Seismic Upgrade of a Reinforced Concrete Building using Friction-Pendulum Isolators

Christos Giarlelis, Dimitrios Koufalis

Equidas Consulting Engineers, Greece

Panagiotis Antoniadis

Structural Engineer, Greece

Constantinos Repapis

Department of Civil Engineering, University of West Attica, Greece

Contact: giarlelis@equidas.com

Abstract

The seismic upgrade of an existing 5-storey reinforced concrete (RC) building through the use of friction-pendulum system (FPS) bearings is presented. The selected building is a model structure, representative of the buildings designed in the period 1960-1980 in Greece. The seismic assessment following the EC8-3 code provisions indicates the structural incapacity of the frame elements. For the seismic upgrade of the structure, FPS bearings are placed above the foundation level. A series of linear and non-linear analyses are performed and the results show that minimal interventions are required only at the ground floor level. The overall seismic performance and the limitations in the applicability of the proposed solution are discussed. It is concluded that seismic retrofitting through seismic isolation of existing buildings is a very good approach and should be widely used.

Keywords: Seismic assessment; Retrofit; Seismic isolation; Friction-pendulum bearings

1 Introduction

Seismic isolation becomes more and more popular as with the increasing number of projects incorporating the technique, its advantages become widely known, valuable experience is gained and the cost drops. Regarding buildings, seismic isolation is mainly used for new ones while examples of retrofitting existing ones by this technique are very few [1-5]. This can be attributed to misconceptions about the architectural and structural requirements, the cost and the applicability of the concept. In Greece to date, despite the high seismicity of the area, only three new seismic isolated building projects have been completed [6-8].

In this work, a typical residential, 5-storey RC building, located in Athens, representative of the ones designed in the period 1960-1980 according to older seismic codes is selected. The seismic assessment of the structure is performed following the provisions of the Greek Code of Interventions

(GCI) [9], which is the Greek annex of EC8-3 [10], and the results indicate the vulnerability of the structural elements on all floors. Therefore, seismic upgrade is required. Instead of selecting conventional techniques such as RC jackets, FRPs or shear walls, seismic isolation is adopted. For this end, FPS bearings are implemented at the foundation level and the seismic performance of the isolated structure is investigated through a series of analyses.

2 Building assessment

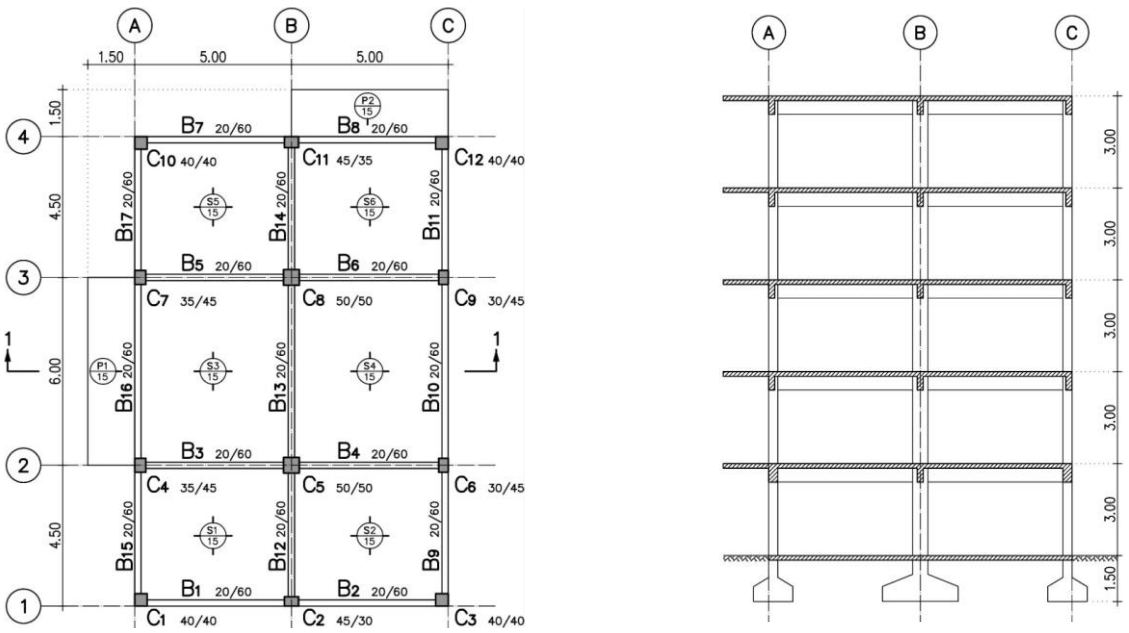
2.1 Building description

The selected building is a RC beam-column frame structure with plan dimensions of 10,00 m by 15,00 m (Figures 1, 2). The lack of strong columns at the corners and the absence of shear walls suggest a design focused on gravity loads, which is typical of the period of construction. Slabs have a depth of 0,15 m and there are two balconies on each floor. The floor height is 3,00 m. Soil conditions are considered as good with soil category defined as B, as for the majority of buildings in Athens. Regarding the foundation, it consists of spread footings without tie beams, as was the common practice at the time the structure was assumed to be designed. Double brick, external, infill walls are considered to be present at all levels (including the ground floor). According to GCI provisions, the material investigation for normal data reliability (knowledge level KL2) concluded that the concrete class is C16 (f_{ck}=16 MPa, f_{cm}=25 MPa) and steel class is S400 (St III).

2.2 Response spectrum analysis

For the assessment of the building dynamic response spectrum (RS) analysis is performed and the design response spectrum of EC8-1 [11] is taken into account (Figure 3). Athens has a reference peak ground design acceleration α_{gR} equal to 0,16g, the soil factor S is equal to 1,20 for soil category B, the building importance factor γ_I is equal to 1, corresponding to an importance class of II for residential use, and the damping factor η is equal to 1 (for ξ=5%). The behavior factor q is equal to 1,70 as proposed by GCI provisions for this type of buildings. Performance level B1, i.e. an average return period of approximately 475 years for life safety level is considered, as defined in both GCI and EC8-3.

The structure is considered a space frame subjected to gravity and earthquake loading. The beam-column frame system is modeled with linear elements located along the centroidal axes



Figures 1, 2. Plan view of ground floor and section 1-1 of the building

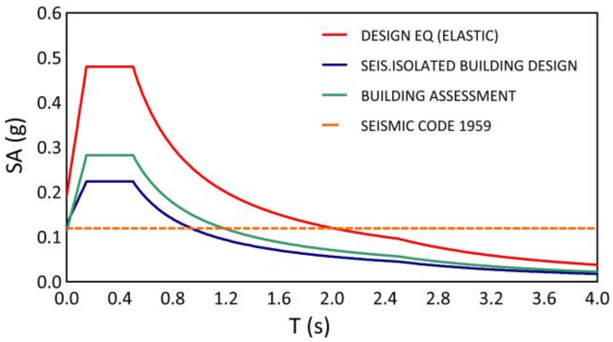


Figure 3. Horizontal acceleration response spectra

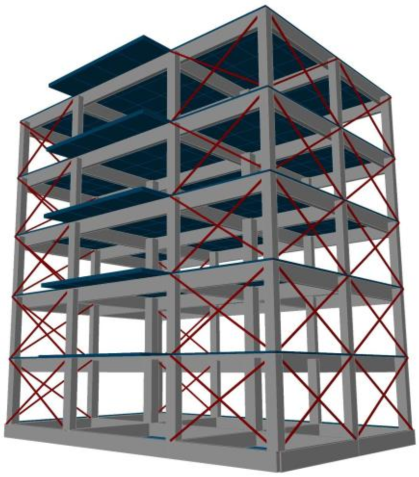


Figure 4. 3D building model

of the members. Slabs are modeled as finite elements and considered as rigid diaphragms. The analysis considers cracked properties for the members with stiffness values at 20% and 30% of the uncracked, for beams and columns respectively, as evaluated from the moment-curvature diagrams. Soil flexibility is taken into account through the use of springs placed at the base of the columns corresponding to the spread footings' properties [12]. Infill walls are taken into account as diagonal braces connecting the joints of the frames only where there are no openings. The simulations are performed using the computer code ETABS (Figure 4).

The fundamental periods of the structure are T_y equal to 0,70s and T_x equal to 0,66s for the transversal (short) and the longitudinal (long) directions of the structure, respectively. A number of 11 modes (in total) are required to encompass 100% of the total effective mass in each direction.

Despite the presence of the infill walls the building is relatively flexible. This can be attributed mainly to the slender vertical elements and the absence of shear walls. Consequently, interstorey drifts reach relatively high values as can be derived from Table1. The structural assessment is performed through the use of failure indices, λ, defined for each structural member as:

$$\lambda = S_E / R_m \qquad (1)$$

where S_E is the action, bending moment or shear, on the member resulting from the seismic load combinations and R_m is the corresponding available resistance of the structural member.

For all members, the highest values of failure indices are located on the ground floor. In Table 2, the failure indices of the columns of the ground floor, at their base, in bending and in shear, respectively, are presented. It should be noted that the failure indices in shear are in general lower to the ones in bending due to the important contribution of the infill walls on all levels.

Overall the analysis results indicate that the building is rather underdesigned for the seismic demand imposed by the current codes, therefore a seismic upgrade is required in order to increase the capacity of the structural members. A seismic upgrade should also aim to lower the interstorey drifts thus resulting in an improved performance of non-structural members during an earthquake.

3 Seismic upgrade

3.1 Introduction

A conventional approach for the seismic retrofitting of the structure would require a combination of implementing new shear walls along with RC jackets on several structural members. The authors have worked on such an approach by placing four shear walls with a length of 3,00 m on perimeter of each side of the building and a limited number of RC jackets, where needed, on beams and columns on all floors. A rigid, mat foundation is required, not only to enhance the seismic performance of the structure by connecting the base of the columns but also to accommodate the large moments developed at the shear walls.

Table 1. Interstorey drifts

Storey	Existing structure (assessement) RS analysis		Seismic isolated structure RS analysis		Seismic isolated structure t-h analysis	
	$\gamma_{X,max}$ ($^{o}/_{oo}$)	$\gamma_{Y,max}$ ($^{o}/_{oo}$)	$\gamma_{X,max}$ ($^{o}/_{oo}$)	$\gamma_{Y,max}$ ($^{o}/_{oo}$)	$\gamma_{X,max}$ ($^{o}/_{oo}$)	$\gamma_{Y,max}$ ($^{o}/_{oo}$)
5-4	1,59	1,44	0,41	0,38	0,39	0,33
4-3	2,75	2,36	0,53	0,49	0,54	0,49
3-2	3,59	3,00	0,73	0,66	0,68	0,61
2-1	4,16	3,39	0,91	0,82	0,78	0,71
1-0	3,15	2,67	0,89	0,79	0,71	0,67

Table 2. Failure indices of ground floor columns at their base due to bending and shear

Columns	Existing Structure		Seismic isolated structure							
	R-S Analysis		R-S Analysis		El Centro		Kern		Lower California	
	Bending	Shear	Bending	Shear	Bending	Shear	Bending	Shear	Bending	Shear
C1	1,15	0,49	0,47	0,29	0,45	0,19	0,41	0,19	0,38	0,20
C2	0,82	0,86	0,54	0,31	0,53	0,23	0,55	0,22	0,54	0,26
C3	0,77	0,82	0,45	0,29	0,36	0,20	0,36	0,20	0,38	0,20
C4	1,03	0,89	0,63	0,44	0,64	0,38	0,60	0,37	0,61	0,38
C5	0,55	0,57	0,40	0,17	0,41	0,15	0,37	0,15	0,39	0,16
C6	0,90	1,41	0,65	0,52	0,67	0,42	0,64	0,40	0,65	0,40
C7	1,02	1,12	0,64	0,47	0,60	0,44	0,61	0,40	0,62	0,39
C8	0,60	0,63	0,40	0,19	0,40	0,15	0,36	0,14	0,38	0,13
C9	1,32	1,21	0,68	0,50	0,70	0,41	0,65	0,40	0,65	0,39
C10	1,37	0,80	0,50	0,34	0,40	0,24	0,40	0,22	0,40	0,22
C11	1,22	1,36	0,58	0,43	0,50	0,30	0,53	0,29	0,54	0,29
C12	1,30	0,91	0,52	0,41	0,40	0,40	0,40	0,40	0,44	0,34

The alternative approach, which is the focus of this study, is the implementation of seismic isolation of FPS type at the foundation level as shown in Figure 5. For this end, a rigid diaphragm above and below the isolation level is required. A mat foundation, encasing the existing spread footings, may serve as a diaphragm below the bearings while new beams and slabs, constructed at the ground floor level, may function as a diaphragm above. The methodology for the design of the seismic isolated structure, performed in three stages includes: (a) simple analyses using a single-degree of freedom (SDOF) model for initial scheming, b) dynamic RS analyses for detailed design and c) non-linear time history analyses using selected earthquake records for verification.

3.2 Seismic action

The response spectra of EC8-1 presented in Section 2, for the design of the superstructure and the foundation of the seismic isolated structure are derived. A behavior factor q equal to 1,50 is applied for the superstructure design spectrum corresponding to quasi-elastic behavior. Following the principle of capacity design, the substructure and the isolation system are designed using a behavior factor q equal to 1. Regarding the seismic components, two horizontal and the vertical ones should act simultaneously. For the horizontal components a damping correction factor η =0,70 (ξ=15.4%) is used. It should be noted that the latter is a rather conservative value, as for the time-history (t-h) analyses, the real values, corresponding to the friction properties of the isolators, are considered; these values correspond to a lower damping correction factor (Table 3). For the vertical component, no damping reduction factor is applied, since FPS bearings provide damping only in the horizontal directions.

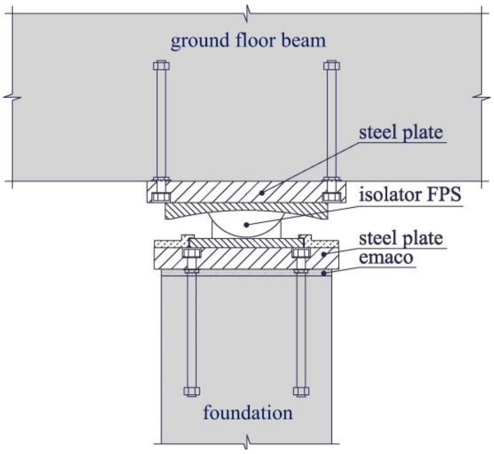


Figure 5. Sketch of the FPS isolator

3.3 Equivalent SDOF system analysis

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 a bilinear model [13], [14], [15]. If D denotes the displacement of the system then the effective period, T_{eff} is given by Equation 2:

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

The critical dynamic property is the fundamental period: as the period increases, the spectral acceleration decreases. However, there is a lower limit to spectral acceleration; according to EC8: it should always be higher than $\beta\ a_g$, where β, the lower bound factor for the horizontal design spectrum, is 0,20. So in this case, the lower bound for the design acceleration is 0,032g corres-ponding to a period of 2,95s. Furthermore, as the period increases there is a corresponding increase in the displacement i.e. the size of the seismic joint.

However, following the Greek Annex of EC8-1 the displacement is considered as constant for periods above T_D=2,5s and equal to 105 mm. So considering all the above limits, a value of T=2.5s is selected for the fundamental period which, corresponds to a design acceleration of 0,045g for the superstructure. Also the displacement after applying the magnification factor γ_x equal to 1,50 corresponds approximately to a displacement of 158 mm for the seismic joints. For the isolator units the displacement comes from the combi-nation of the horizontal components combining the seismic force in one direction with the 30% of the seismic force in the orthogonal direction which results, after applying the magnification factor, γ_x, in a displacement of 165 mm. Then for the friction coefficient, μ, a mean value of 0,025 is selected to ensure a high spectral reduction factor due to damping. According to EN15129 [16], two sets of design properties of the system of devices shall be properly established: upper bound design properties (UBDP) and lower bound design properties (LBDP) which should have values ±30% of the mean. So for a mean value of friction coefficient of 0,025, a range between 0,0175 and 0,0325 must be considered. Those properties are necessary for the non-linear analyses and are shown in Table 3.

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. Also for the LBDP, the corresponding period is 2,83s which is lower than the period of 2,95s corresponding to the lower bound for the design acceleration.

Table 3. Isolator design properties

PROPERTY	LBDP	MEAN	UBDP
D (mm)	105	105	105
μ	0,0175	0,025	0,0325
R (mm)	2980	2980	2980
T_{eff} (s)	2,83	2,65	2,50
ξ	0,211	0,264	0,305
n	0,619	0,565	0,531

3.4 Response spectrum analyses

Dynamic RS 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 uplift and finally (d) do the final design of the new structural elements and the assessment of the existing ones.

The 3D finite element model used for the assessment is used for the dynamic response spectrum analyses after some modifications: new beams and slabs are implemented on the ground floor level while the spread footings are encased in a mat foundation. In the vertical direction the superstructure is considered to be elastically supported on the isolators using spring values that reflect the stiffness of the foundation and the soil. In the horizontal direction, the total effective stiffness of the isolation system is distributed to each bearing proportionally to the normal load it carries. The analysis considers cracked properties for the members with stiffness values at 50% of the uncracked, for beams and columns, as suggested by EC8-1 for new structures, since it is estimated that the superstructure will behave elastically.

The results show that the fundamental period is equal to 2,54s. in both horizontal directions, which is almost identical to that found previously in SDOF analysis. So modeling the structure as a SDOF system is a valid assumption. Also torsional effects are remarkably reduced, as the two translational modes of vibration stimulate the entire mass of the structure. In the vertical direction, the structure is rather stiff and large number of modes is required in order to achieve a total participation factor 100% with the periods found to be lying in the interval of 0,03-0,35s.

The horizontal displacements D_X and D_Y are equal to 103 mm so, as has been explained above, the displacement of the isolator unit is equal to 161 mm. This value is slightly lower to the one found previously from SDOF calculations and is expected to be further reduced after performing the more accurate non-linear analyses. Interstorey drifts are calculated and reach values approximately at the 25% of the ones before the intervention, as presented in Table 1. Also in the vertical direction there is no uplift present. Using the RS analysis results, the new structural elements of the ground floor and the mat foundation are designed. For the existing elements, their assessment, performed using failure indices, shows that there are no elements, beams or columns at any level that require strengthening. In Table 2, the failure indices of the columns of the ground floor, at their base (where they get their maximum values), in bending and in shear, respectively, are presented. Comparing the results with the ones of the structure before the seismic upgrade, the improvement of the capacity of the structural members is remarkable.

3.5 Non-linear dynamic analyses

In order to check the isolation system behavior time history analyses is performed examining two structural models: 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. For this end, the 3D finite element model used previously is modified after modeling isolators as non-linear elements using the friction-pendulum isolator link property of the ETABS code. The link accounts for the real damping of the system, corresponding to the damping correction factors presented in Table 3. Three sets of ground motions, each consisting of two horizontal and one vertical component are used: (a) the El Centro record (Imperial Valley EQ, May 18, 1940, Station #117), (b) the Lower California record (L. California EQ, December 30, 1934, Station #117) and (c) the Kern record (Kern County, California EQ, July 21, 1952, Station #095). Those records correspond to the seismic profile of the region and are amplitude and frequency manipulated, using the code Seismomatch [17], to be compatible with the elastic response spectrum for 5% damping (Figure 6).

Results from the LBDP models, critical for displacements, indicate maximum displacements of 89 mm after combining the displacements in the two horizontal directions using the SRSS rule. After applying the magnification factor of γ_x=1,5 this results to 134 mm. This value is, as expected, lower than the one found from RS analyses.

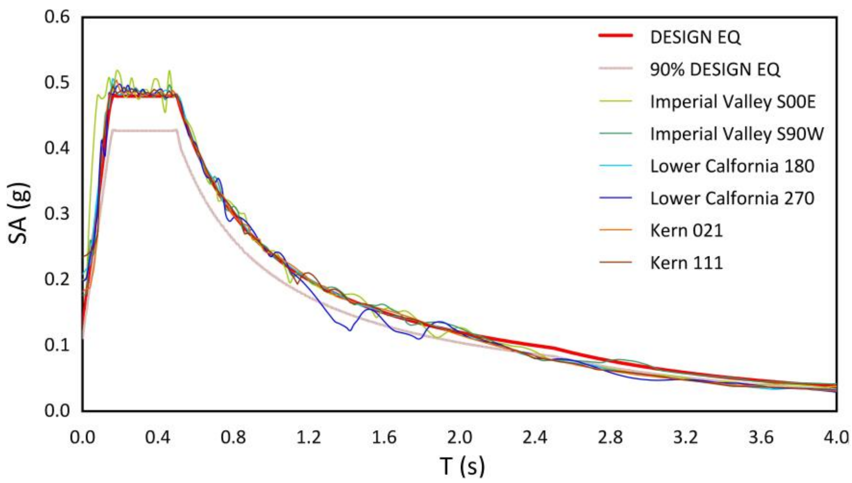


Figure 6. Elastic horizontal acceleration response spectra (ξ=5%) of the records for t-h analyses

Results from UBDP models show that there is no uplift present. Regarding the existing structural members the assessment, performed using failure indices, shows approximately similar values as the ones calculated from the RS analysis. In Table 2, the failure indices of the columns of the ground floor, at their base, in bending and in shear, respectively, are presented.

4 Implementation of the seismic isolation system

In order to implement the seismic isolation system the following requirements should be satisfied: (a) a diaphragm above and below the isolation level is required; a rigid, mat foundation, encasing the existing spread footings, may serve as a diaphragm below the bearings while new beams and slabs, constructed at the ground floor level, may function as a diaphragm above (b) a minimum height of 1,50 m from the upper part of the ground floor slab to the foundation is needed; if this is not available, underpinning the foundation could be an option (c) the building should be detached; this is important not only for the creation of the seismic joint but also to provide working space during construction. Regarding the construction sequence, this can be done in two stages. In the first stage the existing spread footings will be encased into a mat foundation and the new beams and slabs at the ground floor level will be constructed. Also a retaining wall at the perimeter of the building, from the foundation to the ground level, will be raised to satisfy the seismic joint requirement. Particular care is required to ensure the free movement of the structure at the seismic joint in order not to compromise its seismic performance. In the second stage the columns between the two diaphragms will be cut and the bearings will be implemented using hydraulic jacks.

The bearings should pass a series of tests, following EN15129, which will ensure their longevity and good performance in case of an earthquake. However, should they need replace-ment, the superstructure can be appropriately supported using jacks. Furthermore specially designed covered openings on the new ground floor slabs will provide access to the isolators for inspection and replacement.

5 Conclusions

A feasibility study for the seismic upgrade of an existing building through the use of seismic isolation is presented. It is a 5-storey RC building designed according to the Greek seismic code of 1959. After the assessment of the structure, performed following the EC8-3 code provisions, the seismic upgrade of the structure is achieved not by a conventional approach such as the use of new shear walls or RC jackets but through the use of seismic isolation, of FPS type. The following conclusions are derived: (a) the incorporation of the isolation system into the existing building leads to substantially lower design accelerations due to changing the structure's fundamental period but also due to the increased damping. So all the existing structural members above the isolation level have adequate capacity. In addition, interstorey drifts are significantly lower than the ones of the original structure but also than those of the conventionally strengthened one. (b) by contrast to a conventional seismic upgrade approach, no structural interventions are required above the seismic isolation level. This means that the original architectural concept can be preserved and the seismic upgrade can take place in a shorter time and in a smooth way for the residents (c) from the simplest approach (SDOF system) to the more sophisticated type of analysis (non-linear time-history on 3D FEM model) the results e.g. periods, displacements etc show good agreement.

Overall, the seismic retrofitting of existing buildings through seismic isolation is a good solution and should be widely used in the future.

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