ABSTRACT

Systems for seismic isolation of engineering structures are not yet seriously considered in many seismically active countries. A typically used conventional aseismic design is based on the concept of increasing resistance capacity of ductile structural members, which results in overall stability during moderate and severe earthquakes. Most structural engineers apply well known antisismic guidelines in the process of dimensioning structural elements rather than using some of the passive or active systems for seismic isolation. High cost of most of these systems is the main culprit for their little practical use. This paper presents the results of a nonlinear response of a reinforced concrete building, after applying one of the simplest and easiest to use passive systems for seismic protection. This is a base isolation system in combination with devices for energy dissipation. Rubber-steel isolators are the main dampers in vertical direction, while lateral damping is entrusted to the system of double steel separated plates with welded steel bolts of different sizes circularly oriented. Depending on the seismic input, one row or two rows of steel bolts are activated in damping lateral movements of the upper structure, giving it a reduced seismic input. Various comparative diagrams are obtained using ETABS code after the nonlinear response of an excited multi-story RC building. These concern the followings: required reinforcement in the same structural model with and without base isolation, frame displacement, inter-story drift and sequence of plastic hinge formation with displayed influence changes in structural elements.

Keywords: Base isolation; energy dissipation; nonlinear response; inter-story drift; plastic hinge

1. INTRODUCTION

Engineering structures designed in seismically active regions, such as the entire Balkan region, require special attention in a structural model. From all load cases, correct seismic loading is the most significant input regarding global stability and optimization of the structural model. The use of computers has enabled a much larger number of iterations in the model, during which engineers gain better insight into the behaviour of the structure under all variable loads. In this way, they often enter the field of nonlinear analysis and plastic deformations, especially in the case of seismically isolated reinforced concrete structures; in order to maximize the potential of all structural materials applied (Naeim and Kelly, 2011).

Modern computers enable engineers to work not only with linear static and dynamic analyses, but with nonlinear structural calculations as well. Still, linear analysis can provide the correct results for dimensioning typical structures. Whether it is a linear analysis (Fig.1) based on strength (linear-static analysis), or on the basis of response-spectrum (dynamic linear analysis), the method of calculation is much simpler compared to non-linear analysis, with a shorter calculation time.

By using nonlinear analysis types, such as static-pushover or time-history-analysis, we can iteratively monitor structural behaviour through many iteration steps. The use of such iterative steps quickly
results in a new stiffness matrix at each stage of the nonlinear analysis. Since the computer software is
the most predictive tool in an engineer’s hands, critical points in a structure, e.g. plastic hinges, and
their position can be predicted. This fact is important because it directly affects the complete structural
rigidity and the entire structural behaviour. The best results are obtained by using time-history-
analysis, because the load in this case is a function of time, and so the equations of motion can be set
for each time step. The application of time-history-analysis can simultaneously take into account
material nonlinearity and P-Δ effects.

![Analysis Type]

<table>
<thead>
<tr>
<th>Analysis Type</th>
<th>Linear</th>
<th>Nonlinear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>Strength-based</td>
<td>Static-pushover</td>
</tr>
<tr>
<td>Dynamic</td>
<td>Response-spectrum</td>
<td>Time-history</td>
</tr>
</tbody>
</table>

Figure 1. Analysis methods.

Since the earthquake primarily represents a process of transmission, absorption and dissipation of
energy where structures are mainly demolished due to the high input energy, this fact has led to a new
philosophy in seismic engineering, which consists of developing methods for identifying possibilities
for modification and reduction of seismic effects on structures (Clough and Penzien, 1993). As
important public buildings, (like hospitals, schools, police and fire stations, bridges), should remain
functional after the earthquake event, the concept of protection using base isolation, i.e. passive
control of structures, has been increasingly applied (Yang et al., 2003). Flexible structures are more
susceptible to seismic actions, especially when there is no possibility of additional energy dissipation.
Depending on the control mode, there are three systems of protection against the earthquakes: passive,
active and hybrid (semi-active), see e.g. (Naeim and Kelly, 1999), (Ikonomou, 1989). Passive systems
do not use additional external energy for their work, while active systems use controllable systems
which induce additional energy (CEN-EN 1998-1, 2003). Hybrid systems combine passive and active
systems of protection.

In this paper, passive system has been used for numerical modelling, combining base isolation and
mechanical dissipation of seismic energy (Mayes et al., 2001). Base isolation is installed at the base of
the building, uncoupling a structure from the ground and reducing thereby the transmission of seismic
forces from the ground. Comparative analysis of 3D structural behaviour of reinforced concrete
building is also provided with and without using passive system of protection and includes rubber
isolators and steel dissipaters.

2. MATERIALS AND METHODS

2.1 Performance based design

After several strong earthquakes in major cities, e.g. Loma Prieta (1989) and Northridge (1994) in
California, as well as the 1995 Kobe earthquake in Japan, human casualties with material damage
(especially technical equipment in buildings), as well as costs of repairs and relocation of business and
commercial activities in densely populated urban areas became unacceptably high (FEMA356, 2000).
Seismic design based on performances represents new flexible philosophy (Tubaldi et al., 2014) and
modern comprehensive approach to seismic design of buildings and other structures, which enables
structural performances to be ensured for several different levels of seismic hazard (Wight and
McGregor, 2012). In 1995, under the supervision of the Federal Emergency Management Agency,
recommendations and guidelines were issued for the first time in the United States for seismic design
as a document FEMA-273, along with the later released documents the US/FEMA-350, FEMA-356,
FEMA-440, ASCE-31 and ASCE-41 (FEMA356, 2000). Although these recommendations were
initially intended to assist engineers in assessing the state of existing structures after very strong
seismic activity and their post-elastic behaviour, they are later included in all major international standards for seismic design of new structures (Figure 2).

With dynamic nonlinear analysis (Time history) a detailed calculation can be generated using either Fast Nonlinear Analysis (FNA) based on modal analysis, or the method of direct integration, where the equations of motion are set at each iterative step (Avramidis et al., 2016). By assigning seismic loads on the structure through ground displacement, speed or acceleration based on time histories records, behaviour of connections, elements and the structure as a whole under the effect of a given earthquake can be accurately calculated (Cosic et al., 2014). Depending on the type of building (residential, hospitals, schools, public institutions, ancillary buildings, temporary facilities) and estimated intensity of potential earthquakes, a designer determines how the structure should be "protected" or which degree of damage is permissible for permanent deformations (Spyrakos et al., 2009).

2.2 Capacity design method

The process in aseismic engineering, where the structural engineer determines which structural elements are allowed to be nonlinear (ductile components), while others fail (brittle components), is the Capacity design method. If the calculation is run by a capacity design, structure’s performances are deliberately determined by an engineer, and presented in a secondary manner by computational tools.

Figure 2. Qualitative definition of seismic performance levels (Avramidis et al. 2016).

Figure 3. (M-ϕ) curves for different axial load (left) with 3D interaction diagrams for square section (Stanojev et al. 2016).
Capacity Design enables creation of a more reliable computational model, which should lead to a better structural design. Also, when an engineer knows which elements will be permitted to yield while other elements will behave elastically, material nonlinearity needs only to be modelled for ductile components, while components which will not yield need only to consider elastic stiffness properties.

Particularly sensitive main structural elements are the RC columns, which have to be ductile, especially on the lower floors. For example, Figure 3 shows a comparative diagram of (M-\(\phi\)) curves for a 40x40 RC column cross-section with 1% reinforcement, where the cross-section is exposed to normal forces of varying intensity. Depending on the analytic method, the influences on columns behaviour, in particular of axial forces, may vary very much. So, even tensile forces can be generated in lower columns using linear analysis under the impact of severe earthquakes. Still, by using nonlinear analysis, with formation of hinges in joints, variations in axial forces are less noticeable (Stanojev and Folić, 2016). This provides smaller cross-sections with smaller amount of reinforcement, especially in tall buildings and large structures. Column cross-sections are dimensioned so that gravitational normal force corresponds to the limit at the level of balance point, where the possibility of enduring a bending moment is highest.

![Figure 3. Comparative diagram of (M-\(\phi\)) curves for a 40x40 RC column cross-section with 1% reinforcement.](image)

For example, Figure 4 shows three identical column cross-sections, but with different reinforcement ratio. It can be seen that in the case of symmetrical reinforcement, the balance point is at the same axial value for both cross-section sides. In the case of asymmetrical reinforcement (Stanojev and Folic, 2016), the balance point moves depending on stiffness. So, the stronger side of reinforced cross-
section later yields in the reinforcement and concrete, but on the weak side the yield occurs earlier. Thus the case of asymmetrically reinforced cross-section is not suggested for cyclical, harmonic and stochastic loads, such as an earthquake. This is one of the reasons why the columns should be designed having a square or round section, with symmetrically distributed reinforcement along the section.

Unlike columns, the beam bending stiffness should be at least 25 to 40% lower than that of the corresponding column (Folie, 2015), to ensure the proper development of plastic hinges in them. Their behaviour in the post-elastic range is easier to predict than in columns, where it ensures interaction between axial forces and bending moments (P-M2-M3). In beams, plastic hinge represents a concentrated post-plastic (post-yield) behaviour in one or more degrees of freedom (mostly around horizontal axis 3 - M3), while other degrees of freedom remain elastic.

2.3 Designing base isolation in buildings

The philosophy behind the base isolated structure is aimed at extending the fundamental period of vibration, which reduces the earthquake induced shear force at the base, providing additional damping of or reducing the relative displacement along the isolator itself (Buckle, 2000). More benefit from base isolation system is achieved in rigid structures where the fundamental period of vibration of the fixed structure is less than 1.0 second. In these structures, the fundamental period can be extended to 1.5 to 2.5 seconds by installing a base isolation system (Yang et al. 2003). This technique is applicable in low and medium-high buildings, and is less effective in very tall buildings, given that the fundamental periods of vibration increase with the height of the building.

In solid ground, soil acceleration consists mainly of high frequency components, with low frequencies prevailing in soft ground. Therefore, base isolation is avoided in founding on soft ground because it is more harmful than helpful. In this case, the viscous and hydraulic dampers are used, (Tubaldi et al. 2014).

3. A CASE STUDY OF AN 3D RC BUILDING: STRUCTURAL RESPONSE WITH AND WITHOUT BASE ISOLATION

Using all previous research and methods, a numerical model of an RC multi-storey building is made here by using the ETABS software. A comparative analysis of results provides better insight in nonlinear response of building structure, with useful conclusions and remarks. Results will be given for modelling with and without base isolation.

Figure 5. Building plan and isometric view.
The case study concerns a multi-story RC building shown in Fig. 5. The building has a basement with height of 3.0m, ground floor with height of 4.5m and 4 stories each with height of 3.0m. The building appears a mostly regular story plan, with 5 frames on 5.0 m in the X direction, and 5 frames on 4.0 m in the Y direction.

The section dimensions of all columns are 40·40cm, while the cross-section of beams is 25/40cm. RC slabs have thickness of 20cm. Every structural element is made of concrete grade C30/37 and reinforcement grade B500. In the first model there is no base isolation at the basement level, while in the second model there are rubber isolators and energy dissipators above the ground slab.

The nonlinear analysis was conducted in the CSI ETABS 2015 software (CSI Knowledge base, 2014) using time-history record for El Centro earthquake (Imperial Valley, California, 1940), for duration of 12s, with the maximum acceleration peaks occurring between 1.5s and 2.5s of the earthquake duration. The time-step of applying the acceleration is 0.01 sec.

![Figure 6. Force-displacement diagram used in ETABS 2015 (left), with points of acceptance criteria for seismic performance on M-φ diagram (right).](image)

Plastic hinges at beam ends were set for the bending moment around the major axis M3, while in columns concentrated plastic hinges at the nodes were set for three degrees of freedom (P-M2-M3). The behaviour of structural elements in the plastic range is defined using a force-displacement diagram (Figure 6), with the marked points that correspond to the limit states defined through performance-based seismic design (PBSD) as: immediate occupancy (IO), life safety (LS) and collapse prevention (CP).

Firstly, a modal analysis was conducted on both models – without and with base isolation. Rubber isolators are modelled with linear effective stiffness and nonlinear stiffness and yield strength in X and Y direction, while in Z direction they behave linearly and have only effective stiffness. These isolators are placed under each supporting column. The modal analysis results in a very different behaviour and response values.

![Figure 7. First vibration mode on model without isolation (left) and with isolation (right).](image)
As can be seen on Figure 7, the forms for the first vibration mode are very different for the regular structure in comparison to the seismically isolated structure. The main difference is in the first modal period, since it is expected that devices for base isolation enlarge the horizontal movement of the entire structure. The second and third periods of vibration are also similar in comparison, see Table 1.

<table>
<thead>
<tr>
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<th>First period of vibration (sec)</th>
<th>Second period of vibration (sec)</th>
<th>Third period of vibration (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure without isolation</td>
<td>0,659</td>
<td>0,611</td>
<td>0,598</td>
</tr>
<tr>
<td>Structure with isolators and dissipaters</td>
<td>0,980</td>
<td>0,730</td>
<td>0,689</td>
</tr>
</tbody>
</table>

When both models are subjected to gravitational and to seismic load (El Centro earthquake), various changes in effects are registered between the models. Firstly, a change in maximum and minimum axial forces, where minimum values means more pressure, results in a larger domain of pressure forces in case with no isolators than with seismic isolators (Figure 8).

Figure 8. Axial forces in central frame of two models, depending on minimum and maximum values (Left: model without isolation, right: model with isolation).
This means that with seismic isolation we have minimal changes in pressure forces on the bottom columns, which are mostly vulnerable in the event of an earthquake. When there is no seismic isolation, the difference between minimum and maximum values of pressure forces in the bottom column are enormous (up to 3 times in central columns), which in combination with bending moments results in larger column dimensions and reinforcement in them.

The differences between designing conventional and seismically isolated structure can be seen also in the required longitudinal reinforcement (Figure 9). In the model without isolators, the required longitudinal reinforcement reaches 2% of column cross section, while in the model with isolators the main reinforcement is accepted as 1% of column cross section. At the beams, the situation is the same: although at the middle span the required reinforcement is almost the same in both cases, on supports 2 to 3 times as much reinforcement is required in the model without isolators. Since the structural members are the same in both models, it can be concluded that dimensions and reinforcement of the structural members in combination with seismic isolator can be reduced, which in most cases will justify slightly higher costs of isolation.

![Figure 9. Longitudinal reinforcement in the central frame of the two models (left: model without isolation, right: model with isolation).](image)

The frame displacement differences between the two models can be also analysed. For weak and relatively moderate earthquakes, the displacements on all building floors, and especially on upper floors, are mostly larger when seismic isolators are used. This is the first and main guidance point when an engineer decides which system will be used for reducing seismic input into the building. But, when a structure is subjected to a moderate or strong earthquake, a seismically isolated structure remains within an acceptable domain of lateral movement, while conventional buildings do experience large lateral movement, which, in combination with P-Δ effect, can induce unacceptable influences into structural members, mostly into columns.

The frame displacements of the two models under El Centro earthquake are shown on Figure 10, and it can be seen that they are almost doubled in the model without isolation. But, it is not only that: the inter-story drift is also a problem which appears next to large scale frame displacement on the model without isolators. In Figure 11 the maximum story drifts are displayed in unit less diagrams. On the isolated model, the maximum inter-story drift appears on the ground level (where isolators end) and on the upper floors it is almost negligible. On the model without isolators, the inter-story drifts are at maximum on the first floors above ground. This results in much lower bending and shears forces in
columns of the isolated structure, which is in correlation with the required reinforcement and dimensions of structural members.

Figure 10. Frame C displacements of the two models under the El Centro seismic load case (left: model without isolation, right: model with isolation).

Figure 11. Inter-story drifts of the two models under the El Centro seismic load case (left: model without isolation, right: model with isolation).
Finally, in Figure 12 maximum global story displacements are given for models with and without base isolation. In both cases the maximum story displacement occurs on the top floor (level beneath elevator slab). Displacement of the first floor significantly increases from ground floor in the model with isolation, while on the upper floors top lateral displacement is slightly lower compared to the non-isolated model.

Figure 12. Maximum global story displacements of the two models under El Centro load case (left: model without isolation, right: model with isolation).

4. CONCLUDING REMARKS

The nonlinear response of a reinforced concrete building, after applying a base isolation system in combination with devices for energy dissipation, has been computationally investigated by the ETABS code.

On the example of two 3D structural models exposed to the El Centro earthquake, the main difference is in the first modal period in the two models. As it is expected, devices for base isolation enlarge horizontal movement of entire structure compared to non-isolated model. Second and third period of vibration have also similar comparison.

Changes in maximum and minimum axial forces result in larger domain of pressure forces in case with no isolation then in case with seismic isolators and dissipaters, so this means that in case with seismic isolation we have minimal changes in pressure forces on bottom columns, which are the most vulnerable in event of an earthquake. In the model without seismic isolation difference between minimum and maximum values of pressure forces in bottom column are enormous (up to 3 times in central columns) which in combination with bending moments results in larger column dimensions and reinforcement in them. These results in required longitudinal reinforcement up to 2% of column cross section in the non-isolated model, while in the model with isolation the main reinforcement is accepted as 1% of column cross section. The situation is the same for beams: although at the middle span required reinforcement is almost identical in both cases, on supports there is 2 to 3 times more reinforcement in the model without isolators.
On the isolated model, the maximum story displacement appears on the ground level, where isolation ends, whereas on the upper floors it is almost negligible. On the model without isolators, the drifts are at maximum on first floors above ground. This results in a much lower bending and shears forces in columns of the isolated structure, which is in correlation with required reinforcement and dimensions of structural members. Displacement of the first floor significantly increases from ground floor in the model with isolation, while on the upper floors, the top lateral displacement is slightly lower compared to the non-isolated model. Since the structural members are the same in both models, it can be concluded that structural members in combination with seismic isolation can be reduced in dimensions and reinforcement, which will justify slightly higher costs for isolation system in most cases.

5. ACKNOWLEDGMENTS
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6. REFERENCES