Αποτίμηση Σεισμικής Απόκρισης Υφιστάμενου Σχολικού Κτηρίου
Ωπλισμένου Σκυροδέματος και Ενίσχυση με Τοποθέτηση Μεταλλικών
Στοιχείων

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ΠΕΡΙΛΗΨΗ

Αντικείμενο της παρούσας ερευνητικής εργασίας, η οποία εκπονήθηκε στο Εργαστήριο Στατικής και Δυναμικής των Κατασκευών του Τμήματος Πολιτικών Μηχανικών του Δημοκριτείου Πανεπιστημίου Θράκης, αποτελεί η αποτίμηση της σεισμικής συμπεριφοράς υφιστάμενου σχολικού κτηρίου από ωπλισμένο σκυρόδεμα με τη χρήση της ανελαστικής στατικής ανάλυσης (ανάλυση Pushover), σύμφωνα με τις διατάξεις του Κανονισμού Επεμβάσεων (ΚΑΝ.ΕΠΕ.). Το υπό εξέταση κτήριο βρίσκεται στην περιοχή του Αγίου Λουκά στην Καβάλα και κατασκευάσθηκε σύμφωνα με μελέτη του Οργανισμού Σχολικών κτηρίων (ΟΣΚ), λειτουργώντας για πρώτη φορά κατά τη σχολική χρονιά 1980-81. Η προσομοίωση και ανάλυση της κατασκευής πραγματοποιήθηκε με τη χρήση του λογισμικού SAP2000 v.16. Κατά την ανάλυση, επιλέχθηκε η μέθοδος Pushover με έλεγχο παραμορφώσεων (Displacement Control) και ως εκ τούτου η φόρτιση πραγματοποιήθηκε υπό τη μορφή επιβαλλόμενης μετατόπισης. Πιο συγκεκριμένα, οριζόντιες φορτίσεις επιβλήθηκαν στους ορόφους του φορέα σύμφωνα με τα μοντέλα της τριγωνικής και ομοιόμορφης κατανομής και εκτελέστηκαν συνολικά έξι αναλύσεις κατά τις διευθύνσεις Χ, Υ και –Υ. Ως αποτέλεςμα, το σημείο επιτελεστικότητας της δυσμενέστερης ανάλυσης που αντιστοιχεί στη διεύθυνση X, ορίστηκε στα 1.9 cm και το μεγαλύτερο ποσοστό πλαστικών αρθρώσεων σχηματίστηκε στα υπόστυλωμα. Εφόσον ο υπό μελέτη φορέας αποτελεί σχολικό κτήριο η σεισμική ασφάλεια είναι ιδιαίτερα σημαντική δεδομένων των συνεπειών κατάρρευσης. Λαμβάνοντας λοιπόν ως στάθμη επανελέγχου τη στάθμη «Περιορισμένες βλάβες», το κτήριο αξιολογήθηκε αρχικά ως ανεπαρκές και κρίθηκε απαραίτητη η ενίσχυσή του. Η ερευνητική δραστηριότητα στον τομέα της αντισεισμικής προστασίας των κατασκευών έχει οδηγήσει στην ανάπτυξη ποικιλίας διατάξεων ενίσχυσης με πρόσθετα μεταλλικά στοιχεία. Στην παρούσα κατασκευή επιλέχθηκε η τοποθέτηση δικτυώματος μεταλλικής διατομής (πλατύπελμη HE180A) μορφής Λ σε τέσσερα φατνώματα στα δύο ακραία (περιμετρικά) ανοίγματα των πλαισίων της διεύθυνσης Χ. Επειτά από την ενίσχυση του χωρικού προσομοιώματος με τα προαναφερθέντα μεταλλικά δικτυώματα, εκτελέσθηκαν εκ νέου οι ανελαστικές στατικές αναλύσεις κατά τις διεύθυνσεις Χ, Υ και –Υ. Το σημείο επιτελεστικότητας της δυσμενέστερης ανάλυσης ορίστηκε αυτή τη φορά στα 0.5 cm, και πολύ μικρότερο ποσοστό πλαστικών αρθρώσεων εμφανίσθηκε σε ορισμένα ακραία υποστυλώματα. Ο ενσωματωμένος φορέας αποτιμήθηκε τελικά ως επαρκής για στοχευμένη στάθμη επιτελεστικότητας «Περιορισμένες Βλάβες» και δεν απαιτήθηκε περαιτέρω ενίσχυση με χρήση αποσβεστήρων τριβής. Ως εκ τούτου, η προτεινόμενη μέθοδος μπορεί να αποτελέσει μια αποτελεσματική προσέγγιση αντισεισμικής ενίσχυσης για υφιστάμενα κτήρια ωπλισμένου σκυροδέματος αντίστοιχης γεωμετρίας και χαρακτηριστικών με τον υπό μελέτη φορέα.

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ABSTRACT

This paper aims to evaluate the inelastic behaviour of an existing reinforced concrete school building constructed in 1973 in the city of Kavala, Greece. It examines the subjection of a monotonic load which increases iteratively, through an ultimate condition, and then investigates the need of implementation of retrofitting techniques. The assessment is based on the National Interventions Code and the performance levels are specified in compliance with ATC-40. According to six different analyses, including both triangular and uniform distribution, it is concluded that structure can’t reach the target level of safety, as defined by importance levels criteria that ensure continued functionality of the building. More specifically, the results indicate that plastic hinges occur in a great number of columns. As a result, the utilization of diagonal bracing arrangements in order to increase the seismic capacity is examined. The nonlinear pushover analysis confirms that target displacement is reduced by 73% if the recommended steel bracing system is used. The main conclusion that can be drawn is that, the examined method could provide an efficient approach for strengthening of existing RC buildings.

1 INTRODUCTION

The differences recorded in seismic response during earthquakes between new and existing RC structures over the past decades brought forward the need to quantify their performance characteristics. Particular emphasis must be placed on existing buildings since they exhibit higher vulnerability to earthquake excitation and suffer most of the damage. Their seismic response is also characterized by substantial uncertainty. Focusing on Greece, during past earthquakes many deficiencies were reported, such as low concrete strength, weak column - strong beam behaviour, short column behaviour, inadequate splice lengths, poor confinement of end regions of columns and beams, and improper hooks of the stirrups leading to weaker than desired performance.

A large number of Greek residential buildings dates back to the 60s, 70s and 80s, and as a result they were designed following allowable stress procedures and using relatively low base shear coefficients on the basis of simplified structural analysis models. Capacity design and critical region detailing were also not intentionally incorporated. The quality of structural materials used in these structures, the detailing practices at the time of construction, past seismic loading history as well as possible occupancy changes lead to significant causes of uncertainty in their expected seismic behaviour. Hence, accurate seismic assessment under expected levels of earthquake motion is of great social and economic significance.

The quantification of structural seismic performance based on collapse capacity under extreme events provides a rational and powerful framework for evaluating existing buildings. The term performance, as it is related to natural hazards’ exposure, usually refers to buildings’ condition after disaster signifying a level of expected damage or a load that can be resisted. The structural performance is currently determined by applying nonlinear static procedures in compliance with earthquake design codes and standards such as ATC-40 (1996), FEMA273, FEMA356 and TERDC. These simple nonlinear static procedures, among others the Capacity Spectrum Method (CSM), the Displacement Coefficient Method (DCM) and the N2 Method, which is also employed in Eurocode 8, have been introduced and later improved by various researchers and compared with nonlinear dynamic analyses in order to validate their accuracy. Goel and Chopra
developed a modal pushover analysis procedure, in which the target displacement is determined from nonlinear dynamic analysis of an equivalent single-degree-of-freedom (SDOF) inelastic system and its peak deformation.

The results derive from nonlinear dynamic analyses are generally considered time-consuming but more accurate considering that different nonlinear static approaches may provide substantially different estimates of target displacement for the same ground motion and structure. Further, inelastic time history analyses (ITHA) are progressively becoming more cost effective. Nevertheless, nonlinear static procedures are generally characterized by simplicity with reasonable accuracy. It can be concluded, then, that static monotonic nonlinear analyses (pushover type) provide sufficient insight in the structure’s expected behaviour for design and assessment purposes.

1.1 Objectives and scope of research

The objective of this paper is the seismic performance evaluation of an existing RC school building which is located in the city of Kavala, in Greece. The 2-storey building was built during 1978-1980 and it was designed following the provisions of the first Greek seismic code introduced in 1959 (Greek Royal Decree of 1959). Pushover analysis has been conducted in order to assess its seismic performance and a simplified nonlinear static method is used for the estimation of the target displacement.

Systematic seismic assessment of reinforced concrete buildings which have been designed based on earlier and outdated design codes is imperative in countries with high seismicity such as Greece, since recent strong earthquakes have underscored their vulnerability. The high percentage of substandard, lightly reinforced existing buildings renders the massive use of detailed seismic analyses a very demanding work-intensive task.

Building codes define the minimum design requirements to ensure the safety of occupants during specific design events. However, various past natural disasters have prompted recognition that significant damage can occur even when buildings are compliant with the codes. Many critical facilities, including school buildings, are closed after natural disasters, even if damage is relatively minor, suggesting that satisfying the minimum defined criteria may not be sufficient. Communities also depend on school buildings to provide reliable shelter and critical services. To meet that need, school buildings should be designed and constructed in order to ensure continued and uninterrupted functionality.

Within this framework, schools are given high priority when earthquake strengthening programmes are discussed, nevertheless, due to economic constraints, a very small fraction of the existing school building stock has actually been upgraded within the umbrella of pre-earthquake strengthening programmes worldwide. In recent years, the most extensive efforts in adopting school strengthening programmes has been conducted in Japan.

2 SEISMIC HAZARD IN GREECE

Greece confronts the highest seismic hazard in Europe and one of the highest in the world. More specifically, potentially destructive (Mw 5.5 and larger) shallow earthquakes occur on average as
often as one about every 2 months. Fortunately, the majority of these earthquakes takes place in sparsely populated areas or under the sea, which considerably reduces their destructive capability. Nonetheless, populated areas are affected by damaging to destructive earthquakes quite often. The tectonic regime in the wider area is determined primarily by the convergence, at a rate of about 1 cm/yr, between the Eurasian and African lithospheric plates, and by the counterclockwise rotation, at about 2.5 cm/yr, of the Anatolian plate relative to Eurasia. The Arabian plate seems to only indirectly affect the tectonic situation in the area of Greece.

![Figure 1: Seismicity in Greece 1900-2006 (Pelli, 2014).](image)

3 BUILDING CODES AND STANDARDS

The building analysed in the present study is an RC structure with no plan irregularity. It was designed following allowable stress procedures in compliance with the 1959 seismic design code, using grade B225 equivalent to C16 concrete, and ribbed S400 reinforcement.

Seismic design according to the Greek Royal Decree of 1959 was based on a three-zone classification system with the seismic base shear coefficient in the three zones on hard soil being equal to 4%, 6% or 8% of the structural weight, which was calculated as the sum of unfactored dead plus live loads. Only simplified design models were used for analysis with a special check for perimeter columns and beams, while interior beams were usually designed only for gravity loads. Neither critical region reinforcement for confinement nor any capacity design provisions were used in design. Buildings of this period were typically characterized by dense and regular column spacing with relatively short bay sizes (3.0 to 4.0 m) and no use of shear walls. Buildings constructed during the 1980s were designed according to MOD84, the Interim Modifications of the Royal Decree of 1959, that introduced modifications to the method of analysis and the lateral load distribution from uniform to inverted triangular, as well as introduced ductile detailing provisions. Seismic base shear coefficient remained the same.

4 PUSHOVER ANALYSIS

Pushover analysis constitutes a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased following a certain predefined pattern. It is used for evaluating the real strength of structures and it promises to be a useful and effective tool for
performance-based design. The ATC-40 and FEMA documents define modelling parameters, acceptance criteria and the actions necessary to determine the yielding of frame members during the analysis. Two of these actions, which are used to govern the inelastic behaviour of members, are deformation-controlled (ductile action) and force-controlled (brittle action).

A pushover analysis can be generally described as a series of incremental static analysis carried out to develop a capacity curve for the building. Based on the capacity curve, a target displacement which is an estimate of the displacement that the design earthquake will produce on the building is determined. The extent of damage experienced by the structure at this target displacement is considered representative of the damage experienced by the building when it is subject to design level ground shaking.

Pushover analysis has been developed by many researchers with minor variation in computation procedure. Since the behaviour of reinforced concrete structures may be highly inelastic under seismic loads, the global inelastic performance of RC structures will be dominated by plastic yielding effects and consequently the accuracy of the analysis will be influenced by the ability of the analytical models to capture these effects. In general, analytical models for the pushover analysis of frame structures may be divided into two main types: 1) distributed plasticity (plastic zone) and 2) concentrated plasticity (plastic hinge). In the present study the plastic hinge approach is adopted.

5 MODELLING AND ANALYSIS OF THE BUILDING

The examined building in the city of Kavala is modeled and analyzed using SAP2000 software. This finite element analysis software is utilized to create 3D models and to analyse general structures varying from bridges and stadiums to industrial plants and offshore structures. The two-story reinforced concrete model constitutes a series of load resisting elements. The floor area is equal to 34.29×22.2 m for all levels except of the level corresponding to the ceiling of the first floor which is equal to 29.05×22.2 m. The overall height of the structure is 6.8 m, with all story heights equal to 3.4 m.

![Figure 2: 2-Storey RC school building, Kavala, Greece.](image)

The lateral loads applied on the building have been calculated based on the Eurocodes. The study is performed for seismic zone I and terrain category B as defined in Eurocode 8. The structure
consists of web plates, and beam elements are modelled as rectangular cross sections with a width equals to effective flange width $b_{eff}$ in compliance with Eurocode 2. The frames are regarded as firmly fixed at the bottom and the soil–structure interaction is neglected.

### Table 1: Structure’s characteristics.

<table>
<thead>
<tr>
<th>Structure</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of stories</td>
<td>2</td>
</tr>
<tr>
<td>Storey height</td>
<td>3.4 m</td>
</tr>
<tr>
<td>Type of building use</td>
<td>School building</td>
</tr>
<tr>
<td>Importance factor</td>
<td>1.2</td>
</tr>
<tr>
<td>Seismic zone</td>
<td>1</td>
</tr>
<tr>
<td>Peak Ground Acceleration</td>
<td>0.16g</td>
</tr>
</tbody>
</table>

### Table 2: Material properties.

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade of concrete</td>
<td>C16/20 (B225)</td>
</tr>
<tr>
<td>Weight per Unit Volume</td>
<td>25 kN/m³</td>
</tr>
<tr>
<td>Modulus of Elasticity, $E_c$</td>
<td>29 GPa</td>
</tr>
<tr>
<td>Poisson’s Ratio, concrete</td>
<td>0.2</td>
</tr>
<tr>
<td>Grade of steel</td>
<td>S400 (STIII)</td>
</tr>
<tr>
<td>Modulus of Elasticity, $E_s$</td>
<td>200 GPa</td>
</tr>
<tr>
<td>Poisson’s Ratio, steel</td>
<td>0.3</td>
</tr>
</tbody>
</table>

### RESULTS

In the present study, nonlinear static analysis using SAP2000 under monotonic loading has been performed. The fundamental periods derived by eigenvalue analysis are 0.479 sec and 0.499 sec corresponding to the generated SAP2000 models without and with basement respectively. Pushover analysis’ results are derived based on the simplified without basement model, and base shear $V_{max}$ is attained under uniform and triangular lateral load profile. The value of seismic base shear ($V_{0,x}$) has been calculated following the design provisions of Eurocode 8 and is equal to 1940.81 kN.

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Figure 3: Model of 2-storey building without & with basement, SAP2000.
Table 3: Results from eigenvalue analysis.

<table>
<thead>
<tr>
<th>Frame system</th>
<th>Period (s)</th>
<th>Effective modal mass (fraction of the total mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1&lt;sup&gt;st&lt;/sup&gt; 2&lt;sup&gt;nd&lt;/sup&gt; 3&lt;sup&gt;rd&lt;/sup&gt; 4&lt;sup&gt;th&lt;/sup&gt;</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; 2&lt;sup&gt;nd&lt;/sup&gt; 3&lt;sup&gt;rd&lt;/sup&gt; 4&lt;sup&gt;th&lt;/sup&gt;</td>
</tr>
<tr>
<td>Model with basement</td>
<td>0.499      0.297 0.255 0.17</td>
<td>0.729 0.402 0.299 0.09</td>
</tr>
<tr>
<td>Model without basement</td>
<td>0.479      0.285 0.2445 0.166</td>
<td>0.897 0.515 0.365 0.103</td>
</tr>
</tbody>
</table>

The Base Force-Displacement curves in x, y and -y directions as obtained by the pushover analysis are presented in the following figures.

![Base force-displacement diagram](image)

Figure 4: Base force-displacement diagram, X, Y, -Y -direction respectively.

The performance point of X-direction corresponds to a base force equals to 5044.579 kN and a displacement of 1.9 cm.

6.1 Plastic Hinges Mechanism

Hinges formation, as depicted in Figure 5, reveals that yielding occurs primarily in X-Direction columns and corresponds to the Immediate Occupancy (IO) performance level. In Y-Direction, which presents a higher stiffness expressed by a higher stiffness coefficient ($K_y > K_x$) due to the geometry of the structure, yielding occurs only in a limited number of columns.
Figure 5: Deformation in X (a), Y (b), -Y (c) – direction, corresponding to the performance point.

Structures that meet the Immediate Occupancy (IO) building performance level are expected to sustain minimal damage to their structural elements and only minor damage to their non-structural components. While it is safe to reoccupy a building, which is designed for this performance level immediately following a major earthquake, non-structural systems may not function due to power outage or damage to fragile equipment. Consequently, although immediate occupancy is possible, some repair and restoration of utility services may be necessary before the building can function in a normal mode. The risk of casualties at this target performance level is very low.

Immediate Occupancy performance level should be at minimum the design goal for all school buildings. However, considering that even the smallest disruption of non-structural systems may be detrimental for the continued operation of a school which is also designated as a shelter, an even higher level of protection for critical functions associated with this use should be considered. As a result, structure’s margin safety against collapse can be characterized as inadequate and strength and displacement reserves as insufficient. Therefore, the security margin has to be enhanced in X direction and a retrofitting strategy is examined in order to enhance the overall performance of the structure.

6.2 Seismic Retrofitting

Bracing constitutes a very effective global upgrading strategy to enhance the global stiffness and strength of steel and composite frames. It can increase the energy absorption of structures and/or decrease the demand imposed by earthquake loads. Several configurations of braced frames may be used for seismic rehabilitation of existing steel, composite steel–concrete, and reinforced concrete building structures. The most frequently used systems include concentrically braced frames (CBFs), eccentrically braced frames (EBFs), and the knee-brace frames (KBFs). Common configurations for CBFs encompass V and inverted-V bracings, K, X as well as diagonal bracings. Bracing methods may be inefficient if the braces are not adequately capacity designed and they also may transmit very high actions to connections and foundations.

In this study, an inverted V-bracing system is proposed in order to improve the seismic performance of the building. The inverted V-bracing system, also known as chevron bracing, is composed of two members meeting at a centre point on the upper horizontal member. This system
can significantly reduce the buckling capacity of the compression brace leading to a buckling capacity which is less than the tension yield capacity of the tension brace.

The fundamental period derived by eigenvalue analysis of the strengthened structure is equal to 0.356 sec, and thus, lower comparing to the obtained period corresponding to the non-braced building. This difference can be attributed to the increased stiffness after the inclusion of the bracing system. The results derive from the inverted V-braced model analysis are depicted in the following figures and tables.

Table 4: Inverted-V braced structure, cross section’s characteristics.

<table>
<thead>
<tr>
<th>Cross-section HE180A</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside height (h)</td>
<td>171 mm</td>
</tr>
<tr>
<td>Flange width (b)</td>
<td>180 mm</td>
</tr>
<tr>
<td>Flange thickness (t_f)</td>
<td>9.5 mm</td>
</tr>
<tr>
<td>Web thickness (t_w)</td>
<td>6 mm</td>
</tr>
<tr>
<td>Area (A)</td>
<td>45.25 cm²</td>
</tr>
</tbody>
</table>

Figure 6: Model of inverted-V braced building, SAP2000.

Figure 7: Base force-displacement diagram, X-direction, braced building.

Figure 8: Deformation in X (a), Y (b), -Y (c) - direction, braced building.

Figure 9: Comparison of displacement between non-braced and braced building, X-direction.

Figure 10: Normalized-relative-displacement for non-braced and braced building.
The analysis indicates that yielding occurs solely in columns, corresponding to the Immediate Occupancy (IO) performance level and the plastic hinge formation is 67.5% lower comparing to the initial hinge formation of the non-braced structure. The new performance point of X-direction corresponds to a base force equals to 4412.713 kN and a displacement of 0.5 cm, which is 73% reduced comparing to the initial one. As Figure 10 illustrates, the normalized-relative-floor displacement of the braced structure has also been significantly reduced.

7 CONCLUSIONS

The assessment of the seismic behaviour of the RC school buildings, conducted within the framework of this study, is based on static nonlinear analysis. The initial results indicate that the building presents deficiencies leading to inadequate seismic performance considering the increased importance attributed to school buildings. With the inclusion of the bracing system, it is apparent that the structure performs an improved behaviour in terms of higher deformability and strength and lower relative floor displacements.

More specifically, it can be concluded that:

1. The initial behaviour of even properly detailed reinforced concrete school buildings may comply with the performance level which serve as design goal but considering their high importance is characterized as inadequate.
2. The relative floor displacement corresponding to the non-braced building frame as well as the distribution of hinges in the beams and columns is much higher compared to the inverted-V braced building frame.
3. Other bracing systems, for instance eccentric bracing should be potentially examined in the future. Eccentric bracing systems are heavily used in earthquake zones due to the high ductility they provide. However, the inherent deficiencies in the detailing of the beam-column joints should be thoroughly considered after providing similar systems.
4. The results obtained from pushover analysis in terms of demand, capacity and plastic hinges provide valuable insight into the real behaviour of structures. As a further step, decisions regarding the seismic rehabilitation of existing buildings should require both engineering and economic studies as well as consideration of social priorities.

8 REFERENCES