistance of concrete is taken into account, and (2) the level of bending ductility presumed to have been damaging the member. In case of the mechanical models specifically in EN 1992-1-1 [6] the effect of the concrete is not taken into account, hence these formulas provide a very low value of shear strength. The US formulations are based on experiments, and they do account the effect of concrete and stirrups. The AIC 318-2002 is giving the largest shear strength, while the other formulations acknowledge some reduction of the shear strength due to cyclic plasticization of the end of the member. Therefore, they predict smaller shear strength. A summary of the results is given in Table 7.

Table 7. Shear strength predictions for Col1 (nominal)

	Cadar et al	**EN 1992-1-1**	**FEMA 356 - 2000**	**AIC 318-2002**	**FEMA 273 - 1997**
	all with nominal properties				
h (mm)	400				
b_w (mm)	400				
Shear reinforcement	2xØ6/150 mm				
N_x (N)	600000				
$V_{concrete}$ (N)			109001	141616	52864
$V_{stirups}$ (N)	32236	30348		32515	32515
Vn (N)	**32236**	**30348**	**109001**	**174132**	**174132**

As a further test of the methods used above, the shear strength values predicted by EN 1992-1-1, FEMA 356 and AIC 318-2002 were compared to a set of test results reported by Sezen et al. [14] in Figure 10. It can be observed that AIC 318 provides the best prediction of the shear strength, while both FEMA 356 and EN 1992-1-1 are conservative. The strength predicted by FEMA 356 is systematically half compared to the one given by AIC 318. The results of EN 1992-1-1 are more dispersed, probably because in some cases the share of the concrete in resisting the shear is significant.

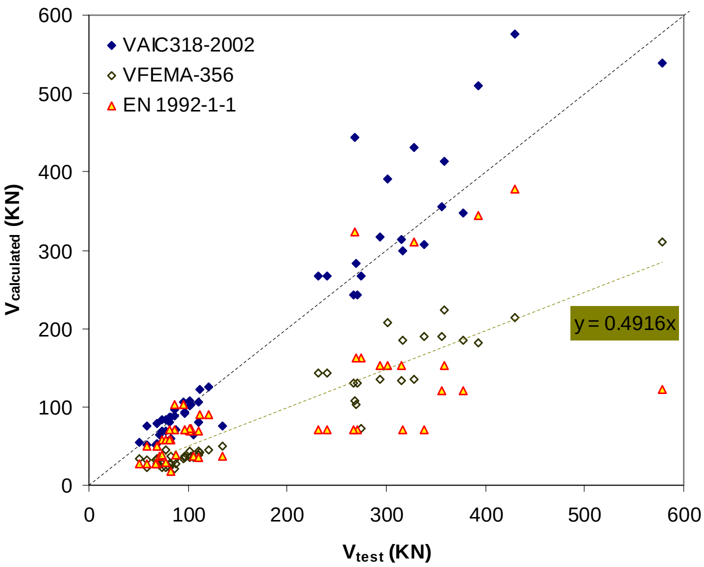


Figure 10. Predicted vs. experimental shear strength of lightly reinforced columns

The significance of the shear strength value can be understood if we analysis the model for columns on the ground floor (Figure 7.a). The horizontal force $F_H=V_{Rd,s}=26.39$kN, corresponding to the shear failure of the column in accordance to EN 1992-1-1 [6] also corresponds to a bending moment at the top of the column of $M=F_H \cdot H=30.34 \cdot 3.3=87.09$kNm. But, it has been shown that the design bending capacity of the column section is $M_{design,cap}=121$kNm, according to EN 1992-1-1 [6]. In other words, shear failure would precede bending failure according to EN 1922-1-1 [6].

This is represented graphically in Figure 11.a. The line corresponding to M=87.09kNm is drawn, and it can be seen that $M=87.09\text{kNm}<M_{cap}=121$kNm, needed to form a plastic hinge in bending at the top of the column. When calculations are carried out, using the design values of the material properties, the N-M curve for all columns Col1 indicate that almost all columns at the ground floor would fail in shear when the base is pinned (Figure 11.a), and they would all fail in shear by a large margin when the base if fixed (Figure 11.b).

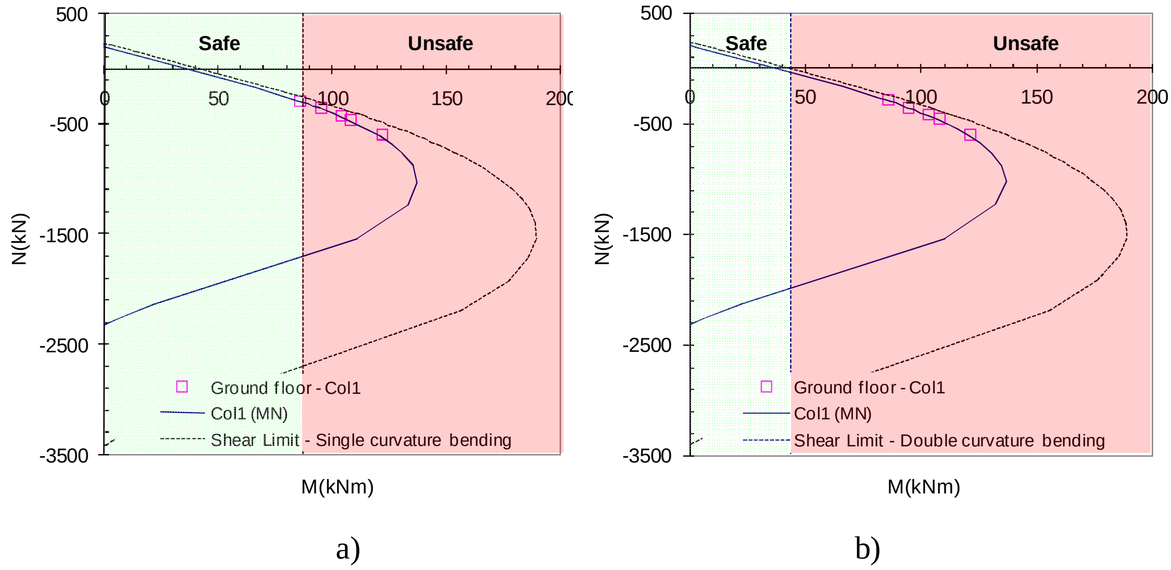


Figure 11. Cut-off given by shear capacity in the N-M interaction curve of Col1 (EN 1992-1-1)

The predicted behavior is better, with one column on the limit, if FEMA 356 [11] prediction is used for the shear capacity (Figure 12). It should also be noted that in Figure 12 the shear cut-off value is set to the FEMA predicted shear strength reduced by a safety factor of 1.5 (i.e. 109kN/1.5=72.7kN). The safety factor 1.5 was chosen somewhat randomly, considering that the bending safety factors are according to EN 1992-1-1, and the FEMA calculation was carried up to this point using nominal properties. (OBS: This kind of difficulties often result from the mixed use of design codes as proposed in the WP2 methodology of STEELRETRO [4]. See conclusions for more comments.)

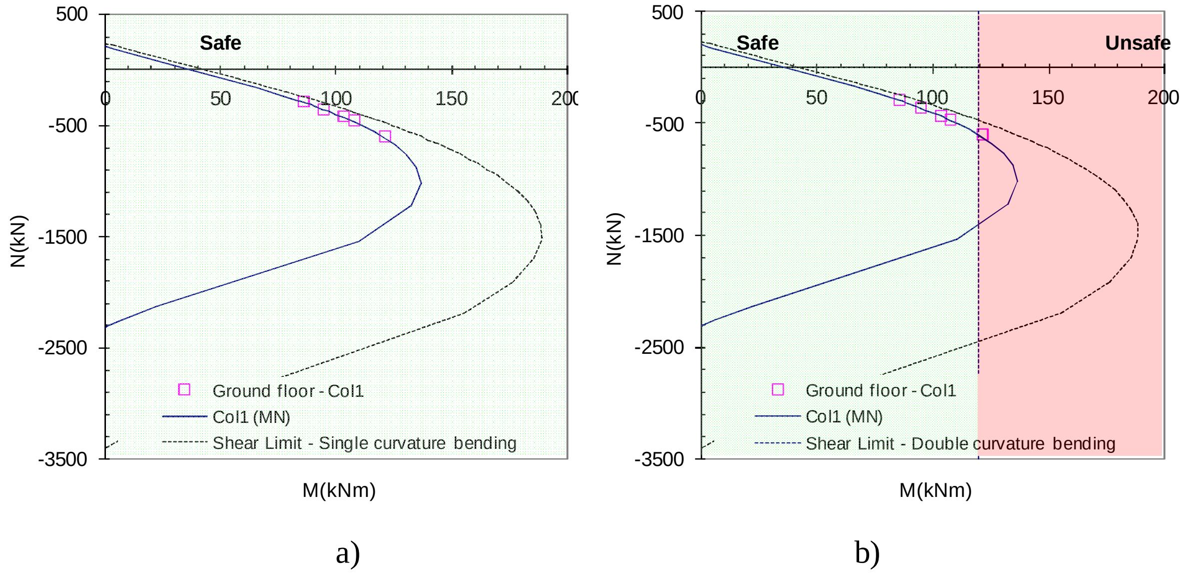


Figure 12. Cut-off given by shear capacity in the N-M interaction curve of Col1 (FEMA 356)

As a conclusion, for the shear strength of members, the FEMA 356 [11] prediction will be used in the following calculations, and we will consider that Col1 is NOT failing in shear. This choice is reasonable also due to the wish to consistently apply FEMA 356.

4 Application of the PBD methodology

The complete application of the PBD methodology proposed in WP2 was carried out for the r.c. benchmark building both (1) in the un-retrofitted and (2) retrofitted form. The details of the procedure and the results are presented below.

4.1 The PBD methodology for rc members

According to WP2 instructions [4], Chapter 6.5.2 "Reinforced Concrete Beam-Column Moment Frames" of FEMA 356 [11] should be applied for evaluating the rotation capacity of members corresponding to the immediate occupancy (IO), life safe (LS) and collapse prevention (CP) limit states. The rotation values in FEMA 356 correspond to "plastic rotation angle in radians". As SEISMOSTRUCT is not supplying the plastic rotation directly as an output, a separate procedure had to be be implemented in EXCEL for post processing the primary nodal and element outputs.

The procedure is carried out for each loading step of the pushover, as both the plastic rotation demand and plastic rotation capacity depends on the internal forces in the members [15]. The outline of the procedure for the calculation of the demand was:

1. Extract the nodal displacements in the pushover direction;
2. Extract the nodal displacements in the vertical direction;
3. From the displacement of each end node of a member calculate the rotation of the members (θ_2);
4. Extract the nodal rotations at the nodes corresponding to the members end ($\theta_{1\text{-START}}$, $\theta_{1\text{-END}}$);
5. From θ_2, $\theta_{1\text{-START}}$, $\theta_{1\text{-END}}$, calculate the total rotation of the member's chord at each end ($\theta_{1\text{-TOT}}$, $\theta_{2\text{-TOT}}$). For columns θ_2 plays an important role, as the floors are drifting, and $\theta_{1\text{-START}}$ and $\theta_{1\text{-END}}$ reduce the chord rotation of the column. For beams it has been accepted that $\theta_2 \approx 0$, and chord rotations are approximately equal to $\theta_{1\text{-START}}$ and $\theta_{1\text{-END}}$ on the two end (Figure 13).
6. The internal forces (N, M, T) were extracted for each members ends.
7. The shear span was evaluated at each end as L_s=M/T (Figure 13)
8. The length of the plastic hinge was evaluated at each end as:
$$L_{pl} = 0.1L_s + 0.17h$$
9. The height of compression zone in the concrete cross-section was calculated, corresponding to the axial forces N and in the case of the member failing due to bending (at each member end).
10. The yield curvature Φ_y was calculated corresponding to the above failure scenario (existing N together with bending failure), both taking into account the (i) yield strain of the reinforcing bars, and (ii) the crush strain of the concrete. (NOTE: For non-symmetrically reinforced members it was distinguished between both positive and negative bending.)
11. The elastic part of the chord rotations, when yield occurs (θ_{y1}, θ_{y2}), was evaluated for both member ends as:
$$\theta_y = \phi_y \frac{L_s}{3} + 0.0013\left(1 + 1,5\frac{h}{L_s}\right)$$
12. The plastic chord rotations were calculated for each end as: $\theta_{PL}=\theta_{TOT}-\theta_y$

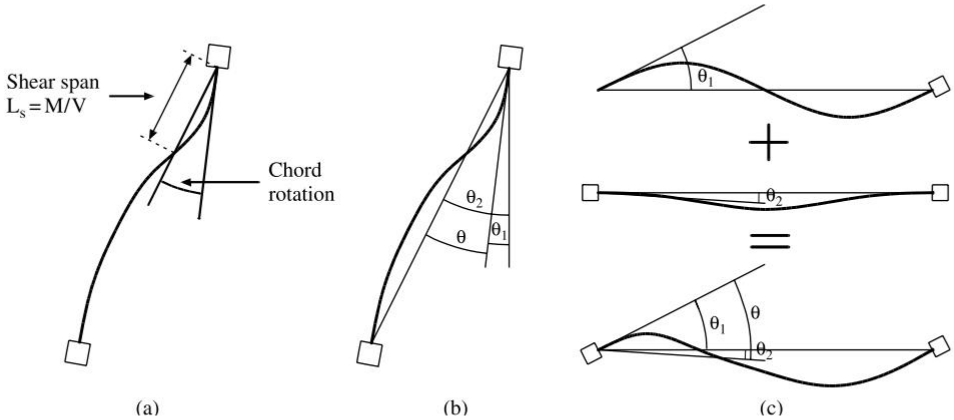


Figure 13. Chord rotation definition for columns and beams [15]

The available plastic rotation capacity was evaluated respecting the FEMA 356 procedure:

1. Identity the type of r.c. element (i.e. beam or column).
2. Identify if it is primary or secondary element (NOTE: all members were treated as primary elements)
3. Identify if the members fail in shear or bending. (NOTE: All members are considered to fail in bending based on the reasoning in the previous chapters of this document.)
4. Asses if the shear reinforcement is “conforming” (C) or “non-conforming” (NC) according to the definition of Table 6-7 and Table 6-8 of FEMA 356. (NOTE: Members in the structure are non-conforming, especially the performance limiting columns. But both the NC and C scenario was evaluated.)

Table 6-7 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[3]			Acceptance Criteria[3]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
Conditions			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01

Table 6-8	Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Reinforced Concrete Columns										
			Modeling Parameters[4]			Acceptance Criteria[4]					
						Plastic Rotation Angle, radians					
						Performance Level					
							Component Type				
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary		
Conditions			a	b	c	IO	LS	CP	LS	CP	
i. Columns controlled by flexure[1]											
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$									
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03	
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024	
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025	
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02	
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015	
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012	
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01	
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008	

5. Calculate the required input parameters for Table 6-7 and Table 6-8. For beams $\frac{\rho - \rho'}{\rho_{bal}}$, $\frac{V}{b_w d \sqrt{f'_c}}$, and for columns $\frac{P}{A_g f'_c}$, $\frac{V}{b_w d \sqrt{f'_c}}$. (NOTE: $\frac{V}{b_w d \sqrt{f'_c}}$ is not independent of the unit system, so it has to be evaluated in US units!!! Please see conclusion for more comments.)
6. Interpolate the available plastic rotation capacity for IO, LS and CP from Tables 6-7 and Table 6-8 of FEMA 356.

At the end, the evaluable and required plastic rotation capacities were compared in each pushover step. The load steps corresponding to the reaching of the plastic rotation limit for IO, LS and CP was retained for each member. The smallest step for any member corresponds to the IO, LS and CP limits for the structure.

As it can be understood from the above description, the procedure utilizes a few simplifications, reported not to affect the results in an important way [15]. These simplifications are:

1. The plastic chord rotations for beams neglect the effect of the beam's spatial rotation.
2. No biaxial effect is taken into account. I.e. the members are supposed to be bent around one axis only, when subjected to any direction pushover. Practically, the beams act only in major axis bending; while columns are only bent in the direction of the pushover load.
3. Due to the symmetry of the building, it has also been accepted that beams perpendicular to the direction of the loading are not active, and the bending of columns in the direction perpendicular to the loading direction is neglected. If there is torsion effect in the building, this simplification is not acceptable.

4.1.1 Example application of the procedure for an r.c. member

A column fixed for rotation at both ends was tested using the above procedure. The cross-section corresponded to the ground floor columns of the benchmark building ("Col1"), and material properties were identical. Nominal material properties were used. An axial compression of N=597kN (compression). The sketch of the test arrangement and the SEISMOSTRUCT model are presented in Figure 14.

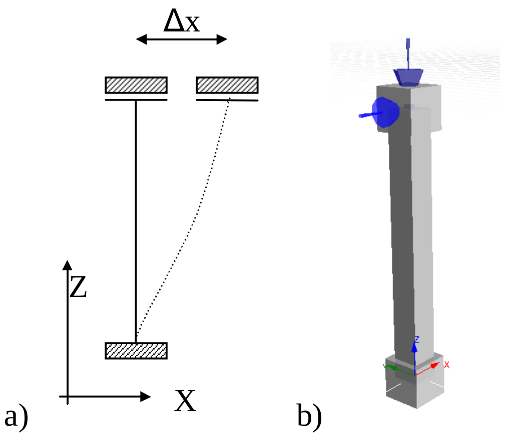


Figure 14. Test case for the PB procedure

In a preliminary analysis, the column was modeled as one Finite Element on the full length. With this preliminary model it was checked that the chord rotation calculations of the EXCEL tool are identical with the direct output of SEISMOSTRUCT.

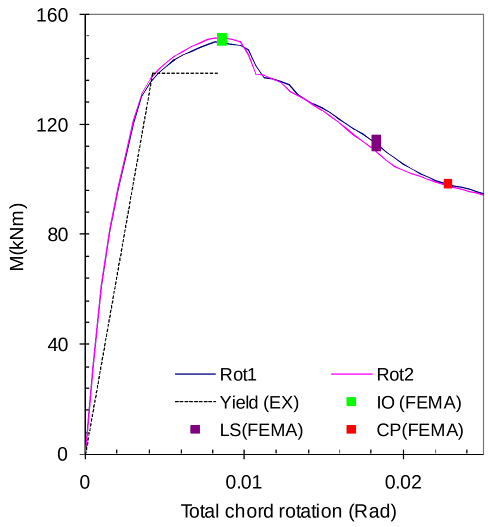
In the second model, the length of the column was divided in 5 segments. It this case the output of SEISMOSTRUCT can not be used directly, because chord rotation is output for each member. Also, the plastic rotation has to be calculated by the EXCEL procedure as the difference between the total rotation and the elastic one, because SEISMOTRUCT does not differentiate between the two, while FEMA 356 acceptance criteria is formulated in terms of plastic rotation only.

The simplified test gave a good opportunity…

1. …to test if the total chord rotation is correctly carried out from the node displacement in EXCEL, and
2. …to estimate the correctness of the elastic chord rotation calculation in EXCEL.

Some results of the testing are presented in Figure 15. essential notes about the results are:

- The elastic stiffness and the bending strength is correctly evaluated by the EXCEL procedure.
- The total chord rotations by the EXCEL procedure were identical to the ones supplied by SEIMSOTRUCT, confirming the correctness of the EXCEL procedure. As the total chord rotation, and the elastic stiffness are correctly evaluated, the EXCEL evaluated plastic chord rotation is reliable.
- In Figure 15, the point corresponding to IO, LS and CP limit states are indicated for conforming (C) and non-conforming (NC) shear reinforcement. In can be seen that the NC choice greatly limit the deformations corresponding to CP.



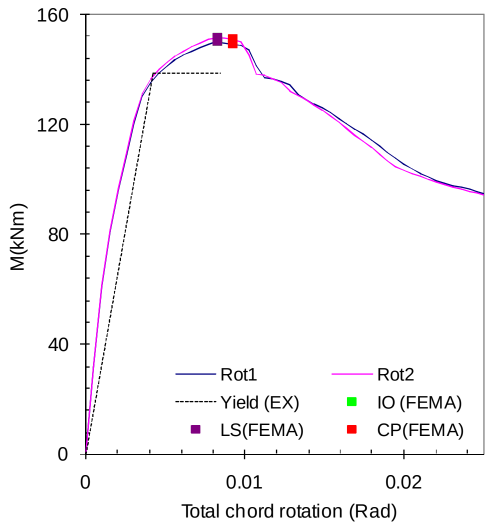


Figure 15. Testing case of one column for the PB procedure

4.2 The PBD methodology for steel members

On the demand side, the same simplifications have been accepted as for r.c. members. Practically, each member is bent only in one direction, around major axis, and the plastic chord rotation of the beams is not affected by the spatial rotation of the beam.

There has been also some difficulty in evaluating the shear span L_s (Figure 13) of the steel members. The usual expression seen in Figure 13 is not proper in case of the members forming the frame of the SW, because they were meat to be used with members not loaded along their length and sustaining small values of the shear compared to the bending.

Since the SW plate transmits large loads to the steel frame members, the application of L_s=M/V resulted in strong anomalies when the two shear spans, calculated from the two ends of each member were reviewed. Usually, from one end the L_s was very small [15], while the sum of the two shear spans calculated from both ends was several times the length of the member. This contradicts the physical model used in the definition of the end section chord rotation, i.e. the angle between the tangent to the member at the end and the line connecting the end to the contraflexure point on the member (where M=0). In order to eliminate these anomalies, it has been accepted that L_s is the half on the length of each member.

The chord rotation at yield has been calculated according to the recommendations of FEMA 356 in §5.5.2.2.2 as:

$$\theta y = \frac{L_s}{3 \cdot E \cdot I} \cdot W_{pl} \cdot f_y \cdot \left(1 - \frac{N^2}{{N_y}^2}\right) \qquad 12$$

Where, the second part of the expression, $W_{pl} \cdot f_y \cdot \left(1 - \frac{N^2}{{N_y}^2}\right)$, is the plastic bending moment capacity (M_{pl}) of the steel cross-section when the axial load is N. In these calculations the value of N was taken from each end of the members separately. W_{pl} is the plastic section modulus, f_y the yield stress and N_y the yield force under axial load. It should be noted that parabolic M-N interaction is used for the member [16], instead of the FEMA 356 proposed linear interaction. The first part of the expression, $\frac{L_s}{3 \cdot E \cdot I}$, is the chord rotation of the member with the length L_s, corresponding to the reaching of M_{pl} at the end section. The above formulation is compatible with Equation 5-2 of FEMA 356, with the observations that we are modeling the member as two half's with length L_s, while Equation 5-2 was considering it as one member of length $2*L_s$, and that we accept the parabolic MN interaction instead of linear interaction. Also, because large axial loads are present in the members of the steel frame, the same formula has been used both for columns and beams.

The plastic chord rotation capacity of steel members was chosen according to Table 5-6 of FEMA 356, corresponding to compact cross-sections:

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components

Component/Action	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
	a	b	c	IO	LS	CP	LS	CP
Columns—flexure [2, 7]								
For $P/P_{CL} < 0.20$								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $d\ \frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{460}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							

4.3 The PBD methodology for SW's

The demand on the SW was evaluated as inter-storey drift between two consecutive levels in the SW; then the storey drift rotation was calculated from this drift. The capacity was chosen according to Table 5-6 of FEMA 356. It should be noted that these values are recommended specifically for shear walls with stiffeners preventing buckling, so their use for the LGS-SW's presented here may be debatable.

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components (continued)

Component/Action	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
	a	b	c	IO	LS	CP	LS	CP
Steel Plate Shear Walls [1]	$14\theta_y$	$16\theta_y$	0.7	$0.5\theta_y$	$10\theta_y$	$13\theta_y$	$13\theta_y$	$15\theta_y$

4.4 Application to the r.c. benchmark structure

The above procedure, implemented in EXCEL was used for the PB evaluation of the benchmark building in two scenarios: (1) when the building is un-retrofitted, (2) when the building is retrofitted with prefabricated and perforated LGS shear wall system as described by Mielonen [3]. The top horizontal displacements when the different performance criteria are reached are summarized in Table 8, for each element type.

Table 8. Top displacement corresponding to the performance points

Top Disp. (m)	Orig-X r.c. NC	Orig-X r.c. C	Orig-Y r.c. NC	Orig-Y r.c. C	LGS-SW-X r.c. NC	LGS-SW-X r.c C	LGS-SW-X Steel	LGS-SW-X SW	LGS-SW-Y r.c. NC	LGS-SW-Y r.c C	LGS-SW-Y Steel	LGS-SW-Y SW
0												
0.01												
0.02												
0.03												
0.04												
0.05												
0.06												
0.065	IO LS CP	IO			IO			IO				
0.07					LS							
0.075					CP	IO						
0.08							IO		IO			
0.085			IO									IO
0.09												
0.095											IO	
0.1		LS CP							LS	IO		
0.105												
0.11			LS	IO					CP			
0.115												
0.12			CP									
0.125						LS						
0.13												
0.135												
0.14												
0.145						CP						
0.15												
0.155												
0.16												
0.165												
0.17												
0.175				LS CP						LS		
0.18												
0.185							LS CP	LS CP				
0.19												
0.195												
0.2												
0.205												
0.21										LS		
0.215												
0.22												
0.225												
0.23												
0.235												
0.24												
0.245												
0.25											LS CP	LS CP

Obviously, if a structure has more than one element types (e.g. r.c. members, steel members and SW) the smallest displacement, corresponding to these element types, from Table 8 should be considered.

These minimums are plotted in Figure 16 on the pushover curves in X and Y directions, for the un-retrofitted and the retrofitted structure. In Figure 16.a the dots represent the reaching of the limits corresponding to the IO, LS and CP limit state according to FEMA 356 definition if all concrete members are considered non-conforming in shear (NC).

Figure 16 b represents the limits if all members are considered conforming (C). The same tendency as in Figure 15 can be observed, the C and NC case are equivalent at IO limit, but the NC case is greatly limited in LS, and CP. The setting of all r.c. members being NC can be considered the lower limit of the performance to be achieved by the intervention, while when all members are C the upper limit.

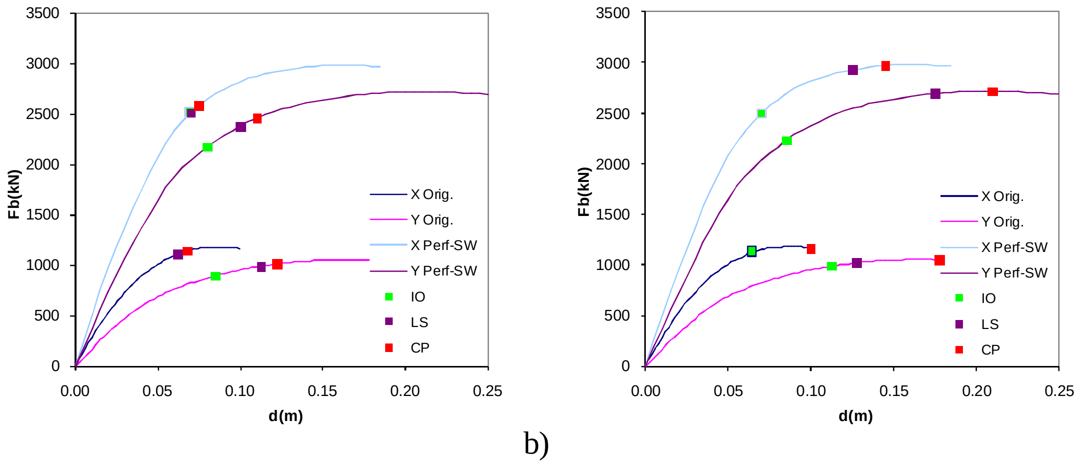


Figure 16. PB evaluation of the benchmark (a) with all members NC and (b) with all members C

It can also be seen from the pushover curves that the base shear capacity increased about 2.5 times in both directions, and the stiffness of the building also increased 1.6-2 times in the two directions.

In Figure 16 a, particularly in the pushover curve in the X direction of the strengthened configuration, it is evident that the CP limit state is reached very early, almost in the elastic range of the curve. This hints to the fact that increasing the strength of the retrofitting shear walls would not improve performance, because some r.c. members would reach CP limit state at the same displacement level, while the overall behavior would. In this case two solutions can be imagined: (1) to increase the stiffness of the retrofitting SW's, or (2) to increase the deformation capacity of the members which limit the X direction performance.

While theoretically possible, in practice the stiffness of the retrofitting measure can not be increased very much. In our model we consider the flexibility of the foundation (usually neglected in the analysis), but we still neglect several sources of flexibility in the connections between the r.c. frame and the retrofitting shear walls. Therefore, we do not believe that even stiffer retrofitting solutions can be realized in practice.

As far as the deformation capacity is concerned, in this case the limiting members are ground floor r.c. columns forming plastic hinges at the two ends due to the soft-storey mechanism reported for the benchmark building. It appears necessary to upgrade the shear capacity of these columns in the plastic hinge region, in order to allow them larger plastic chord rotations in LS and CP limit states. If all r.c. columns on the ground floor were upgraded for shear in the plastic hinge region (at both extremities), then the performance would approach the one presented in Figure 16 b.

The pushover curves of the un-retrofitted building, from Figure 16, are presented as capacity curves in Figure 17 with the last point in the curve being the CP limit for the case of all members conforming. It can be observed that the structure is inadequate to withstand the earthquake load in both directions.

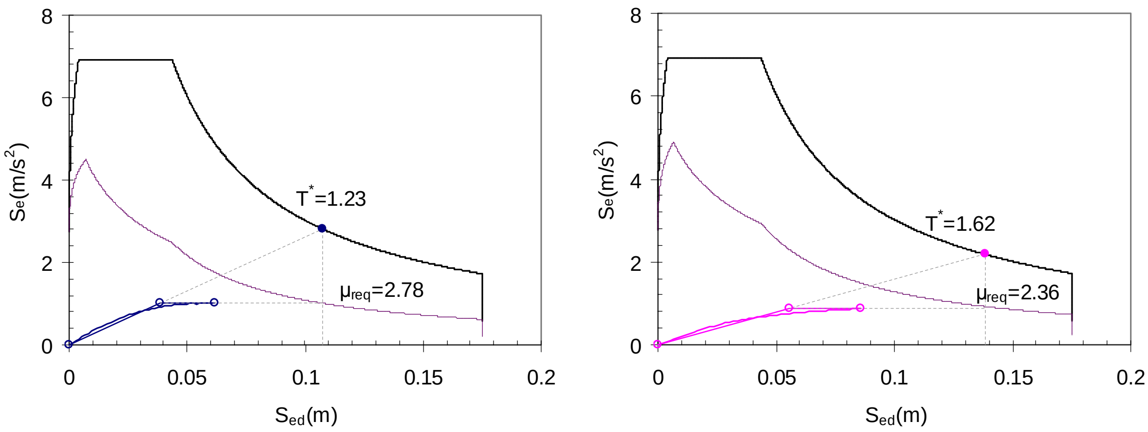
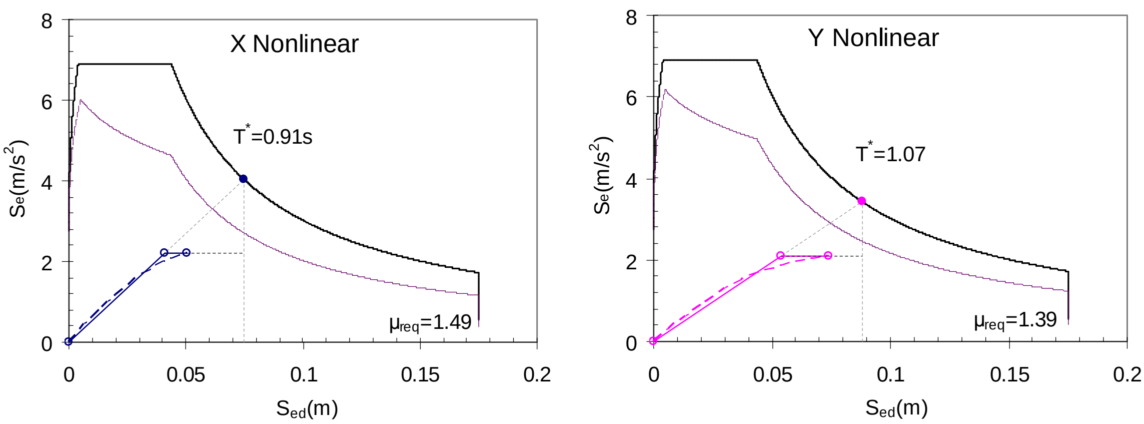
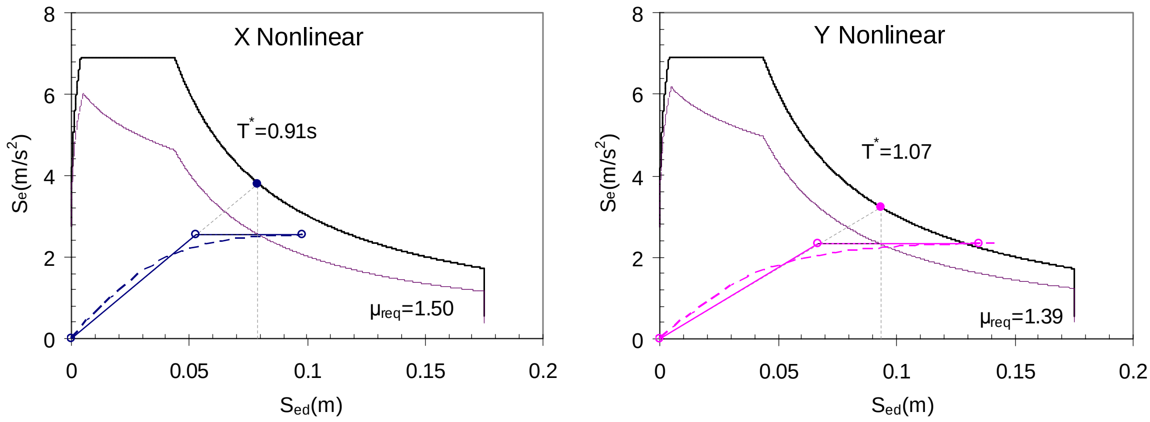


Figure 17. CP performance level for the un-retrofitted building all members C

The curves of the upgraded building are presented in the capacity design format, in Figure 18a if presuming NC members and in Figure 18b, if presuming C members. It can be seen that, as long as some r.c. members are not upgraded in shear from NC to C, the upgraded structure will not fulfill the earthquake requirements in any of the loading direction. On the other hand, if all members are upgraded in shear to C then the retrofit is successful. It should be noted here, that more detailed analysis showed that not all member shave to be upgraded, but all ground floor columns.



a)



b)

Figure 18. CP performance level of the upgraded structure if r.c. members are (a) NC in shear, and (b) if they are C in shear

The PGA corresponding to the performance levels is given in Table 9, while in Table 10 the matrix of performance upgrade is given, corresponding to the conforming case, and requirements defined by the Italian code D.M.2008. The same matrix is repeated for the requirements defined by Eurocode 8. It can be observed that, according to the Italian code the upgraded structure fulfills the earthquake requirements at all performance levels. However, according to the Eurocode 8 requirements, the upgraded structure would fail the CP limit. This is, obviously caused by the 2475 years return period imposed for the CP limit state.

An other observations refer to the IO requirements of the two codes. While for the Italian code the retrofit more than sufficient performance in the IO limit state, for Eurocode 8 the same solution is slightly insufficient. In the wider context, it appears that fulfilling the IO requirement for Eurocode 8 is impossible with anything else but bracing systems which increase the stiffness substantially.

Table 9. PGA in g's for the three performance levels

	Conforming in shear (FEMA 356 – C members)			Non-conforming in shear (FEMA 356 – NC members)		
	IO	LS	CP	IO	LS	CP
Orig-X	0.1	0.13	0.13	0.095	0.095	0.105
Orig-Y	0.13	0.145	0.14	0.1	0.13	0.14
UPE_SW0.5-X	0.145	0.245	0.28	0.145	0.155	0.155
UPE_SW0.5-Y	0.155	0.29	0.33	0.145	0.175	0.19

Table 10. PBD table of the effect of the upgrade with LGS shear walls (D.M.2008).

	Italian Code for construction (D.M.2008)	PGA/a$_g$ (g)	Performance with C members		
			IO	LS (ULS)	CP
Earthquake Hazard Level	Occasional -MRI = 50 years	0.094	INI SW	INI	INI
	Rare - MRI = 475 years	0.23		SW	
	Very Rare - MRI = 975 years	0.292			SW

Table 11. PBD table of the effect of the upgrade with LGS shear walls (Eurocode 8).

	Eurocode 8	PGA/a_g (g)	Performance with C members		
			IO	LS (ULS)	CP
		0.08	INI SW	INI	INI
Earthquake Hazard Level	Occasional -MRI = 50 years	0.176			
	Rare - MRI = 475 years	0.23		SW	SW
	Very Rare - MRI = 2475 years	0.39			

Note: IO - Immediate Occupancy; LS - Life Safety; CP - Collapse Prevention

5 Conclusions

Some general observations on the application of the PBD methodology:

1. The application of the PBD design method in the mixed US EU format is difficult. The inconsistencies brought in by the dual usage many times introduce questions that are not in the reach of researchers, let alone designers, to answer. The use of the US codes also forces the analyst to carry out calculations in US units. This can be difficult for a designer (and potential source of errors). Hopefully, the methodology will be fully implemented in the Eurocode system in the future, for users to stay consistently within one code environment.
2. The evaluation of the chord rotations is difficult, when the FE analysis software is not supporting the direct output of these quantities. The implementation of an EXCEL procedure, similar to the one reported here, is painstaking and error prone. In order for PBD to be useful for everyday work, designers need analysis software with direct outputs.

Concerning the performance of the proposed retrofit solution based on LGS-SW's:

3. The retrofit technique analyzed here performed relatively well. The SW solution's strength can be adjusted by the quality of the steel, the thickens of the plate and the diameter of the perforations. It is very important that the SW is not too strong compared to the original structure. If too much force is attracted by the SW problems appear in the dimensioning of the connections and foundations.
4. We believe that the full prefabrication of each SW module is possible, and the SW walls can be mounted from the exterior.
5. Unfortunately, even the stiff SW solutions did not provide enough performance to completely avoid intervention in the r.c. frame. This is caused by the soft storey behavior of the r.c. frame, which greatly limits the range of drifts which the designer tries to exploit with the retorting technique. It is also known that most older r.c. frame structures are prone to forming soft-storey mechanism, due to the lack of capacity design measures in the design codes at the time when they were built. The need to upgrade the shear capacity of the soft-storey columns might cancel some of the economic advantage given by the use of an external SW, but advantage certainly remain, especially because the same shear upgrade has to be done when using any other bracing based intervention technique.
6. Most probably, there is also a transition area between what FEMA 356 calls non-conforming in shear and conforming in shear (especially that the jump between the two is very big). With a more detailed analysis, e.g. based on EN 1998-1-3, or further experimental tests, it might be possible to show a more gradual decrease of performance from C to NC. In such case some columns might not need the improvement of their shear reinforcement in the plastic hinge area.

Finally, it is argued that the light-gauge steel shear walls with perforations can present a very effective way to retrofit old r.c. frames, because they can be cost effectively prefabricated and installed with minimal effort on site, and interruption of the function of the building.

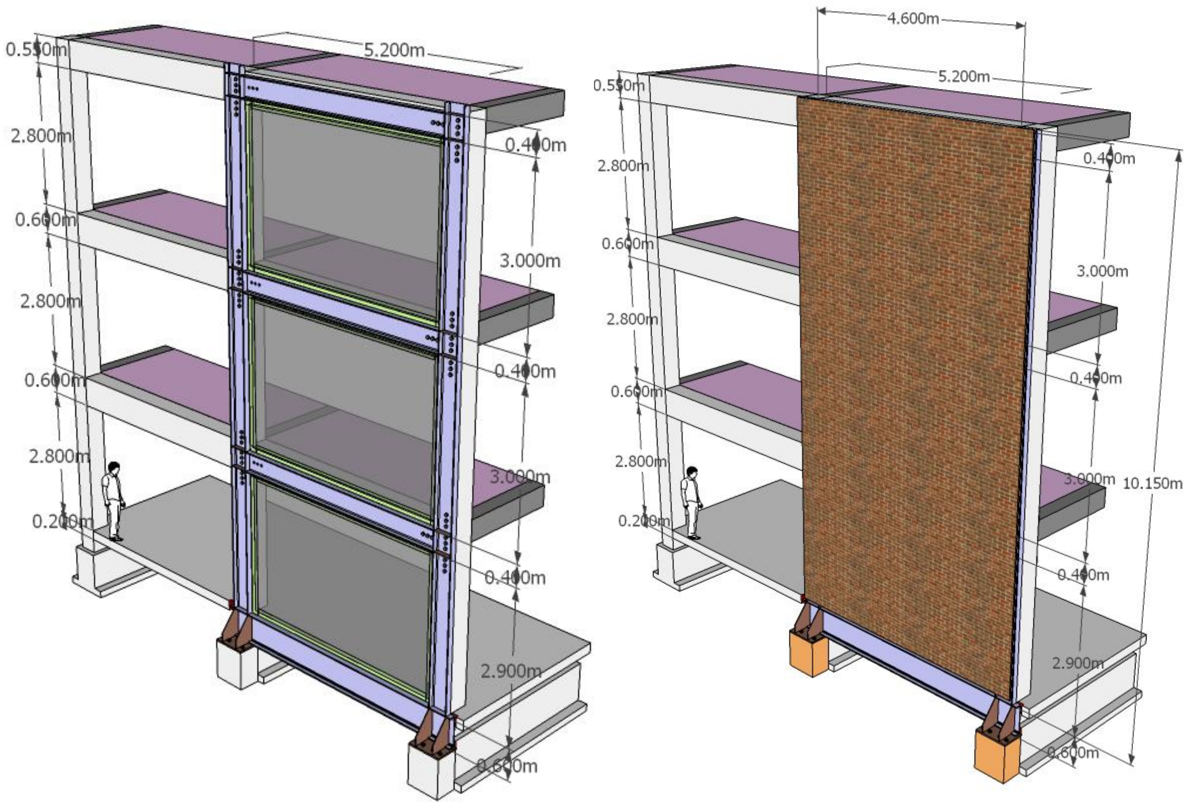


Figure 19. Proposed externally mounted LGS-SW: (a) details, (b) covered with fake masonry façade

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