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Seismic assessment of historic concrete structures: The case of Pedieos Post Office in Nicosia, Cyprus

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ABSTRACT

Monuments are buildings in which historical, archaeological and cultural values are displayed, yet their lack of seismic detailing and design renders them dangerous for public use and prone to collapse and permanent loss, especially in countries located in seismic zones. Furthermore, the existing European regulatory framework does not oblige the owners of such structures to assess and retrofit them against earthquakes, thus leading to dangerous user safety conditions and possible permanent loss of historic concrete structures in a seismic scenario. This paper examines one of the culturally significant historic concrete structures of Cyprus. It assesses the seismic capacity of the structure by means of comparing various analysis methods and highlights the urgent need for a new relevant regulatory framework regarding historic concrete structures. The results are used to formulate recommendations that should be incorporated in future regulations in order to protect both the users and the concrete heritage structures themselves.

1. Introduction

The architectural heritage of a country is usually linked to vernacular buildings and large monumental structures. Historic reinforced concrete structures built in the 20th century are rarely listed as monuments, despite also reflecting the past and usually displaying a multitude of values. This paper deals with the complex issue of historic concrete structures and, in particular, the issue of their seismic assessment. The basic aspects that are hereby explored are: (a) the lack of recognition of the importance of historic concrete structures, their abandonment or poor retrofit practices, which usually result to the loss of their historic features, (b) the lack of proper legislative framework for the assessment and retrofit of such structures, and (c) the insufficiency of the elastic analysis proposed by the Eurocodes for the assessment of existing structures to capture the true failure mechanisms under seismic loading.

1.1. Vulnerability of historic concrete structures

Seismic events are responsible for the extensive damaging, abandonment, collapse and permanent loss of historic reinforced concrete buildings that constitute part of a country's cultural heritage (Walsh et al., 2015). Earthquakes that have occurred worldwide have explicitly revealed the disadvantages and fallacies of previous design practices (i.e., poor detailing with low strength and ductility materials (usually without proper shear/confinement), discontinuous load paths, lack of proper lap splicing, weak column-beam joints, soft stories), that are usually the main causes of failure in old substandard structures (fib Bulletin No. 24, 2003). One of the most important parameters that might had affected proper structural design in the past was the adoption of elastic methods of analysis, that did not take into consideration the formation of plastic hinges mechanism, or the brittle failures that could occur due to shear (Foti, 2015), and the consequent redistribution of forces in other members of the structure.

In recognition of the great possibilities of damage to existing substandard buildings, relevant standards for their structural assessment and upgrading have been developed worldwide, namely: Eurocode 8 - Part 3: *Strengthening and Repair of Buildings* (EN1998-3, 2005), ASCE Standard on the *Seismic evaluation of existing buildings* (2001) (ASCE/SEI 41-17, 2017), Japan Guidelines for the *Assessment of Existing Concrete Structures* (Japan Concrete Institute, 2014), *fib* Commission 7 - Task Group 7.1 *Seismic Assessment and Retrofit of Existing Structures* (fib Bulletin No. 24, 2003), the New Zealand *Seismic Assessment of Existing Buildings Guidelines* (NZSEE, 2017), 1997 NHERP *Guidelines for rehabilitation of existing buildings* (FEMA, NEHRP, 1997; F.E.M.A. P154, 2015; ASCE/SEI 31-03, 2004).

Eurocode 8 (EC8) - Part 3 (EN1998-3, 2005), in particular, describes

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the methodology that should be adopted for the verification of the response of a structure under specific ground motion. Both the correct simulation of the geometry and of the materials, as well as the correct characterization of the possible seismic ground motion, are important for the outcome of the overall structural assessment (Roca et al., 2019; Dmochowski et al., 2021). The failure mechanisms are estimated based on the forces or inelastic deformations, at global level, inter-storey level and component level. Nevertheless, as stated in clause 1.1 (4) of EC8 - Part 3 (EN1998-3, 2005), the seismic assessment and retrofitting of historic buildings can differentiate from the code provisions.

Even though historic buildings are often considered afforded waivers or special case-study considerations, this may not be appropriate in the case where public safety is of utmost importance and may thus be prioritized over and above the objectives of historic preservation; therefore, the best-fit solution should be somewhere between legislation for historic construction and seismic codes with exceptions (ASCE/SEI 41-17, 2017).

In 2019, the CONSECH20 JPI-CH project ("CONSErvation of 20th century concrete Cultural Heritage in urban changing environments") initiated, with the aspiration of highlighting the urgent need for the protection of historic concrete structures, outlining existing conservation policies, investigating structural assessment practices and experimentally exploring novel restoration proposals. One of the reports prepared by the CONSECH20 team focused on the State of the Art of existing European regulations regarding the assessment and retrofit against seismic loading of historic concrete structures. The main code of interest was found to be Eurocode 8 - Part 3 (EN1998-3, 2005). Yet, in almost all cases of listed historic concrete buildings, the level of safety is up to the owner to decide, although in some countries (e.g., Italy) when interventions are not undertaken, limitation of use is applied. This practice usually results in vague provisions that do not bridge the need of protection of the monument in a future seismic event and the need of preservation of the materials, systems and architectural design, as parts of a country's cultural heritage.

As per EC8 - Part 3 (EN1998-3, 2005), the identification and elimination of major structural defects is a very important part of the entire retrofitting procedure. All local errors should be appropriately remedied and local ductility must be increased (if needed). Yet, older buildings (especially historic structures) may include materials or systems that may not be removed or altered, as per the ICOMOS Charter (ICOMOS, 2003). In the case of such buildings, some evaluation and retrofit techniques may thus not be acceptable, i.e., (a) condition assessment or material testing that would disturb historic elements, (b) potential architectural damage that might otherwise be found acceptable for normal structures, (c) retrofit measures that involve the removal of architectural components to gain access to the structure, and (d) retrofit measures that permanently alter the external appearance or configuration of the building (ASCE/SEI 41-17, 2017).

Whilst when dealing with ordinary structures, the measures to be adopted may lead to extensive intervention requirements, with increased cost, for the case of historic listed buildings, the decision of no intervention at all, or indeed demolition of the structure, or even extensive transformation of the load-bearing system is not an option. In this case, the priorities of selecting the retrofit strategy change and, while in normal circumstances the important criteria would be (with descending significance) cost, available workmanship and materials, duration and disruption, functionality, aesthetics, reversibility (EN1998-3, 2005), for historical structures this list may be reversed. It is also crucial that the retrofit procedure is compatible with the materials of the existing system, and that it works in harmony with them, not creating any further damages and weaknesses (Garmendia et al., 2018; Thermou and Elnashai, 2006).

The different interventions that may be proposed, nevertheless, affect the overall behavior of the structural system in different ways; some methods increase the strength and stiffness, whilst others increase the ductility of the structure. The final decision is made on the basis of the assessment procedure, and more specifically on the relation between the Force-Displacement curve of the structure and the Performance Point (d_{et}), i.e., the level of deformation imposed by the design seismic action for the case of inelastic analysis, or in terms of forces: capacity vs request, when elastic analysis is employed.

1.2. Assessment of historic concrete structures

As per EC8 - Part 3, for the assessment analysis of a structure, two parameters are required: the possible seismic excitation and loading of the structure, that is site-specific and is given in terms of either acceleration or displacement, based on the elastic spectra, and the model of the structure, that can be either linear or non-linear. The possible analysis procedures suggested by the vast majority of assessment codes are: (a) linear static, (b) linear response spectrum (modal), (c) nonlinear static (pushover), and (d) nonlinear dynamic (time-history). Some of these methods take into consideration the maximum modal response and combine modal maximum results irrelevant to the inelastic behavior (Priestley, 2003). Yet the actual damage is related to strains and displacements and not accelerations; therefore, inelastic approaches have been developed (EN1998-3, 2005; NZSEE, 2017; FEMA, NEHRP, 1997; Priestley, 2003; Moehle, 1992). Having an elastic spectral acceleration and displacement over the entire range of the response does not take into consideration load redistribution and damping phenomena that are generated from cracking, yielding or failure of members and energy dissipation (Priestley, 2003).

Research on historic concrete structures has only recently initiated, since such structures were not appreciated as part of a country's cultural heritage in the past. Most of the relevant studies and publications focus on the damages of the material itself, such as re-bar corrosion, concrete cracking, alkali-aggregate reaction, sulphate attack (Brueckner and Lambert, 2013; Crevello et al., 2015; Heinemann, 2008; Marcos et al., 2016), and methods for its repair, such as the Patch Restoration Method (Valença et al., 2012; Heinmann, 1967; Macdonald, 2020). Additionally, codes and recommendations regarding historic concrete structures are also limited to the evaluation of the defects and the deterioration of concrete, and measures for its repair (Reed et al., 2008; Urquhart, 2014; Lightcycler, 2014; Hanna et al., 1991). Some of the few technical specifications for the seismic rehabilitation of historic concrete buildings have been developed in the USA (Aguilar, 2016).

Researchers have studied the special cases of patented reinforced concrete systems, such as the Hennebique system, concluding that short lapping or anchorage lengths may result in failures, even for static loads (Hellebois and Espion, 2013). Other studies on historic concrete structures in seismic prone areas have highlighted the low ductility of historic concrete elements due to the use of low grade concrete and insufficient percentages of reinforcement (Mosoarca and Victor, 2013), or demonstrated the importance of assessing premature shear failures due to the unsafe design of past construction time (Foti, 2015; Miano et al., 2019). Studies on historic concrete buildings in New Zealand and Japan concluded that force-based seismic assessment under-rates such structures (Walsh et al., 2015), with historic concrete buildings up to three storeys being able to sustain high magnitude seismic events. This, though, may not be interpreted as a rule that can apply worldwide, due to the local characteristics of the design practices in different geographical areas.

This paper examines the procedure for the seismic assessment of historic reinforced concrete structures, in general, through the use of a case study building: the Pedieos Post Office in Nicosia, Cyprus. Firstly, the research focuses on the collection and evaluation of information on the construction practices of the era, and of the specific case study, in particular. It also addresses the pathology of the case study building and the damages inflicted due to inherent deficiencies and environmental exposure. The structure is then simulated with the use of SAP2000 (CSI, 2009) and assessed against seismic loading, based on the methods described in EC8 - Part 3 (EN1998-3, 2005): (a) elastic static, (b) elastic response spectrum, and (c) inelastic static analysis. Even though individual element analysis shows ductile performance of the columns, pushover analysis reveals that the structure collapses due to the formation of a storey-sway mechanism. The results from the various types of analysis carried out are compared in terms of forces, drift and ductility demands. The demands obtained play a crucial role regarding the extent of the repair and strengthening of the structure under study in a possible future retrofit scenario. The case study is, therefore, used to derive conclusions and recommendations that could be used by competent local authorities in European countries adopting the Eurocodes, with regards to the assessment and strengthening of historic concrete structures for seismic stability.

2. Historic concrete structures in Cyprus

2.1. State of conservation

The protection of the cultural and architectural heritage of a country is of paramount importance. In Cyprus, the local architectural heritage is protected by a legislative framework implemented by the Department of Urban Planning and Housing. Monumental constructions on the island, nevertheless, are heavily linked to vernacular load-bearing masonry structures (Georgiou et al., 2021). There are currently only a few examples of listed buildings constructed with contemporary materials, such as reinforced concrete, despite the fact that the materiality of a building should not be a problem for its characterization as a monument.

The non-listing of many historic reinforced concrete structures in Cyprus usually leads to their inadequate protection and preservation, and sometimes even to their demolition. Since 1930s, when reinforced concrete was first used in Cyprus, a multitude of buildings with significant historical, cultural, aesthetic, architectural and social values were constructed on the island, most of which are, unfortunately, not currently listed as monuments. Such buildings include both urban dwellings in city centers and industrial or individual buildings in rural areas.

Some examples of large-scale urban buildings, built with reinforced concrete in Cyprus, are the well-known historic Ledra Palace Hotel (1947–1949) (Fig. 1a), the Municipal Market (1965) (Fig. 1b), and the offices of SPEL (1954) (Fig. 1c) in Nicosia. An example of an industrial reinforced concrete building on the island is the carob store in Carnagio, Limassol (1961) (Fig. 1d), while an example of a single building constructed with reinforced concrete is the Pavilion of Hala Sultan Tekke in Larnaca (Fig. 1e), which is part of the larger Hala Sultan Tekke Muslim monument complex. All the above buildings have been listed, as opposed to a large number of smaller individual urban buildings, which nevertheless make up a large portion of the infrastructure of Cyprus and are certainly part of the history of the island. One such building is the Pedieos Post Office in Nicosia, which was built in 1964 (Fig. 1f).

The aforementioned historic concrete structures were designed and built in an era when seismic design provisions were not implemented in the local code provisions and are, thus, currently prone to seismic events (Georgiou et al., 2019), since Cyprus is located in the border between tectonic plates and the seismic activity, especially in the southern part of the island, is high. In fact, in Cyprus, all buildings constructed until

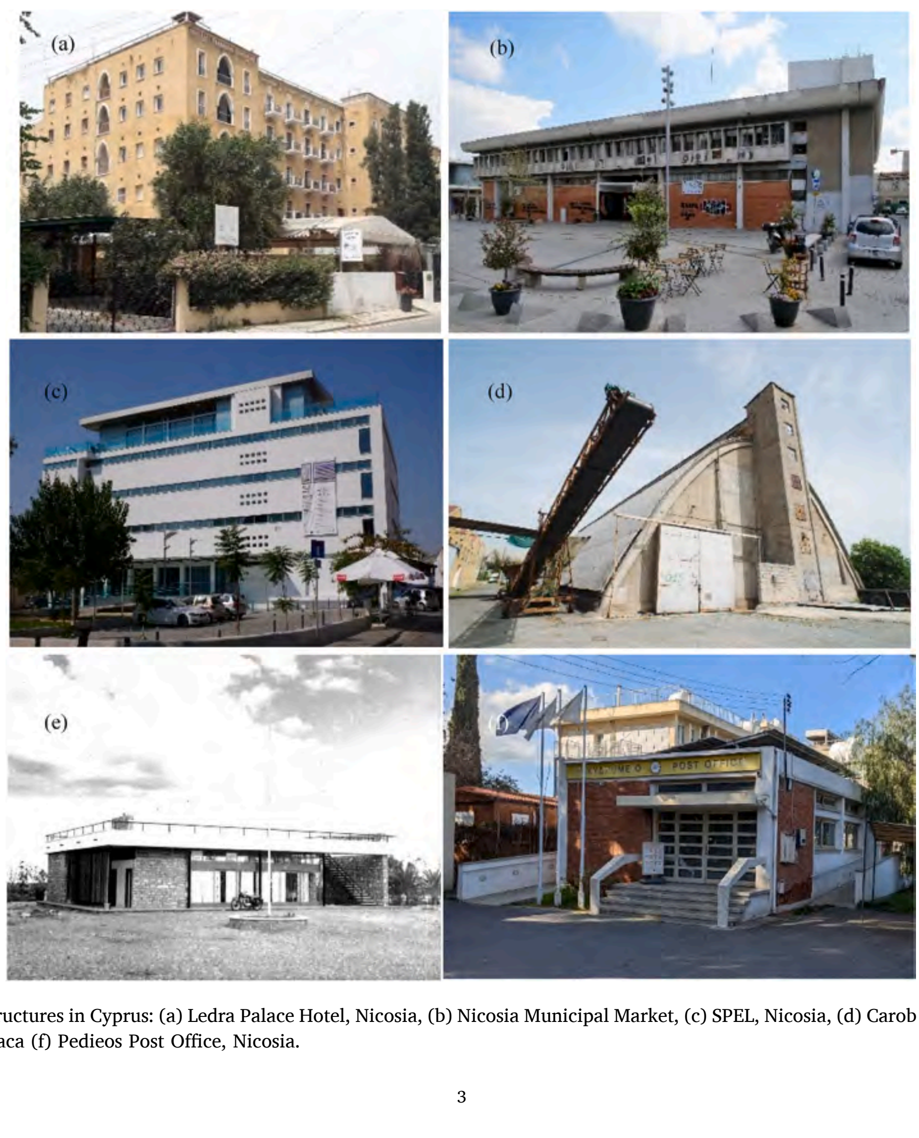

Fig. 1. Historic concrete structures in Cyprus: (a) Ledra Palace Hotel, Nicosia, (b) Nicosia Municipal Market, (c) SPEL, Nicosia, (d) Carob store, Limassol, (e) Pavilion at Hala Sultan Tekke, Larnaca (f) Pedieos Post Office, Nicosia.

1994, when the first Cypriot Seismic Standard became mandatory, were designed without any seismic provision and detailing. The establishment of the first anti-seismic measures on the island was initiated in 1979, following a destructive earthquake with various casualties that took place in Thessaloniki, Greece. The lack of local authorities (the island was under British administration until 1960) and/or universities and research centers (first university department in civil engineering established in 2003), however, were probably some of the factors that delayed so long the application of local regulations concerning the stability and safety of reinforced concrete structures. The first local Seismic Zone Map was issued in 1986, while in 1992 the Cypriot Seismic Code was introduced, initially as an option, before it became mandatory in 1994. In 2012, the Eurocodes (incl. EC8 - Part 3 (EN1998-3, 2005) superseded all other national documents on the island, and are thus now solely used for the construction of new, or the seismic assessment and repair/strengthening of existing, structures on the island.

According to EC8 – Part 3 (EN1998-3, 2005), the desired Performance Level of a structure in different future seismic scenarios is decided mutually by the engineer and the owner of the structure. The number of Limit States to be considered, as well as the return period of seismic actions under which the Limit States should not be exceeded, are defined as Nationally Determined Parameters. As a result, in Cyprus, since 2012 when EC8 was implemented, the level of assessment of an existing structure is decided by the owner of each building, leading in most cases of retrofit to the implementation of simple aesthetic repairs.

2.2. Case study: the Pedieos Post Office

The Pedieos Post Office was one of the first post office buildings constructed on the island. According to archival research carried out by the authors, it was the first project among a number of development projects announced by the Department of Transportation and Works on October 5, 1959, in the framework of a five-year development program with projects throughout Cyprus. The cost for the construction of the building amounted to £3000 (Cyprus pounds) and its construction, according to the State Archive, was completed in 1964 (Postal Department, 1959; Ministry of Communications and Works, 1964). Immediately after its completion, the construction of other important postal buildings followed.

The building is a single-storey structure that covers an area of 140 m^2 and is accessible only from the south side of the plot. It is located next to a number of other public buildings and facilities, such as the Municipal swimming pool, the Departments of Forestry and Agriculture, and the Ministry of Agriculture, Rural Development and the Environment, and it is built at a distance of about 300 m from the Pedieos river. The wider area is characterized by a multitude of important public buildings, such as the House of Representatives, the Nicosia District Court, the Cyprus Museum, and the Department of Urban Planning and Housing.

Unfortunately, no original construction plans were found in the State Archives; this may be attributed to the transitional stage of state services from the British Administration to the Republic of Cyprus at the time of the construction of the building. Plans dated from 1999 were, nevertheless, acquired from the Department of Public Works (Fig. 2). These relate to interventions made to the building under study for renovation purposes at the time. The file concerning the actions for the renovation of the Pedieos Post Office is still not in the State Archive and cannot be inspected by the public, at least not before thirty years after its closure.

The Pedieos Post Office is currently operating normally, without interruption, since its inauguration, and it is thus one of the oldest branches of the Cyprus Post Offices. The building, though, requires regular maintenance, despite being completely renovated in 1999, with significant changes to its internal division of spaces, but without

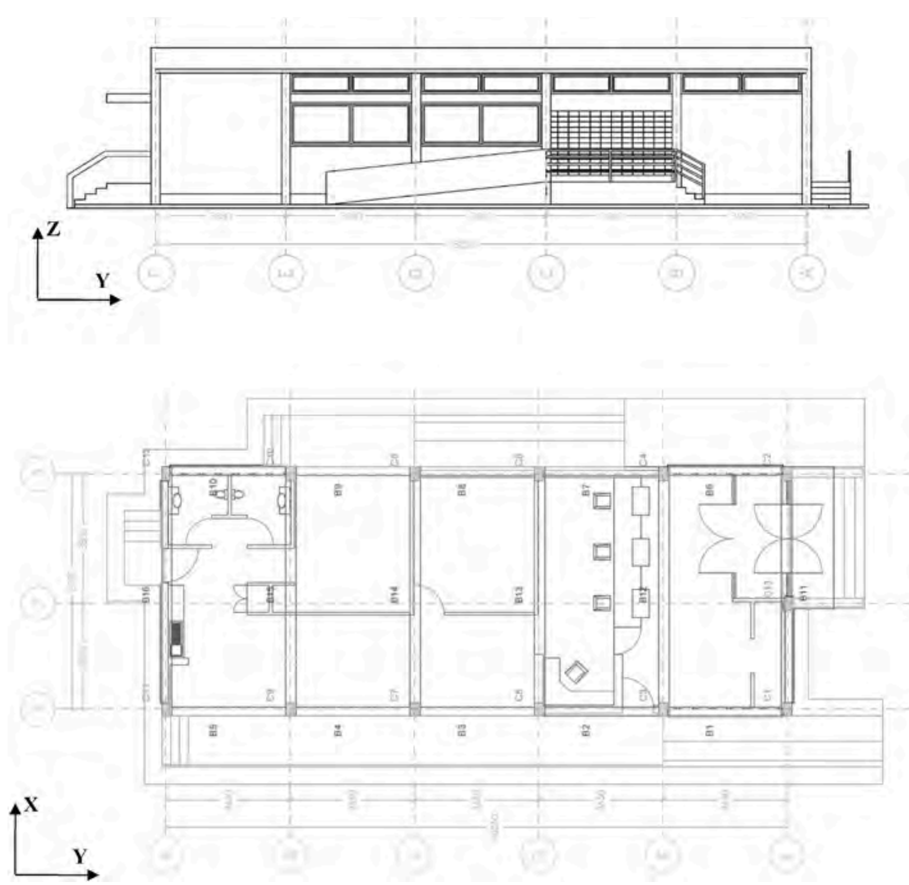

Fig. 2. Side (top) and Plan (bottom) view of the Pedieos Post Office.

alteration of its structural system or its architectural character. The latter is dominated by the presence of fired red clay solid bricks on its facades, a material which was widely used in Cyprus during the period of British Administration.

3. On site investigation

Evaluation of any existing building may follow three levels of escalating degree of complexity, as specified in (ASCE/SEI 41-17, 2017), (JBDPA, 2005): (1) screening, (2) assessment based on identified deficiencies, and (3) systematic evaluation. In historic structures, it is additionally important to identify the location of historically significant features and fabric, so that care is taken in the design and investigation process to minimize the effect of interventions on these features. Most of the National Authorities issue Rapid Assessment Documents that can identify possible global defects of the structure and possible collapse during future seismic events (NZSEE, 2017; F.E.M.A. P154, 2015; JBDPA, 2005; NRC, 1993). Some of the critical structural weaknesses that are usually observed in old type constructions are: (a) irregularities in plan (T-, L-, U- or E-shaped plan, etc.), (b) irregularities in elevation, (c) short columns, (d) site, soil and foundation (potential for landslide, liquefaction etc.) issues and pounding with adjacent buildings. Additional factors that pertain to the long-term effects of ageing and corrosion induced damage need be also evaluated, as they may prove to be the controlling parameters of the structural condition.

Of crucial importance for the building configuration is the as-built information that should identify the load-resisting components (both structural and non-structural) that participate in resisting the seismic loads. This information assists the engineer in identifying potential seismic deficiencies in load-bearing components, such as load path discontinuities, weak links, irregularities, inadequate strength and deformation capacities. Especially in the case of Eurocode 8 (EN1998-3, 2005) the "knowledge level" defines both the confidence factors that are used in the material parameters and also the possible methods for the analysis of the structure. Variations in actual performance from the assessment results is associated with unknown parameters, such as geometry and member sizes, deterioration of materials, incomplete site data, variation of ground motion, incomplete knowledge and simplifications related to modelling and analysis.

3.1. Geometrical verification

The detailed plans from the restoration works that took place in 1999 show the changes made to the building, largely reflecting its current state. The plans concern architectural, but not structural details. The verification of the geometry of the structural and architectural elements was performed through a detailed survey of the building, carried out during four on-site visits that took place between May–Feb 2019.

The building is of rectangular shape (Fig. 2), and comprises reinforced concrete frames with beams and columns, with total plan dimensions measuring 18.25 × 6.90 m, and a net height of 3.75 m. It consists of a vestibule with a parcel counter and a small letter collection room, a corridor leading to an office, a storage room and ancillary areas (kitchen and toilets). At the entrance, there is a small cantilever providing shading, whilst on the upper part of the perimeter of the building there are skylights (Fig. 2). The post office is partially elevated, so there are stairs to the entrance. To the left and right of the latter, there are two ramps leading to the private mailboxes. Besides the main entrance, there are two more doors, one on the west and one on the north side of the building.

The structural system shown in Fig. 2 consists of thirteen columns with dimensions 400 × 250 mm, and two types of beams with dimensions 600 × 250 mm and 450 × 300 mm, in the Y and X directions, respectively. From the on-site inspection, a slab thickness of 15 cm was measured, both for the cantilever and for the slabs. There are also twelve metal drainpipes passing through the core of the columns.

3.2. Pathology

Historic concrete structures also suffer from the ever changing environmental conditions in their long life span, adding up to their pre-existing poor construction practices; these may induce corrosion of the reinforcement, carbonation, rising humidity, lack of maintenance and abandonment (Redondo et al., 2021; Marcos et al., 2021). An integral part of the evaluation of the building under study was the recording and analysis of its pathology. During the on-site inspections, it was observed that the building presents several damages, some of which concern the structural system. Whilst internally it does not indicate the existence of serious damage or load-bearing insufficiencies, externally damage is evident, with the presence of moisture, cover delamination, cracks, and material disintegration clearly noted.

Rising damp can be observed around the lower part of the masonry infills of the building, resulting to significant loss of the masonry material (fired red clay solid brick) and detachment of plaster (Fig. 3a–d). The loss of rebar cover in the reinforced concrete elements is also evident. It is worth noting that the on-site concrete mixing practice in small batches, the lack of compaction equipment, and the low strength of the cement of the time (Georgiou et al., 2021), together with the honeycombing and concentration of aggregates observed at the base of the columns, suggest the existence of low quality concrete. In some columns, there are cracks parallel to the longitudinal axis of the elements, indicating corrosion of the longitudinal reinforcement (Fig. 3e–f). Small cracks are also observed in various other parts of the structure, such as beams (Fig. 3g) and columns. It is important to mention that, during the site visits, it was observed that rainwater flows through metal drainpipes embedded in the core of the corner concrete columns. Therefore, through potential leaks from these pipes, moisture is allowed to enter the load-bearing elements, with detrimental effects.

3.3. Non-destructive testing

In order to collect data regarding the detailing of the structural members and the material properties, non-destructive tests were performed by the use of Schmidt rebound hammer and rebar scanner (Fig. 4).

3.3.1. Rebound hammer test

The Rebound Hammer method is the most common non-destructive method for estimating the strength of concrete. A gauge measures the bounce of the mass ejected by a spring on the concrete surface (R value). The test was performed as per EN 12504–2 (EN 12504-2, 2013) with 9 readings at each location. The locations selected were those where concrete was visible. The resulting average value was correlated to the surface strength of the concrete element through charts provided by the manufacturer of the equipment, that are dependent on the direction of testing. A total of thirteen concrete members were measured (Fig. 4a). The readings between the various concrete members examined showed a significant scatter, with R-values ranging from 23 to 39. An average compressive strength of 22.4 MPa was recorded, which is deemed quite high for the concrete mixes of the era. While EN 13791 (EN 13791, 2019) as well as EC8 - Part3 (EN1998-3, 2005) also suggest the use of direct measurements of the compressive strength of concrete with core extraction, this was not allowed by the owner of the property. This is also the case in most cultural heritage monuments. The unavailability of direct determination of concrete strength leads to the use of Low Knowledge Level and a high safety factor, in the order of 1.4. Therefore, the compressive strength of concrete that was used for capacity calculations of the members was 16 MPa.

3.3.2. Cover meter and rebar detector

Reinforcement scanning is a non-destructive method for determining the position and size of structural steel, without the need to remove concrete cover. A HILTI rebar detector was used on the building under

Fig. 3. Building pathology: (a) rising damp and salt crystallization damage to the fired red clay solid brick facade, (b) plaster detachment on painted infill walls, (c and d) plaster detachment on columns, (e and f) longitudinal cracks due to corrosion of the reinforcement, (g) vertical crack at the center of the entrance beam. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

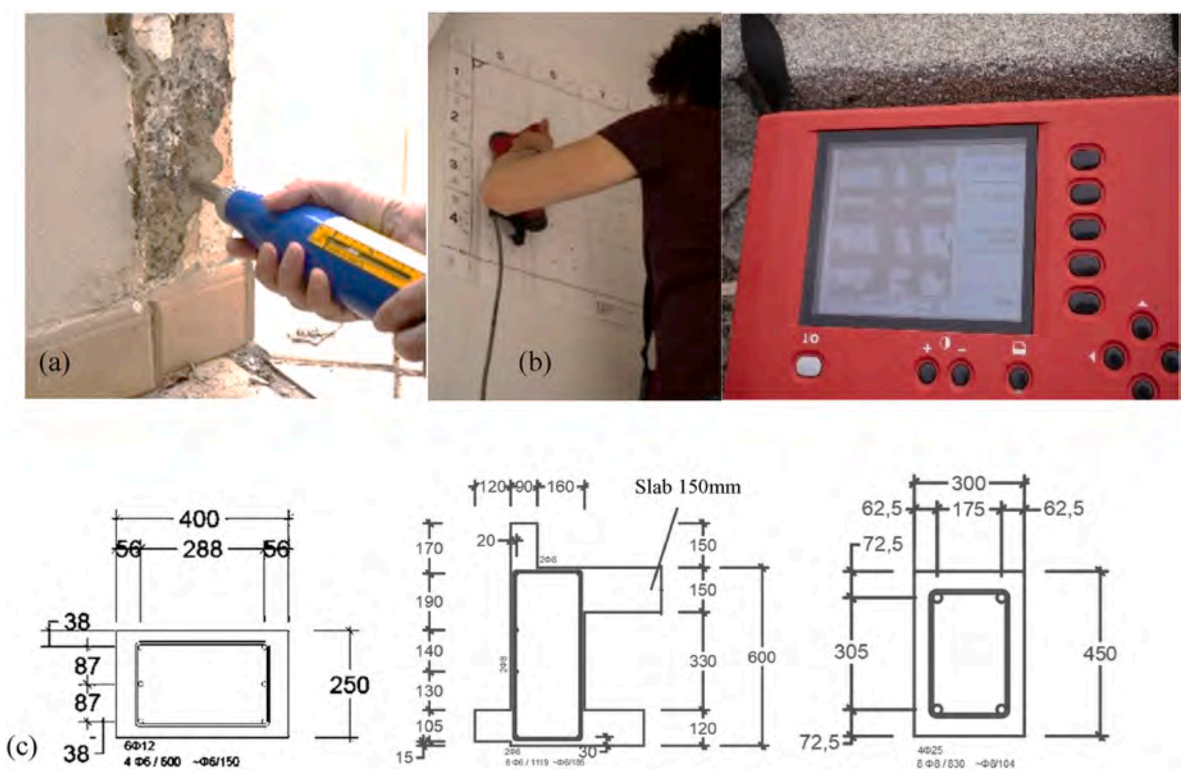

Fig. 4. (a) Schmidt rebound hammer test on column, (b) rebar detector, (c) column and beams detailing.

study to determine the position, depth and size of the reinforcement. For the correct scan of the reinforcement, a grid printed on large surfaces of paper was used; this was placed on the surface of the member to determine the path to be followed by the scanner. Scans were carried out in both horizontal and vertical paths (Fig. 4b) at various representative points of the building, such as end-columns, beam ends in X and Y direction, interior beams and roof slab. Fig. 4c shows the detailing of the columns and beams, as measured. The results show that the building is detailed with small sections of reinforcement, as expected due to the design practices of the era, which took into account only vertical loads.

4. Seismic assessment

4.1. Level of knowledge

When collecting the data to be used in the analysis, EC8 - Part 3 (EN1998-3, 2005) requires assessment of their reliability, based on three sets of information: geometry, structural details and material properties. This aims at finding the safety factor for the types of analyses to be used. The reliability of the data collected in the case of the Pedieos Post Office was assessed as Low (KL 1), mainly because of the absence of destructive test results. Since the level of complexity of the analysis chosen must be compatible with the reliability of the data collected to accurately capture the simulation, the only permitted analysis for the level of knowledge hereby achieved was linear analysis, either static or spectral. Hence, the two respective elastic analyses allowed by EN 1998–3 (EN1998-3, 2005) were performed: elastic static analysis and elastic spectral analysis; inelastic static analysis was additionally employed merely for comparison purposes.

4.2. Performance level and assessment spectra

As per the Cypriot Annex of EN 1998–3 (CYS EN1998-3, 1998), clause 2.1, (2)P and (3)P:

(2) P *Buildings of importance class IV (as defined in* Table 4.3 *of CYS EN 1998-1:2004) should be checked for all three Limit States defined in 2.1(1)P of CYS EN 1998-3:2005. For the other importance classes, the number of limit states to be checked shall be agreed between the owner/owners and the designer.*

(3) P *The return periods specified for the various Limit States shall be agreed between the owner/owners and the designer.*

The project engineer, together with the owner, thus decide on the Performance Level of the structure under study, which should correspond to a design earthquake. This national annex does not set minimum values for the assessment of existing structures; therefore, the project owner may choose, for example, the level of maximum damage in a very frequent earthquake (i.e., 50% in 50 years). In contrast, the Greek national code for structural interventions, KAN.EPE. (KANEPE, 2017), requires minimum safety criteria, and sets limits based on the importance of the structure under study.

The Pedieos Post Office may be classified as a structure of Importance Class II, which corresponds to objective (C1) of KAN.EPE (KANEPE, 2017) (Annex 2.1). Objective C1 refers to a collapse prevention performance level for the design earthquake, with a probability of exceedance of 10% in 50 years. The coefficient of significance γ_I is equal to 1. The soil is classified as Type C (Table 3.1, (EN1998-1, 2004)), with $S = 1.15$, $T_B = 0.2$s, $T_C = 0.6$ s and $T_D = 2$ s. The Peak Ground Acceleration is determined as 0.2g (Map of Seismic Zones of Cyprus (CYS EN1998:1, 2007)).

4.3. Simulation of the model in SAP2000 and types of analysis performed

The structure was modeled in SAP2000 (CSI, 2009), with linear frame elements for the beams and columns and shell elements for the slabs (Fig. 5a). The compressive strength of the materials was set as 16 MPa, based on the low knowledge level obtained. The frames' stiffness was reduced to half the value of the of the uncracked frames, in order to take into consideration stiffness at yielding during the seismic event (EN1998-3, 2005). The columns were connected rigidly to the ground and the nodes of the slab were assigned with diaphragmatic action. The dead and live loads were added to the self-weight of the slab (2 kN/m^2 + 0.35 kN/m^2 and 3 kN/m^2, respectively (CYS EN1991:1, 2007)). The vertical loads, combined with the seismic action, were G+0,3Q. Mode shapes are depicted in Fig. 5b and corresponding periods in Table 1. The first three modes of the structure, with periods $T_1 = 0.76$ s, $T_2 = 0.52$ s and $T_3 = 0.43$ s, are much larger than those that would have been obtained by current practices, due to the very slim vertical members that were designed in previous eras.

4.3.1. Elastic static analysis

In the elastic static analysis, the members of the structure are considered to behave elastically at all times, while the center of mass node on the slab is loaded in the two directions of motion with the

Table 1
Modal Analysis results.

	Mode	Period (sec)
MODAL-Primary X	1	0.759793
MODAL-Primary Y	2	0.519718
MODAL-Torsion	3	0.433454

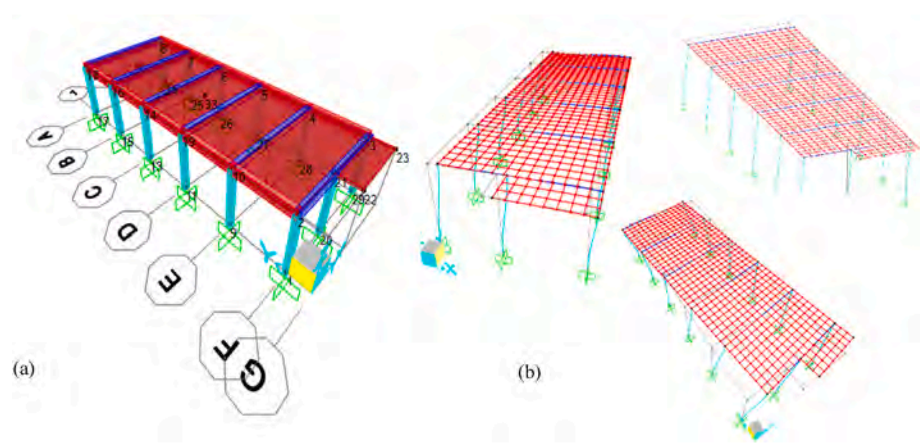

Fig. 5. (a) Model of the structure, (b) 1st, 2nd and 3rd mode shapes.

seismic shear force corresponding to the fundamental period in each direction and the mass of the structure. The mass for the G+0.3Q combination, found to be 208 tn, was used, corresponding to a base shear applied for the static elastic analysis in the two directions as per Eqs. (1) and (2). The deformations of the structure for the application of the base shears are depicted in Fig. 6.

$$F_{d,x} = 5.64 \text{ m/s}^2 \bullet 208 \text{ tn} = 1173 \text{ kN} \tag{1}$$

$$F_{d,y} = 4.45 \text{ m/s}^2 \bullet 208 \text{ tn} = 925.6 \text{ kN} \tag{2}$$

4.3.2. Elastic modal analysis

In the case of the elastic modal analysis, the loads are applied on the structure based on the mode shapes and the response spectrum incorporated in the analysis program. The results from each mode are then combined by the SRSS method. In this case, the results indicate much lower deformations and resulting forces in the members of the structure, compared to the elastic static analysis, in both the X and Y directions, as shown in Fig. 7.

4.3.3. Non-linear static analysis (pushover)

For the non-linear static analysis (pushover), plastic hinges were added at the edges of the columns, based on the moment-curvature properties of the cross-sections determined by the use of RESPONSE2000 (Bentz, 2000). The detailing, materials and axial load were imported in the program, and M_y, M_u, φ_y, φ_u were obtained for each column (Fig. 8 left). The moment-curvature was translated into shear-chord rotation diagrams, while the shear capacity of the members based on the stirrups was compared to the yielding shear (Fig. 8 right). All the members were found to fail with a ductile manner by yielding of the flexural reinforcement. Yet, the failure mechanism of the columns initiated prior to the yielding of the beams, contrary to the well-established capacity design.

The non-linear static analysis used a constantly increasing force applied on the node corresponding to the center of mass, in the X and Y directions. The displacement of the monitoring node (center of mass) was monitored for each step, along with the base shear corresponding to the sum of the shear forces of all vertical members of the ground floor. The base shear vs control node (center of mass) displacement diagram comprises the pushover curve of the structure. The pushover curves in the two directions are shown in Fig. 9. These were then bi-linearized, according to EC8 - Part 1 (Annex B) (EN1998-1, 2004), as shown in Fig. 8 (right) for the X-direction. The pushover curve gives the maximum base shear that can be obtained by the structure, the displacement capacity, as well as the demand according to the elastic design spectrum.

5. Results and Discussion

The shear capacity of all members for yielding of the flexural reinforcement (V_y), failure in flexure (V_u) and shear failure (V_r) was compared to the shear request, based on the three types of analysis performed: elastic static (V_{ES}), response spectrum (V_{RS}) and inelastic static ($V_{pushover}$) (Fig. 10 top). In terms of forces, both elastic analyses resulted in failure of the columns of the structure and increased shear demand (3.5 times the capacity of the members). By adopting the aforementioned assessment methods, any potential retrofit scenarios will result in excessive demand for stiffness and strength. The lowest forces are obtained by the non-linear pushover method, as this takes into consideration the actual yielding of the cross-section and the redistribution of load to the rest of the members.

The opposite applies for the case of the chord rotations, as shown in Fig. 10 bottom. The pushover analysis shows that most of the columns will be required to reach their Life Safety (θ_{sd}) threshold in the design seismic scenario.

Other than the correct redistribution of loads by this inelastic analysis, compared to the two elastic ones, the pushover analysis (Fig. 11 left) also sheds light to the prevailing type of failure that seems to be the formation of a soft storey mechanism (Fig. 11 right), which leads to collapse of the structure, prior to the members reaching their full deformation capacity. Even though the ductility demand in this case, $\mu_{dem} = 2.33$, is much closer to the ductility capacity, $\mu_{cap} = 1.66$, the proposed retrofit scheme would still have to increase the ductility of the structure by 1.5 times in order to prevent collapse.

6. Conclusions

The preservation of the cultural and architectural heritage of a country is among the most important guardians of its history. The appraisal, maintenance and restoration of historic buildings, regardless of materiality, must be a matter of paramount importance. In the context of the protection of historic reinforced concrete monuments, recording becomes important. Cyprus, unfortunately, is still reluctant to declare such monuments as listed buildings. The research hereby presented aims at contributing towards this direction and at enhancing knowledge regarding the seismic assessment and retrofit of historic concrete structures.

This study performed a detailed recording of the historic value, architectural type, structural system and materiality of a case study: the Pedieos Post Office, a modernist building located in Nicosia, Cyprus. Furthermore, it assessed its seismic capacity through various types of methods. The results indicate that elastic analyses require excessive retrofit interventions, whilst inelastic methods are better suited to historic reinforced concrete structures of architectural significance.

The case study results may be used to derive some basic recommendations that should be adopted by the regulatory bodies in Cyprus and in the other countries of the European Union, with regards to the safeguarding and preservation of historic concrete structures against earthquakes. These are listed below:

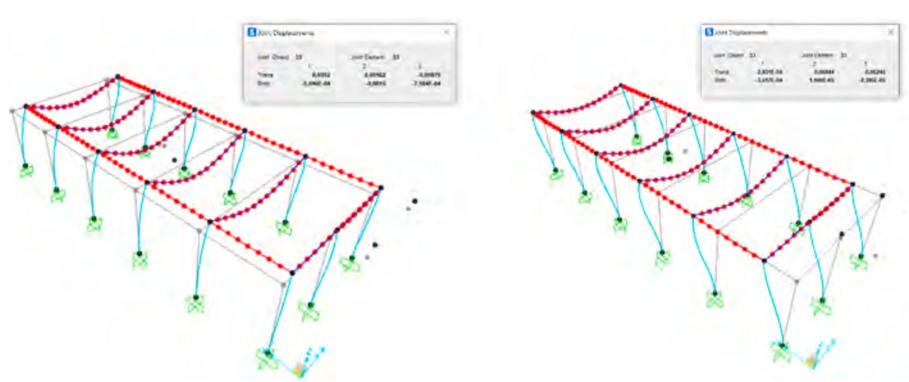

Fig. 6. Deformations of elastic static analysis in the Ex (left) and Ey (right) application of loads.

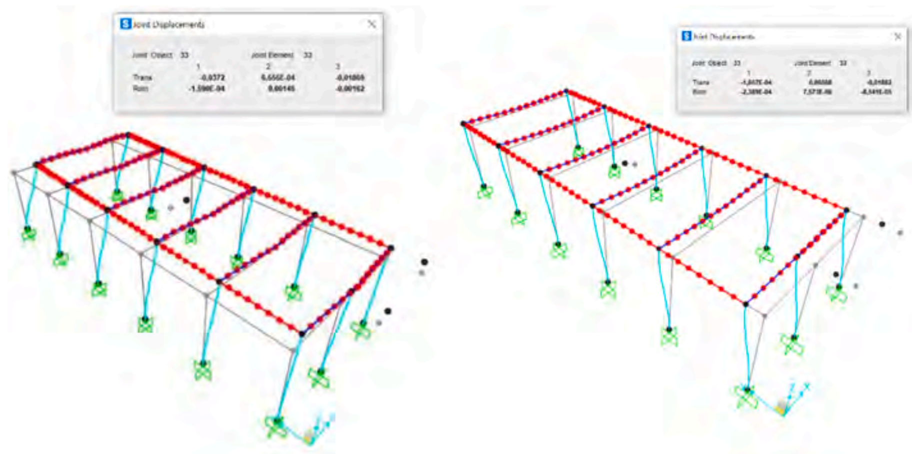

Fig. 7. Deformations of elastic modal analysis in the Ex (left) and Ey (right) application of loads.

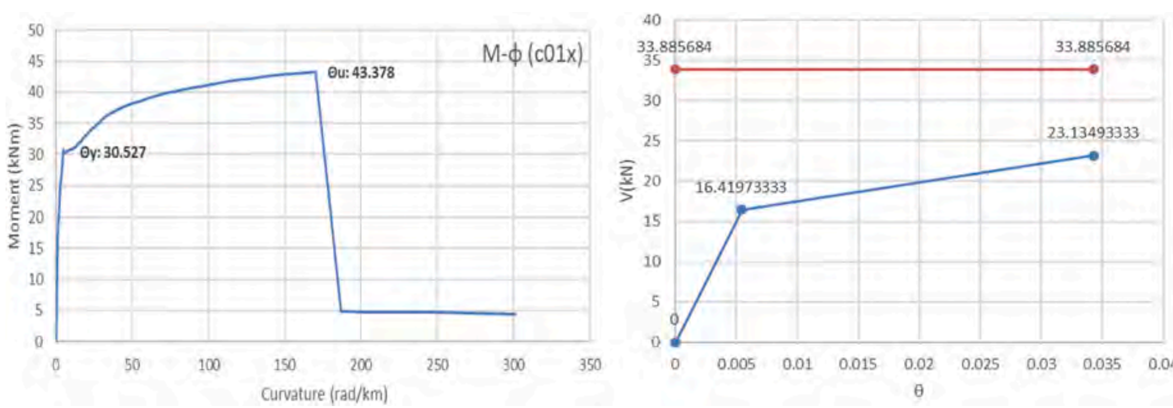

Fig. 8. Moment-curvature (left) and Shear-drift (right) for column K1 in the X-direction.

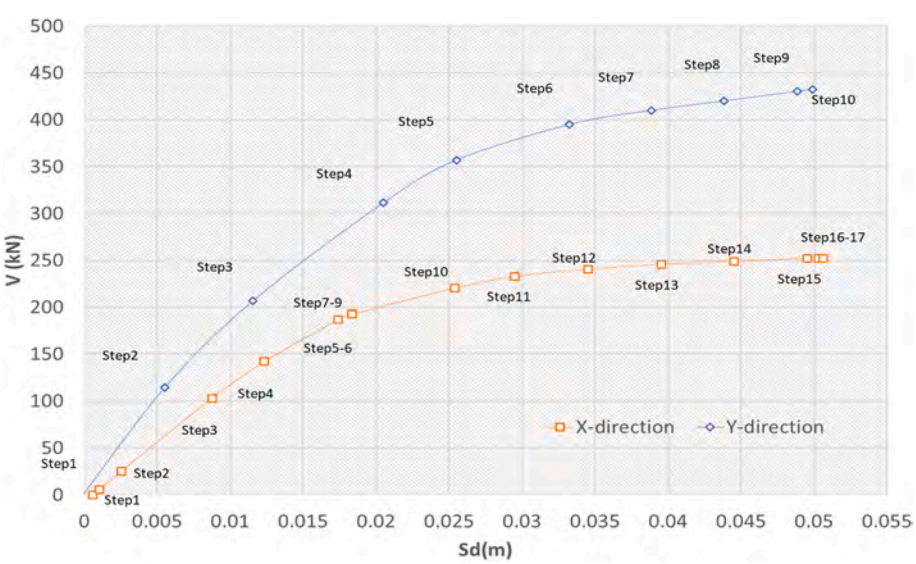

Fig. 9. Pushover curves in the X and Y direction.

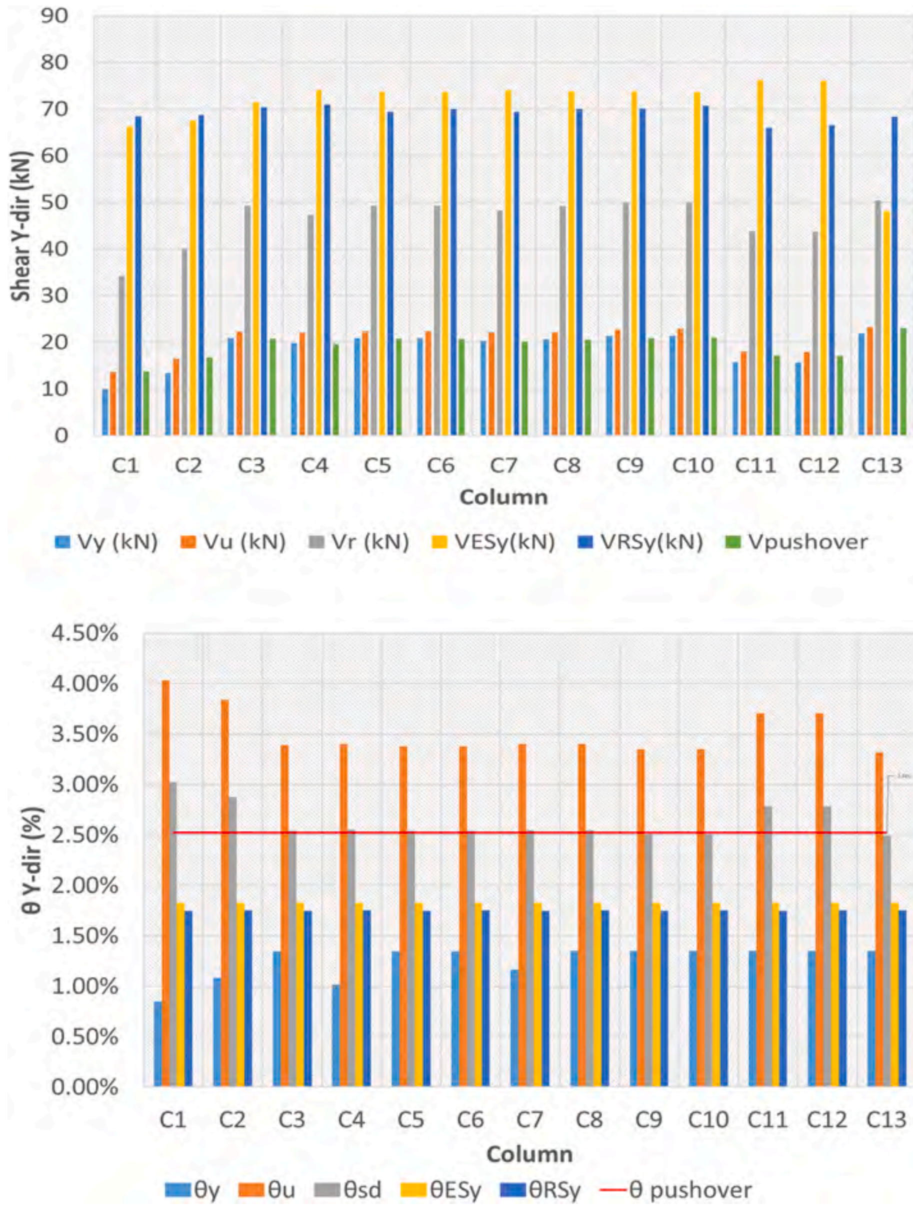

Fig. 10. Shear demand and capacity (top) and chord rotation demand and capacity (bottom).

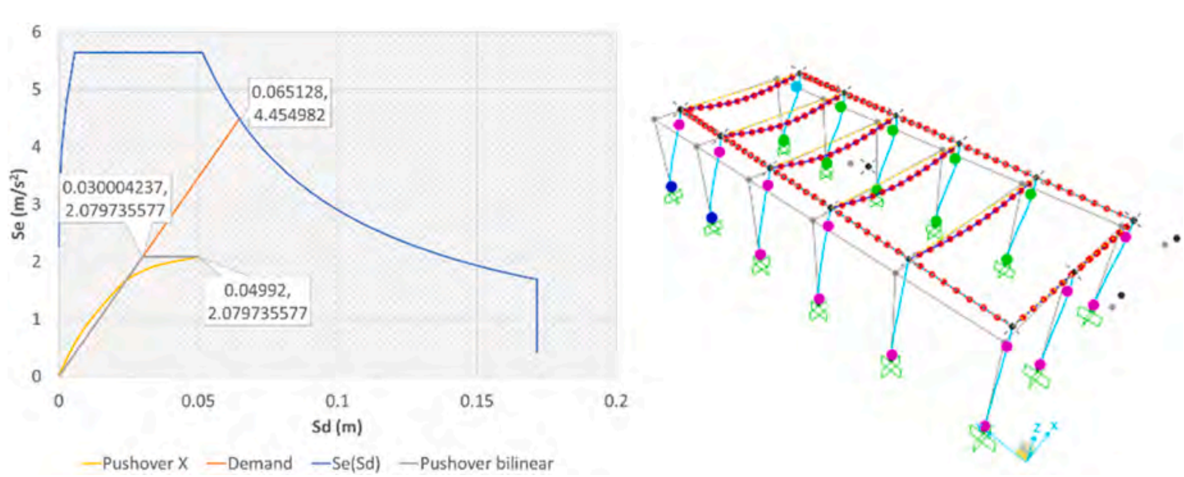

Fig. 11. Pushover curve and seismic demand (left) and formation of plastic hinges at the end of pushover (right) for the X direction.

1. The seismic assessment of historic concrete structures in seismic prone areas should be mandatory, especially in those cases where the structures were originally designed without any seismic provisions.
2. The decision of the minimum safety level that will be assessed should not be left on the owner. Minimum provisions should be stated in the codes.
3. Elastic types of analysis should not be an option for the assessment and retrofit of historic concrete structures, due to their inadequacy in determining the correct modes of failure and the extensive retrofit they will lead at, due to force-based checks.
4. Inelastic types of analysis, such as pushover, must be allowed for the case of historic concrete structures, even when the knowledge level is limited due to restrictions in destructive testing. The use of inelastic analysis may show mechanisms of failure that cannot be captured by elastic analysis, such as storey-sway mechanism formation.
5. Educational activities, seminars and workshops should be implemented for the practicing engineers in order to understand and appreciate the cultural significance of historic concrete structures and learn new approaches and best practices for their assessment and retrofit.

Data availability

The raw/processed data required to reproduce these findings cannot be shared at this time, as the data also forms part of an ongoing study.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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