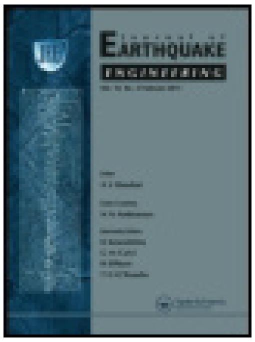

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Case Study of Seismic Assessment of a Short Irregular Historic Reinforced Concrete Structure: Time-History Vs. Pushover Nonlinear Methods

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ABSTRACT
The selection of appropriate analysis method for the seismic assessment of existing structures and the reliability of its results are of utmost importance, as they determine the magnitude of potential damage in a future seismic event, as well as the demands for strengthening and ductility during retrofitting. This paper examines a historical concrete building in Nicosia, Cyprus, that exhibits strong irregularities, both in plan and elevation. The aim is to compare the results of the static pushover to the nonlinear time-history analysis method, to better understand the applicability limits of the pushover analysis when applied on irregular, historic concrete structures.

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KEYWORDS
Historic concrete; irregular building; seismic assessment; static pushover; time-history analysis

1. Introduction

Global warming and the need for resource efficiency have led the global community in recent years towards a shift to sustainability. Sustainability in the field of infrastructure is currently mostly related to the use of insulation or other means to reduce energy consumption or to the adoption of recycled materials or low carbon cements in new construction. Yet, sustainability must be comprehended in a more holistic way since re-use of the existing building stock can dramatically decrease the consumption of resources and the corresponding carbon footprint. According to a survey conducted for the Buildings Performance Institute Europe (BPIE), approximately 38% of Europe's building stock was constructed prior to 1960, while another 45% was built between 1960 and 1990 (BPIE 2011). This large portion of existing building stock, especially in regions that are prone to earthquakes, has been constructed in many cases without seismic provisions, or with older presumptions, such as the strong beam-weak column mechanism, that are now considered dangerous when subjected to seismic excitations. Assessment and retrofit of existing infrastructures are therefore of utmost importance for the safety of the users; the 2nd Generation of Eurocodes will, in fact, include technical rules to fully and more broadly cover the assessment, reuse and retrofitting of existing structures for structural safety requirements (Luechinger et al. 2015). Additionally, some countries (e.g. Cyprus) have adopted a new approach in their legislation for energy efficiency renovations, which requires assessment and strengthening against seismic loads, prior to receiving any funding for energy upgrading; this approach is also proposed to be included in the European Directives (ETEK 2021).

In the case of historic reinforced concrete (RC) structures built during the first half of the 20th century, potential structural problems increase exponentially, compared to buildings built after 1970s, since apart from their patented construction techniques, long life span, special architectural features, experimental use of materials of low strength (e.g. $f_c = 12$ MPa and $f_s = 220$ MPa) and quality (rebars without ribs), small cover of rebars, lack of or sparse stirrups, irregularity in plan and elevation, these structures have also been exposed to weathering, environmental changes, and seismic actions, for

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which they were not originally designed. Their study, assessment, and retrofit are, therefore, of paramount importance, as they are part of the global cultural and architectural heritage, while the choice of the assessment method affects the extent of repairs and strengthening required for their preservation and reuse (Georgiou, Ioannou, and Pantazopoulou 2019). The level of performance considered for their assessment (i.e. damage level corresponding to seismic events of a specified return period) varies, depending on the uses and importance of the monument itself, and must be selected on a case-by-case basis. Depending on the conservation principles, the ownership status and the number of users, a historic RC structure may be assessed for safety levels corresponding to ordinary buildings ($\gamma_I = 1$), or for higher safety levels, if the structure has high social value and occupancy. Lower performance levels may be considered to avoid damaging the historic form due to reinforcement interventions; higher levels of performance may be required to reduce repair interventions following a potential earthquake. In any case, the safety of users is the most crucial factor and, ultimately, the retrofit scheme must preclude the loss of human life, while at the same time respecting the architectural form by allowing some exceptions to the regulations (ASCE 2018; EN1998–3 2005).

1.1. State of the Art on the Assessment of Inelastic Analysis of Short Irregular in Plan and Elevation Structures

One of the most important issues that the engineering community is called upon to address during retrofitting projects, is the selection of the appropriate analysis method for the assessment of the load-bearing capacity of existing irregular reinforced concrete buildings under seismic excitation (Freeman, Nicoletti, and Tyrell 2010). Irregularity in plan is a common phenomenon in most old-substandard buildings and usually leads to an increase in stresses of the load-bearing elements and makes the analysis more complex than for regular structures (Fajfar, Marušić, and Peruš 2005; Herrera and Soberón 2008). The choice of assessment method will affect the requirements of strengthening in terms of strength and ductility, as well as the magnitude of damage that is likely to develop in a future seismic event (Mahdi and Gharaie 2010). Inelastic static analysis (static pushover), due to the assumptions it contains, is considered more suitable for short, rigid, and symmetrical buildings, while it is considered unsuitable for buildings that are tall, torsionally sensitive, non-rectangular, with coupling between translational and torsional oscillations, or very non-linear with degradation of strength (NEHRP 2011). In particular, some researchers discourage its use in cases where irregularity or phenomena related to higher modes occur (Carvalho, Bento, and Bhatt 2013; Katsanos, Sextos, and Elnashai 2014; Mehdi, Khoshnoudian, and Moghadam 2014; Oyguç 2012). Nevertheless, inelastic static analysis continues to be an option, even in these cases, under Eurocode 8-Part 3.

In the case of asymmetric structures that shift during seismic loading from the elastic to the inelastic region, the effect of structural irregularities on their response becomes more complex, since the distribution of strength and stiffness constantly changes with increasing damage (Ghayoumian and Emami 2020). Elastic static analysis methods were developed primarily for the assessment of structures, the response of which is mainly translational, so their adoption in structures with irregularities in the plan presents applicability issues, especially in terms of accurate evaluation of the response (Rofooei and Mirjalili 2018). Despite extensive research to investigate the assessment of asymmetric structures with static inelastic analysis, the evaluation of the ductility and damage indices in different planar loading directions has not yet been investigated (Ghayoumian and Emami 2020). Therefore, the issue of choice of the analysis method for asymmetric buildings has not yet been fully studied, while normative documents, such as the Eurocode, also seem to be incomplete in this matter. In addition, ongoing research carried out worldwide for the study of this phenomenon, adopts models of standard constructions with current reinforcement detailing and materials (Das, Chandra Dutta, and Kumar Datta 2021).

Presently, in the European Union, engineers use Eurocode 8, Part 3 to assess the structural capacity of existing structures against seismic loading. The most used method from the ones proposed in the code is the non-linear pushover analysis, since the time-history analysis is found to be too complicated and time-consuming (Bhatt and Bento 2014) and difficult to use its results for retrofit purposes. Yet, the use of the pushover (Nonlinear Static) analysis on existing irregular structures has been studied only by very few authors up until now, e.g. Chopra, Goel, and Chintanapakdee (2004), Fajfar, Marušić, and Peruš (2005), Bhatt and Bento (2014), Carvalho, Bento, and Bhatt (2013). This renders its use for existing reinforced concrete structures with irregularities in plan and elevation problematic. Another parameter of variability is what the different researchers use for determining the actual seismic demands. Bhatt and Bento (2014) compared four different non-linear static procedures, i.e. 1) Capacity Spectrum Method- CSM-FEMA440, 2) N2 method proposed in the Eurocode 8 (CEN 2004, 3) Modal Pushover Analysis (Chopra and Goel 2002; Chopra, Goel, and Chintanapakdee 2004), and 4) Adaptive Capacity Spectrum Method (Casarotti and Pinho 2007), considering only three real records in the two planar directions (without using a vertical time-history). It is found that the modal load pattern results in lower values of base shear than the uniform load pattern and the adaptive capacity spectrum method, while in terms of displacement, all the static analyses presented led to higher values than the time-history results, assuming that the Eurocode N2 and CSM-FEMA440 lead to more conservative predictions.

Furthermore, there are variations on the way the time-histories are applied to the structure since the codes usually permit separate applications on the principal axes of the structure and later on a combination of the results, i.e. Ex + 0.3 Ey. D'ambrisi, De Stefano, and Tanganelli (2009) studied the performance of an irregular in plan and elevation school building built in 1974, with the extended N2 procedure (Fajfar, Marušić, and Peruš 2005), since the extended N2 methodology is intended only for considering the effects of in-plan irregularity only. They used a set of seven bi-directional time-histories, scaled to the desired PGA, yet they applied each acceleration record separately to the X and Y directions, and no vertical acceleration time-history was applied. Finally, they concluded that the modified N2 procedure predicts conservatively the drifts and rotations in relation to the inelastic dynamic analysis.

The results obtained by the Non-linear Static Analysis should be more conservative than those obtained from time-history, in order not to underestimate structural response and retrofit requirements. Recently, Daei and Zarrin (2021) have proposed a new enhanced multi-modal pushover procedure by enveloping the results obtained by different single-run pushover analyses; yet, their procedure was validated on three steel special moment frames with different heights.

1.2. Objective and Scope

Various studies presented in the bibliography have concluded that the non-linear static analysis as per EC8-Part 3 derives conservative results for the assessment of existing irregular structures. Yet, (a) most of the studies were conducted on relatively new constructions, built with more recent materials, or even steel structures, (b) the results were compared with time-history inelastic analysis applied separately in the principal directions, and even not considering the vertical component, (c) the structureswere either irregular in plan or in elevation (not both) (Das, Chandra Dutta, and Kumar Datta 2021), (d) the models used were either only elastic, and hence their results were not valid in the non-linear range (Manoukas 2019), or they presented ductility and no brittle (shear, lapping, and node) failures.

A case study of a specific historic building completed in 1965 in Cyprus, the Nicosia Old Municipal Market, is adopted in this paper. The building exhibits irregularities in plan and elevation, peculiarities in the materials and the typology of construction, and very high flexibility (long natural period). It was constructed with the available materials of the time, based on the then typical design practices, which did not include any particular seismic provisions. All these characteristics provide this research with a rare combination of aspects that have not been previously addressed in tandem.

Prior to modelling of the structure under study, extensive on-site investigation was performed for the verification of its current condition, while destructive and non-destructive testings was used to assess the materials' properties. After the verification of the geometry of the members, the structure was modelled in SAP2000. Assessment of any possible brittle failures was employed, and proper inelastic hinges (plastic or brittle) were incorporated into the model. The structure was then analysed using inelastic static analysis and inelastic time-history analysis, based on the provisions of Eurocode 8-Part 3. The results from the two types of analysis were compared in terms of drift to assess the capacity of the non-linear static procedure to capture the full range of demands on the particular members, as calculated by the time-history inelastic analysis, which is hereby considered as the reference. A set of seven records were used, with all their three-directional components, applied simultaneously to the structure.

2. The Nicosia Old Municipal Market Building

The Old Municipal Market is located in the central walled city of Nicosia, Cyprus. The building served the public as a market from its inauguration until 2017, when it was abandoned for some years. It is listed as a concrete heritage building in Cyprus and has monumental character and a modernist architectural style. At the moment, retrofit works are undertaken to reuse it as a research center. According to the original plans, it was built between 1964 and 1965 and it had significant socio-economic influences in the walled city. It consists of a basement on a small section of the total floor plan, a ground floor area of 1968 m^2, and a first floor, which hosted offices of the Municipality of Nicosia. The building is made of reinforced concrete, cast on site, and it is divided by expansion joints measuring ≥3 cm into 4 statically independent parts. The four individual parts consist of RC slabs, beams, columns, and walls; the latter, according to exploratory excavations, are supported on connecting beams without footings. Most frames on the perimeter of the building have brick infill walls that do not extend to the full column height, thus creating captive columns (Fig. 1). Additionally, there is a great irregularity in the elevation of the structure in terms of stiffness, due to the different height of the floors, a large variation of mass distribution and increased masonry infill percentage on the first floor. Only the part of the structure marked with blue colour in Fig. 1 was selected for analysis. This part has an irregular L-shape, indicating that the center of rotation (CR) is offset from the center of mass (CM). The L-shaped area was chosen as it was the only irregular part of the structure. The other three parts were rectangular in shape, and eccentricities in plan did not exist. Therefore, the pushover analysis is considered more accurate as the primary mode shapes do not include any coupling between the lateral and torsional components. In the other three parts, there is no torsional modal shape.

Most of the columns continue with the same dimensions on the first floor, while the reinforcement remains the same or is slightly reduced. Some of the first floor columns are supported on beams and do not continue to the ground floor. This requires using a vertical earthquake component for the seismic assessment of the building, which is not taken into account in static inelastic analysis. The cross-sectional width of the beams varies from 0.2 m to 0.515 m, while the beam height is 0.7 m. The ground floor has a height of 4.45 m, while the height of the first floor is 3.10 m. The floor slabs are 0.2 m thick.

It is worth noting that the Republic of Cyprus, which was established in 1960, had no universities or research centres until about 30 years ago, while no local regulations for the design of concrete structures existed prior to 1992. Cypriot engineers, who studied in various countries abroad, were mostly designing according to the foreign regulations. Moreover, in the absence of local enforcement of international design practices against seismic excitations and without any measurements of the local intensities of earthquakes, the buildings were designed only for gravity loads. It is also important to note that, at the time, there were no batching plants in Cyprus, and concrete was thus prepared on site in small quantities, approximately 2 tn at a time. This led to great variability in the quality of the material in various parts of the structure, even along the height of a column, as there was also no equipment for vibration and proper compaction and consolidation.

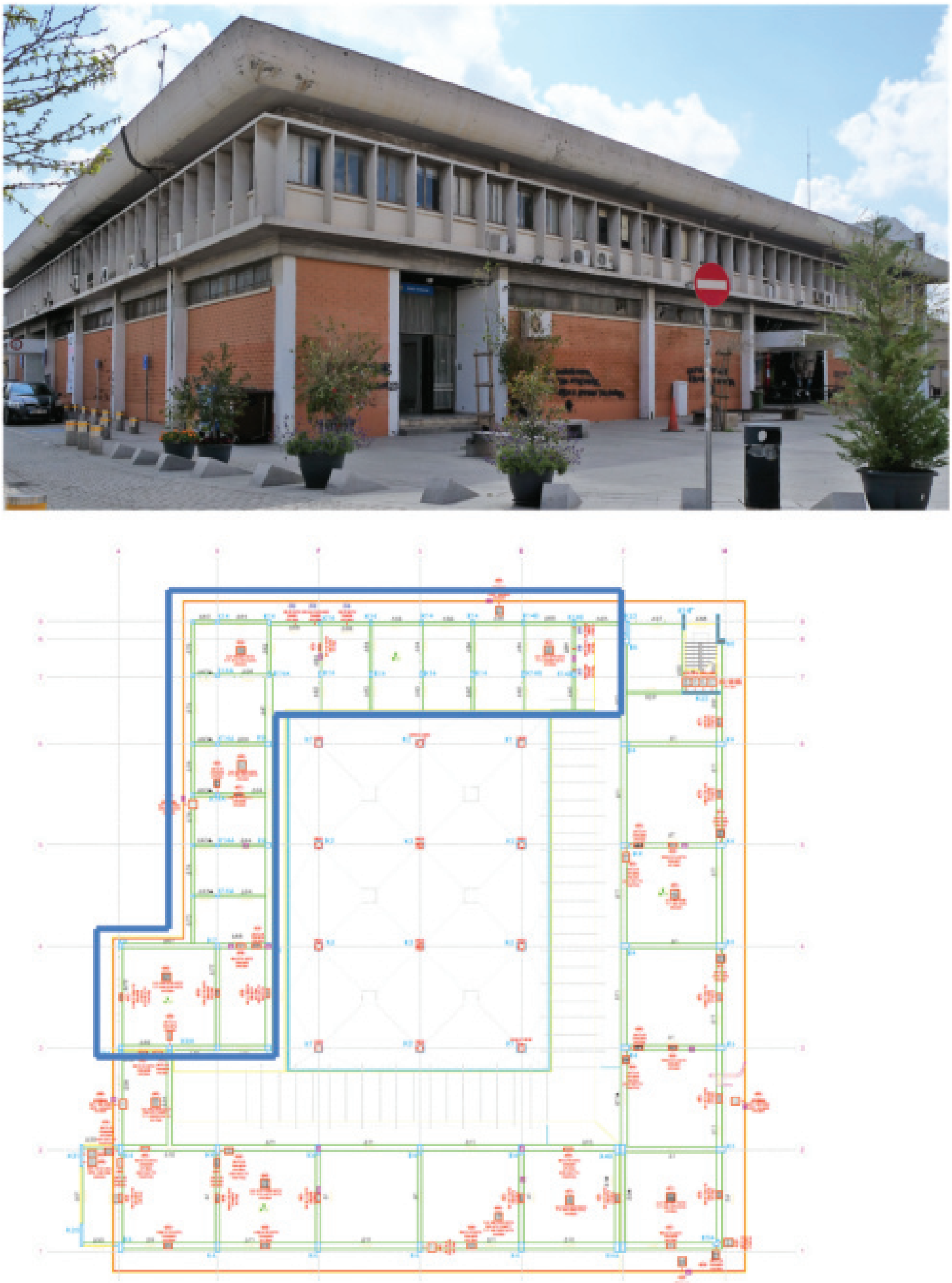

Figure 1. Exterior view of the structure (top) and floor plan (bottom) showing the part that was assessed in the present study (in blue).

2.1. Building Pathology and Verification of Current Condition

Typical damages were recorded and are shown in Fig. 2. Cracks are visible in most of the beams. Drainage from the roof passes through some of the external columns of the building, inducing moisture to them and potential corrosion to the reinforcement. The base of the columns with embedded drains, in fact, shows extensive cracking, probably due to the corrosion of the reinforcement. Nevertheless, the overall state of the building is rather good, despite its abandonment and lack of maintenance for many years.

Figure 2. Recorded damages in the old municipal market (2019–2020): (a) Rust stains, (b) Cover delamination and rebar corrosion, (c) Water drainage passing through columns with extensive stains and cracking, (d) Poor cover – exposure of reinforcement – extensive erosion, (e) Mold, stains, efflorescence internally under slab, (f) Deflection cracks on beams, (g) Deformations in the cantilever slab, (h) Horizontal cracks at the connections of the infill walls with the structural system.

Corrosion is evident in some of the slabs of the first floor, especially at locations where rainwater is ponding on the roof, and in the columns of the ground floor, where the cover to the reinforcement has spalled and the latter is exposed. The slabs at these locations have black stains and show cover delamination. Any deterioration in reinforced concrete is related to moisture, carbonation, and corrosion of the reinforcement. The walls in one of the shops also show some diagonal cracking, which suggests a minor settlement of the foundations. Mosses and blemishes are extensive on the exterior surfaces of reinforced concrete.

In general, there are no visible cracks from seismic loads on the reinforced concrete elements. In order to investigate the seismic events that may have influenced the structure in its life span, the PGA

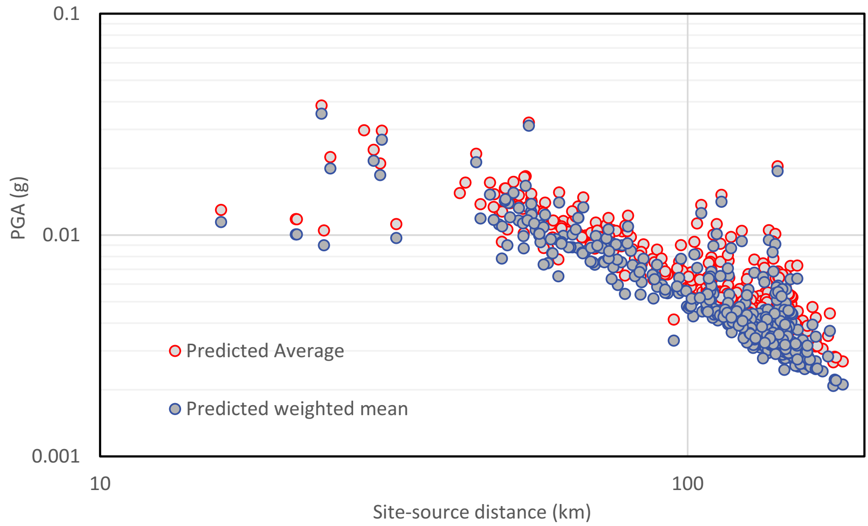

Figure 3. Estimated PGA values at the site, for seismic events with $M_w > 4$ in the period 1963–2019.

felt at the specific site in the period from 1963 (beginning of construction) to 2019 (beginning of retrofitting) was estimated for the seismic events appearing in the catalogue of the Cyprus Geological Survey Department. Earthquake magnitude conversion from M_L to M_s and M_s to M_w was carried out using the empirical equations of Tselentis (1997) and Mohamed et al. (2012), respectively.

The PGA was estimated based on earthquake magnitude and source-to-site distance as an average (simple and weighted) of the predictions of the attenuation relationships of Theodulidis and Papazachos (1992), Danciu and Akis Tselentis (2007), Tento, Franceschina, and Marcellini (1992), Joyner and Boore (1981), Boore and Atkinson (2008), with weight factors 0.3, 0.3, 0.1, 0.15, and 0.15, respectively. The averages of the predicted PGA (simple and weighted) are depicted in Fig. 3. The estimated PGA values at the site location, during the life span of the building, have a maximum value of roughly 0.035 g, corresponding to lateral loads equal to 3.5% of the weight of the structure. The force applied on the structure is well below the Base Shear for yielding, and thus none of the cracking observed on the structural members can be attributed to earthquake shaking.

2.2. Building Survey and Testing of Materials

An extended survey was performed for the verification of the member geometry and sizes. Only some of the original construction drawings were found. Examination of these drawings revealed significant differences with the as-built investigation in some parts of the structure, especially in the one under investigation. In order to increase the detailing information, a rebar detector was used to detect the steel bar reinforcement, the bar cover, and diameter in the beams and columns. The steel used was mild S220 without ribs. Based on the old drawings of the structure, the column concrete cover was specified at 2–5 cm, in line with the results of the on-site investigation, while the lap splices were $40d_b$ long. Yet, in some columns, the latter was actually not applied, with the lapping lengths being substantially smaller.

The detection of the reinforcement position was also used to determine the locations for non-destructive rebound tests (EN 12504–2) and the possible positions for concrete core sampling (EN 12504–1). The core samples were enough to determine the Knowledge Level of the strength of materials as extensive (KL3), according to the requirements for nonlinear assessment of EC8-Part 3 (EN1998–3 2005). Core samples were also used to determine the carbonation depth, the aggregate size, and the chloride content of concrete. All the tests were performed by the Laboratories CERS Cyprus

Table 1. Compressive strength of concrete.

Member	Average (N/mm^2)	Standard deviation (N/mm^2)
Columns ground floor	17.8 (10.60 at lower parts)	3.5
Columns first floor	12.8	4.0
Beams ground floor	17.7	2.6
Beams first floor	16.5	6.7

and Geoinvest Ltd, and the results were kindly provided to the authors by the owner of the building, Nicosia Municipality.

The carbonation depth results indicate that, in most cases, the carbonation depth is less than the cover of the reinforcement, with only a limited number of members exhibiting values up to the cover depth. The original concrete mix design, as described in the old drawings, was 1:1.5:3 by volume (cement:sand:coarse aggregates) for the columns and 1:2:4 by volume for the beams and slabs. In the case of the 1:2:4 mix design, based on oral communication, 1 part of water was used if the aggregates were wet, while 1.5–2 parts of water were used if the aggregates were dry. Local crushed diabase coarse aggregates and natural sand were used in all mix designs. From the concrete cores, aggregates with diameter >20 mm could be observed. The compaction of the concrete was deemed average to good, while the air voids in the concrete, due to entrapped air and incomplete compaction, were small (0.5–3.0 mm).

Uniaxial compression (EN 12390-3 2009) tests were performed on a number of cores extracted from members of the structure. The results are listed in Table 1. An average of 17.8 MPa was found from the compression tests (EN 12504–1) on samples from the ground floor columns at higher levels, with a standard deviation of 3.48 MPa, while, when the tests were performed at the lower parts of the columns, the results only reached 10.6 MPa. The mean compressive strength of the columns on the first floor was 12.8 MPa. Beams and floor slabs had higher compressive strengths.

The reinforcement was in close agreement with the original detailing drawings. Exploratory excavations showed that the columns continued up to 2 m underground, in order for the foundation (consisting of isolated footings) to reach a firm ground. Some voids were found below the ground floor slabs. The columns at the foundation level were connected with beams with very sparse reinforcement. Many of the columns had drainage pipes embedded in them (something that is now forbidden by the building codes). Stirrups in beams are sparse and not suitable for ductile structures and/or the formation of plastic hinges at the edges of members. The lap splicing of the longitudinal reinforcement was also found to be less than what is required in current seismic codes. Additionally, the reinforced concrete walls shown in the original drawings were connected only on beams at the foundation level (Fig. 4), leading to the conclusion that they cannot attain flexure due to the lack of foundation system.

3. Assessment of the Seismic Capacity of the Structure

3.1. Modelling of the Structure

The structure was modelled in the commercial program SAP2000 (CSI 2009) in order to assess its capacity under seismic conditions (Fig. 5). The reinforced concrete beams and columns were simulated using 2-node frame elements. The average strengths were used to determine the properties of the various materials. The modulus of elasticity of concrete was determined indirectly from its compressive strength, based on EC2 (EN 1992-1-1 2004). For all the elements in the structure under study, modifiers were used to reduce the stiffness of the cross-sections to the actual cracked stiffness, assessed from the moment-curvature diagrams obtained with the use of RESPONSE2000 (Bentz and Collins 2000), in order to consider the stiffness degradation during the seismic event (KANEPE 2017; NZSEE 2017). Diaphragmatic action was applied to all the nodes of the floor levels. At foundation level, the columns were resting on linear links (springs and dashpots), according to the properties of the foundation system and the soil, while the walls in the basement were assigned with compression-

Figure 4. Exploratory excavations and concrete cover removal during retrofit works: (a) lack of foundation slab, (b) holes and ancient ruins under the foundation system, (c) drainage pipes within columns, (d) insufficient stirrup spacing, (e) insufficient lap splice length, (f) lack of foundation of the R/C wall, (g) insufficient flexural and shear reinforcement on foundation beams.

only springs that allow free movement away from the soil (formation of a gap). All the floor slabs were assigned with the load combination of G + 0.3Q, with additional load for the finishing of the floor surfaces. The live load was chosen based on the Cypriot National Annex to Eurocode 1 (Standard 2003). This load combination was also used to derive the mass of the structure for the modal analysis. Some columns of the ground floor were assigned with hinges (zero moment) at their bottom node, due to the increased axial load ratio $v = N_{G+0.3Q}/(A_c f_c)$ (Fig. 5). Note that these columns were subjected to a value of v that is higher than the limit of 0.4 that corresponds to balanced column failure, which

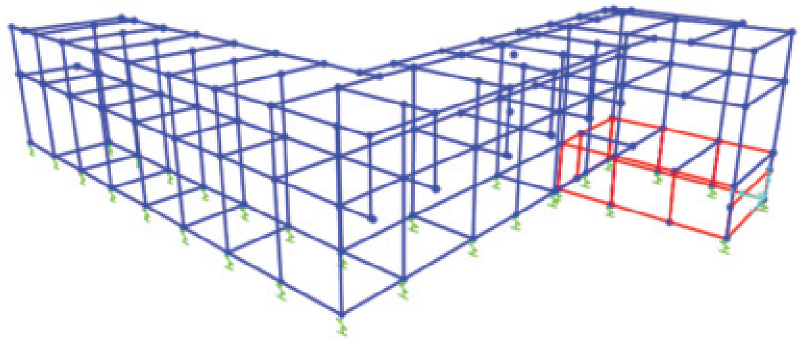

Figure 5. 3-dimension model of the structure in SAP2000.

identifies the limit of brittle response in the Axial Load vs. Moment Interaction Diagram. This load ratio was estimated from service life loads only, without considering the additional axial load that the seismic overturning action imposes on the columns. On account of the high value of ν and the reported corrosion of reinforcement at the base of those columns, it is concluded that no moment can be sustained at their base; thus, a hinge was assigned in the model.

3.2. Geological and Seismological Characteristics

According to the geotechnical study carried out for a nearby plot, at a depth of 2 m from the ground surface, a soil layer of alluvial deposits begins, which is overlain by old coarse grained backfill; the layer of alluvial deposits extends to a depth of at least 12 m. None of the exploratory drillings carried out in the framework of the aforementioned geotechnical study reached the marls of the Nicosia geological formation, which constitutes the bedrock in the area. According to the geotechnical study, the properties of the soil on which the structure is located correspond to fine alluvium with $\varphi = 28–30o$, $N = 20$, $\nu = 0.35$, and $\rho = 1.8$ tn/m^3. The soil type is characterized as Type C as per EC8–1 (soil amplification factor S = 1.15). According to the Cypriot National Annex to EC8–1, the peak ground acceleration a_{gR} is 0.20 g (for the event with 10% probability in 50 years). Taking into account the socio-economic impact of a potential collapse (both for the former and future use), the building's importance class is set to III, with a value of $\gamma_I = 1.2$.

3.2.1. Link Properties for the Foundation System

Footings were grouped based on their dimensions and depth. The soil–structure interaction was taken into account with the use of links connected to the base of the columns. These incorporate both the deformability of the soil and the corresponding radiation and hysteretic damping provided by the ground (Pitilakis et al. 1999). The spring and dashpot properties were determined according to Gazetas (1991) and the results are listed in Tables 2 and 3.

3.3. Assessment of Brittle Failures and Plastic Hinges

Potential brittle failures that may occur in old substandard members, designed without any seismic provisions, are a crucial parameter for the assessment and retrofit of historic reinforced concrete structures (Pardalopoulos, Thermou, and Pantazopoulou 2013). The hierarchy between the individual failure mechanisms must be assessed in order to determine any prevailing brittle failure. The mechanisms of column failure hereby examined were the following:

- Yielding of the flexural reinforcement and failure in flexure, ($\mathbf{V_{flex}}$)
- Shear failure, ($\mathbf{V_v}$)

Table 2. Dynamic stiffness K of spread footings of columns.

Footing group	Length L (m)	Width B (m)	$K_{z,emb}$ (kN/m)	$K_{y,emb}$ (kN/m)	$K_{x,emb}$ (kN/m)	$K_{rx,emb}$ (kNm)	$K_{ry,emb}$ (kNm)	$K_{t,emb}$ (kNm)
K14	1.4	1.4	311024	469095	469095	231691	251464	367031
K14A	1.5	1.5	324657	482653	482653	265275	286846	432027
K14B	1.7	1.7	351988	509450	509450	341847	369027	581905
K9 (Ground floor)	1.7	1.7	351988.	509450	509450	341847	369027	581905
K9 (Basement)	1.7	1.7	573603	1007959	1007959	3247522	4118874	2028682
K7	2.0	2.0	6150165	1054541	1054541	4003847	4683232	2893500

Table 3. The total damping coefficient C of spread footings of columns.

Footing group	Length L (m)	Width B (m)	Total $C_{z,emb}$ ($kNsm^{-1}$)	Total $C_{y,emb}$ ($kNsm^{-1}$)	Total $C_{x,emb}$ ($kNsm^{-1}$)	Total $C_{rx,emb}$ (kNms)	Total $C_{ry,emb}$ (kNms)	Total $C_{t,emb}$ (kNms)
K14	1.4	1.4	8638	13521	13521	6013	6013	9301
K14A	1.5	1.5	9096	14019	14019	6892	6892	10949
K14B	1.7	1.7	10036	15020	15020	8900	8900	14750
K9 (Ground Floor)	1.7	1.7	10036	15020	15020	8900	8900	14750
K9 (Basement)	1.7	1.7	15651	34082	34082	93188	93188	51420
K7	2.0	2.0	17130	36917	36917	115162	115162	73355

- Anchorage and Lap splice failures, ($\mathbf{V_{\alpha}/V_{lap}}$)
- Joint shear failure, ($\mathbf{V_j}$)
- Formation of plastic hinges in the adjacent beams (ductile behavior) $\mathbf{V_{by}}$

The capacities of the members for the different mechanisms of failure, in terms of equivalent column shear, are shown for the X-direction in Fig. 6, with the denomination including first the floor level and then the particular member-column, i.e. K1–13 is a specific column at the ground floor. All the columns of the first floor continue to the second floor, yet on the second floor some additional columns appear that are supported only on beams, i.e. ΦK2–1 to ΦK2–6. The first conclusion to be made is that, in almost all cases, the failure of the columns will precede yielding of the beams. This is attributed to the erroneous design concept for RC members of the era that required "strong beams-weak columns." The failures observed during earthquakes worldwide in the years that followed showed that this kind of design was responsible for the collapse of structures. Hence, modern seismic codes, through the capacity-based design approach, have rectified this approach by promoting a "strong column-weak beam" design.

Additionally, the results indicate that most of the columns on the ground floor will behave in a ductile manner with flexural failures, except for some columns with very high axial load that will show brittle failure of the compressive zone prior to yielding reinforcement and some cases of brittle shear failure for loads parallel to the weak axis of the members. On the contrary, most of the columns on the first floor will fail first due to brittle shear failure, caused by their short length, in combination with the very sparse stirrups, having a spacing of more than 300 mm.

The inelastic behavior of the concrete members is imported into the model by the use of plastic hinges through the introduction of their moment-curvature or moment-drift response. The plastic hinges are able to consume energy through the initiation of cracking in concrete and yielding of the reinforcement and are mainly located at the edges of beams and columns. Based on the prevailing type of failure, two types of hinges were used to determine either ductile behavior (ductile plastic hinge based on deformation) or brittle failure (hinge based on member shear force).

3.4. Dynamic Characteristics of the Structure

Three different models were used: (a) Fixed base without stiffness reduction, (b) Fixed base with stiffness reduction, and (c) Springs-dashpots at the foundation level to account for soil–structure

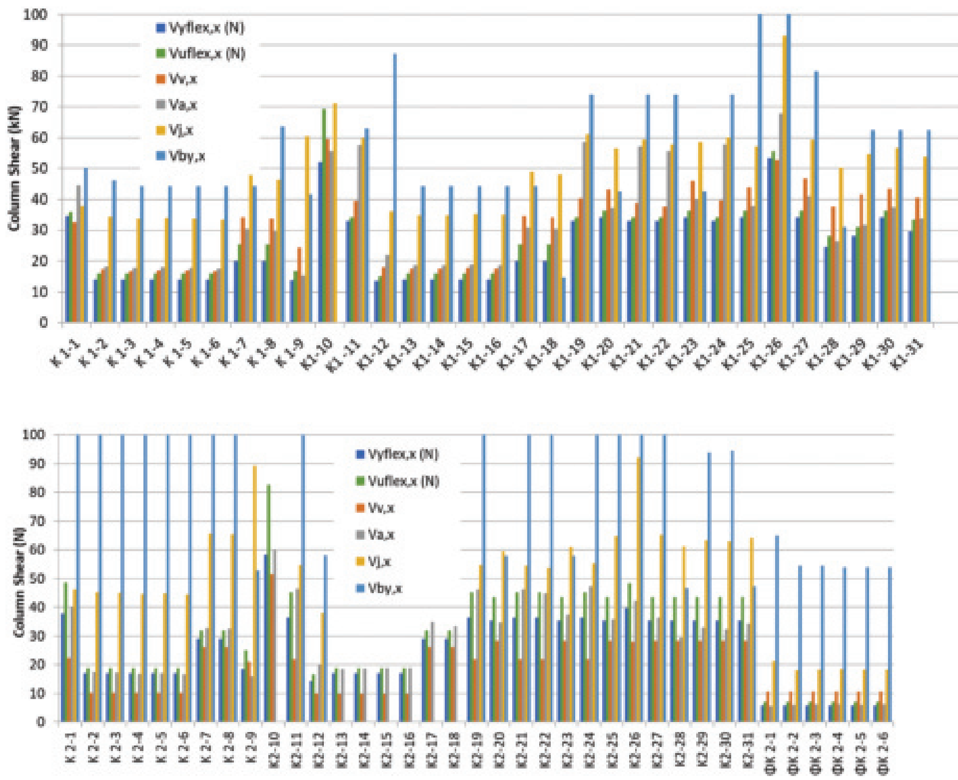

Figure 6. Shear force for different mechanisms of failure for the X-direction of seismic action for the ground floor columns (top) and for the first floor columns (bottom).

interaction (SSI) and stiffness reduction. The stiffness reduction in the models is related to the cracking of the structural elements in the case of earthquakes. The EC8-Part 3 related to the assessment of structures suggests that the stiffness of members is reduced when seismic assessment is performed. This reduction in the stiffness may be determined either by using half the stiffness of the uncracked cross-sections, or in a more detailed manner, by using the stiffness that corresponds to the yielding of the flexural reinforcement determined by the ratio of yielding moment to drift at yield (My/θy). In our case, the latter was used to determine the cracked stiffness of the structural components and was implemented in the model by an appropriate reduction factor of the members' moment of inertia. The incorporation of stiffness reduction coefficient results in a significant increase in the fundamental period of the structure from 0.62 sec to 0.99 sec (Table 4). However, the use of links has negligible effect on the fundamental period, due to the stiff soil characteristics and the relative flexibility of this specific structure. It is interesting to note that the value of 0.62 sec is approximately three times larger than what is expected for a two-storey building designed based on modern codes (such as Eurocode 8).

The first three modal shapes of the structure are depicted in Fig. 7 and their characteristics are recorded in Table 5. While the first mode of the structure is primarily translational in the X-direction,

Table 4. Fundamental period for different models.

	T_1 (sec)
Model A	0.62
Model B	0.99
Model C	1.00

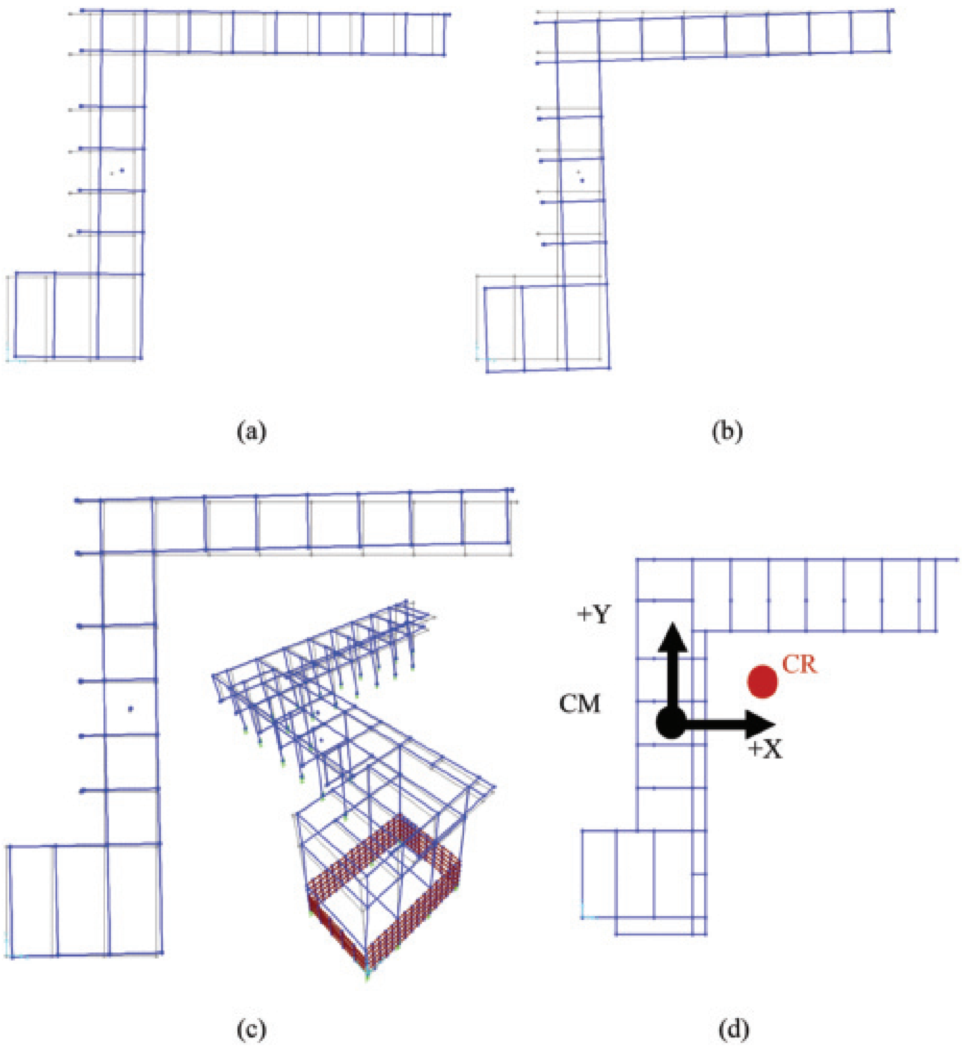

Figure 7. Results of modal analysis: a) first mode T = 1.003 sec, b) 2nd mode T = 0.95 sec and c) 3rd mode T = 0.495 sec in 2-D and 3-D view, d) eccentricity between CM and CR.

Table 5. Results of modal analysis (period and cumulative mass percentage).

Mode	Period (sec)	$SumU_X$	$SumU_Y$	$SumR_Z$
First	0.995	0.826	0.045	0.05029
Second	0.936	0.897	0.614	0.2555
Third	0.459	0.897	0.892	0.87396

the second and third modes are combined translational in the Y-axis and rotational around the Z-axis. This agrees with the finding that the center of mass (CM) has high eccentricity in the X-direction with respect to the CR.

Additionally, when the mode shapes are plotted height-wise (Fig. 8), it is clear that the structure behaves as a pilotis, with a soft ground floor storey, indicating that during seismic loading, most of the displacement that will be induced in the structure due to the motion will be undertaken by the ground floor columns, leading to increased levels of ductility demand. This is also evident by the floor characteristics, such as their height, mass, and stiffness (Table 6).

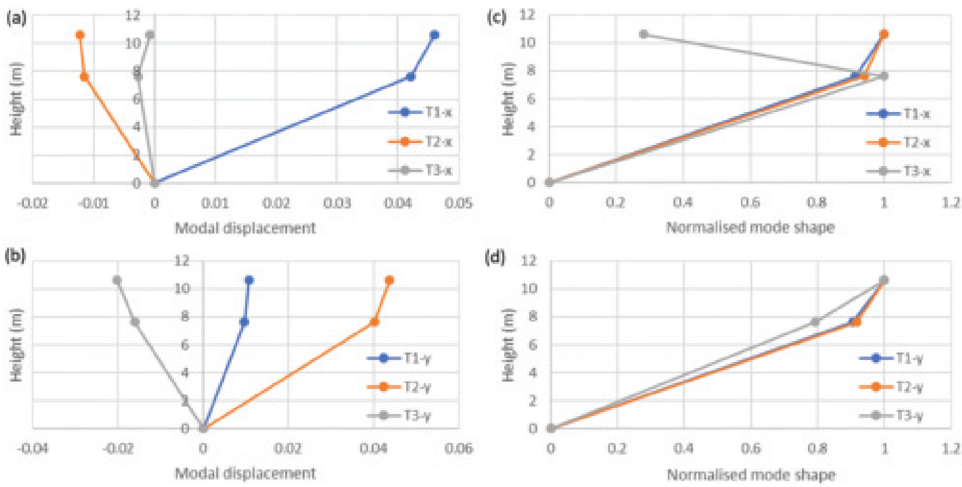

Figure 8. Displacements from modal analysis at the CM (a) X-direction, (b) Y-direction and mode shapes heightwise (c) X-direction, (d) Y-direction.

Table 6. Floor characteristics: height, mass, stiffness (K).

Floor	Height (m)	Mass (tn)	K_x (kN/m)	K_y (kN/m)
1	4.49	346.19	21316.07	45517.91
2	2.98	149.71	99216.22	165818.8

Based on the modal analysis of the structure, it is evident that:

(a) The structure has an irregular shape in plan (L-shape), which leads to torsion and is not recommended according to modern seismic regulations.
(b) There is an irregular distribution of stiffness on the two floors, as the ground floor columns operate at a greater free length than the floor columns while maintaining in most cases the same cross section and reinforcement.
(c) There is an unequal distribution of mass between the floors, which reinforces the phenomenon of irregularity in height and the formation of a soft ground floor.

3.5. Pushover Vs. Time History Analysis

According to previous research studies (Deierlein, Reinhorn, and Willford 2010; Inel, Tanik Cayci, and Meral 2018; Mahdi and Soltan Gharaei 2011), dynamic inelastic time-history analysis determines more realistically the response of a structure in a seismic event, and its bearing capacity with greater reliability, in comparison to any other method (Mahdi and Soltangharaie 2019; Moghadam and Tso 2000). Dynamic inelastic analysis has the advantage of taking into account the effect of higher modes in the structural response, especially in cases with many floors. However, the long time of analysis required and the human factors involved in the process of selection of accelerograms makes the use of this method impractical for most routine design applications (Mahdi and Gharaie 2010). Therefore, through a series of studies (Chopra and Goel 2002; Elnashai 2001; Faella, Giordano, and Mezzi 2004; Kalkan and Kunnath 2006; Moghadam and Tso 2000), the static inelastic analysis (static pushover) was developed, which is a more practical method and achieves a satisfactory balance between the reliability of the results and the applicability for simple projects. In addition, it provides information on the displacement demand of the building and an estimation of the load-bearing elements that may develop major damage (Mwafy and Elnashai 2001). Research has shown that this method of analysis provides an adequate estimate of the displacement demand, especially for buildings where the first mode prevails (Inel, Tanik Cayci, and Meral 2018). However, the actual levels of displacement, for the particular case of structures with irregularity in plan and elevation, are not accurately determined, due to premature termination of the analysis, while the capacity of the structure is also overestimated (Cavdar and Bayraktar 2014; Papanikolaou, Elnashai, and Pareja 2006; Thermou and Pantazopoulou 2011).

In this research, both types of analysis have been used for the assessment of the historic reinforced concrete structure under study, which has irregularities in both plan and elevation; the results are compared in terms of ductility, drift capacity, and demand.

3.5.1. Inelastic Static Analysis (Pushover)

The inelastic static analysis is based on three main assumptions: (a) the load-bearing system has two main axes of symmetry, X and Y, (b) there is one primary horizontal seismic component of force exerted parallel to each plane of symmetry, and (c) the dynamic behaviour is determined from the main oscillation mode shape that activates the maximum mass percentage. Based on the aforementioned assumptions, elastic static analysis cannot be applied to the evaluation of drifts or local phenomena, in the case where the effect of higher modes is crucial or in the case of irregular structures.

In the case of inelastic static analysis (pushover), an increasing force was imposed on the floor levels (centre of mass – CM) for each direction of motion, based on two different mode shape distribution patterns ("fundamental mode" and "uniform") used when irregularities in elevation exist (as per EC8-Part 3, § 4.4.4.2 (EN1998–3 2005)). The choice of the distribution of forces along the height of the building in this case plays a critical role in the analysis of the structure, as it remains unchanged throughout the load range, despite changes in the stiffness of the floors due to cracking. Furthermore, the actual levels of displacement of the floors are not accurately determined, due to premature termination of the analysis from non-convergence issues when local failures occur; thus, redistribution of forces is required.

3.5.2. Inelastic Dynamic (Time History) Analysis

In order to determine the seismic displacement demand, a time history analysis was performed with a set of seven natural accelerograms that were selected based on the provisions of EC8-Part 1 (CYS 2007). The accelerograms (Table 7) were selected from the PEER strong motion database (PEER Strong Motion Database on Line 2005), to correspond to geological conditions similar to those of the site of the building under investigation (Type C) and stem from earthquakes with magnitudes between 6.0 and 7.0 at epicentral distance in the range of 5–60 km (i.e. approximately the magnitude and potential source distances of the design earthquake for the wider Nicosia region). The average PGA of the accelerograms was targeted to be the same as the design ground acceleration of EC8 ($\gamma_I \cdot a_{gR} \cdot S$ = 0.28 g), while the average spectral acceleration in the range of periods between $0.2T_1$-$2T_1$ was above 90% of the elastic response spectrum, in each direction of motion. Figure 9 shows the acceleration spectra for the selected accelerograms and the corresponding EC8 response spectrum. The accelerograms were applied in each direction simultaneously and in combination, while the average values of all load combinations were derived for the assessment of the response.

4. Results and Discussion

4.1. Pushover Analysis

Figure 10 records the drift demands of the floors for the different cases of the pushover analysis, i.e. X and Y direction (positive and negative) and uniform and triangular distribution of forces. The drift demands were directly calculated from the displacement demand (as per Annex B, EC8-Part 1) divided by the free height of the columns on each floor. In all cases, the drift demands on the first floor are less than the yielding drifts of the columns (approximately 0.5%), while the ground floor

Table 7. Selected accelerograms (peer ground motion database).

Record Number	Earthquake Name	Year	Station Name	Magnitude	Mechanism	R_{jb} (km)	R_{rup} (km)	V_{s30} (m/sec)
6959	"Darfield_ New Zealand"	2010	Christchurch Resthaven	7.00	strike slip	19.48	19.48	141.00
949	"Northridge-01"	1994	Arleta - Nordhoff Fire Sta	6.69	reverse	3.30	8.66	297.71
3965	"Tottori_ Japan"	2000	TTR008	6.61	strike slip	6.86	6.88	139.21
8124	"Christchurch_ New Zealand"	2011	Riccarton High School	6.20	reverse oblique	9.43	9.44	293.00
759	"Loma Prieta"	1989	Foster City - APEEL 1	6.93	reverse oblique	43.77	43.94	116.35
8130	"Christchurch_ New Zealand"	2011	Shirley Library	6.20	reverse oblique	5.58	5.60	207.00
5814	"Iwate_ Japan"	2008	Furukawa Osaki City	6.90	reverse	31.07	31.08	248.19

columns demand drifts that require a ductility of 4, something practically impossible for historic concrete structures without seismic provisions.

The damage stage of all columns for the case of the ±Y triangular distribution pattern is depicted in Fig. 11. Damages seem to concentrate on the ground floor; this coincides with the modal shape of pilotis found earlier and leads to decreased damages in the first floor level, mostly in a series of columns that do not extend to the ground floor. The formation of plastic hinges on the edges of the ground floor columns leads to the creation of a floor failure mechanism and consequently to the collapse of the building.

4.2. Time-History Analysis

Figure 12 depicts the drift demands along the height of the structure, from all load combinations, as well as their average (red dotted line), and Fig. 13 the levels of damage to the members from two of the time history analysis. Even though the first floor columns undergo a very low level of drift well below 0.5% (related to the yielding of the flexural reinforcement), it is seen that most of them fail and are in levels of damage not accepted by the assessment performance objectives. This is due to the previous conclusion that the first floor columns are susceptible to collapse in brittle shear failure prior to yielding the flexural reinforcement. Additionally, the drift demands at the ground floor, in the order of 2–2.5%, require great drift ductility of the ground floor columns, in the order of 4. Yet, the members seem able to perform adequately, albeit at the level of significant damage, something that is nevertheless accepted by the Performance Objective.

Figure 14 shows the spatial displacement of the control node, which is the same as the center of mass (CM), from all the time-history analyses. Displacements are characteristic of a structure, which is irregular in plan, with high torsional effects; most of the displacements happen in the first and third quadrants.

4.3. Pushover Vs. Time History Analysis

The hysteretic curves (base shear vs. displacement of the first floor CM) resulting from two sets of accelerograms are compared to the pushover curves in Fig. 15, for the two horizontal directions of motion (X and Y). While some of the time-history results are within the limits of the pushover curve, in others, either the total displacement capacity or the base shear capacity exceeds what is obtained from the pushover analysis. This is attributed to the fact that, in the pushover analysis, the applied forces on the floors have a constant ratio throughout, while in the dynamic analysis, redistribution allows the structure to sustain greater total displacements and base shear.

The drift demand of the columns, in relation to their in-plan location, differs for the two types of analysis. The comparative results between the responses of the ground floor four corner columns and

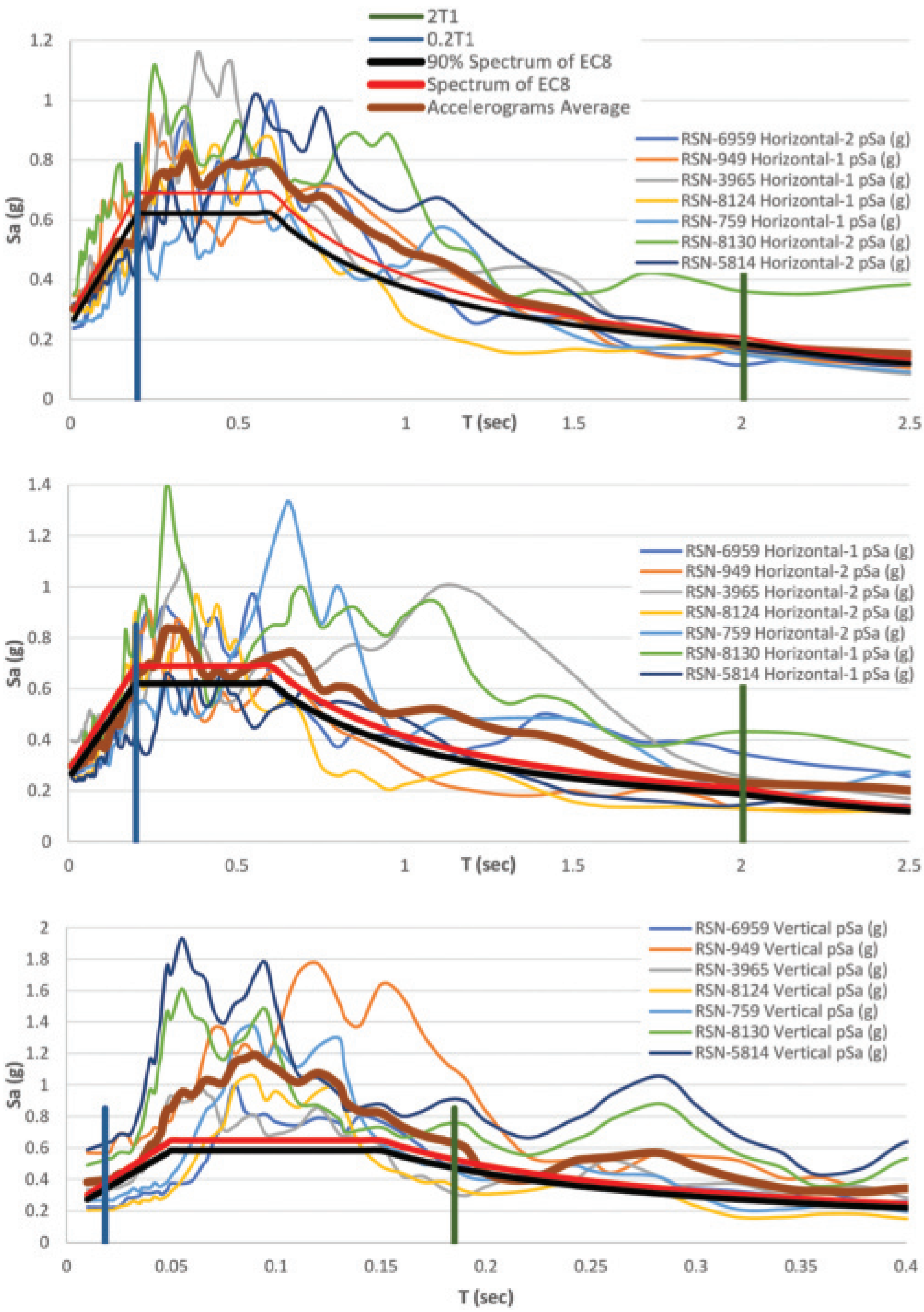

Figure 9. Acceleration spectra in the X (top), Y (middle) and Z (vertical) (bottom) directions for the selected accelerograms compared to the corresponding EC8 response spectra.

the column located closest to the CM (K1–23) are depicted in Fig. 16. In particular, for the +X-direction of motion, it is obvious that the pushover analysis demands the same drifts from all columns, regardless of their position in plan (black and gray lines), in contrast to the time-history analysis, where the central column (K1–23) demands the minimum drifts (1.5%), while the extreme columns drift demand reaches up to 2.5%. This phenomenon is even more pronounced in the –X

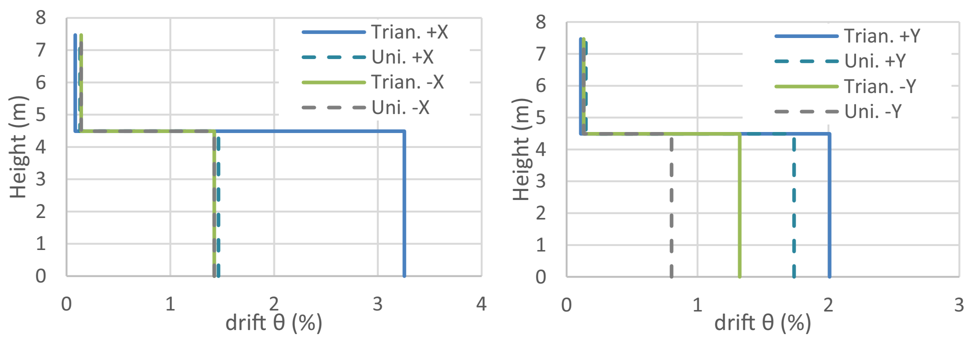

Figure 10. Drift demands from pushover analysis in X-direction (left) and Y-direction (right).

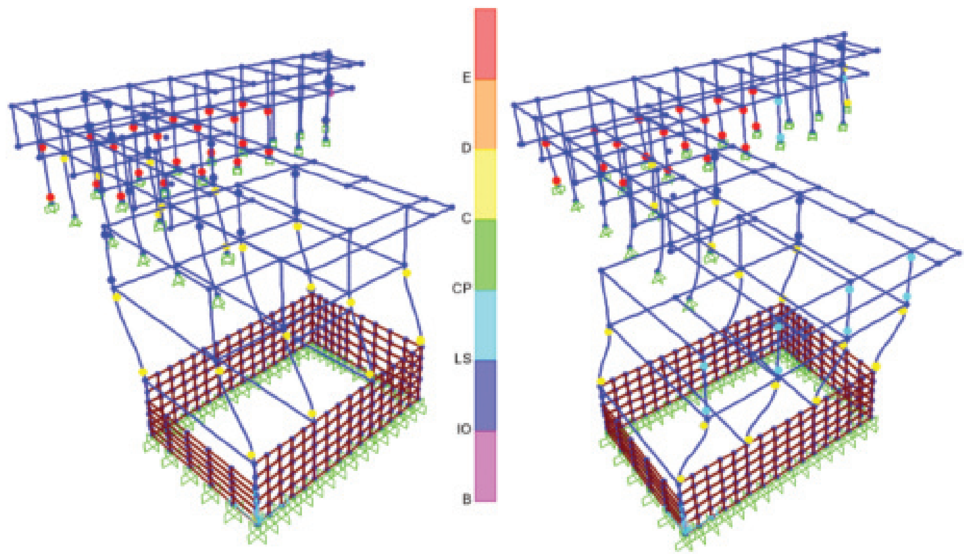

Figure 11. Damage stage in the plastic hinges (+Y triangular and -Y triangular) (IO: damage limitation, LS: significant damage, CP: collapse prevention, yellow: yielding, orange: hardening, red: residual strain).

direction (the CM moves away from CR), where the demand from the pushover (either with uniform or triangular distribution) is much smaller than the demand resulting from the time-histories. In the Y direction, where the eccentricity between the CM and the CR is even greater, there is a systematic underestimation of the drift demand by pushover analysis, which is greater in the −Y direction of loading (the CM moves away from CR). In most directions, the drift demands resulting from the pushover analysis with the triangular distribution of forces is closer to the results obtained by the time-history analysis, or even higher (on the safe side for assessment purposes).

Last but not least, the absolute accelerations and relative displacements of the first floor (1F) and ground floor (GF) center-of-mass, for one set of time-history accelerograms (RSN 6959), are depicted in Fig. 17. In the same figure, the displacements of the mass-centers (absolute and relative) are also depicted. The maximum relative displacements of the structure in the Y-axis is +0.04 m and −0.06 m, and in the X-axis +0.16 m and −0.07 m. What is also evident from the displacement graphs on both axes of the structure is that the relative displacements of the ground floor and the first floor are exactly the same, indicating that the first "weak" floor takes up all the deformation that the earthquakes impose on the structure. This is more pronounced in the case of the time-history analysis since the formation of the plastic hinges in the soft floor from the early stages affects the distribution of seismic

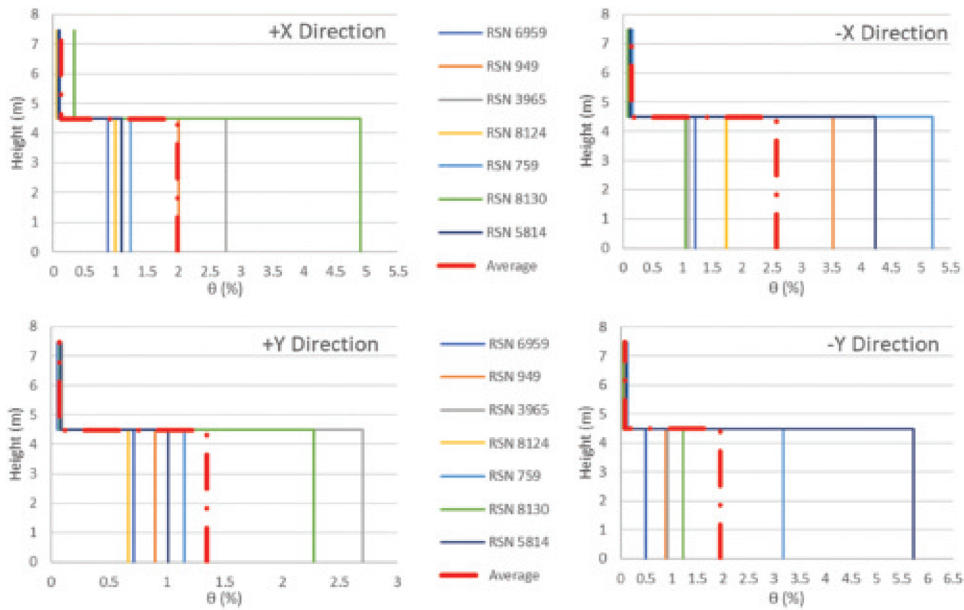

Figure 12. Drift demands from time-history from all accelerogram combinations.

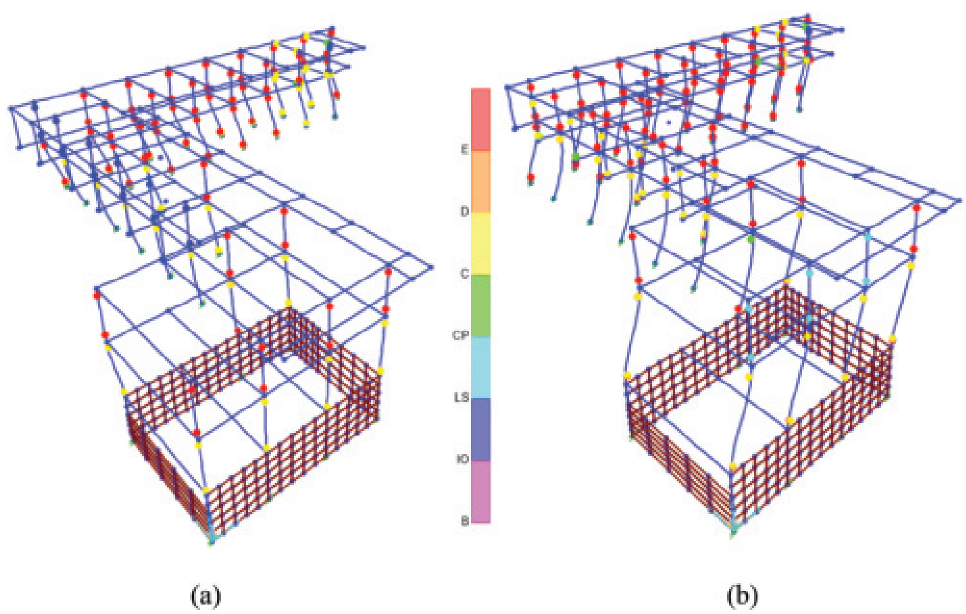

Figure 13. Damage level of the structure for seismic event (a) RSN 759 and (b) RSN 8130.

forces of the members in plan and elevation. In comparison, the seismic force distribution pattern in elevation remains constant throughout the pushover analysis.

A comparison of all cases (in terms of total-absolute-CM acceleration) is shown in Table 8. The absolute accelerations are greater in the Y-direction, with a maximum value of 3.307 m/sec^2, and lower in the X-direction, with 2.35 m/sec^2. In any case, these values are less than the accelerations resulting from the regulation spectrum (0.7 g) and the pushover analysis (1.5-2 g). This may be attributed to the extensive dissipation of energy that takes place in the plastic hinges during the cycles of the dynamic analysis.

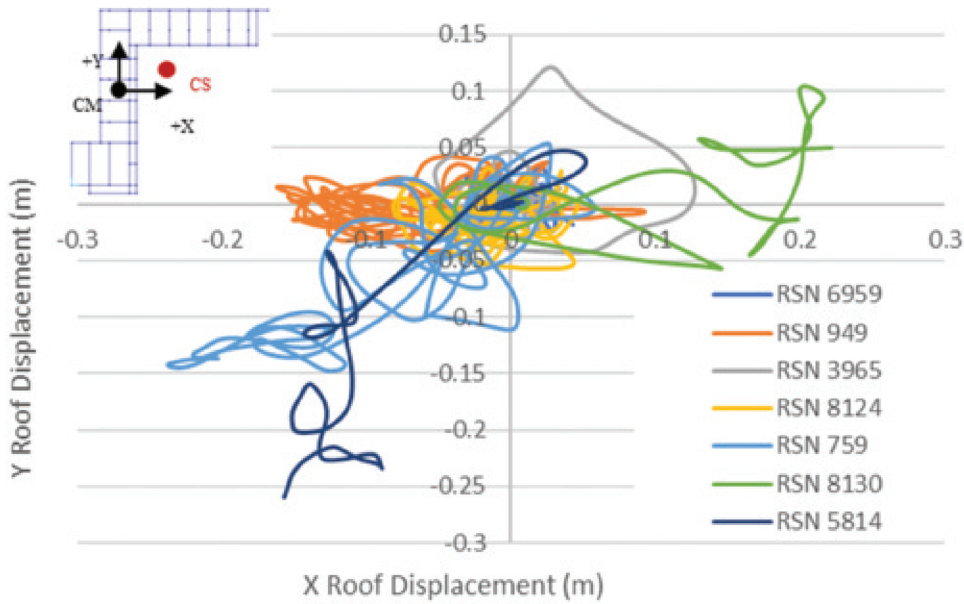

Figure 14. Spatial displacements from all time-history analyses.

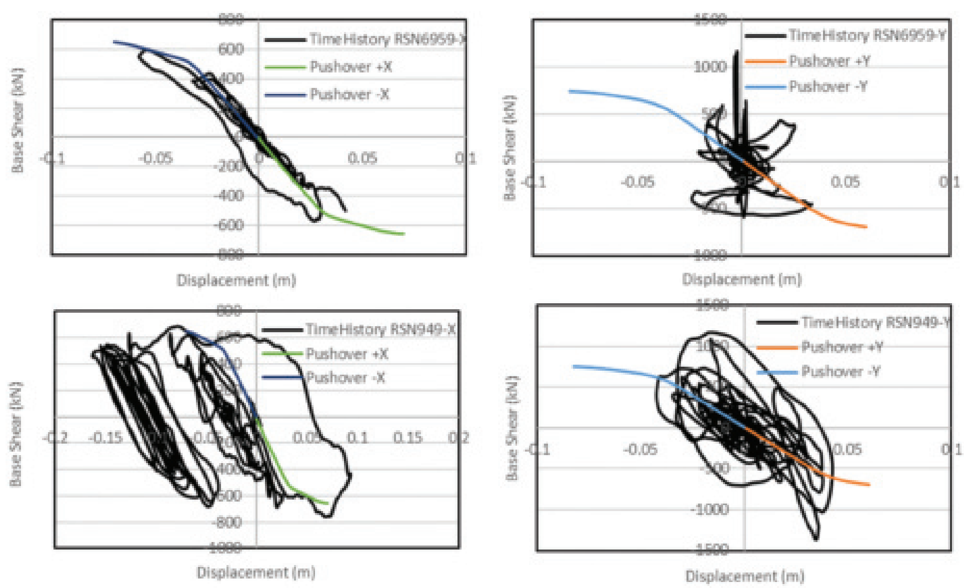

Figure 15. Base shear vs displacement curves for seismic events RSN 6959 and RSN 949 compared to pushover analysis curves.

5. Conclusions

The characteristics employed in the simulation model of an existing historic concrete structure and the type of analysis used for its assessment determine the extent of the retrofit requirements in terms of strength and plasticity increase. The analysis of 3D models, which are more appropriate for irregular structures (§ 4.4.4.1(2) (EN1998–3 2005)), may be performed either with static or dynamic analysis (ASCE 2018; CYS EN 1998–3 2005; KANEPE). The structural assessment of buildings requires a good understanding of the various components of the structure, their

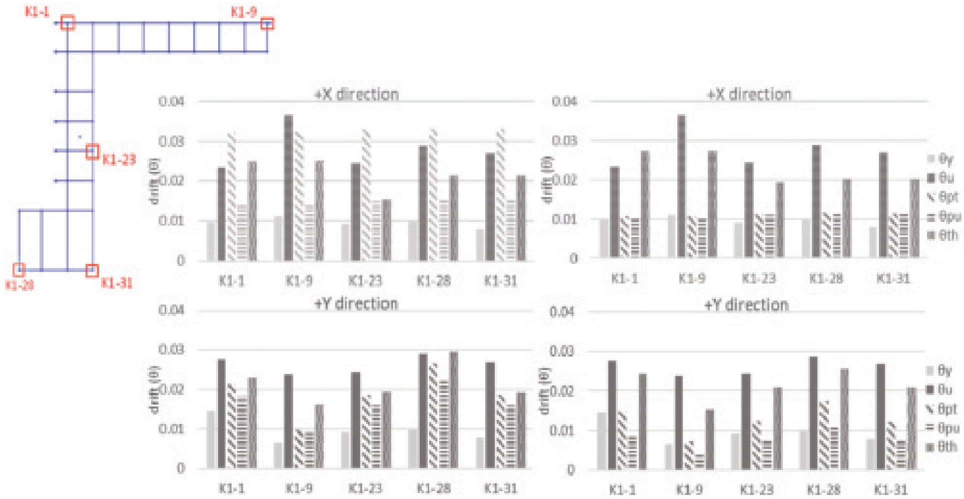

Figure 16. Drift capacity/demand for columns located in different positions at the ground floor, in the X and Y direction (θ_y = drift at yield, θ_u = drift at failure, θ_{pt} = drift demand for triangular pushover, θ_{pu} = drift demand for uniform pushover, θ_{th} = average drift from time-history series).

interconnection and material mechanical properties, and finally of the global behaviour under seismic excitation. At the local components level, the task of assessing the properties of members is becoming even more challenging in the case of historic structures. In such cases, thorough member analysis must be explicitly performed and all possible failure mechanisms must be taken under consideration. While EC8 specifically states that, for the case of historic structures, the assessment and retrofit "often requires different types of provisions and approaches," there are no other guidelines that can be applied and/or are considered legitimate for the practicing engineer to use. Therefore, the assessment procedure suggested in EC8-Part 3 is often the only available option.

This research paper explored a specific case study, a poorly maintained listed reinforced concrete structure having intense irregularities in plan and elevation, as an example for carrying out seismic assessment following the provisions of EC8-Part 3. Non-linear time-history and static pushover analysis were used for this purpose. The results from the assessment procedures show the possibility of brittle shear failure in the first floor columns due to their intrinsic characteristics: sparse stirrups and low concrete strength. Additionally, the assessment stemming from both methods shows that extensive repair is required for the overall structure due to carbonation, corrosion, and other types of damages that have been induced by environmental conditions and the many years of lack of maintenance and abandonment of the building. Hence, seismic strengthening of historic concrete structures is needed in areas prone to earthquakes, such as Cyprus, in order not to let these types of buildings collapse and vanish in a potential future major seismic event.

Inelastic time-history analysis is the most accurate method available for determining seismic requirements; however, its use is not widespread among practicing engineers, as it requires specialized knowledge and skills, and long computational times. The irregularity in plan and elevation of the structure under study led to great differences in the drift demand obtained from the two assessment procedures hereby used. This demonstrates that the non-linear static analysis (i.e. static pushover) described as a possible method of assessment in EC8-Part 3 is totally unsuitable for such types of buildings because it leads to the under-estimation of the demand and the required retrofit, especially in the extreme columns of the structure. Therefore, the inelastic static analysis based on the pushover curve of a structure must be adapted accordingly to different load distributions than those proposed by the existing code (EN 1992-1-1 2004), so that the evaluation of an asymmetric reinforced concrete

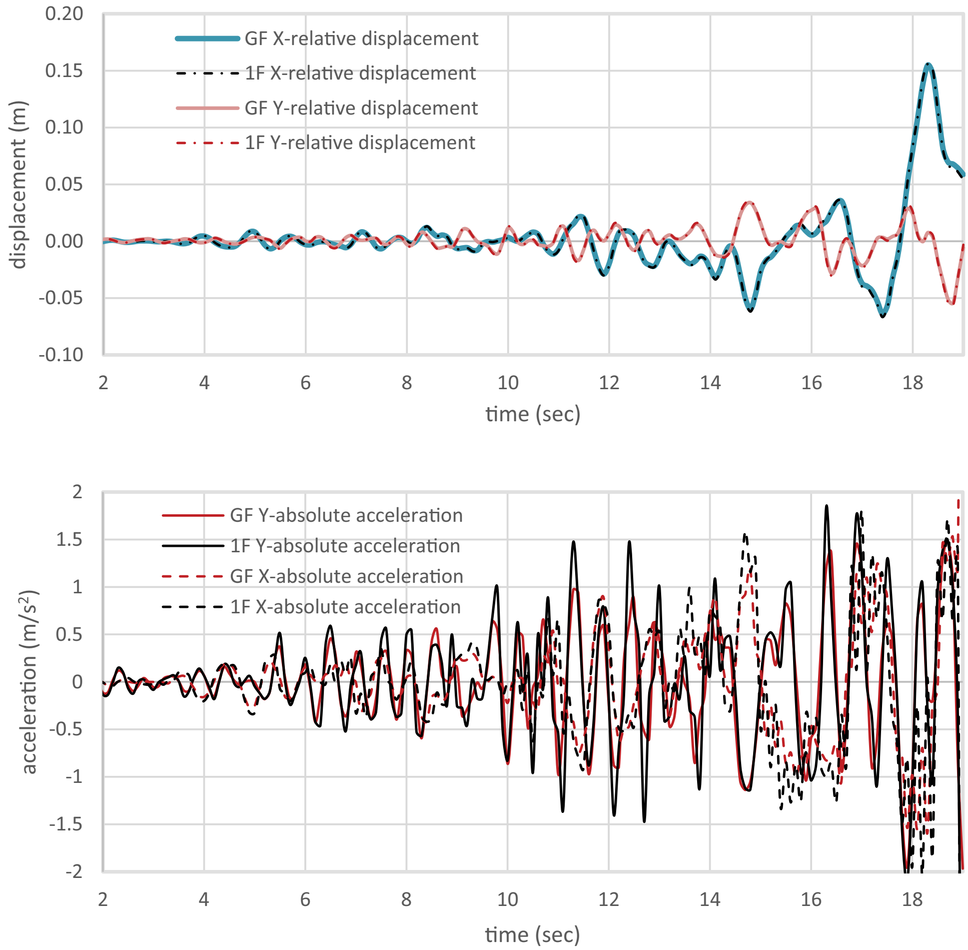

Figure 17. Absolute acceleration and relative displacement time-histories of Ground Floor (GF) and first floor (1F) in X and Y-direction (seismic event RSN 6959).

Table 8. Absolute acceleration (m/s^2) of first floor CM for all time-histories.

Eq. event	"+X"	"−X"	"+Y"	"−Y"
RSN6959	1.780	1.263	1.487	1.924
RSN949	2.058	1.948	2.766	2.867
RSN3965	1.942	1.706	3.074	2.216
RSN8124	2.234	1.924	2.526	2.435
RSN759	1.863	1.798	2.751	3.307
RSN8130	2.746	2.354	2.574	3.036
RSN5814	2.189	1.403	2.734	3.259
Ave	**2.116**	**1.771**	**2.616**	**2.711**
Max	**2.746**	**2.354**	**3.074**	**3.307**
Min	**1.780**	**1.263**	**1.890**	**1.857**

building produces more realistic results, corresponding to those of the dynamic inelastic analysis. The results indicate that the soft floor developed in the model is shortly presented after the yielding of the GF columns. In this case, a multimode analysis taking into consideration higher modes is not the solution for structures with brittle failures and the formation of soft floor mechanism.

Disclosure statement

No potential conflict of interest was reported by the author(s).

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