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CHOICE OF THE OPTIMAL STRENGTHENING TECHNOLOGY

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SUMMARY

Modeling of the true behavior of existing buildings is a difficult step in the analysis of its state, as there are many unknown variables: building methods, various materials, stiffness and mass distribution, structural degradation and so on. These unknown variables can make complicated numerical procedures obsolete. Bad input data lead to non-reliable output data although the methods used could be very precise. Simplified methods based on some experimental data could give better results and reliably be used for safety factor evaluation.
Dynamic characteristics represent the real structural behavior and could be used for evaluation of structural state, calibration of the mathematical model and analysis of seismic risk or safety factor. Calibrated numerical model is used for choice of the optimal strengthening technology that should pay special attention to inclusion of elements that eliminate unfavorable behavior under strong seismic effects. After strengthening works are finished quality of the performed works could be verified by repeting the dynamic tests.
The methodology proposed in this paper could be used for quick evaluation of the structural state, modeling of the true structural behavior, choice of optimal strengthening technology and for verification of the performed strengthening works. It combines experimental data about building material, ground and structural behavior with analytical methods for estimation of the seismic vulnerability of masonry and reinforced concrete buildings. It roughly covers three vulnerability assessment levels required in the EC8: general stability, strength capacity and lateral displacements capacity.

EVALUATION OF THE STRUCTURAL CHARACTERISTICS

Preliminary structural investigation

Preliminary structural investigation includes building inspection with records of the structural geometry, structural system, and observed damages. The standard non-destructive material and structural element tests are used for determination of the basic building material characteristics.

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Measuring of the dynamic characteristics

Ambient vibrations or micro-tremor measurements are tests performed for obtaining the fundamental frequencies, mode shapes and damping values of existing buildings. Its advantages are that they do not require heavy and expensive equipment to introduce the excitation forces, can be conducted without traffic interruption and enable identification of vibration modes with frequencies bellow 1 Hz, which is difficult to achieve with forced vibration tests on large structures. However, ambient vibration tests also have disadvantages mainly related to the lack of control and quantification of the excitation forces. This brings some difficulties in the evaluation of the damping factors or in the identification of the dynamic properties associated with vibration modes poorly excited by the ambient vibration.

The source of ambient noise is external, of weak and unknown amplitude, in random sequence. Measured are structural vibrations caused by the ambient (wind, traffic, machines working near the structure, etc.) and the signals are than processed and analyzed in frequency domain. Duration of the recording has to be long enough in order to eliminate possible non-stationary forces that might appear during the test.

Dynamic response of the structure, excited with low intensity forces with flat amplitude spectrum, contains vibrations in all their modes. Each mode is presented with peak in the amplitude response spectrum. Amplitude response spectra at each measuring point are averaged (minimum 32 times) in order to decrease the variance caused by the FFT, to increase deterministic part of the signal (structural response) and thus decrease the accidental part (noise). We obtain natural forms by measuring the response at various places and normalizing them to take into account different excitation levels.

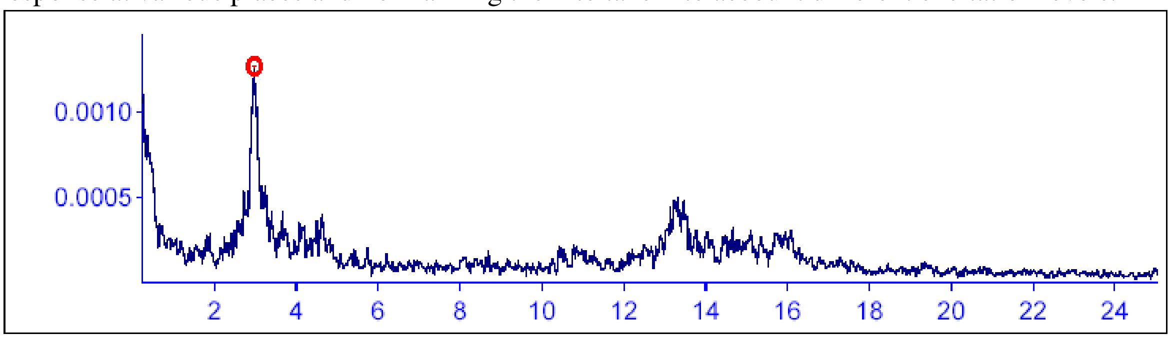


Figure 1. Amplitude response spectra

EVALUATION OF THE STRUCTURAL SAFETY FACTORS

Dynamic experiments performed on the structure give us the insight into its state. By knowing dynamic characteristics (natural frequencies, forms and damping values) we are able to exactly determine structural stiffness, masses and to take into account such problematical things as torsion, stiffness changes, wall-slab stiffness, accumulated damage, ground-structure interaction, etc. Measured frequencies and mode shapes (horizontal and vertical) define horizontal and vertical distribution of earthquake forces. Their intensity is determined on the basis of estimated mass intensity and code defined response spectra for particular building location. Modal participation factor (γ = participation factor for the first-mode shape normalized so that the value at the top level is unity) and modal ordinates at each level (φ_I) are obtained by measurement.

Strength capacity safety factors <u>Iss</u>

There are three elements to be taken into account for story strength capacity safety factor (Iss) evaluation.

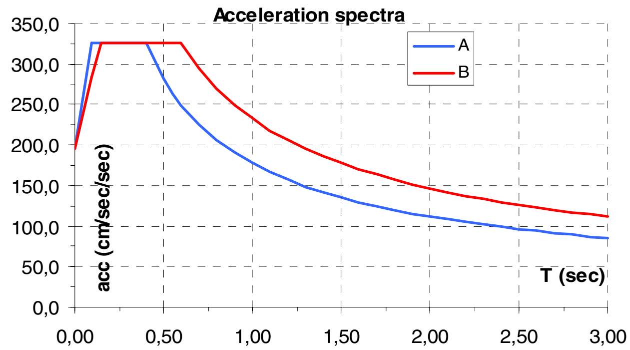
Site geological parameter G

When the measured fundamental frequency of the structure (Ts) and measured fundamental frequency of the adjacent soil (Tg determined by Nakamura (H/V) method) are close to resonance:

$$0{,}8 \leq Ts/Tg \leq 1.2$$

then calculated expected earthquake forces are to be increased by G=1.25, otherwise G=1.00.

Expected horizontal forces



The expected horizontal forces for the chosen return period are calculated on the basis of the measured natural frequencies, forms, damping values and EC8 design response spectra for the respective ground. Total horizontal seismic force is distributed into story forces along the height according to the measured vertical natural forms.

Figure 2. EC8 response spectra for ground type A, B, q=1.5, α=200gal

Shear capacities of the stories

Shear capacity of the story **(Vst)** is a sum of shear capacities of columns, walls and bracing systems located in the particular story. The estimate of the elements' shear capacities are based on the conservative estimates of their shear capacities.

For each story proper story strength index **Iss** is determined as

Iss=Vst/(Veq*G) _ 1

Where Veq=expected horizontal shear force caused by earthquake, Vst=shear capacity of the story's structural element, G=site geological parameter 1 or 1.25.

The building overall strength index **Iso** is

Iso=min (Iss) _ 2

On the basis of Iso we conclude that if:

- Iso > 1.0 the structure has the required safety level.
- 0.75 < Iso < 1.0 the structure needs more detailed analysis in order to properly establish its safety.
- Iso < 0.75 the structure is unsafe for expected ground motions.

General overturning stability safety factor:

When the seismic forces probable to act on the building are known (from EC8 response spectra and natural frequency) and their vertical distribution along the height is determined on the basis of measured forms the overall overturning stability safety factor **Ios** is calculated as follows:

Ios=Mo/Ms _ 3

*Where Mo is overturning moment provided by foundation (Mo=WBmin) and Ms is overturning moment caused by earthquake forces (Ms=ΣG*Ei*Hi); Ei=earthquake force at Level I; Hi=height of the level I from the base; W=total weight of the structure, Bmin=minimum width of the foundations, G=site geological parameter 1 or 1.25.*

The building is safe against overturning if **Ios>1.5**.

Lateral displacement safety factor (damage index Id)

This part could be understood as the performance index calculation. Building behavior (damage) during a possible earthquake is estimated as a function of the expected story drift at each level. The expected damage in a building depends mainly on the story drift, material quality, and structural system arid the construction details of the different components and it's connections. The nonstructural damages do not compromise the structural stability but they affect its functionality and the allowable drift ratio is defined in view of the contents and function of the structure.

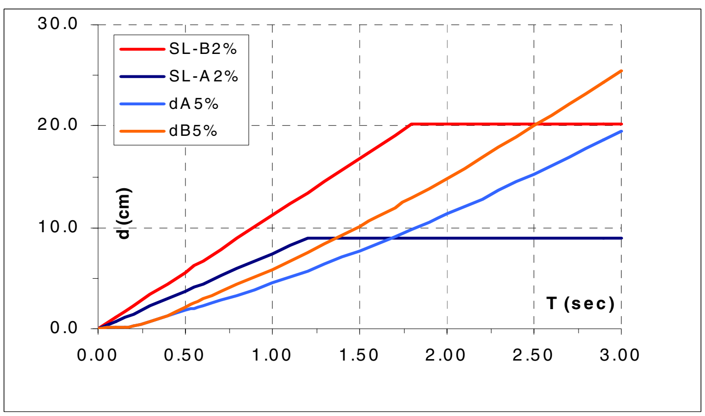
Expected nonlinear drift is calculated by the methodology outlined in (Lepage & Sozen, 1997) which states that linear spectral analysis could be used for evaluation of the expected nonlinear drifts during earthquakes as expected nonlinear drifts are lower or equal to the drifts calculated by linear spectral analysis for 2% damping. The following relation applies:

$$DR = \begin{cases} 1/TR & \text{for } TR<1 \\ 1 & \text{for } TR\geq 1 \end{cases} \qquad \_\ 4$$

**Where: TR=period ratio= (To* $\sqrt{2}$)/Tg (earthquake period); DR=drift ratio= (nonlinear drift)/(linear drift for 2% damping).*

For a MDOF system, with a reasonably uniform distribution of story mass and stiffness, the maximum displacement at any level I (Dmax,i) may be estimated using:

$$\text{Dmax,i} = \gamma * \varphi i * \frac{Fa * \alpha * g * Tg}{(2*\pi)2} * Teff \qquad \_\ 5$$



**Where: γ =participation factor for a given mode shape (obtained from the measured natural form); φi=ordinate defining the assumed mode shape at level i (also obtained by measurement); Fa=acceleration amplification factor (usually 3.75); α =peak ground acceleration expressed as a coefficient of the acceleration of gravity; g=acceleration of gravity; Tg= measured characteristic period of the ground motion; Teff=Ts(measured period of vibration)* $\sqrt{2}$ is effective structural vibration period for the first mode.*

Figure 3. EC8 and Sozen-Lepage nonlinear displacement spectra

Base shear strength plays a minor role by drift evaluation, but it should be above a minimum value defined by an equation:

$$Cy = \alpha*(1-TR) \geq \alpha/6 \qquad \_\ 6$$

So, using the outlined methodology, story damage index **Idi** is determined in two steps:

1. Using Tg (measured or determined on the ground of geotechnical characteristics), Teff (Ts(measured)*1.41), γ (modal participation factor-measured) and EC8 design spectra for the particular location and return period we can determine expected maximum drift at the top level and estimate lateral deformation at each level. From these we can calculate the upper bound of the expected nonlinear drift story drift at each level. (Δi)

2. Damage index for each level (Idi) is calculated as:

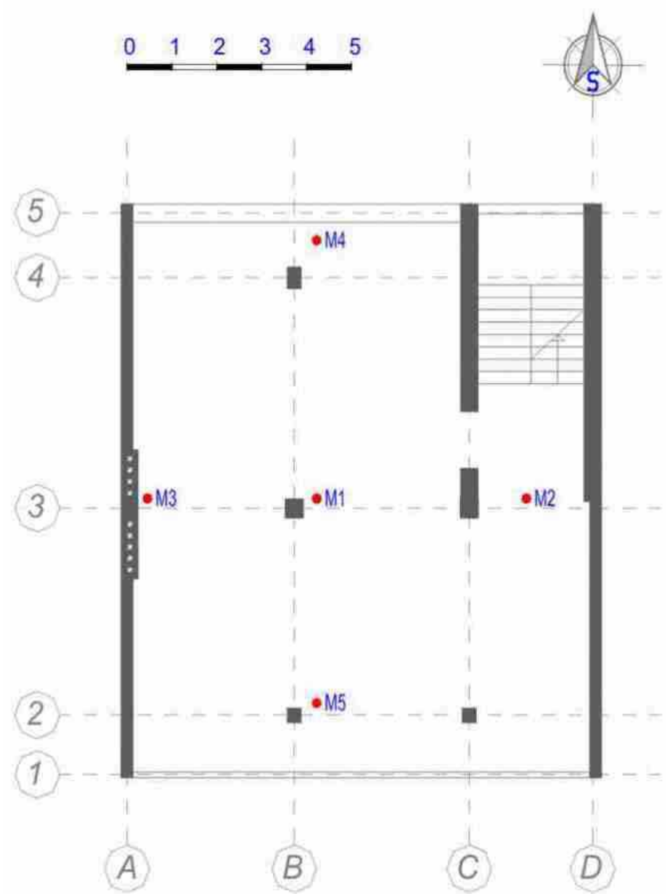
$$Idi = (\Delta pi) / (\Delta i) \qquad \_7$$

where: Δi- evaluated story drift at level I; Δpi - permissible story drift defined in view of contents and function of the structure.

The structural performance (damage index Idi) is acceptable if **Idi>1.0**. If the damage index **Idi<=1.0** then story has an unacceptable behavior and we have to increase story stiffness through strengthening.

EXAMPLE STRUCTURE

The office building in Osijek was built in the year 1957. as typical masonry structure at that time



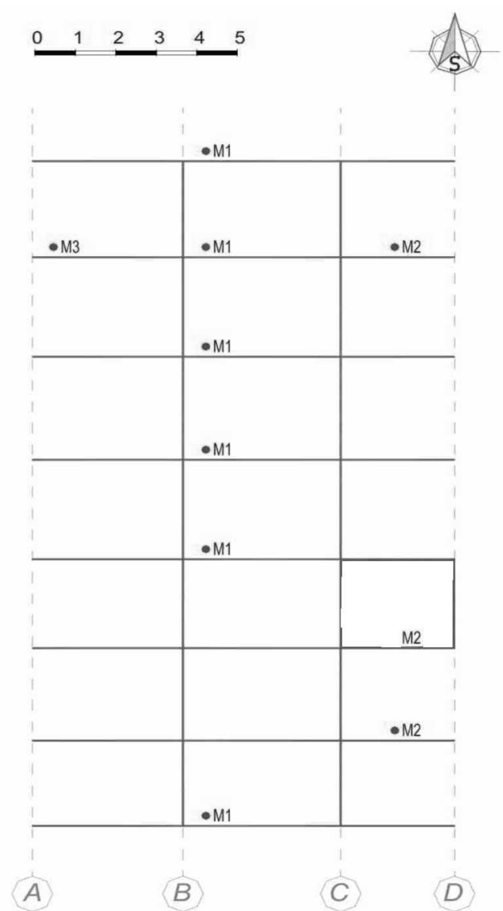


Figure 4. Plan and vertical cross-section of the example structure

designed for vertical and wind loadings only. It consists of basement, ground floor with gallery and four floors (A=10.69x12.06m) with an average height of 2.8m. Total building is 17,0m above the ground. Various owners have adapted the building several times and the works were especially intensive at the ground floor where structure lost some of its resisting system for horizontal seismic loads.

In order to properly establish the real structural state and define the best strengthening method investigation of material and structural characteristics were necessary. They consisted of: (a) evaluation of material characteristics; (b) check of the walls homogeneity by non-destructive methods; (c) ambient vibration measurement in both main directions. Seismic safety indices were calculated after the material

characteristics (shear capacity), natural frequencies and forms of the structure, natural frequency of the surrounding ground and the EC8 design response spectra were known (Table 1.).
Strength capacity safety factor Iss for four floors were lower than accepted and the structure had to be strengthened.

Table 1. Evaluation of the safety indices for existing (HLK0) structure

Story	H	X1	Qk	Qk*X1	Qk*X1^2	Eta	Eta1	Ei	Veq	h1	M
	(m)		(kN)					(kN)	(kN)	(m)	(kNm)
Roof	16,80	1,00	1081,4	1081,4	1081,4	1,26	0,21	394,49			
4.floor	13,94	0,97	1417,9	1375,4	1334,1	1,22	0,21	382,66	777,15	2,86	1128,2
3.floor	11,14	0,92	1530,3	1407,9	1295,2	1,16	0,20	362,93	1140,08	2,80	3304,3
2.floor	8,34	0,7	1439,4	1007,6	705,3	0,88	0,15	276,14	1416,23	2,80	6496,5
1.floor	5,54	0,51	1762,3	898,8	458,4	0,64	0,11	201,19	1617,42	2,80	10461,9
Gallery	2,74	0,42	1603,0	673,3	282,8	0,53	0,09	165,69	1783,11	2,80	14990,7
Ground floor	0,00	0,19	456,0	86,6	16,5	0,24	0,04	74,95	1858,06	2,74	19627,8
o.oo	0,00	4,71	9290,3	6530,9	5173,7	5,95	1,00	1858,06			

Hs= 0,792 γ= **1,262**

Overturning safety stability factor

Ios= 7,44

Strength capacity safety factor Is

Story	Veq	Ax	Wx	tauX	Vst	I0	K	Iss
	(kN)	(m2		(kN/m2)	(kN)			
Roof	394,5							
4.floor	777,2	0,59	35,1	1317,2	957,2	1,23	0,90	**1,11**
3.floor	1140,1	0,59	35,1	1932,3	957,2	0,84	0,90	**0,76**
2.floor	1416,2	0,59	35,1	2400,4	957,2	0,68	0,90	**0,61**
1.floor	1617,4	0,70	35,1	2310,6	1135,6	0,70	0,90	**0,63**
Gallery	1783,1	0,71	35,1	2507,2	1153,8	0,65	0,90	**0,58**
Ground floor	1858,1	1,89	35,1	983,1	3066,2	1,65	0,90	**1,49**

Iso= 0,58

Shear capacity concrete **Vu1= 1622,34**

Damage index Id

α	(PGA)=	0,3	g	Cy=	0,33005
Tg	=	0,6	sec	α/6=	0,05

Dmax	γ=	Dt
0,060	1,262	0,107

Story	H	X1	Eta	h1	Di	Δi	Δpi	Idi
	(m)			(m)	(m)	(%)	(%)	
Roof	16,80	1,00	1,26		0,107			
4.floor	13,94	0,83	1,05	2,86	0,088	0,634	0,9533	**1,50**
3.floor	11,14	0,66	0,83	2,80	0,070	0,647	0,9333	**1,44**
2.floor	8,34	0,49	0,62	2,80	0,052	0,647	0,9333	**1,44**
1.floor	5,54	0,33	0,42	2,80	0,035	0,609	0,9333	**1,53**
Gallery	2,74	0,16	0,20	2,80	0,017	0,647	0,9333	**1,44**
Ground floor	0,00	0,20	0,26	2,74	0,022	0,167	0,9133	**5,46**
o.oo	0,00	3,67	4,64					

Where: H=story height from the reference plane; X1=measured natural forms normalized to 1 at the top level; Qk=story weight; Eta=X1γ; Eta1=amount of the total seismic force attributed to the respective story; Ei=story seismic force; Veq=story shear force; K=corrective facto, Iss=strength capacity safety factor of the story; Idi=lateral displacement safety factor of the story; Ios=general overturning stability factor.*

Combining structural characteristics for both directions and after several trials the best strengthening method, from the strength and economic point of view, has been chosen. A new reinforced-concrete wall has been added in the axis 3-3 between axis C-D. Beams and columns along axis 3-3 and B-B have been strengthened, masonry wall in the axis A-A has been homogenized by filling in the chimney holes, the wall in axis C-C has been strengthened by adding the reinforced concrete jackets. Additionally, the new reinforced-concrete slab 6cm thick has been added at the 1st and 3rd floor level.

Measurements of the dynamic characteristics were done at several stages during strengthening performance (Table 2. and 3.). The quality of the performed works and chosen strengthening method has been continually checked in that way.

Table 2. Experimental stages

Model	**Date**	**Description**
HLK0	01.04.2003.	Existing structure
HLK1	05.06.2003.	Existing structure with removed nonstructural walls
HLK3	19.07.2003.	New wall in axis 3:C-D added up to the 3rd floor
HLK5	02.09.2003.	Finished strengthening works.

Table 3. Measured dynamical properties

	N-S			E-W		
Model	Calc. F1	Meas. f1	Damping%	Calc. F1	Meas. F1	Damping%
HLK0	5,0	5,1	3%	2,5	2,8	7%
HLK1	5,0	5,5		2,1	3,4	
HLK3	5,0	5,5		2,4	3,3	
HLK5	4,8	5,1	3%	2,4	3,2	3%

In the Figures 5. to 8. presented are measured and calculated natural forms for E-W direction, as well as characteristic infill walls in the axis 3-3. Modified numerical model, that included measured foundation flexibility, has been used for correlation analysis. Various building stages were measured and numerically analyzed (3D ETABS model). The results are presented in the form of natural frequencies and oscillation forms shown in the tables 2 and 3 and figures 5 to 8.

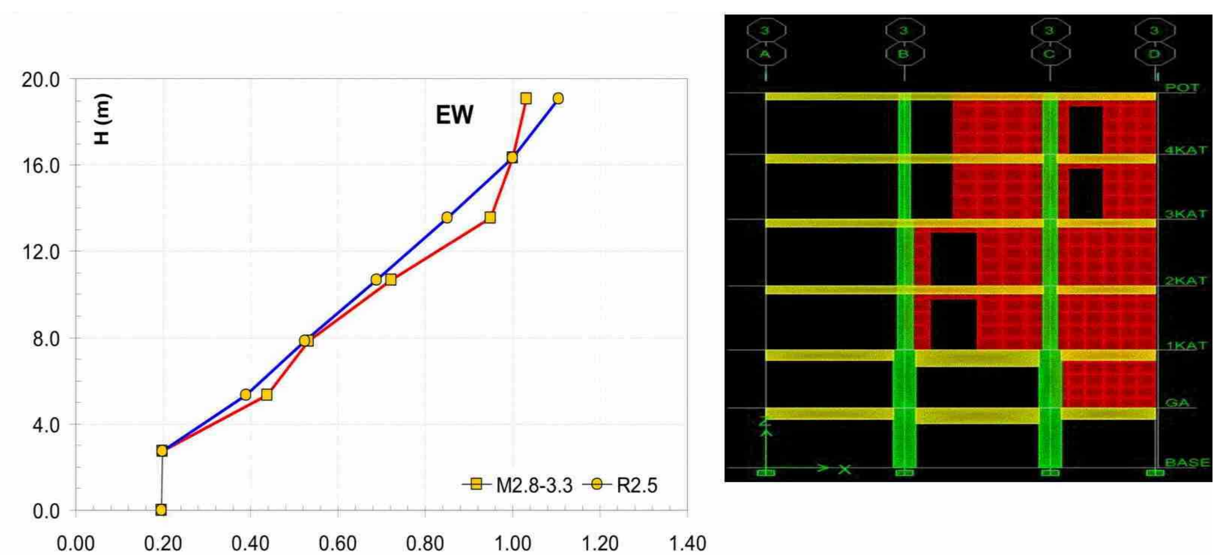


Figure 5. HLK0 Measured (M) and calculated (R) natural forms

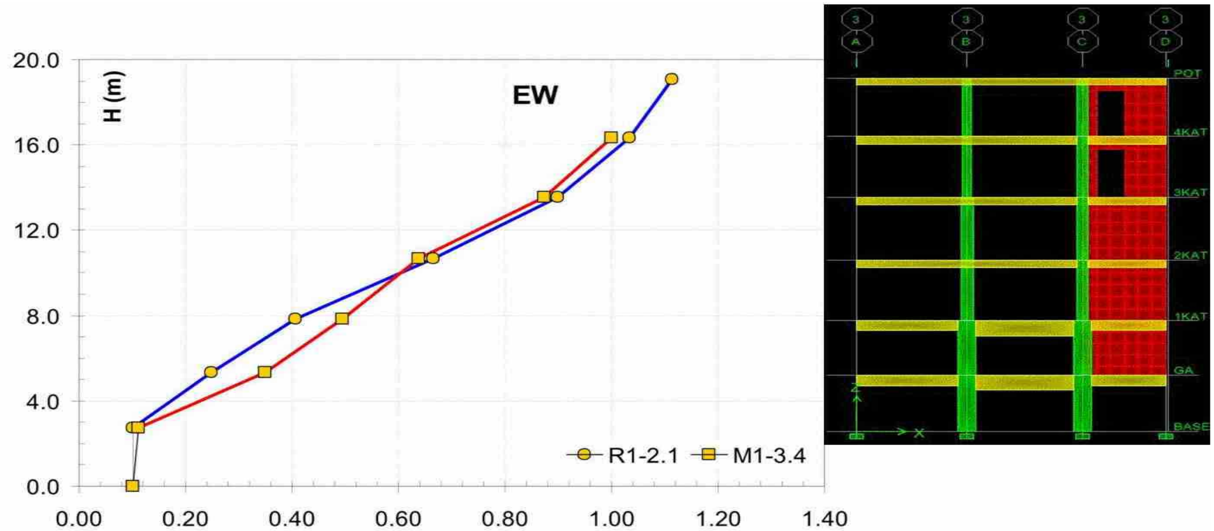


Figure 6. HLK1 Measured (M) and calculated (R) natural forms

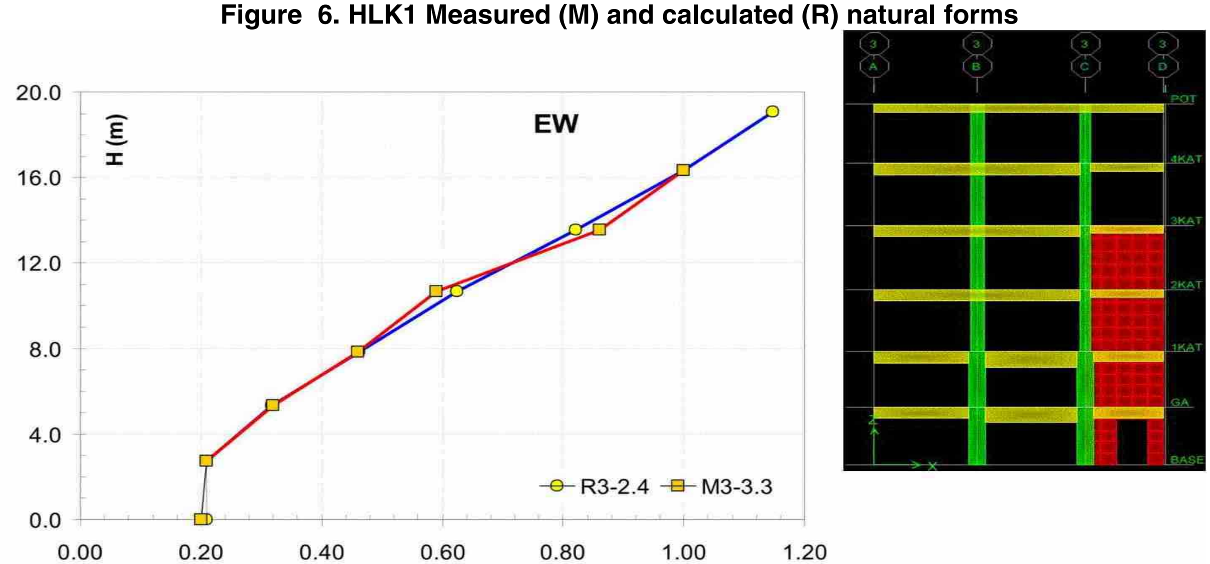


Figure 7. HLK3 Measured (M) and calculated (R) natural forms

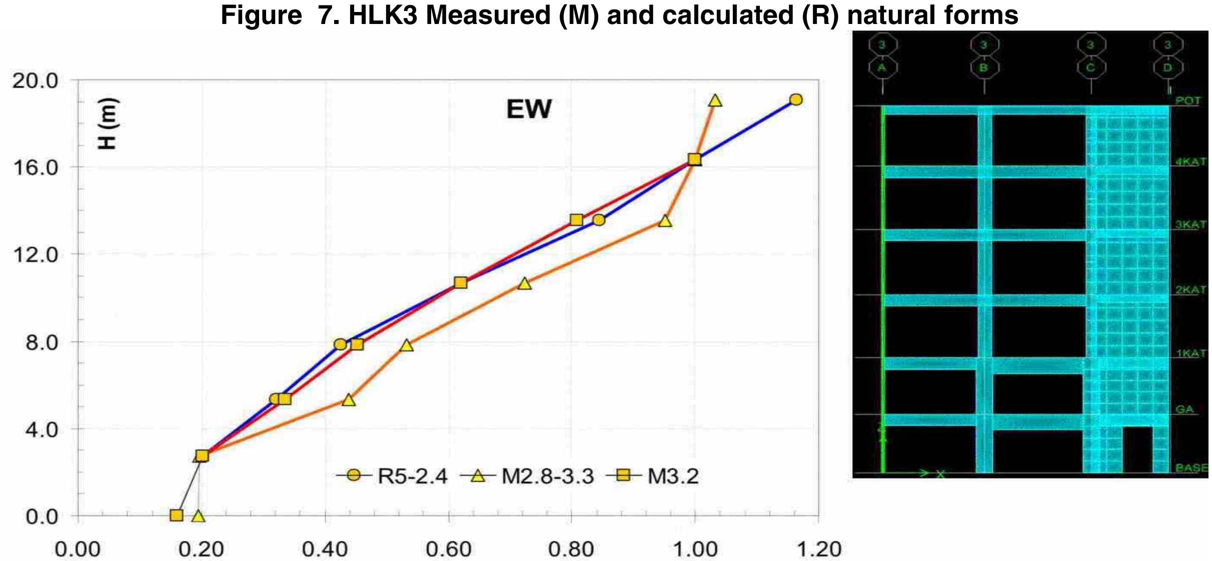


Figure 8. HLK5 Measured (M) and calculated (R) natural forms

It can be observed that numerical models, no matter how precise, poorly represent the true structural behavior and state. At the initial stages (HLK-0 and 1), when we have many uncertainties, numerical model represents the true structural behavior only by calibration with the experiments. Discontinuities in the measured vertical forms and high damping values indicated lack of vertical stabilizing element and high-energy dissipation at the story levels. As the structural state improved by advancing the strengthening works numerical results approached to the real structural behavior (HLK-3 and especially HLK-5).
Seismic safety indices were calculated again after the works were finished (Table 4.) using the measured dynamic characteristics (HLK-5) and the EC8 design response spectra. Results are presented in the form of table used for their calculations.

Table 4. Evaluation of the safety indices for strengthened (HLK5) structure

Story	H (m)	X1	Qk (kN)	Qk*X1	Qk*X1^2	Eta	Eta1	Ei (kN)	Veq (kN)	h1 (m)	M (kNm)
Roof	16,80	1,00	1081,4	1081,4	1081,4	1,36	0,24	437,19			
4.floor	13,94	0,95	1417,9	1347,0	1279,7	1,29	0,22	415,33	852,52	2,86	1250,4
3.floor	11,14	0,77	1530,3	1178,3	907,3	1,05	0,18	336,64	1189,16	2,80	3637,4
2.floor	8,34	0,59	1439,4	849,2	501,1	0,80	0,14	257,94	1447,10	2,80	6967,1
1.floor	5,54	0,43	1762,3	757,8	325,8	0,58	0,10	187,99	1635,09	2,80	11019,0
Gallery	2,74	0,32	1603,0	513,0	164,1	0,44	0,08	139,90	1774,99	2,80	15597,2
Ground floor	0,00	0,19	456,0	86,6	16,5	0,26	0,04	83,07	1858,06	2,74	20305,0
0.00	0,00	4,25	9290,3	5813,4	4275,9	5,78	1,00	1858,06			

Hs= 0,74 γ= 1,36

Overturning safety stability factor

Ios= 7,15

Strength capacity safety factor Is

Story	Veq (kN)	Ax (m2	Wx	Taux (kN/m2)	Vst (kN)	I0	K	Iss
Roof	437,2							
4.floor	852,5	1,87	35,1	455,9	3033,8	3,56	0,90	3,20
3.floor	1189,2	1,87	35,1	635,9	3033,8	2,55	0,90	2,30
2.floor	1447,1	2,00	35,1	723,6	3244,7	2,24	0,90	2,02
1.floor	1635,1	2,00	35,1	817,5	3244,7	1,98	0,90	1,79
Gallery	1775,0	2,85	35,1	622,8	4623,7	2,60	0,90	2,34
Ground floor	1858,1	2,87	35,1	647,4	4656,1	2,51	0,90	2,26

Iso= 1,79

Shear carried by concrete Vu1= 1622,34

Damage index Id

α (PGA)= 0,3 g Cy= 0,50118
Tg = 0,6 sec α/6= 0,05

Dmax γ= Dt
0,05233 1,360 0,10032

Story	H (m)	X1	Eta	h1 (m)	Di (m)	Δi (%)	Δp (%)	Idi
Roof	16,80	1,00	1,36		0,100			
4.floor	13,94	0,83	1,13	2,86	0,083	0,596	0,9533	1,60
3.floor	11,14	0,66	0,90	2,80	0,066	0,609	0,9333	1,53
2.floor	8,34	0,49	0,67	2,80	0,049	0,609	0,9333	1,53
1.floor	5,54	0,33	0,45	2,80	0,033	0,573	0,9333	1,63
Gallery	2,74	0,16	0,22	2,80	0,016	0,609	0,9333	1,53
Ground floor	0,00	0,20	0,28	2,74	0,020	0,157	0,9133	5,80
0.00	0,00	3,67	4,99					

All seismic safety indices were satisfactory. Damping values decreased and were now in the expected range due to the wall homogenization and continuity of the vertical oscillation forms. Building behaved continuous and homogeneous system with decreased torsional effects.

CONCLUSION

In order to evaluate and improve seismic capacity of existing buildings ‘seismic safety evaluation’ and ‘retrofitting’ are very important tasks. Engineers make these evaluations with different methods, from too simple to too sophisticated ones. The performed investigations showed that it is possible to predict the behavior of buildings due to seismic loading if a suitable testing procedure is sensibly combined with common engineering knowledge and, if necessary, with sophisticated finite element software. Good results can be obtained even if only data obtained on a limited number of measurement points are available.

The outlined method is simple, combines experimental data and engineering knowledge for evaluation of the seismic safety factors and expected structural performance under strong events. It allows distinction between the structures without problems and those with severe problems and is a quick way to check the behavior of structures against seismic demands contained in design codes, using the design spectra for the zone in which the building is located. It can also be used for choice of the optimal strengthening method and for verification of the quality of performed strengthening works. Proposed methodology can be considered as a useful engineering tool to provide a base for the planning of measures of restoration and reinforcement and to check their success.

The proposed methodology could be used for quick evaluation of the structural state, modeling of the true structural behavior, choice of optimal strengthening technology and for verification of the performed strengthening works. It combines experimental data on material, ground and structural behavior with analytical methods for estimation of the seismic vulnerability of masonry and reinforced concrete buildings. It roughly covers three standard vulnerability assessment levels required in the EC8 as it considers: general stability, strength capacity and lateral displacements capacity.

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