DYNAMIC BEHAVIOR OF THE SEISMICALLY ISOLATED SNF CULTURAL CENTER IN ATHENS

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ABSTRACT - *The Stavros Niarchos Foundation (SNF) Cultural Center, houses both the National Opera and National Library of Greece. The structural design had to accommodate demanding architectural requirements as well as high performance seismic specifications. The latter included significantly higher seismic actions than those prescribed by basic Eurocode provisions, as well as a requirement to protect the valuable contents of the library during an ultimate limit state earthquake.*

In order to fulfill these needs a seismic isolation system was incorporated, using 323 friction pendulum type isolators (SIP), placed at a uniform level beneath the ground floor slab. Soil-structure interaction effects also had to be considered due to poor silty sands that bordered on liquefiable.

Key features of the design, site constraints and design parameters, as well as the choice of seismic isolation technique, are presented. The paper then focuses on the methodology for the seismic design which was performed in three stages using 3D finite element models, including: a) simple analyses using single-degree of freedom model – for initial scheming, b) dynamic response spectrum analyses – for detailed design and c) non-linear time history analyses using selected earthquake records – for verification of non-linear effects. The results demonstrate good correlation between different analysis techniques and provide a valuable insight into the behavior of two complex, seismically isolated structures under seismic loads.

Keywords: *Dynamic Analysis, Seismic Isolation, Friction Pendulum System (FPS, SIP), Nonlinear Time-History Analysis*

1 INTRODUCTION

The Stavros Niarchos Foundation Cultural Center (SNFCC) is currently under construction in the bay of Phaliron in Athens and, when complete in 2016, will house the National Opera and Library of Greece. The Stavros Niarchos Foundation is funding the project and commissioned executive architect Renzo Piano and a team of highly qualified engineering firms to design the various individual structures within the development.

The main structural challenges of the project are the very poor soil conditions, the high seismic performance expectations and the need to satisfy a demanding architectural concept. Thus the incorporation of a seismic isolation system of friction pendulum type was deemed necessary. This makes the project the third seismic isolated building project in Greece after the Onassis Cultural Center and the Acropolis Museum [Anagnostopoulos 2007, Huber et al. 2007, Giarlelis et al., 2008]. However the combination of scale, number of isolators, level of seismic demand and soil-structure interaction effects (SSI) make this project unique. Also in the aforementioned projects, IBC code provisions for the design of seismic isolation were used while for SNFCC, the brand new Eurocode 8 (EC8) was in effect and used for the first time.

This paper focuses on the methodology followed for the analysis and the design of the structures. The results show good agreement, from the simplest approach (analysis using a SDOF system) to the more sophisticated (non-linear time-history analysis), allowing a good insight into the dynamic behavior of the buildings to gradually be gained. Structural choices and solutions to problems rising from the application of seismic isolation are also briefly presented.

2 PROJECT DESCRIPTION

Architect Renzo Piano envisaged the SNFCC rising out of the landscape, with the green roofs of the structures being the continuation of a large park which covers the majority of the site (Fig. 1). The complex consists of various structures, the main ones being the seismically isolated buildings of the National Opera and Library of Greece and a non-isolated car park.

2.1 *National Opera*

The Opera (Figs. 1, 2) is the largest building on the site. It is a reinforced concrete (RC) structure, the dominant element of which is a cruciform shape formed by five adjacent cubes of the main and

Figure 1 - Visualization of the completed complex: Opera (left) and Library (right)

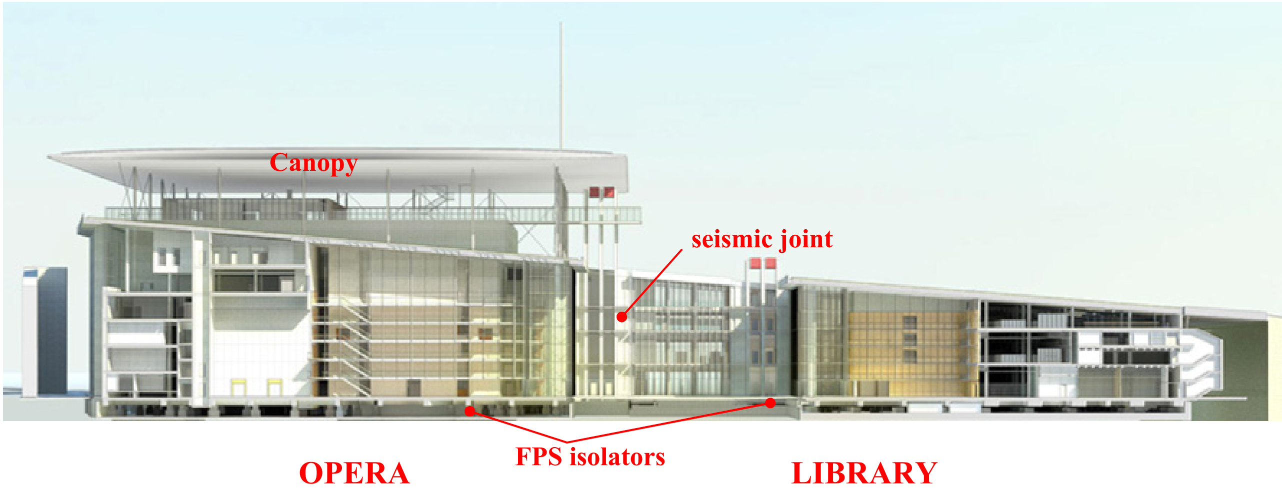


Figure 2 - Longitudinal Section of the National Opera and Library

rear stage, side stages and the auditorium. This cruciform forms the heart of the building – both architecturally, where it provides a home for the national opera's activities, and structurally, where the five cubes provide the lateral stability core. This core uses RC walls to provide stiffness in orthogonal directions as well as robust torsional resistance. Additional shear walls throughout the building further increase the overall stiffness.

The fly tower roof supports the terrace which is accessible to the public and has dimensions of 32.5m x 22.5m; the roof slab lies some 45m above the stage pit slab below. Structural walls define the perimeter of the fly tower and substantial openings at the base of these provide links between the main stage and the side stages, rear stage, and auditorium. To prevent buckling under axial loads and resist bending under seismic loads, RC piers provide additional out of plane restraint to the walls. Due to the need to ensure the isolators are in the same horizontal plane, as described in section 3.4, the stage pit is hung from the adjacent structure, free to move within the surrounding box of diaphragm walls. To create a column free auditorium, profiled RC ribs cantilever to support the main balconies.

In the foyer, steel balconies span between the external envelope of the auditorium and hangers from the roof above. The roof in this area is clear spanning to a perimeter column line, where high strength steel is used to allow the circular columns to span up to 26m without intermediate restraint. Cast-in, cruciform head details ensure that any lateral loading is adequately transferred into the inclined, voided roof slab which forms the continuation of the surrounding parkland.

On top of the Opera lies the Solar Collector, a ferrocement structure supported on 30 steel columns lying on the perimeter of the building. This is an innovative structure; its description is beyond the scope of this paper.

2.2 *National Library*

Like the Opera, the Library (Figs. 1, 2) is an RC building with flat slabs supported on an arrangement of columns and shear walls. It is a three-storey structure with predominantly open-plan floor space and a sloping, fully landscaped roof, using the same voided construction as previously mentioned. Storey heights are generally six meters, with local mezzanine levels for intensive book and document storage as well as an additional fourth storey where the roof slope permits.

A key architectural and structural component of the library is the 'book-castle', a 20m clear span space with waffle slabs and solid walls on all four sides. Continuity between the walls and slabs permits a reduced floor depth for such a span and allows sufficient head height for intermediate walkways within the reading rooms. Due to the open plan nature of the building, the shear walls are largely concentrated to the rear of the structure and therefore the book castle is a crucial element in bringing the torsional centre of the building closer to the centre of mass.

The book castle is adjacent to the full height atrium which forms the main entrance space of the library and elegant steel cantilever walkways are again supported from the book castle walls and roof hangers. The atrium columns span 20m and, as with the Opera, to minimise the diameter of the columns, high strength steel is used.

2.3 *Foundations and Soil-Structure Interaction*

As it has been previously mentioned the site presents poor soil conditions. A series of in-situ geotechnical investigation works included [Edafomichaniki 2008] (a) borehole drilling, (b) sampling for soil and water (c) geophysical tests (Down-hole tests). The project site is characterized by a flat terrain with a mild inclination towards the seafront. The soil stratigraphy consists of: (a) man made deposits up to a depth of 1.30m, which is underlain by (b) a coastal clay with a thickness of 1.30m, (c) a silty sand layer encountered from the depth of 2.60m up to 8.80m which involves a dense upper-half and a loose lower-half sub-layer, (d) the 'Alipedon' formation, a sandy silty clay layer which extends up to a depth of 12.60m, and (e) a marl formation encountered up to the maximum depth of the investigation (~45m). The marl formation is weathered in the upper 5-6 meters. The water table lies at a depth of 2.50m. It should be mentioned that this is a simplified but representative soil stratigraphy; there are differences in the thicknesses of soil layers from one part of the project to another that were taken into account for the foundation design and the assumptions for soil spring values used for considering the SSI effects.

A series of further geotechnical investigations were conducted using CPT, unconfined compressive strength tests and quick triaxial tests (UU). Also evaluation of the liquefaction resistance of soil materials was conducted according to the ASTM Standard [Athanasopoulos G. and Vlachakis V. 2010], with the outcome indicating a very low possibility of liquefaction for the loose sand material. The shear modulus and the shear wave velocity were defined for each soil layer. Shear wave velocities vary from 160m/s at the alipedon (sandy silty clay) to 500m/s for the non weathered marl.

Given these soil conditions, a piled foundation was deemed necessary for both structures. For the Opera, in addition to the piles, a series of diaphragm walls linked by a base slab were used to form an outer concrete box into which the stage pit is hung. Piles and diaphragm walls extend deep into the marl layer to an approximate depth of 20m.

3 SEISMIC DESIGN OF THE PROJECT

3.1 *Seismic specifications*

High performance seismic specifications were set by the client, reflecting the monumental aspirations for the development. A site specific spectrum was created [Gazetas, 2009] accounting for both the faults in the greater area and the local soil conditions. Thus, while Athens, according to EC8, is in zone I with a ground design acceleration of a_g=0.16g, a value of a_g= 0.267g was selected, incorporating an allowance for the soil factor S. In addition, the building importance factor was taken as equal to γ_I = 1.4, resulting in an increased return period for the design earthquake, normally considered as 475 years. The spectral acceleration amplification factor was taken as 3.0 instead of 2.5 reflecting again the soft soil conditions. Finally, a value of q=1.5, corresponding to quasi-elastic behavior, was assigned to the behavior factor in order to minimize damage to structural elements.

3.2 *Seismic isolation*

Preliminary studies indicated that the architectural concept and these high-performance seismic specifications could not be met using standard seismic design techniques. Thus the incorporation of a seismic isolation system was deemed necessary in order to:

- liberate the architectural concept from the limitations of seismic design

- achieve operational structural performance levels
- protect the contents of the structures and continue its function in case of a moderate to strong earthquake
- reduce foundation loads which is critical for the poor soil conditions of the project

With specific consideration of the architectural concept, seismic isolation allows for:
- considerably lower seismic acceleration for design of finishes, fittings and secondary elements
- a delicate façade for both the Opera and the Library which could not have been otherwise accomplished
- reduced interstory drifts
- reduced floor accelerations resulting in an increased protection for the recording equipment of the Opera and the artefacts housed in the Library
- reduced size and number of structural elements thus making the structures more elegant

3.3 *Selection of the seismic isolation system*

Three types of base isolation systems were considered: elastomeric bearings, steel spring bearings and pendulum bearings. The selection was based on the advantages or disadvantages of each system for the particular project. Analytically:

3.3.1 *Elastomeric bearings*

Elastomeric bearings are the most widely used type of seismic base isolation mainly because in general they have the lowest initial cost. The bearings often contain a lead core to increase energy dissipation and hence damping of the seismic response of the building. The two principal disadvantages of elastomeric bearings, particularly in regards to this specific project are:

i) The behavior of a base isolation system using elastomeric bearings varies with the vertical load applied to the bearing. For most buildings the self weight of the building structure is much larger than the transient live loads and this effect can be neglected. In this instance, the potentially high live loads associated with book storage in the Library and the highly variable loading in the Opera House mean that this is not the case.

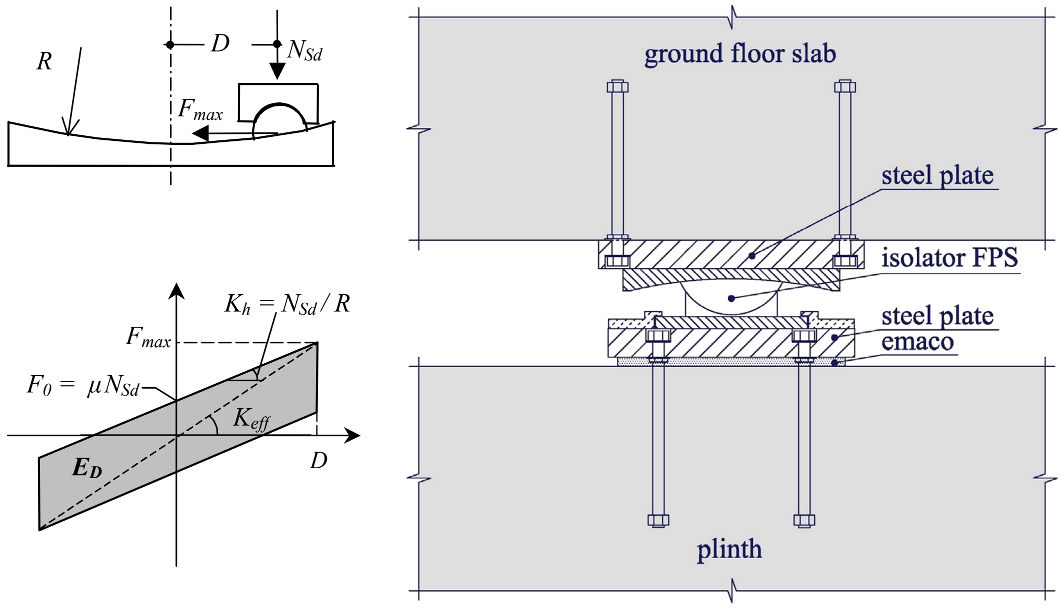
ii) The properties of these bearings change over time. To remain effective elastomeric bearings should be replaced periodically and this is a disadvantage for an iconic project intended to have a very long life span.

3.3.2 *Steel spring bearings*

Steel spring bearings are commonly used to support mechanical equipment and work in conjunction with separate damping units. They are more effective in the isolation of vertical accelerations than elastomeric or pendulum bearings but could also be beneficial for the rocking modes of vibration of the structures. However they are not as efficient as other systems in increasing the fundamental period of the structure, thus lowering the induced seismic accelerations. In addition they have a limited load bearing capacity, so for this project a significant number would be required making them uneconomical in regards to both financial and spatial considerations.

3.3.3 *Friction - Pendulum bearings (FPS)*

For FPS SIP according to EC8) the period of the isolation system is determined by the radius and friction properties of the bearing surface, so the behavior of the isolation system is independent of the vertical load. The bearings are therefore well suited to uses where the live load contributes a large proportion of the vertical load. Friction between the sliding surfaces dissipates energy providing damping so, in general, additional dampers are not required. Pendulum bearings are made from steel and therefore offer the same levels of durability as steel spring bearings. Whilst the initial



Figures 3, 4 - Bilinear model & sketch of FPS type isolator (inverted configuration as placed)

build cost of pendulum bearings may be higher than that of elastomeric ones, the former do not require periodic replacement so the total cost during the desired life cycle of the project is lower.

3.4 *Incorporation of the seismic isolation system*

Following the review of bearing types summarized above, FPS bearings are selected as they provide the greatest durability and longevity combined with predictable behavior irrespective of the vertical load.

The isolation units should be placed at the same vertical level to ensure that they move in unison. Therefore, it is desirable to have the entire ground floor diaphragm at a continuous level, without significant steps. This also helps minimize the required thickness of the ground floor diaphragm; the transfer of lateral forces from the walls to a distributed arrangement of isolators results in significant forces through the ground floor diaphragm. Any steps in this diaphragm would produce complementary vertical forces and require a substantial amount of additional concrete sections to resolve these. For this reason, the stage pit of the Opera is suspended from the ground floor slab as described further in Section 7.

4 ANALYSIS OF AN EQUIVALENT SDOF SYSTEM

4.1. Dynamic properties of a SDOF system based on a FPS type isolator

The structure can be modeled as a SDOF system of mass, m, based on an FPS type isolator. The main characteristics of the isolator are the radius of curvature of the sliding surface, R, and the friction coefficient, μ. The behavior of the system is described by the bilinear model of Fig. 3 (Mokha et al. 1991, Naeim and Kelly 1999]. The effective stiffness, K_{eff}, is therefore:

$$K_{eff} = \frac{N_{sd}}{R} + \frac{\mu N_{sd}}{D} \tag{4.1}$$

where D is the horizontal displacement of the system and N_{sd} is the normal load on the bearing:

$$N_{sd} = m\,g \tag{4.2}$$

The first term of Eq. 4.1 is the restoring force due to the rise of the mass:

$$K_{\mathrm{H}} = \frac{N_{sd}}{R} \tag{4.3}$$

The fundamental period of the system is:

$$T_{eff} = 2\pi\sqrt{\frac{m}{K_{eff}}} \tag{4.4}$$

Eq. 4.4 after substituting parameters K_{eff} and m from Eqs. 4.1 and 4.2 respectively, results to:

$$T_{eff} = 2\pi\sqrt{\frac{R\,D}{g\,D + \mu\,g\,R}} \tag{4.5}$$

Thus the fundamental period depends on the radius of curvature of the sliding surface, the friction coefficient and the horizontal displacement of the system. For the calculation of damping coefficient, ξ, the following equation from EC8-2 is used:

$$\xi = \frac{1}{2\pi}\left[\frac{\sum E_{\mathrm{M}}}{\mathrm{K}_M D_M{}^2}\right] \tag{4.6}$$

Where K_M is K_{eff}, D_M is the displacement of the structure in direction X or Y and E_M is the dissipated energy on a bearing in each cycle of displacement given by the area of the hysterisis loop of the bilinear model in Fig. 1:

$$E_M = 4\mu N_{sd} D_M \tag{4.7}$$

For a SDOF system, Eq. 4.6 after substituting E_M from Eq. 4.7 reduces to:

$$\xi = \frac{2}{\pi}\frac{\mu}{D/R + \mu} \tag{4.8}$$

The spectral reduction factor due to damping can then be calculated according to the equation 3.6 of EC8-1:

$$\eta = \sqrt{\frac{10}{5+\xi}} \tag{4.9}$$

4.2 *Analysis response spectra*

In order to proceed with the calculations for the seismic isolation properties, the final design spectrum should be available. As has been already mentioned, the superstructures are designed using as a basis a site specific spectrum for an earthquake with a return period of 475 years [Gazetas, 2009]. The derived design spectrum is defined by a ground acceleration of a_g=0.267g, which incorporates the soil factor, *S*. Importance factor is considered as 1.40 -increasing the design earthquake return period- and behavior factor, q=1.5, corresponding to quasi-elastic behavior. The

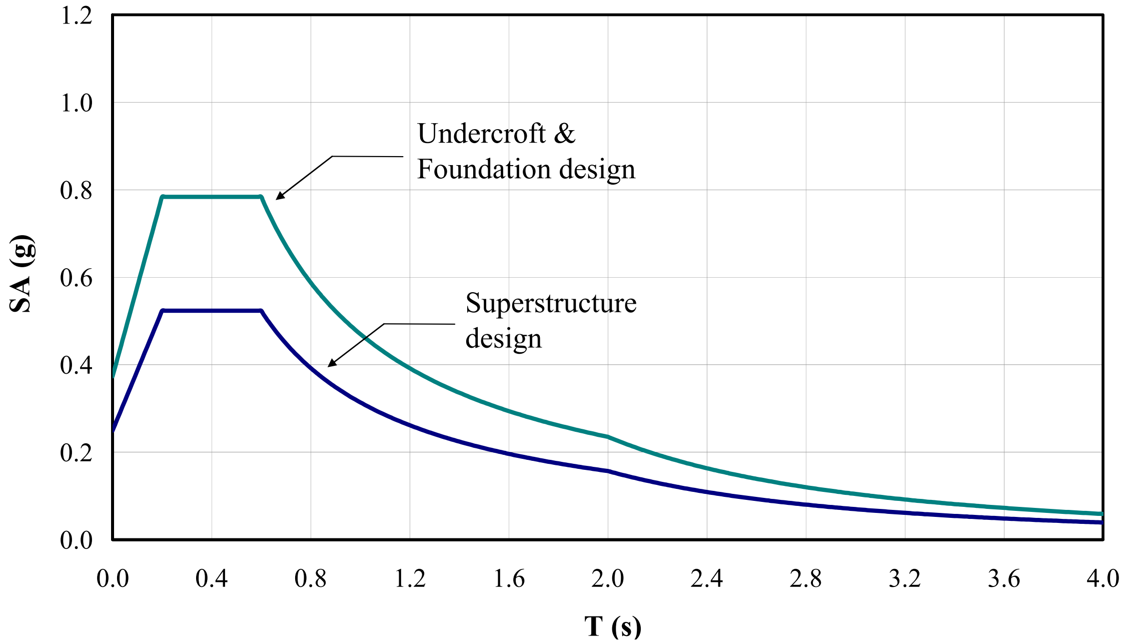


Figure 5 - Horizontal acceleration response spectra for the design of the structures

spectral acceleration amplification factor is taken as 3.0 instead of 2.5 reflecting the soft soil conditions. Finally a damping correction factor of η=0.70 is applied to the spectrum. It should be noted that the latter is a rather conservative value selected for the response spectrum (R-S) analysis in order to design the structures with an extra margin of safety. In order for the time-history (t-h) analyses to check the performance of the isolators in terms of displacements, the real values, corresponding to the friction properties of the isolators are used; these values represent a lower damping correction factor.

Following the principles of capacity design, the substructures (the undercroft and the foundations) are designed with increased seismic specifications when compared to the superstructures which translates into using a behavior factor of q=1. For the design of the isolation system, according to EC8, displacements are checked using a magnification factor of γ_x=1.5. It should be noted that this is the EC8 equivalent of using the Maximum Considered Earthquake (MCE) spectrum as defined in IBC code. It should be further noted that this value of γ_x comes from the Greek National Annex to EC8 and it is 25% more conservative than the standard value proposed by the code. The spectra used for the design of the structures are presented in Fig. 5.

The vertical acceleration response spectra are also site specific, using the design vertical ground acceleration which is taken as a_{vg}=0.216g and then following the recommendations of EC8 for a type 1 spectrum. It should be mentioned that there is no damping reduction factor in the vertical direction. Similarly to the design for the horizontal directions the superstructure is designed using a behavior factor of q=1.5 while the undercroft and the foundation are designed with a behavior factor of q=1.

4.3 Selection of Seismic Isolation system properties

The seismic isolation system was designed for the structure to specifically demonstrate the desired dynamic behavior. The critical dynamic property is the fundamental period: as period increases, spectral acceleration decreases. There is a lower limit to spectral acceleration; according to EC8, it should always be higher than β a_g, where β is 0.2. As period increases, there is a corresponding increase in displacement however, according to the site specific spectrum -and also EC8-, displacement is considered as constant for periods above T_D=2s.

Considering all this, a period of T=2.59s was selected for the design of the structures corresponding to a design acceleration of 0.093g (Fig. 5) and a displacement of SD=234mm. This

Figure 6 - Seismic isolator as placed and seismic joint on a staircase

value corresponds approximately to a maximum displacement of 350mm for the seismic joints and isolator units after taking into account the magnification factor, γ_x, which is considered as an upper limit for practical purposes and cost effectiveness. The displacement of 234mm is used as a starting point for the calculations; the final displacement is expected to be lower due to a lower damping correction factors as presented in Section 4.2.

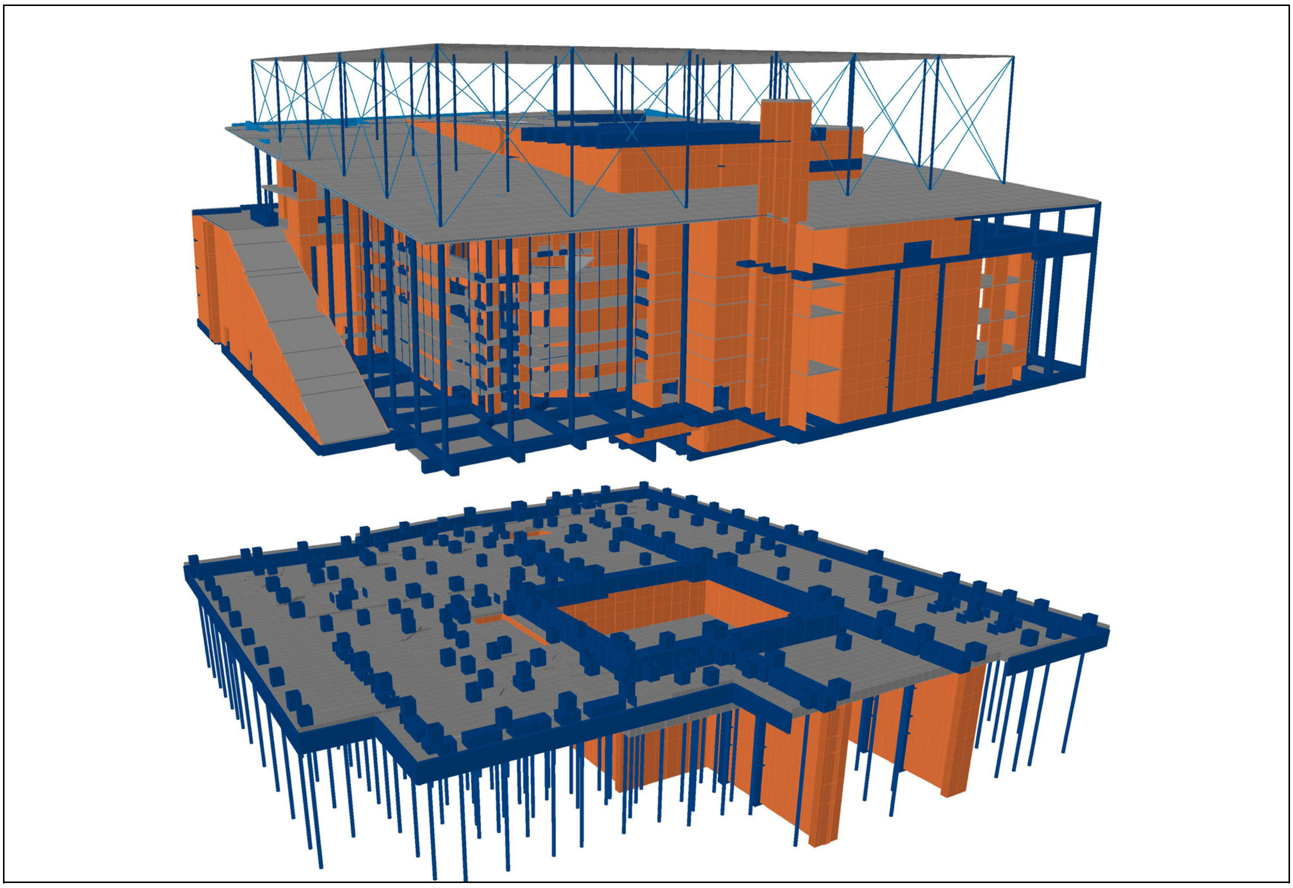
For the friction coefficient, μ, a mean value of 0.05 is selected to ensure a high spectral reduction factor due to damping. According to EN 15129 (4.4.2) two sets of design properties of the system of devices shall be properly established: (a) Upper bound design properties (UBDP): (b) Lower bound design properties (LBDP). So for a mean value of friction coefficient of 0.05 a range between 0.035 and 0.065 must be considered. This means that, in order to check the isolation system using t-h analyses, two structural models should be examined: one for which the higher value of friction coefficient is used (UBDP) and is critical for reactions and another for which the lower value of friction coefficient is used (LBDP) and is critical for displacements. Conversely, the required radius of the isolator actually results from the use of the higher coefficient of friction when calculating the spectral acceleration of the structure, since a higher value results to a smaller effective period. The range of values of the isolation system, as calculated from the equations of Section 4.1, is presented in Table 1.

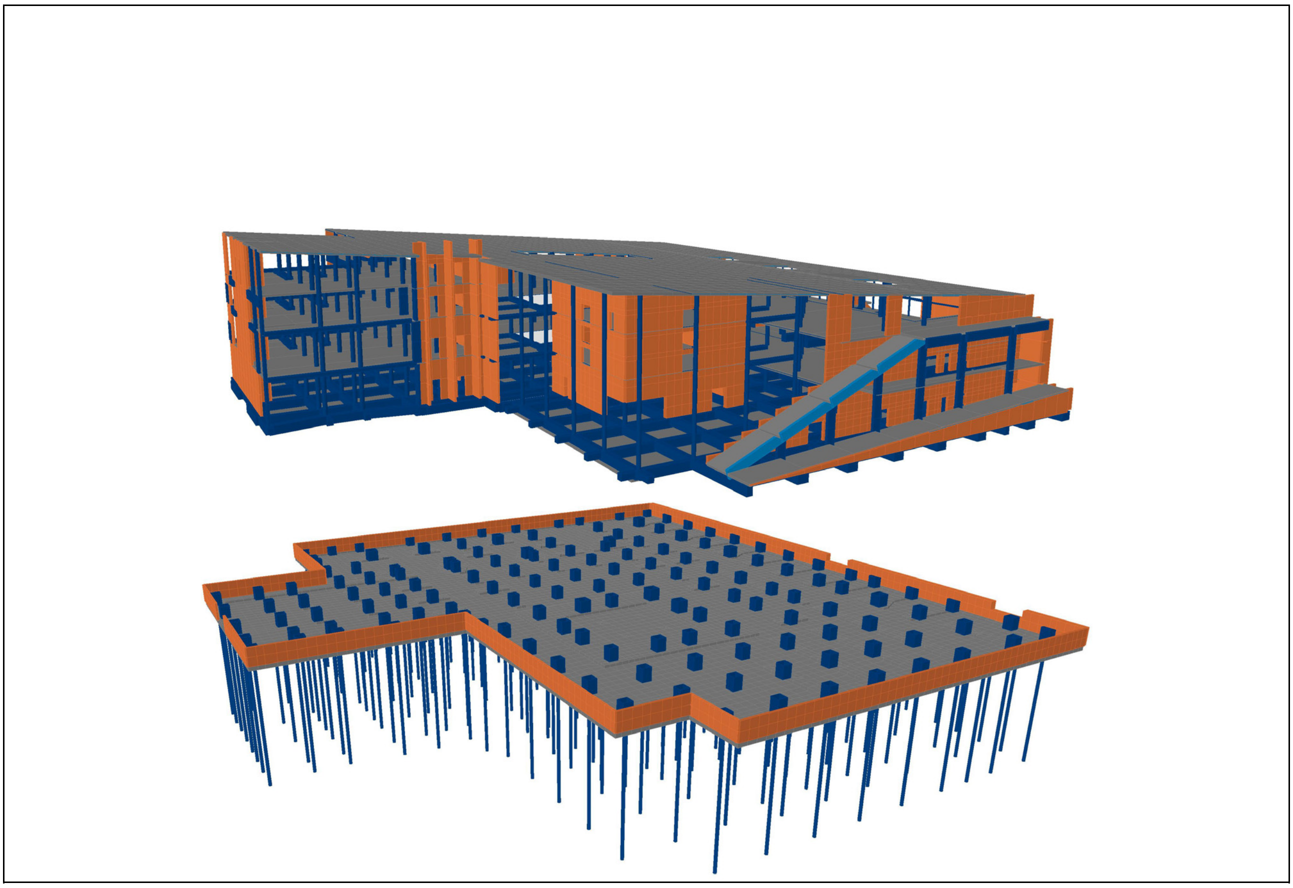
5 DYNAMIC RESPONSE SPECTRUM ANALYSIS ON 3D MODELS

Dynamic R-S analysis on 3D finite element models is the next step of the design methodology aiming to: (a) verify the fundamental period (b) Calculate maximum displacements (c) check for

TABLE 1. Isolator design properties

ISOLATOR PROPERTIES	MODEL		
	LBDP	MEAN	UBDP
D (mm)	234	234	234
μ	0.035	0.050	0.065
R (mm)	3102	3102	3102
T_{eff} (s)	2.92	2.74	2.59
ξ	0.202	0.254	0.295
n	0.630	0.574	0.539





Figures 7, 8 - 3D finite element models of the National Opera and Library uplift and finally (d) do the final design of structural elements.

For both structures, seismic isolation at the ground floor level separates superstructure from substructure. The load transmission between the two sections is carried out through friction pendulum isolators (SIP, Mageba) placed on plinths underneath the ground floor slab. There are 172 bearings for the Opera and 151 for the Library, all placed in an inverted configuration (Figs. 4, 6), with the spherical surface on top. This ensures that, during seismic movement, there will be no additional moments induced in the supporting structural elements due to eccentricities, thus avoiding additional loading in the piles or diaphragm walls of the foundation. Instead, the robust series of beams supporting the ground floor slab are designed to account for these additional moments.

Analysis is performed using the ETABS software. For each structure two separate 3D finite element models (Figs. 7, 8) are created; one for each of the superstructure and the undercroft /foundation respectively. This increases the accuracy and simplifies the analysis.

The superstructure is considered to be elastically supported on the isolators in the vertical direction using spring values that reflect the stiffness of both the soil and substructure. These values are derived from the convergence of a consecutive iterative process for the system of the superstructure and the soil/substructure. The procedure is as follows: initial vertical springs are placed on the bearing positions on the superstructure model and then from static analysis for gravity loads, reactions on the bearings are found. These reactions are imposed on the substructure model which incorporates soil springs on piles as defined by Mylonakis and Gazetas [1999]. Vertical displacements on the bearings position are found from which new vertical springs are defined and placed on the superstructure model and the procedure is continued until convergence.

In the horizontal direction, the total effective stiffness of the isolation system is distributed to each bearing proportionally to the normal load it carries. As the isolator stiffness is proportional to the axial load the nodal springs are adjusted during the iterative process described above. For the horizontal components of the excitation, the response spectra of Fig. 5 are used while for the vertical one, the spectra described in Section 4.2 are used.

For both structures R-S analysis, results in two translational modes of vibration –X and Y direction- which have fundamental periods of 2.61s for the Opera and 2.59s for the Library. In the vertical direction, the structures are stiff by comparison and a large number of modes is required in order to achieve a total participation factor of 100%. The periods found lie in the interval of 0.35-0.05s for the Opera and 0.25-0.05s for the Library. However only 16% of the mass for the Opera and 17% for the Library is mobilized with periods that lie on the horizontal part of the vertical acceleration spectrum i.e. between 0.05-0.15s.

For both structures horizontal displacements are found to be D_x=D_y=235mm. The final displacement comes from the combination of the horizontal components with the seismic force in one direction combined with 30% of the seismic force in the orthogonal direction. The resultant is found to be D=245mm and, after applying the magnification factor γ_x=1.5, leads to a final design displacement of Δ=368mm. This displacement is marginally higher -as anticipated- than the value of 350mm used for the seismic joint and movement of the isolator units but is expected to reduce after performing the more detailed t-h analyses that take into account the actual values for the friction of the isolators.

From the R-S analyses, structural elements of the basement and the superstructure are designed. It should be noted that no uplift is present demonstrating that the plan arrangement of the isolators and stability system has been appropriately adjusted to avoid uplift. It can be demonstrated from this part of the analysis that the fundamental periods found from R-S analysis on a 3D finite element model are almost identical to the ones from the equivalent SDOF system calculations. This shows that the structure is not flexible so modeling as SDOF system is a valid assumption. Correspondingly, the displacements found, show also good agreement: 235mm from R-S analysis vs 234mm from initial SDOF analysis.

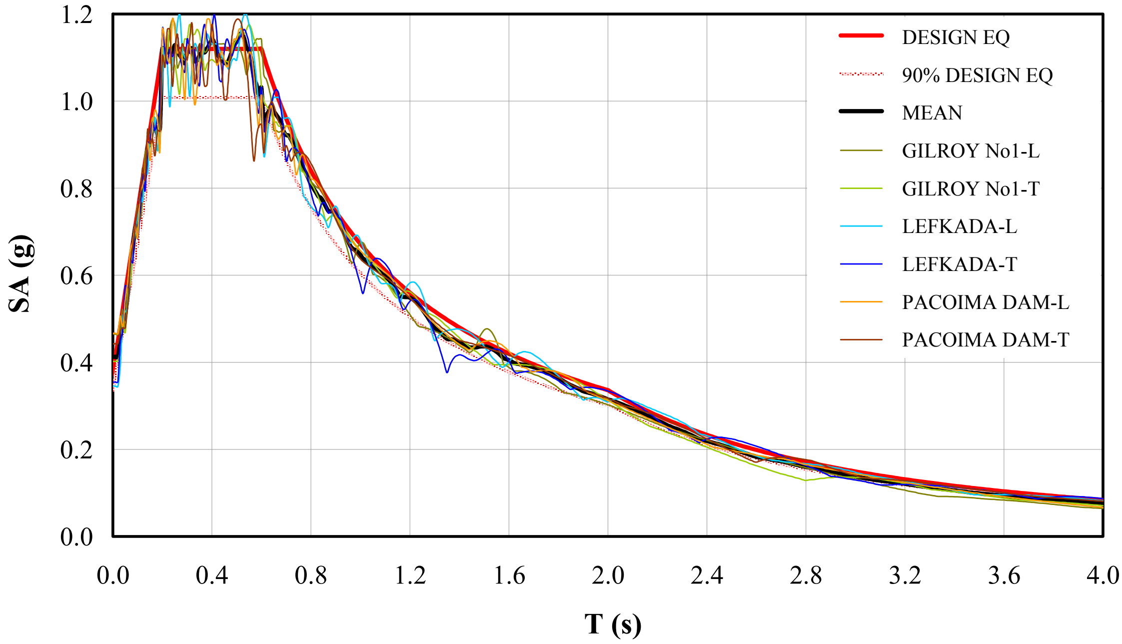


Figure 9 - Elastic horizontal acceleration response spectra (ξ=5%) of the records for t-h analyses

6 NONLINEAR ANALYSES

To further refine the understanding of the performance of the structure and also in order to check the displacements of the structures at the isolation system level, non-linear time-history analyses are performed. To achieve this, the 3D finite element models used previously (Figs. 7, 8) were modified after modeling isolators as non-linear elements in the horizontal direction while in the vertical, the same linear springs as in the R-S analyses are used.

Non-linear properties differ in regards to the friction coefficient: as mentioned in Section 4.3 two sets of design properties of the system of devices are established: UBDP and LBDP with friction coefficients of 0.065 and 0.035 used to check reactions and displacements respectively. This means that two sets of models are created and two sets of t-h analyses are performed.

For this purpose three sets of ground motions, each consisting of two horizontal and one vertical component, are used. The records correspond to the seismic profile of the region considering soil conditions (soft soil), fault distance and magnitude of event. These are: the Gilroy No1 record (Loma Prieta earthquake, 1989), the Lefkada record (Lefkada earthquake, 2003), the Pacoima Dam record (San Fernando earthquake, 1971). The motions are amplitude–and–frequency manipulated to be compatible with the design response spectrum (Fig. 9). According to EC8, in the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram is applied, no value of the mean, 5% damping, elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum. A scaling factor is applied to the horizontal accelerograms to account for the damping. This takes the value of 0.630 for LDBP and 0.539 for UBDP models respectively. There is no damping correction factor for the vertical earthquake.

For each model, 8 combinations of application for each of the motions are considered – combining the horizontal directions with varying factors whilst keeping the vertical constant - thus resulting to a total of 24 combinations examined for the 3 sets.

Results from LBDP models, critical for displacements, indicate maximum displacements of 225mm for the Opera and 220mm for the Library which after applying the magnification factor of γ_x=1.5 result to 338mm and 330mm respectively. These values are, as expected, lower than the ones found from R-S analyses. Results from UBDP models show additionally that there is no uplift present in any of the design scenarios.

7 INCORPORATION OF SEISMIC ISOLATION IN THE STRUCTURES DESIGN

The incorporation of a seismic isolation system necessitated a number of specific structural decisions and presented some particular design challenges. These are briefly presented below:

7.1 *Suspending elements*

In order to keeps isolators on the same level, several design modifications are required. The most significant of these is the suspension of the stage pit of the Opera from the ground floor slab. Acoustic isolation requirements and the mechanical systems installed in the orchestra and stage pit require the pits to be part of the base isolated portion of the building. To reap the benefits of base isolation, the pits must only be attached to the building at ground floor, and must not contact the under croft or the diaphragm walls. Therefore, this requires the pits to be hung from the structure above. The stage elevator lifting equipment and orchestra lifts are then built off these suspended concrete stage pits. As the total pit area is a large and highly loaded space this is a particularly difficult challenge which is achieved by making use of the proscenium wall. The majority of the pit weight is carried on two very large bearings.

7.2 *Joints*

A seismic joint runs around the entire perimeter of the Opera and the Library buildings and over the full height of the interface between the opera house and the library. Particular care is required to ensure free movement at the seismic joint to avoid compromising the base isolation system. If the base isolation system were unable to move freely, the seismic performance of the base isolated superstructure would be significantly compromised. It is therefore not permissible for either structures, or finishes to restrict the movement of the joint.

At floor level proprietary bridging plates span the joints to accommodate pedestrian traffic. The joint details at roof level have been developed to coordinate the structural, landscaping and services requirements. The structural requirements are particularly onerous for these joints and great care is required to ensure that the base isolation action is not compromised

A lot of emphasis is put to the seismic joint of the glass facade between the Opera and the Library. This is a specially designed element that can accommodate small differential displacements between the two structures. It should be noted that the structures share the same dynamic characteristics so if the excitation is synchronous there should be no differential displacements. An asynchronous excitation, however, would bring differential displacements that can be accommodated by the façade system up to 100mm. In the unlikely event that this is exceeded, a small number of façade elements near the joint will be sacrificed in a controlled way and can be replaced afterwards.

Also special attention is required in the design of the access staircases to the undercroft which, by definition, pass through the level of seismic isolation (Fig. 6). Apart from the free movement of the joints good performance of the staircases is crucial since these constitute the escape routes after an earthquake.

Similarly, increased care must be taken for services bridging the joints.

7.3 *Replacement of the bearings*

The bearings pass a series of tests which ensure their longevity and good performance in case of an earthquake. However should they need replacement, the superstructure can be appropriately supported using jacks. The resulting stress condition has been taken into account in the dimensioning and the reinforcement of the affected structural elements.

8 CONCLUSIONS

Dynamic behavior of SNF National Opera and Library of Greece is presented. The study focuses on the methodology followed for the seismic design which, performed in three consecutive steps, included simple calculations using an equivalent SDOF system, dynamic response spectrum analyses and non-linear time-history analyses using selected earthquake records on a 3-D finite element model. The following conclusions are derived:

a. From the simplest approach (SDOF system) to the more sophisticated type of analysis (non-linear time-history on 3D FEM models), the results e.g. periods, displacements etc. show good agreement. This can be attributed to the high stiffness of the structures.

b. Had the building been designed without seismic isolation the story drifts would be unacceptably high and the structural system would not be as elegant as that required by the architectural concept.

c. In addition, the incorporation of the isolation system leads to substantially lower design accelerations. A conventional structure would have to be designed for acceleration five times greater. Similarly, the contents of the building would experience significantly greater accelerations in a conventional structure and therefore be more susceptible to damage in a seismic event.

References

Anagnostopoulos S.A. [2007] "Applications of base isolation and energy dissipation devices in structures in Greece", *Proc. of the 10th World Conference on Seismic Isolation, Energy Dissipation and Active Vibration Control of Structures,* Istanbul, Turkey.

Athanasopoulos G., Vlachakis V. [2010] *Evaluation of liquefaction resistance, strength & dynamic moduli/damping ratios of soil formations underlying the SNFCC project site,* Athens, Greece.

Edafomichaniki [2008] *Geotechnical survey and study for SNFCC*, Athens, Greece.

Eurocode-8, *Design of structures for earthquake resistance*, EN 1998-1, CEN Brussels.

Gazetas G. [2010] *Requirements for the antiseismic design of the SNFCC– Final Phase Report*, Athens, Greece.

Mylonakis G., Gazetas G. [1999] "Lateral Vibration and Internal Forces of Grouped Piles in Layered Soil" *Journal of Geotechnical and Geoenvironmental Engineering,* 125 (1), 16-25.

Giarlelis C., Kostikas C., Lamprinou E., Dalakiouridou M. [2008] "Dynamic behavior of a seismic isolated structure in Greece", *Proc. of the 14th World Conference on Earthquake Engineering*, Beijing, China.

Huber P., Medeot R., Tuncer M. [2007] "Seismic Protection of three recently constructed Buildings by seismic Isolation with Sliding Isolation Pendulum Devices. *Proc. of the 10th World Conference on Seismic Isolation, Energy Dissipation & Active Vibration Control of Structures*, Turkey.

Kircher C., personal communication.

Koumousis V. [2003] *Notes on seismic isolation*, Athens, Greece.

Mokha, A., Constantinou, M., Reinhorn, A., Zayas, V. [1991] "Experimental study of friction-pendulum isolation system", *Journal of Structural Engineering* vol. 117: 4, pp 1201-1217.

Naeim F., Kelly J.M. [1999] *Design of seismic isolated structures: from theory to practice*, John Wiley & Sons, Inc., New York.

Tsopelas, P., Constantinou, M., Kim, Y., Okamoto, S. [1996] "Experimental study of FPS system in bridge seismic isolation", *Earthquake Engineering & Structural dynamics*, vol. 25, no 1, pp 65-78.