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SEISMIC ANALYSIS OF IREGULAR REINFORCED CONCRETE COUPLING WALLS SYSTEMS CONSIDERING DUCTILITY EFFECTS

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Summary: *This paper presents results of experimental and numerical analysis of seismic resistance of reinforced concrete coupling system considering the plastic capacity in accordance of standard requirements STN ENV 1998 (1999), ENV 1998 (2003) and Önorm B4015 (2002). The plastic capacity of the structure can be established by parameter q in the case of the spectral analysis to determine the seismic response. The experience from dynamic analysis of a hospital structure in accordance with standard requirements is presented in this paper. Dynamic parameters of the building structure are checked by experiment and the calculation model is modified on the basis of the experiment. The nonlinear analysis of the coupling system was realized in the program CRACK (Králik and Cesnak 2001) under system ANSYS for Kupfer's failure condition and Červenka's model of the concrete strength reduction.*

1. Introduction

The new seismic-resistant construction standards ENV 1998 (2003) and STN ENV 1998 (2005) enable to consider the seismic load effect to structures with regard to their partial damage, eventually collapse.

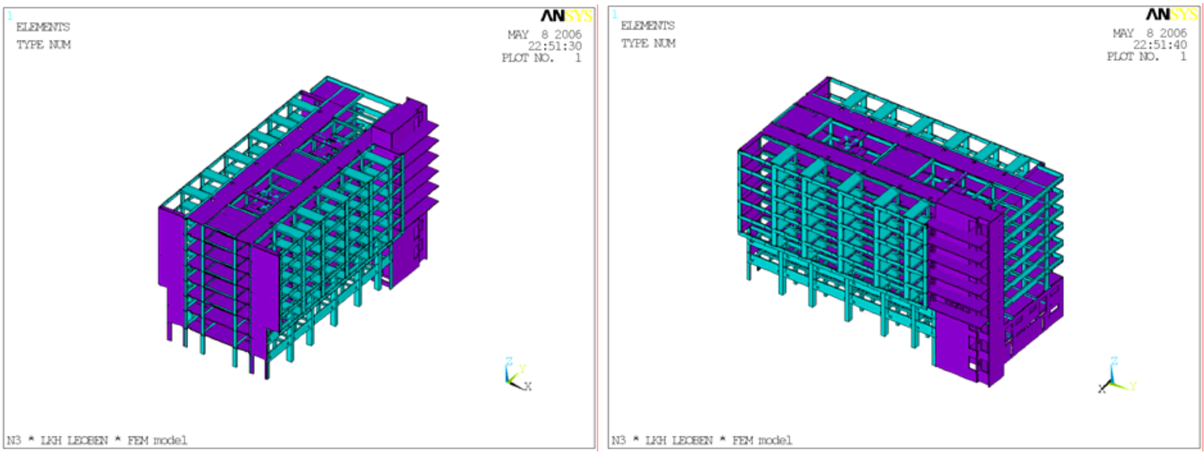

Fig. 1: The calculation model of reinforced concrete structure of the hospital facility

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With respect to computational and economical complexity there was a method established, which permits to transform a nonlinear dynamic calculation to a linear domain. The process of the failure is described by so-called ductility factor. The evaluation of the seismic response based on spectral analysis performs the ductility factor to design spectrum, as main feature of the seismic load.

Ductility factor is well described in standards, although nonessential ignorance of physical background could tend to incorrect results and performance errors (Flesch 1993). Incorrect and unsuitable interpretation can occur especially when hybrid structural systems, irregular geometry shapes and masses are considered.

Nonlinear behavior of a hybrid system became an aim of this analysis. The framework of the hospital facility consists of the combination of frames, shear walls and a core wall system. Nonlinear analyses of walls coupled with frames were realized by software called CRACK, using Kupfer's biaxial stress failure criterion and also reduction of concrete strength described by Červenka (Červenka 1985). Program CRACK runs as subroutine of the software package ANSYS (Kralik and Cesnak 2001).

2. Inelastic design spectrum

If structure or its elements occur in the plastic domain, the design spectrum can be reduced by ductility factor (Chopra 2001). The standardized ultimate strength characterizes elasto-plastic behavior (Fig.2).

$$\overline{f}_y = \frac{f_y}{f_o} = \frac{u_y}{u_o} = \frac{1}{R_y} \tag{1}$$

where f_o and u_o are peak values of a seismic response force and deformation at the linear behavior system corresponding to Elasto-plastic system. The value of the parameter f_y is evaluated as a function f_o through the decrease factor R_y of the ultimate strength value.

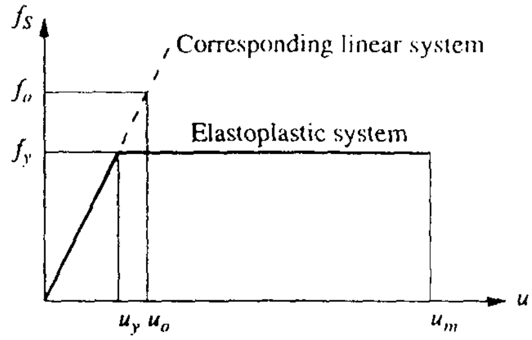


Fig. 2: Elasto-plastic system and its corresponding linear system

u_m value defines either peak or absolute value of a seismic displacement induced in the elasto-plastic system. Ductility factor is defined (Chopra 2001) as a ratio which normalizes the displacement in relation to the maximum displacement value in the elastic domain. (For instance there is a case when the first plastic hinge occurs in the frame.):

$$\mu = \frac{u_m}{u_y} \tag{2}$$

The ductility factor is equal to 1 (μ = 1) if $f_y = f_o$ in the case of elasto-plastic behaviour of structure. Moreover is valid follow

$$\frac{u_m}{u_o} = \mu.\overline{f}_y = \frac{\mu}{R_y} \tag{3}$$

One of the most often used simplified interpretation of $\overline{f}_y$ or R_y (Chopra 2001) is the following:

$$\overline{f}_y = \begin{cases} 1 & T_n < T_B \\ (2\mu-1)^{-1/2} & T_B < T_n < T_C \\ \mu^{-1} & T_n > T_C \end{cases} \quad \text{or} \quad R_y = \begin{cases} 1 & T_n < T_B \\ (2\mu-1)^{1/2} & T_B < T_n < T_C \\ \mu & T_n > T_C \end{cases} \tag{4}$$

Factor q described by Eurocode (EC) – standards and also by a national standard denoted STN 73 0036 is comparable to decrease factor R_y of the ultimate strength value. Factor R_y is used by the U.S. technical literature and standards (FEMA 302).

3. Seismic load based on Eurocode

The term behavior factor q, presented by national standards STN ENV (1999) and ENV 1998 (2003) devoted to seizmicity, reflects: failure intensity of the structure depended on their importance, a subsoil, a ductility and structural regularity classes. Its value equals 1.0 in the case of linear behavior.

Basic relations dedicated to an estimation of the elastic and design acceleration response spectrum are compiled in table 1 and 2, where $S_e(T)$ denotes ordinate of the design spectrum, T – vibration period of a linear single degree of freedom system, a_g (a_{vg}) – horizontal (vertical) design ground acceleration in return period of the occurrence, T_B, T_C - limits of the constant spectral acceleration branch, S – soil parameter, q – behavior factor depended on importance of a structure. η- damping correction factor with reference value η=1 for 5% viscous damping.

Tab. 1: Elastic and design response spectrum Type 1 by ENV 1998-1

Period	Elastic response spectrum	Design response spectrum
	ENV 1998-1 (2003)	ENV 1998-1 (2003)
	Horizontal spectrum	
$0 \le T \le T_B$	$S_e(T) = a_g.S.\left[1+\frac{T}{T_B}.(\eta.2,5-1)\right]$	$S_d(T) = a_g.S.\left[\frac{2}{3}+\frac{T}{T_B}.\left(.\frac{2,5}{q}-\frac{2}{3}\right)\right]$
$T_B \le T \le T_C$	$S_e(T) = a_g.S.\eta.2,5$	$S_d(T) = a_g.S.\frac{2,5}{q}$
$T_C \le T \le T_D$	$S_e(T) = a_g.S.\eta.2,5\left[\frac{T_C}{T}\right].$	$S_d(T) = a_g.S.\frac{2,5}{q}\left[\frac{T_C}{T}\right] \ge 0,2.a_g.$
$T_D \le T$	$S_e(T) = a_g.S.\eta.2,5\left[\frac{T_C.T_D}{T^2}\right].$	$S_d(T) = a_g.S.\frac{2,5}{q}\left[\frac{T_C T_D}{T^2}\right] \ge 0,2.a_g.$
	Vertical spectrum	
$0 \le T \le T_B$	$S_{ve}(T) = a_{vg}.S.\left[1+\frac{T}{T_B}.(\eta.3,0-1)\right]$	$S_{vd}(T) = a_{vg}.S.\left[\frac{2}{3}+\frac{T}{T_B}.\left(.\frac{2,5}{q}-\frac{2}{3}\right)\right]$

$T_B \le T \le T_C$	$S_{ve}\left(T\right)=a_{vg}.S.\eta.3,0$	$S_{vd}\left(T\right)=a_{vg}.S.\frac{2,5}{q}$
$T_C \le T \le T_D$	$S_{ve}\left(T\right)=a_{vg}.S.\eta.3,0\left[\frac{T_C}{T}\right].$	$S_{vd}\left(T\right)=a_{vg}.S.\frac{2,5}{q}\left[\frac{T_C}{T}\right]\ge 0,2.a_{vg}.$
$T_D \le T$	$S_{ve}\left(T\right)=a_{vg}.S.\eta.3,0\left[\frac{T_C.T_D}{T^2}\right].$	$S_{vd}\left(T\right)=a_{vg}.S.\frac{2,5}{q}\left[\frac{T_C T_D}{T^2}\right]\ge 0,2.a_{vg}.$

The recommended values of parameters for spectrum type 1 in the table 1 are following

$$a_{vg}=0,9a_g, \quad T_B=0,05s \quad T_C=0,15s \quad T_D=1,0s \tag{5}$$

The q factor established in STN ENV 1998 1-1 to 3 is expressed in the following form:

$$q=q_o.k_w>1,5 \tag{6}$$

q_o – basic value of the behavior factor, depends on a type of the structure, k_w – factor reflecting prevailing failure mode in a structural system with walls.

Tab. 2: Comparison of q - factor in various standards

		Ductility factor „q“		
Standard	Level	Reinforced Concrete		
		Frames multistory	Shear walls	Frames & shear walls
STN P ENV (1999)	DCL	2,5	2,00	2,25
	DCM	3,75(3,00)	3,00(2,40)	3,38(2,70)
	DCH	5,00(4,00)	4,00(3,20)	4,50(3,60)
ENV 1998-1 (2003)	DCL	1	1	1
	DCM	3,90(3,12)	3,00(2,40)	3,90(3,12)
	DCH	5,85(4,68)	4,40(352)	5,85(4,68)
FEMA 368 (2001)		3-6	4-5	5,5
ÖNORM B4015(2002)	DCM	1,5	1,5	2,9
	DCH	3,0	2,5	3,0
CRACK (nonlinear calcul.)		-	-	2,37/2,84

Eurocode 8 advises for various structures following ranges of values for behavior factor: steel structures, concrete structures, non-bearing structures.

In table 2 there is shown a comparison of behavior factor values, depended on system of the structure and on criterion ductility. Wide scatter of ductility factor q values contributes to enhanced demand to verify their accuracy in respect to a type of the structural system and boundary conditions. The ductility magnitude of the structural system depends mostly on a load level.

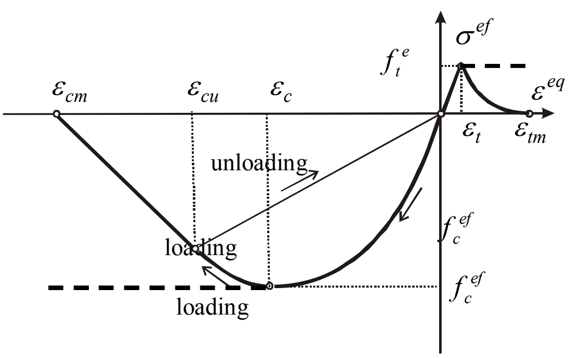
4. Nonlinear analysis of the reinforced concrete wall

Nonlinear analysis of seismic resistance is determined by the weakest member of a coupled frame – shear wall hybrid bearing system (Fig.1). It means that: resistance of the entire structure depends on the critical member's resistance. The failure in critical crosssections can evoke collapse of the entire construction (Wawrzynek et al. 2001, Juhásová et al. 2000). In the case of hospital facility (Flesch et al. 2003), there is a behavior factor determined by bearing capacity of the horizontal bracing system located on the 48th axle, the place of coupling shear walls and bracing frame.

Numerical model of the coupled frame - shear wall system is based on Červenka's reduction model (Červenka 1985) of a concrete strength and also on Kupfer's biaxial stress plasticity criterion in the plane of the principal stresses. A new program was developed called CRACK (Králik et al. 2001, 2005, 2006) containing before mentioned techniques. This program cooperates with the system ANSYS. The constitutive model presented is a further extension of the smeared crack model.

The smeared crack model, used in this work, results from assumption, that the field of more micro cracks (not one local failure) brought to the concrete element will be created. A validity of this assumption is determined by size of finite element, hence its characteristic dimension, where A is the element area (versus integrated point area of element).

One concrete layer was considered as orthotropic material for which the direction of a crack is the same as the direction of a principal strain.
In this model the Kupfer's bidimensional failure criterion of concrete is considered. The concrete compressive stress f_c, tensile concrete stress f_t and shear modulus G are reduced after the cracking of the concrete.



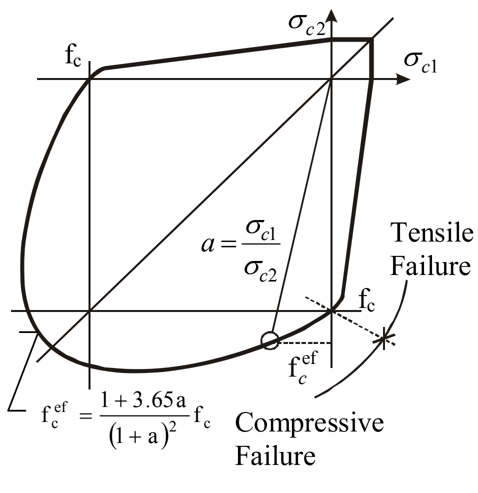


Fig. 3: Stress-strain concrete diagram

Fig. 4: Kupfer's plasticity function

The stress-strain relation (Fig.3) is defined following ENV 1992-1-1 (1991)

Loading in compression region

$$\varepsilon_{cu} < \varepsilon^{eq} < 0\,,\ \sigma_c^{ef} = f_c^{ef}.\frac{k.\eta - \eta^2}{1.+(k-2).\eta}\,,\ \eta = \frac{\varepsilon^{eq}}{\varepsilon_c}\ ,.(\varepsilon_c \doteq -0.0022, \quad \varepsilon_{cu} \doteq -0.0035) \qquad (7)$$

Softening in compress region

$$\varepsilon_{cm} < \varepsilon^{eq} < \varepsilon_{cu}\,,\ \sigma_c^{ef} = f_c^{ef}.\left(1 - \frac{\varepsilon^{eq} - \varepsilon_c}{\varepsilon_{cm} - \varepsilon_{cu}}\right) \qquad (8)$$

Tension region

$$\varepsilon_t < \varepsilon^{eq} < \varepsilon_m\,,\ \sigma_c^{ef} = f_t.\exp(-2.(\varepsilon^{eq} - \varepsilon_t)/\varepsilon_{tm}) \tag{9}$$

In the case of plane stress the strength function in tension and in compression was considered as equivalent values. In the plane of principal stresses can be defined the relation between the one and two stresses state due to plasticity function by Kupfer (Fig.4).
Compression-compression

$$f_c^{ef} = \frac{1+3.65.a}{(1+a)^2}.f_c\,, \qquad a = \frac{\sigma_{c1}}{\sigma_{c2}} \tag{10}$$

Tension-compression

$$f_c^{ef} = f_c.r_{ec}\,, \qquad r_{ec} = \left(1.+5.3278\frac{\sigma_{c1}}{f_c}\right), \qquad r_{ec} \geq 0{,}9 \tag{11}$$

Tension-tension

$$f_t^{ef} = f_t.r_{et}\,, \qquad r_{et} = \frac{A+(A-1).B}{A.B}\,, \qquad B = K.x + A\,, \quad x = \sigma_{c2}/f_c \tag{12}$$

$$r_{et} = 1. \Leftrightarrow x = 0\,, \qquad r_{et} = 0{,}2 \Leftrightarrow x = 1. \tag{13}$$

The shear concrete modulus G was defined for cracking concrete by Kolmar in the form

$$G = r_g.G_o\,, \qquad r_g = \frac{1}{c_2}\ln\left(\frac{\varepsilon_u}{c_1}\right), \qquad 0 < p < 0{,}02 \tag{14}$$

$$c_1 = 7.+333(p-0{,}005), \qquad c_2 = 10-167(p-0{,}005) \tag{15}$$

where G_o is initial shear modulus of concrete,ε_u is strain in the normal direction to crack, c_1 and c_2 are constants dependent on ratio of reinforcing, p is ratio of reinforcing transformed to the plane of crack.

The limit of damage at a point is controlled by the values of the so-called crushing or total damage function F_u. This is defined in the principal strain space in terms of the plastic strains at the point, the limit equivalent of plastic strain ε_{up}.

4.1 Function of concrete failure

Function of concrete failure (loss of integrity) can be defined in dependency to the components of principal stresses in the crack plane of layer l by the function of failure surface F_u^l. Thus

$$F_u^l = F_u^l(\varepsilon^p, \varepsilon_u^p, \xi) = 0\,, \quad F_u^l = \sqrt{\alpha_u\left[\left(\frac{\varepsilon_1^p}{\xi_1}\right)^2 + \left(\frac{\varepsilon_2^p}{\xi_2}\right)^2\right]} - \varepsilon_u^p = 0; \quad \frac{2}{3} \leq \alpha_u \leq 1 \tag{16}$$

where $\xi_i = 1$ *in compression*, $\xi_i = \left(\dfrac{\varepsilon_{tu}^p}{\varepsilon_{cu}^p}\right)$ *in tension* (17)

For the membrane and bending deformation of the reinforced concrete shell structure, we have chosen the SHELL91 layered shell element, on which we propose a plane stress on every single layer.

The stiffness matrix of reinforced concrete for the layer "l^{th}" can be written in the following form

$$\left[\mathbf{D}_{cr}^{l}\right]=\left[\mathbf{T}_{c}^{l}\right]^{T}\left[\mathbf{D}_{c}^{l}\right]\left[\mathbf{T}_{c}^{l}\right]+\sum_{j'=1}^{n}\left[\mathbf{T}_{s}^{l}\right]^{T}\left[\mathbf{D}_{s}^{l}\right]\left[\mathbf{T}_{s}^{l}\right] \tag{18}$$

where $\mathbf{T_c}$, $\mathbf{T_s}$ are the transformation matrices for concrete and reinforcement separately.

$$\left[\mathbf{D}_{cr}^{l}\right]=\begin{bmatrix} B^l E_x^l & B^l \mu_{xy}^l E_x^l & 0 & 0 & 0 & 0 \\ B^l \mu_{xy}^l E_x^l & B^l E_y^l & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & G_{xy}^l & 0 & 0 \\ 0 & 0 & 0 & 0 & \dfrac{G_{yz}^l}{k_s} & 0 \\ 0 & 0 & 0 & 0 & 0 & \dfrac{G_{zx}^l}{k_s} \end{bmatrix} \tag{19}$$

$$B^l=\frac{E_y^l}{E_y^l-\left(\mu_{xy}^l\right)^2 E_x^l} \tag{20}$$

where E_x^l (or E_x^l) is Young modulus "l^{th}" layer in the direction x (or y), G_{xy}^l, G_{xy}^l, G_{xy}^l are shear modulus l^{th} layer in plane *xy*, *yz* and *zx* ; k_s is coefficient of effective shear area

$$k_s=1+0{,}2\frac{A}{25t^2}\geq 1{,}2 \tag{21}$$

A is the element area, *t* is the element thickness.

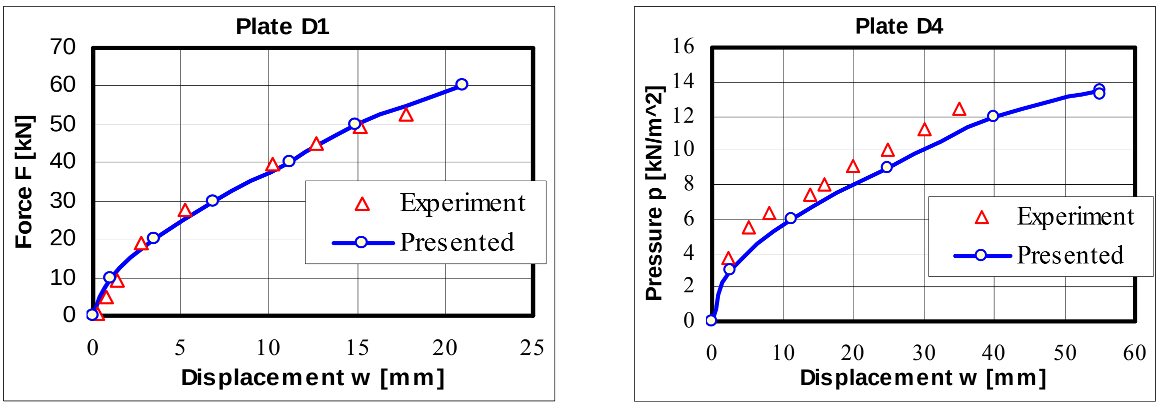


Fig. 5: Comparison of experimental and nonlinear numerical analysis of plates

The program CRACK was checked and the results were compared with experimental results of Hájek (Hájek et al., 1983, Križma et al., 2002).

The reinforced concrete plate D1 were loaded by force F in centrum and plate D4 by pressure *p* on the area of plate.

A critical structural fragment (side shear walls) was estimated by a linear spectral analysis. The nonlinear behavior of the hospital building was investigated in dependency of the ductility capacity of the reinforced concrete shear wall on the building side.

Estimated values of equivalent horizontal forces were calculated by constraint equations which conveniently match the reference assumption of horizontal degree of freedom values along the height of the building. Afterwards nonlinear analysis was carried out just for a selected part of the structure (Fig.5).

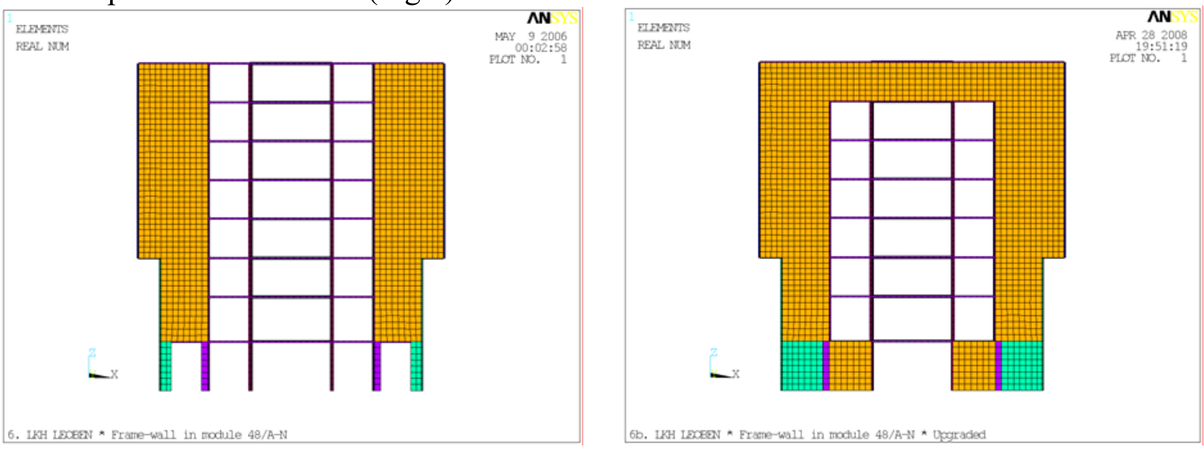


Fig. 6: FEM model of frame-wall system in modul 48/A-N

a) Original model b) Upgraded model

The ductility factor for the original model is follow

$$\mu_{\Delta}\Big|_{a_g=0.64} = \frac{\Delta_{nonlin}}{\Delta_{lin}} = \frac{0,02906}{0,02246} = 1,29 \qquad \mu_{\Delta}\Big|_{a_g=1.25} = \frac{\Delta_{nonlin}}{\Delta_{lin}} = \frac{0,10237}{0,04320} = 2,37 \qquad (22)$$

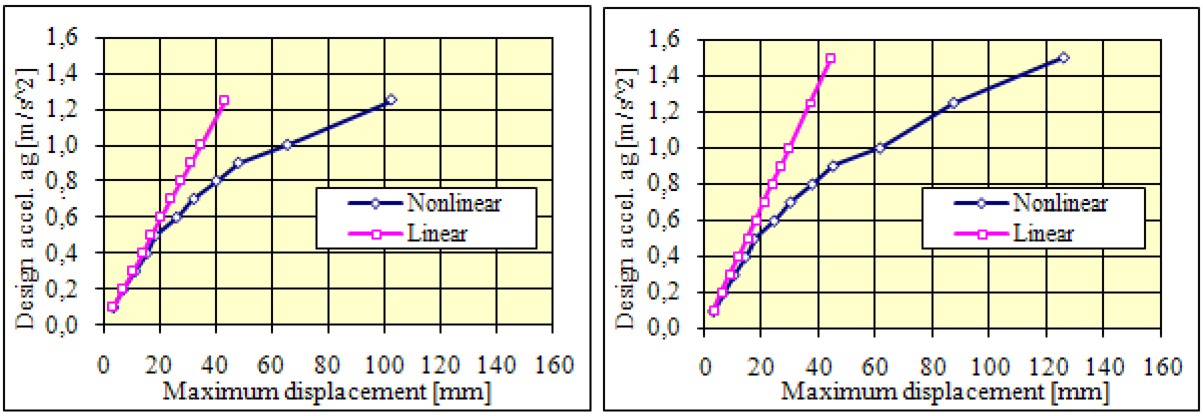


Fig. 7: Maximum displacement and design acceleration dependence

a) Original model b) Upgraded model

The ductility factor, described by the maximum load equation (2) according to (Flesch 1993, Chopra 2001), is semantic identical to the behavior factor q for period interval $T_B \leq T \leq T_C$. The value of the ductility factor obtained by calculations is evidently lower than the limit value defined by the Eurocode standard. But its value is the most similar to an Austrian national standard called ÖNORM B4015 (2002) recommended value.

The seismic load described by acceleration spectrum in Eurocode standard is calculated by equations compiled in table 1 in accordance to defined behavior factor. A comparison of acceleration design inelastic spectra on Fig. 8 shows that values at the range of their peaks (it means the frequency range between 2-7 Hz) are significantly reduced, i.e. frequencies which describe main mode shapes of the structural system.

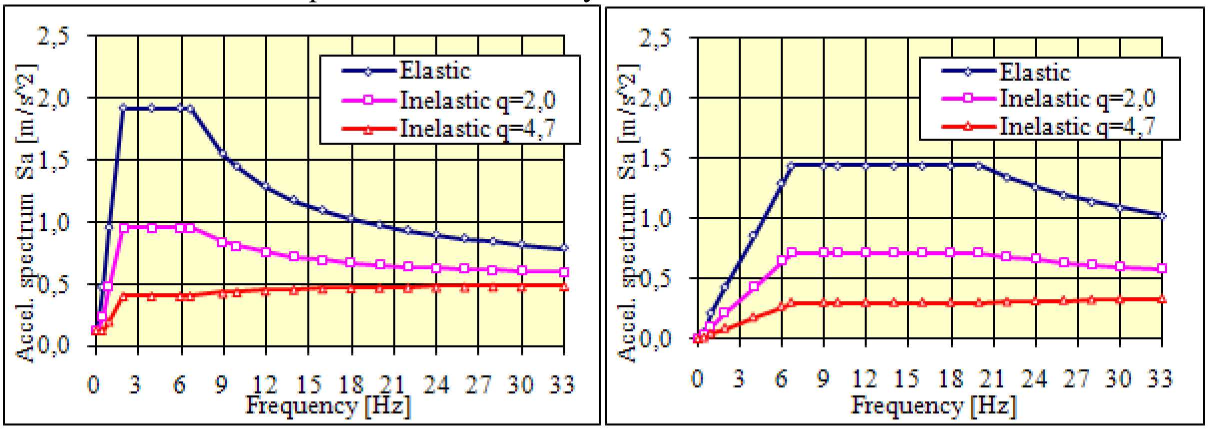


Fig. 8: Horizontal and vertical acceleration response spectrum

5. EXPERIMENTAL MODAL ANALYSIS

An experimental modal analysis (Fig.10-12) of the construction and basement was performed by Arsenal Research (Flesch et al. 2003). The numerical modal analysis (Fig.9) was realized in the program ANSYS.

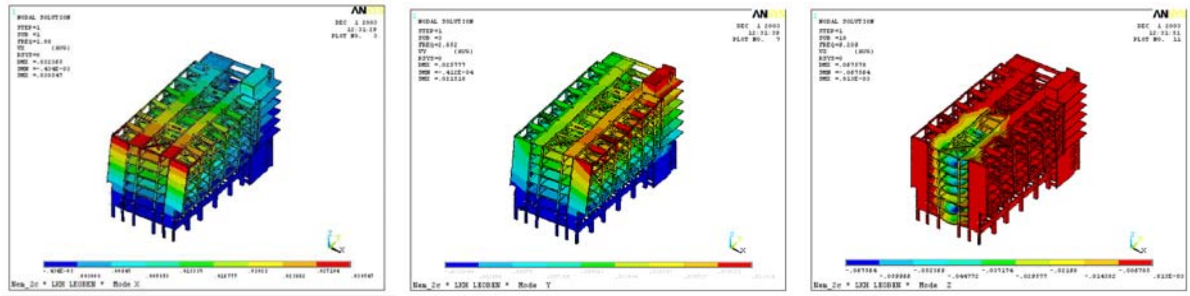


Fig. 9: The shape of dominant modes of the hospital structural model in X, Y and Z direction

Tab. 2: Critical modal shapes of hospital building

Model	Subsoil	Direction X		Direction Y		Direction Z	
	Vert./Horiz.	Frequency [Hz]	Effect.mass ratio [%]	Frequency [Hz]	Effect.mass ratio [%]	Frequency [Hz]	Effect.mass ratio [%]
Nem1	R/R	1,12866	68,095	1,52316	68,533	7,39420	59,346
Nem2	R/EH	1,06114	71,799	1,51682	69,231	7,39416	59,551
Nem3	EH/EH	0,86421	58,735	0,99064	80,775	5,54769	59,372
Nem4	EM/EM	0,69891	79,256	0,78654	91,525	2,77618	93,210
Nem5	EL/EL	0,62927	79,610	0,69355	92,000	2,00706	98,464
		Model with internal brick wall elements					
Nem3m	EH/EH	1,87976	68,022	2,85207	69,998	5,54867	47,241
		Experimental measured critical eigen-frequencies					
Experiment		1,88	-	3,12	-	5,56	-

Notes: – R – Rigid subsoil, EH – elastic with the high rigidity, EM – elastic with the medium rigidity, EL – elastic with the low rigidity

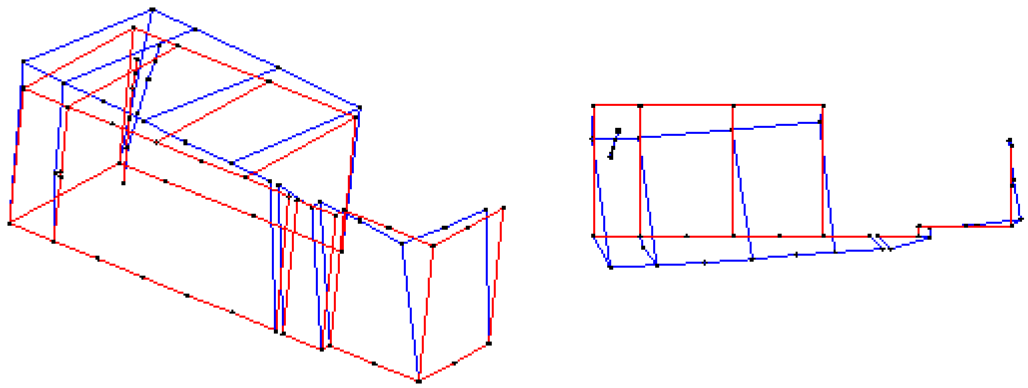


Fig. 10: The mode in X direction found experimentally for frequency value equals 1,88 Hz, viscous damping 11,4 % (Flesch, 2003)

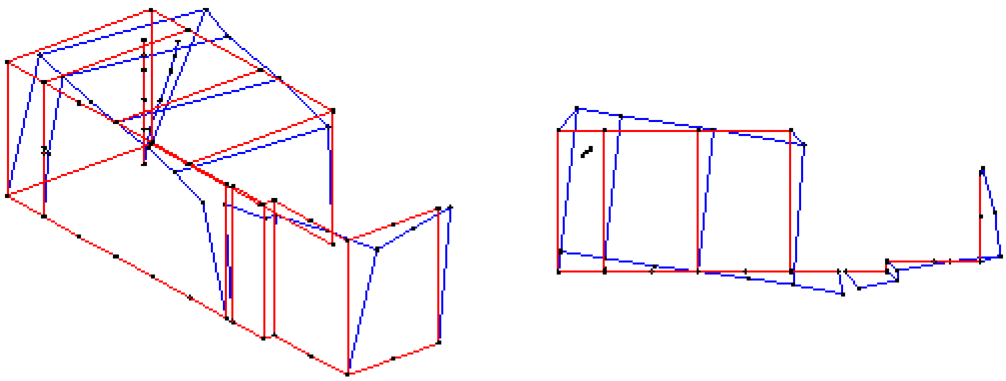


Fig. 11: The mode in Y direction found experimentally for frequency value equals 3,12 Hz, viscous damping 4,18 % (Flesch, 2003)

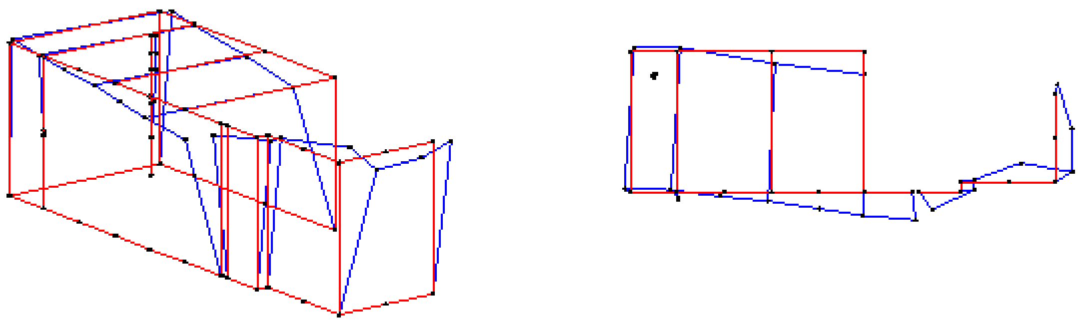


Fig. 12: The mode in Z direction found experimentally for frequency value equals 5,56 Hz, viscous damping 2,99 % (Flesch, 2003)

Stiffness parameters of the basement were specified (v_P = 1261 m/s - velocity of longitudinal waves v_S =600 m/s – velocity of transverse (shear) waves). The subsoil class was defined as B category of Eurocode scale.

The numerical modal analysis was realized for various subsoil rigidity. The comparison of the modal characteristics between the numerical and experimental analysis is presented in the table 4.

6. Conclusions

The application of behavior factor with respect to the failure of the structure significantly affects the design of structures in seismic regions (Flesch 1993). Projects of the structures become economically efficient. In the case of more variable structures, it means structures with irregular geometry in the horizontal as well as the vertical plane, hybrid structures combined with various bearing systems etc. there is necessary to verify the accuracy of the behavior factor value. This factor was achieved by nonlinear calculation method performed on the weakest element of the structure (Chopra 2001).

Factors of the behavior, described in recent standards, are in case of some irregular structures unsuitably defined. The performance of those values would result into incorrect conclusions, as the article’s shown. The nonlinear analysis, of the 2D critical substructures (wall, frame, core wall,..) subjected to the quasi load, presents the acceptable accurate view of its resistance.

In this paper was presented the nonlinear analysis of the concrete structures considered the concrete cracking and crushing, layered approximation of the shell elements with various material properties, orthotropic material depending from the direction of the rotated cracks and the orientation of reinforced steel, modified Kupfer's yield function, degradation of the shear modulus by Kolmar depending on the properties of the reinforcement (Králik et al. 2001). The design of structures with regard to an elastic response spectrum for return period of 475 years is ineffective.

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