

Base Isolation Compared To Capacity Design For Large Corner Periods And Pulse-Type Seismic Records

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Research Article

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Posted Date: October 25th, 2021

DOI: <https://doi.org/10.21203/rs.3.rs-949488/v1>

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Base isolation compared to capacity design for large corner periods and pulse-type seismic records

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ABSTRACT: Southern Romania experiences special soil conditions, leading to rather long corner periods and to an enlarged plateau of the response spectrum, with associated large displacement demands. Pulse-type ground acceleration records complete this unique seismic area. Research on the seismic behavior of structures built under these special conditions is limited and engineers are not comfortable with alternative solutions such as base isolation. This study investigates the seismic performance of a hospital building with the following two anti-seismic solutions: 1) stiffening, in line with the capacity design method and 2) base isolation. Base shear, structural drift and structural acceleration are compared for both approaches.

Keywords: seismic behavior, hospital building, dynamic nonlinear analysis, push-over analysis, large corner period area

Declarations

Funding: The authors want to acknowledge the financial support and the technical background offered by the Maurer Söhne Foundation.

Conflicts of Interest / Competing Interests: Not appropriate.

Availability of data and material: Framework and reinforcement drawings for the analyzed structure were kindly provided by S.C.Consild S.R.L, Bucharest, Romania.

Code availability: Etabs software, Perform software

Author's contributions: case study on the structural efficiency of base isolation in Southern Romania

1 INTRODUCTION

The paper addresses the issue of base isolation efficiency in long corner period areas under pulse-type seismic records, (see chapter 3). Southern Romania experiences intermediate-depth earthquakes (depths between 60 and 200 km) with low frequency content, seismic conditions that brought severe damage or collapse to a wide range of flexible vulnerable buildings during past earthquakes. Similar frequency content is typical only for Mexico City area, but there it is coupled with bright-band seismic records, (Pavel et al.,2018). Under these rather unique seismic conditions, the confidence of local practical

engineers in the efficiency of base isolation is low. Bucharest, the capital city of Romania counts nowadays only 4 base-isolated buildings: the Victor Slavescu Building of the Management Studies Academy, the City Hall, the Arc de Triomphe, the INFLPR Magurele. Worldwide the issue of base-isolated structures in near-fault areas (although for corner period values T_c of up to 1,0 s) has been addressed for example in California, at the Loma Linda Medical Center (Nielsen et al., 2017) and Christchurch Women's Hospital (Kuang et al., 2016). The authors have chosen an existing hospital building in Bucharest (corner period $T_c=1,6s$, see figure 3), plan and elevation irregular, for which construction drawings were available (see chapter 2). Advantages and structural impact of base isolation are outlined.

Base isolation of buildings is considered worldwide a suitable alternative to the classical design approach based on the capacity method, (Lewis, 2017; Mayes et al., 2019; De Domenico et al., 2020). Due to decoupling of the super-structure from the foundation ground the earthquake input for the isolated building was reduced dramatically. In this way not only structural elements, but especially the building content (nonstructural elements, equipment, furniture) are protected from damage. Currently, an increasing number of people choose base-isolation for the seismic protection of their homes. For example 2017 in Japan, approximately 4300 commercial and multi-family residential buildings and more than 5600 single family homes were provided with base isolation systems, (Mayes et al., 2019).

Especially, hospital buildings should remain operational after important seismic events, in order to shelter and help injured people, (Arranz et al., 2017). Immediate occupancy requirements can be achieved by aiming an elastic seismic response. Two possible structural solutions are investigated: 1) a stiffening design approach – expecting high response acceleration and damage (Moehle, 2014; Derecho et al., 2001; Fajfar, 2018; Postelnicu et al. 2012) and 2) base isolation (Mazza et al., 2018; Reddy et al., 2019; Sorace et al., 2014).

Despite the advantages that base isolation offers (reduced damage expectation, functionality after important seismic events), its efficiency is still questionable in high corner period regions, which request high displacement demands, (Pavel et al., 2018; Oliveto, 2020). Therefore the authors conducted a study on an existing hospital building built in Bucharest, the capital city of Romania (it is a high corner period region, $T_c=1,6s$, Romanian Seismic Design Code P100-1/2013; Pavel et al., 2020; Sokolov et al., 2009; Grecu et al., 2018).

Two design approaches, a classical stiffening solution and a base isolation design were analytically compared. Modal analysis (for design) and static as well as dynamic nonlinear analysis (for checking the structural behavior) were performed (Wilson, 2000; Narasimhan et al., 2006; Spacone et al., 2007). For estimating the building's modal frequencies in the case of elastic behavior, ambient vibration measurements were performed on the hospital building, under normal occupancy conditions (Köber et al., 2019). Further experimental investigations, for example for the validation of analytical results were not possible, especially because of the analysed structure large dimensions.

Due to their special characteristics (reduced sliding path due to double curvature and enlarged damping by properties of sliding surface) Curved Surface Sliders were chosen for the base isolation solution, (Saitta et al., 2018; Arranz et al., 2017; Weber et al., 2018). Other slider types (like High Damping Rubber Bearings or Lead-Plug Rubber Bearings) were not investigated due especially to their larger sizes, characteristic that enlarges the costs of the base isolation structural solution.

2 Hospital building

The analyzed RC frame structure is part of a state hospital in Bucharest, has a plan layout of approximately 12x28 m and a building height above ground equal to 16,55m. The building has one underground and five over ground levels with level heights between 3,2m and 3,5m. A setback of up to 30% affects the first two levels in X-direction (see red area in Figure 1a), indicating elevation irregularity (P100-1/2013).

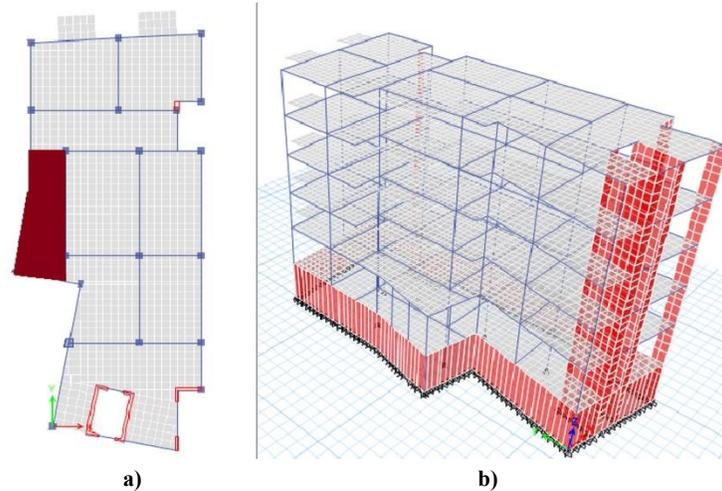


Fig. 1 ETABS structural model (CSI 2013): a) ground floor plan layout; b) 3D-view

Plan irregularity is also present, especially due to the position of the vertical circulation wall assembly. Displacements along the structural perimeter for earthquake in X-direction are amplified by 37% compared to the mean floor displacement, (P100-1/2013).

The original hospital building was designed according to the capacity spectrum method (P100-1/2006, the former version of P100-1/2013) and was built in 2012. Limited research on base isolation, in long corner period areas for pulse-type accelerograms, has been performed so far (Pavel et al. 2018). Therefore, to help practical engineers gain confidence in the base isolation alternative to classical design methods, even under special seismic conditions, the current study has been planned. Bucharest was chosen for the study as it is situated in the southern part of Romania, in a seismic area with corner period $T_c=1,6s$ and pulse-type accelerogram registrations. So the original building layout has also been designed with an isolated base. Dimensions for current structural elements are shown in Table 1 for both design approaches.

Table 1. Current structural element dimensions (cm)

Element	Classic design	Design with base isolation
beams ($b_w \times h_w$)	40 x 50, 40 x 60	25 x 50, 25 x 60
columns ($b_c \times h_c$)	60 x 60	50 x 50
walls (b_w)	20	20
slabs (h_{sl})	15	15

A 60cm thick foundation slab supports both structures. The isolation level is placed at the bottom of the underground level. Isolators are supported by the foundation slab and connect to a foundation beam girder, which supports the structure. This structural solution turned out to be more cost efficient than a double foundation slab, although it preserves less usable area.

The design of both structural solutions was performed using the ETABS software (CSI 2013), considering modal analysis. Once designed, the seismic performance of the structures was investigated in the nonlinear range of

behavior by using the PERFORM software (CSI 2006). Static and dynamic nonlinear analysis has been performed. A 3D-view of the PERFORM software structural model is shown in Figure 2. The PERFORM software offers the advantage of fast dynamic nonlinear computation whereas the ETABS software represents the structural system more accurately.

Base shear, structural displacement and structural acceleration are compared for both design approaches. Sliding isolators with double curvature were considered to reduce the dimensions of the isolators and to optimize the structural design. The isolator design was performed to obtain a minimum response spectrum ordinate for a chosen Eigen period of the isolated structure equal to 4,4s. According to P100-1/2013 the design earthquake has 225 years return period. The maximum expected earthquake was considered according to EN 1998-3: 2005, having a return period of 475 years. An isolator displacement capacity of 64,2 cm was designed.

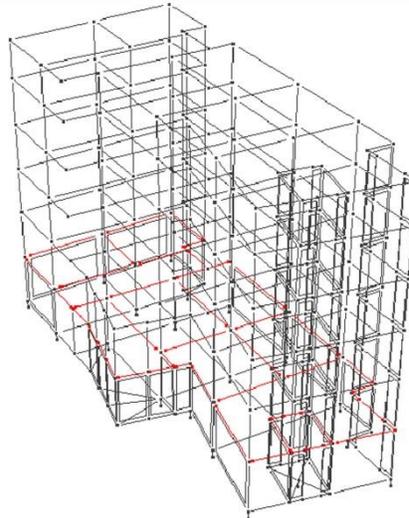


Fig. 2 3D structural view, CSI 2006

3 Seismic input of Bucharest region

The earthquake source Vrancea affects most of the Romanian territory, including Bucharest and produces intermediate depth seismic events with low frequency content. Therefore long corner period values (of up to $T_c=1,6s$) are typical for the southern part of the country (Figure 3a), whereas design ground acceleration values of up to 0,3g considering a return period of 225 years, are expected (Figure 3b).

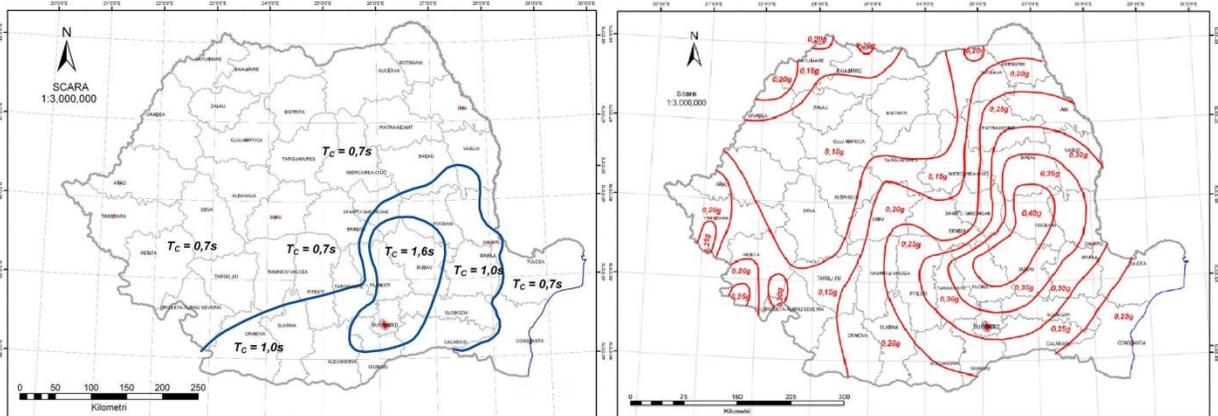


Fig. 3 Seismic area information for Romania, (P100-1/2013): a) Corner period values, T_c distribution; b) Design ground acceleration values for earthquakes with return period of 225 years

Despite long corner period values and a high design ground acceleration expectation, a third characteristic joins the group of particularities, making the southern part of Romania a rather unique seismic area (Pavel et al. 2018). It is the pulse-type ground acceleration record that is typical for strong Vrancea seismic events (Figure 4). Such earthquakes concentrate more than 90% of their energy in only one pulse.

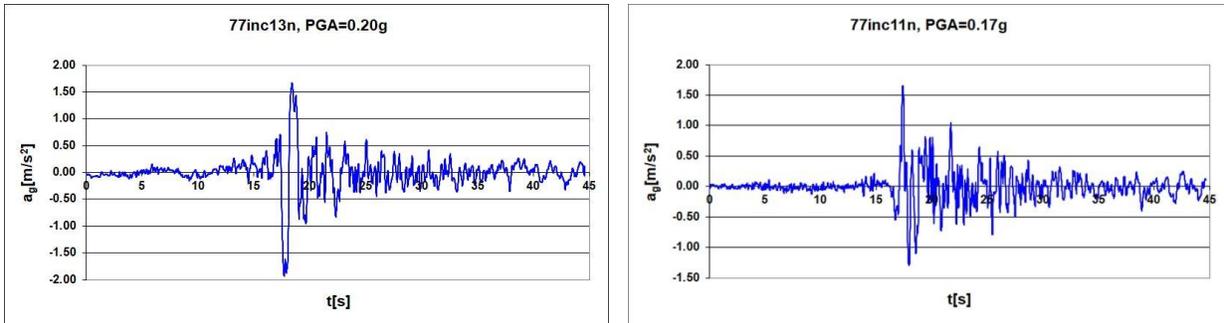


Fig. 4 Horizontal acceleration records from the Vrancea earthquake of March,4 1977 ($M=7,4$ Richter Scala), INCERC monitoring station: a) NS direction; b) EW direction

These special seismic input characteristics provide extremely high displacement demands (over 45 cm in Bucharest and over 60 cm in areas with expected design acceleration of $0,4g$), which buildings should withstand when a design earthquake occurs. If capacity design is applied, the high displacement demand directly influences the strength and ductility of the lateral force resisting structural elements.

According to the limited knowledge of the authors, such long corner periods linked to high expected horizontal acceleration occur otherwise only in the Mexico City area. The earthquakes in Mexico City area produce wide band acceleration records. This means that the earthquake energy is distributed over a large frequency range and so the influence on the structural elements is diminished compared to a pulse-type accelerogram, (Pavel et al., 2018).

Therefore, the current study on strong earthquakes generating pulse-type acceleration records in long corner period areas has been considered relevant.

4 Finite element modeling

To comply with the different stages of the current investigation, two softwares were used:

- the ETABS software (CSI 2013) for design (modal analysis and vertical load distribution);
- the PERFORM software (CSI 2006) for structural performance check (static and dynamic nonlinear analysis)

The ETABS model (Figure 1) considers beams and columns as frame elements and slabs and walls as surface elements. Vertical loads were applied as uniform loads on slabs and linear distributed loads along the perimeter beams (to account for the weight of nonstructural closure walls). Wind loads were neglected, due to their inferior influence on the structural behavior, compared to the effect of horizontal earthquake loads.

The PERFORM model (Figure 2) provides frame elements for beams and columns, having concentrated plastic hinges at their ends. The behavior of the plastic hinges was defined with the help of trilinear bending moment-rotation curves. The rotation capacity of the plastic hinges was computed according to EN 1998-3, considering mean strength values for concrete and steel. Walls were modeled as surface elements with fiber section. The geometry and material characteristics control their nonlinear behavior (for bending and axial force). Their shear behavior was considered elastic. In order to model the wall-beam connection (especially when beams connect to a perpendicular wall) columns with fiber section were defined at wall ends. These columns model the axial behavior of wall ends, whereas the contributing wall area assures the bending behavior of the columns perpendicular to the wall. As

alternative for modeling columns at wall ends, frame elements having axial plastic hinges were considered, but convergence problems were encountered.

The stiffness of structural elements was reduced at 50% to account for the cracked concrete stage (P100-1/2013).

Details regarding the first six Eigenmodes for the over ground structure are presented in Table 2.

Table 2 Eigenmodes overground structure, ETABS software

mode number	1	2	3	4	5	6
frequency [Hz]	1.75	2.03	2.30	5.56	7.04	8.06
mass ratio [%]	66%	→ 65%	↑ 15%	9%	→ 10%	↑ 6%
	translation	translation	→ translation	2% translation	4% translation	7% torsion
			51% torsion	torsion	torsion	

5 Curved surface slider as base isolator

Given the particularities of the seismic source Vrancea (see also chapter 3) CSS-Curved Surface Sliders were chosen for the isolation system. The two most important advantages of these sliders are the following: 1) the reduction of the isolator displacement (due to the double curvature) and 2) the high damping of the earthquake-induced movement (due to friction along the curved sliding surface). In this way, the isolator dimensions are reduced to a minimum and reduced dimensions of the isolator supporting structural elements (for example the underground columns) are achieved, making the solution more cost-efficient.

Base isolation devices need to be designed for the design earthquake (DBE) and their sliding capacity must be checked under maximum expected earthquake (MCE) conditions (EN 15129).

The Romanian Standard for earthquake design (P100-1/2013) indicates for the seismic area of Bucharest a design earthquake with return period of 225 years and a maximum expected design ground acceleration of 0,3g. The characteristics of the isolator devices designed for DBE are shown in Figure 5.

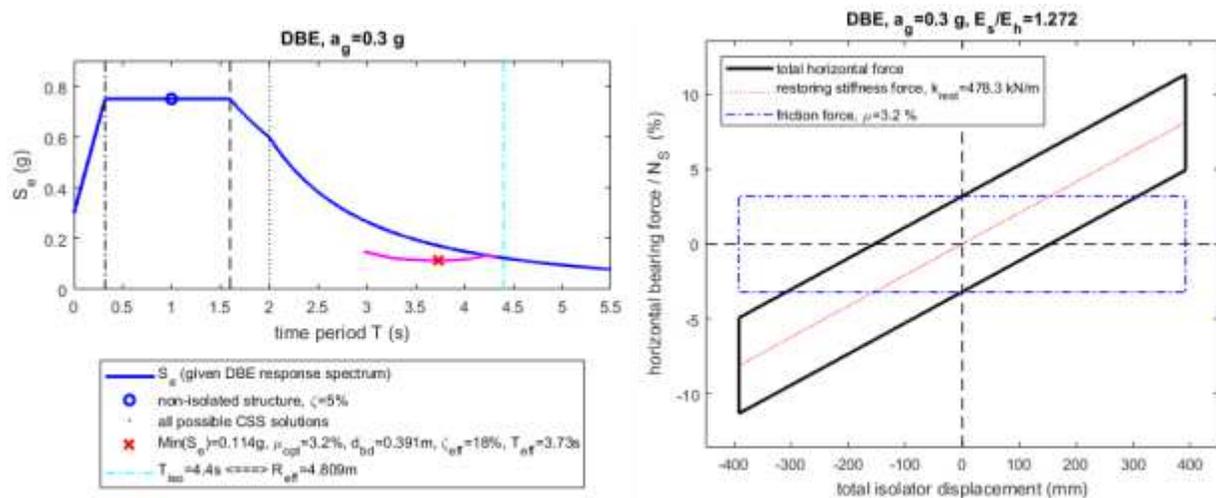


Fig.5 Design CSS: a) optimization; b) displacement capacity

An earthquake with return period of 475 years and maximum expected horizontal ground acceleration of 0,375g was considered as MCE event (EN 1998-1, P100-3/2019). The characteristics of the isolator devices designed for MCE are shown in Figure 6.

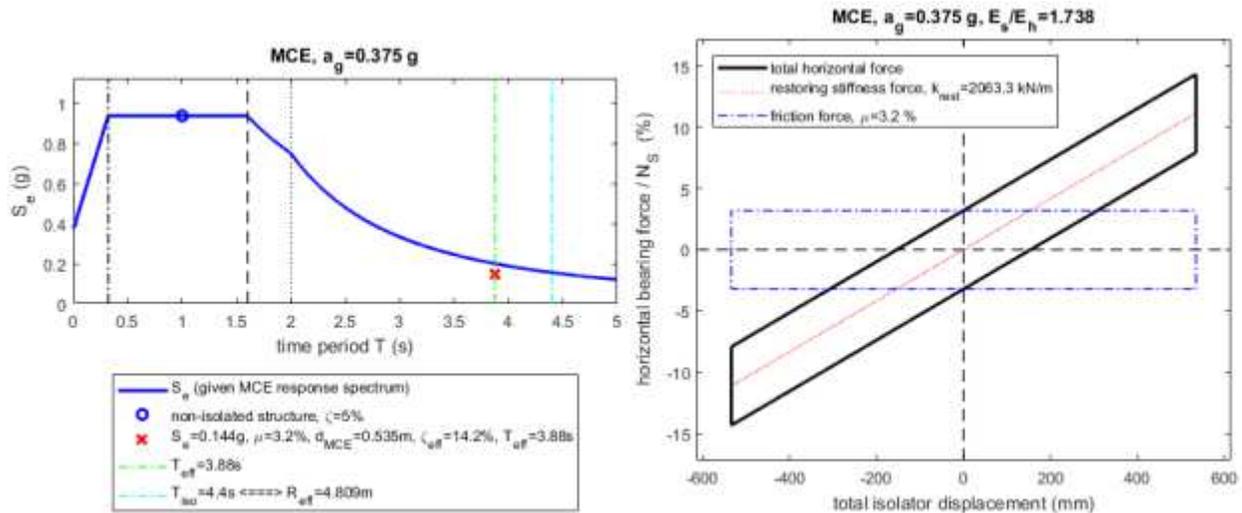


Fig.6 Behavior CSS for MCE: a) response spectrum; b) displacement capacity

In the PERFORM software the sliding isolation devices were considered with the help of horizontal force-displacement curves, indicating the restoring stiffness, the displacement capacity and the initial stiffness (until friction is overrun). The restoring stiffness and displacement capacity result from the isolation device design. The initial stiffness was chosen 100 times larger than the restoring stiffness (to limit the initial displacement, until friction is overrun). Supplementary, the following sliding isolation device characteristics were taken into account: the friction coefficient, the radius of the sliding surface and the stiffness for axial compression loads.

Because of the building layout (columns close to each other in some parts of the building) and in order to catch up with the elevator shaft, isolation devices were placed at the bottom of the underground level. A flat slab supports them and they support a foundation beam girder, to reduce material consumption for the foundation system.

6 Structural performance at design earthquake

The following two analysis were performed using the PERFORM software in order to check upon the nonlinear structural behavior:

- one static nonlinear (push-over) analysis for estimating the overstrength and the deformation capacity of the structure
- dynamic nonlinear computations to follow the structural behavior for the entire seismic record

For the dynamic nonlinear analysis, three original accelerograms (Incerc Bucharest station seismic records from the Vrancea earthquakes of 1977, 1986 and 1991) and three spectrum compatible accelerograms (considering the design spectrum of P100-1/2013 for corner period $T_c=1,6s$) were chosen. Seismic action combination rules (100% in one main direction and 30% in the other main direction) were taken into account (P100-1/2013, EN 1998-1/2004). The vertical earthquake component was also applied. Results are presented hereinafter for the weak main structural direction (X-direction or small plan layout dimension).

Building designed according to the capacity method

The force-displacement curve (global earthquake force at the bottom of the over ground structure and displacement in the center of mass at last level) is shown in Figure 7 and results are explained in Table 3.

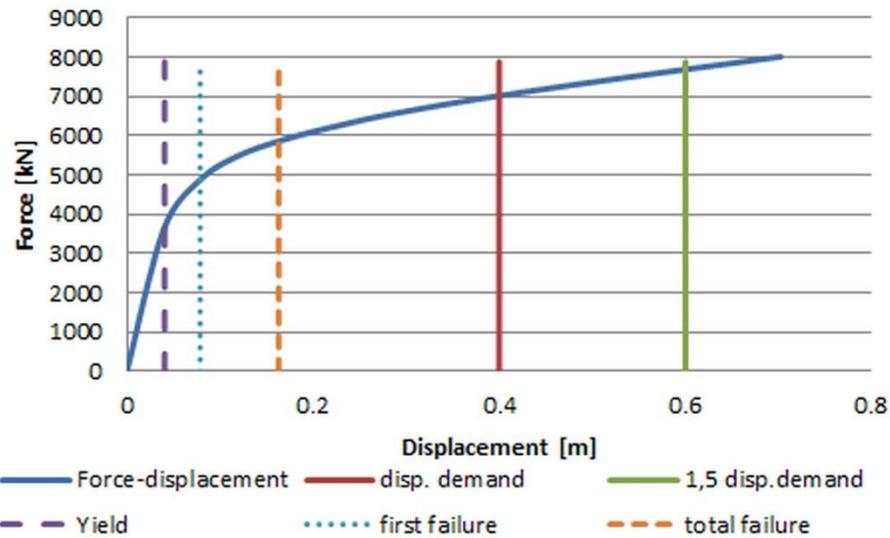


Fig.7 Force-displacement curve for the building designed according to the capacity method, X-direction

The displacement demand was determined according to Annex B of EN1998-1/2004 and equals 40 cm. Because a push-over investigation needs to be performed until 1,5 times the displacement demand, this limit is also represented in Figure 7.

Table 3 Details regarding the force-displacement curve shown in Fig.7

stage	horizontal force [kN]	displacement [m]	Remarks
yield	3655≈ 1,97design force*	0,041	-
first failure	4883≈ 2,7 design force*	0,079	30% of beam plastic hinges have failed
total failure	5861≈ 3,2 design force*	0,163	90% of beam plastic hinges have failed; 25% columns ground floor have failed

* Under design force we understand the horizontal earthquake force considered for structural design.

The overstrength of the building designed according to the capacity method equals 3: the ratio between horizontal failure force and building weight equals $5861/18972=0,3$. This is three times the global seismic coefficient considered for design. Therefore, the displacement demand in this case is not the right measure.

The behavior factor chosen for design considered a ratio $\alpha_w/\alpha_l=1,35$ (P100-1/2013). According to the force-displacement curve, this ratio equals $0,163/0,041\approx 4,0$. This is 2,9 times more than designed.

Results of the dynamic nonlinear calculations are shown in Table 4 for the building designed according to the capacity method. Mean values are shown when considering spectrum-compatible accelerograms.

Table 4 Results from dynamic nonlinear analysis, building designed according to the capacity method

Result	Incerc 1977 original	Incerc 1986 original	Incerc 1991 original	spectrum compatible $T_c=1,6s$
max. F_b [kN]	6108	3256	1260	7840

interstory drift [%]	7,40	2,80	0,86	12,10
	OG1	OG1	OG2	OG2
acceleration underground level [g]	0,20	0,10	0,08	0,36
acceleration building top [g]	0,38	0,23	0,11	0,50

Base-isolated building

Results of the dynamic nonlinear calculations are shown in Table 5 for the base-isolated building. Mean values are shown when considering spectrum-compatible accelerograms.

Table 5 Results from dynamic nonlinear analysis, base-isolated building

Result	Incerc 1977 original	Incerc 1986 original	Incerc 1991 original	spectrum compatible $T_c=1,6s$
max. F_b [kN]	1308	573	550	2068
interstory –drift [%]	*	*	*	*
acceleration underground level [g]	0,19	0,10	0,091	0,26
acceleration building top [g]	0,21	0,06	0,079	0,27

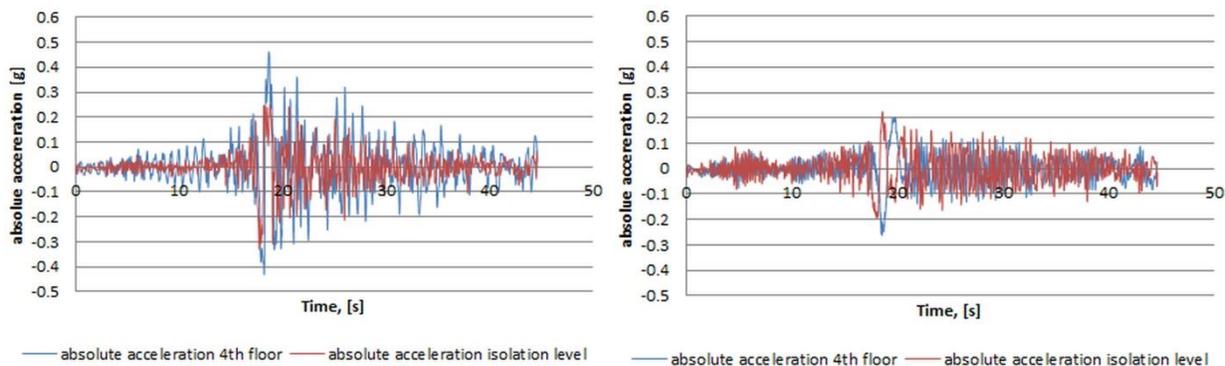
* negligibly small inter-story drift

Considering base isolation implies the following:

- a 45% maximum response acceleration drop (registered at building top)
- the isolation system takes over up to 90% of the earthquake-induced movement. Therefore the structure above the isolation level moves like a stiff box and no yielding (and so no damage) of structural elements is registered.
- a 65%-80% global horizontal earthquake force reduction

Considering the vertical earthquake component brings axial force variations of 4% in the range of high ground acceleration amplitudes. No axial tension forces are registered.

Figure 8 shows absolute acceleration time histories in the mass center for the strongest analyzed earthquake record (Incerc 1977 spectrum compatible) at the base of the structure (or quite over the isolation level for the base isolated structure) and at the building top. The isolated structure consumes most of the earthquake induced movement at the isolation level and just for the first strong pulse 37% acceleration amplification is registered at the building top. As comparison for the building designed according to the capacity method, base horizontal accelerations are amplified by up to 84% until the building top.



a)

b)

Fig.8 Absolute acceleration time histories, Incerc 1977 spectrum compatible ground acceleration record: a) building designed according to the capacity spectrum method; b) base-isolated structure

7 Concluding remarks

Base isolation turned out to be a valuable alternative to stiffening design, even under special seismic conditions (long-corner period area and pulse-type seismic horizontal ground acceleration records).

A 45% reduction of the maximum response acceleration is registered at building top. The isolation level takes over 90% of the earthquake induced movement. Up to 80% global horizontal earthquake force reduction is observed.

Supplementary to the evident structural advantages encountered for the base isolated solution of the analysed hospital structure, an economical comparison (as-built state as well as life-cycle analysis) is planned to be performed.

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