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Development of aluminum load-carrying space frame for building structures

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Abstract

The study presents the development of a residential building with a load-carrying space frame consisting of steel bracings and horizontal floor girders but columns made of an aluminum alloy. This is to the authors' knowledge the first building of its kind developed in Greece and Europe. The development is based on analysis using a space frame model, subjected to EC3 and EC8 specified loads, followed by a design based on the EC7 requirements. The study also discusses difficulties in developing the system and presents solutions to cope with code specified wind and seismic loads. In order to account for seismic loads that in most cases govern the design of the columns, 3-D and 2-D finite element analyses have been performed accounting for nonlinear material behavior of critical column-bay locations. Several alternative column profiles are proposed depending on the intensity of the seismic loads.

Keywords: Analysis; Design; Aluminum building; Seismic loads

1. Introduction

Aluminum has not been widely used for structural purposes compared to structural steel, mainly because of its low mechanical properties and high manufacturing cost. Although use of aluminum in non-structural components is quite familiar, there is only a small number of buildings, primarily in the USA, with a load-carrying structural system made of aluminum alloys [1]. Nowadays, as a result of technological advancements, aluminum has become a very competitive material compared to steel, in terms of both mechanical properties and cost. Nevertheless, aluminum alloy structures can still be considered as a new topic in the field of structural engineering with several aspects under development in order to attain sufficient confidence in the prediction of structural behaviour, e.g., [2–6], as well as in the assessment of codification rules, e.g., [7–9].

This paper presents the analysis and design of a two-story residential building with an aluminium alloy load carrying frame. It is the first structure of its kind built in Greece and Europe. The analysis and design follows the EC3, EC8, EC7 as well as the current Greek seismic code requirements. Since the structure is constructed in a highly seismic area, emphasis is placed on studying the behaviour of its joints using nonlinear analysis as well as on examining alternative column profiles to satisfy the demands imposed by the seismic loads.

2. Structural system

The structural system of the two buildings under consideration consists of space frames made of aluminum [10]. The presentation focuses on one of these two similar residential buildings, see Figs. 1–3. The space frame structural elements consist of an aluminum alloy EN AW 6082(T6) except the diagonals that are made of steel. In order to meet the requirements of the Greek Seismic Code [11] and also to improve the behavior of the structure to wind loads, diagonal steel

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Fig. 1. Aluminum building frame under construction.

braces, SHS 50 × 4 (S355), are used, see Figs. 4 and 5. The location of the diagonal bracings is also indicated in Figs. 2 and 3 outside the plan view. The vertical bracings reduce the structural horizontal deformations, which, because of the pin connections at the bottom of the columns and the conservative assumptions related to the loads, are quite significant. Rigid diaphragm action is achieved at each floor with the aid of diagonally placed horizontal stiffeners Ø25 made of structural steel S500s that run along the bays and along the girders connected to the high-strength plywood shims, a behavior which is desirable in highly seismic areas, see Figs. 2 and 3.

Vertical trusses form the frame bays of the two-story buildings, supported by columns with the profile shown in Fig. 6(a). The top and bottom chords of the first floor vertical trusses consist of a pair of L-shapes, L50 × 6. Single angles, L50 × 6, are also used for the struts and ties (Fig. 4). The struts are spaced at 600 mm intervals in order to facilitate the erection process. The girders running perpendicular to the top chords of the trusses consist of channels, C150/60/20 × 2.0. Also, 20 mm thick plywood shims, weighing 0.15 kN/m^2 with a nominal strength of 5–7 kPa are bolted to the girders. Channels, C150/60/20 × 2.0, are also used as girders running between the bottom chords of the trusses. Plasterboards of 12 mm thickness and weight of about 0.1 kN/m^2 are bolted to the girders.

Purlins, C150/60/20 × 2.0, are placed on the top chords of the inclined roof trusses (Fig. 4). The final roof surface consists of insulating panels weighing 0.40 kN/m^2 that are bolted to the purlins. The truss spacings at the first floor are 500 mm; thus, there is enough space for all the necessary insulation work. The spacing between the trusses of the second floor ranges from 300 to 800 mm, resulting in a 10% slope.

The exterior infill walls are made of double thin cement boards, while the interior walls have a total thickness of 100 mm and weigh 0.35 kN/m^2. Horizontal steel girts, C150/60/20 × 1.5, running between the columns are used to provide support for the walls.

The reinforced concrete foundation consists of spread footings connected with beams in two directions, as specified for highly seismic areas in order to achieve uniform seismic response at the foundation level [11,12].

3. Structural analysis and member design

Finite element analysis is used to study the behaviour of the building (STATIK [13]). The columns were modelled as beam elements with six degrees of freedom per node [14]. Truss members are pinned to allow for rotation, while the supports of the structure are also assumed to be pinned, simulating the connection of the columns to the reinforced concrete foundation [12].

In the 3-D structural model, diaphragm action at each floor level is simulated by bounding the horizontal displacement of all nodes to the displacement of a 'master node' that lies near the center of gravity of the diaphragm of each level. It should be noted that, in the 3-D model the effect of the openings for the staircases is taken into account in order to define the position of the master node [12,15].

The slenderness of the high-strength structural steel braces, SHS 50 × 4, satisfies the relationship (1), which is a requirement of the Greek Seismic Code for vertical

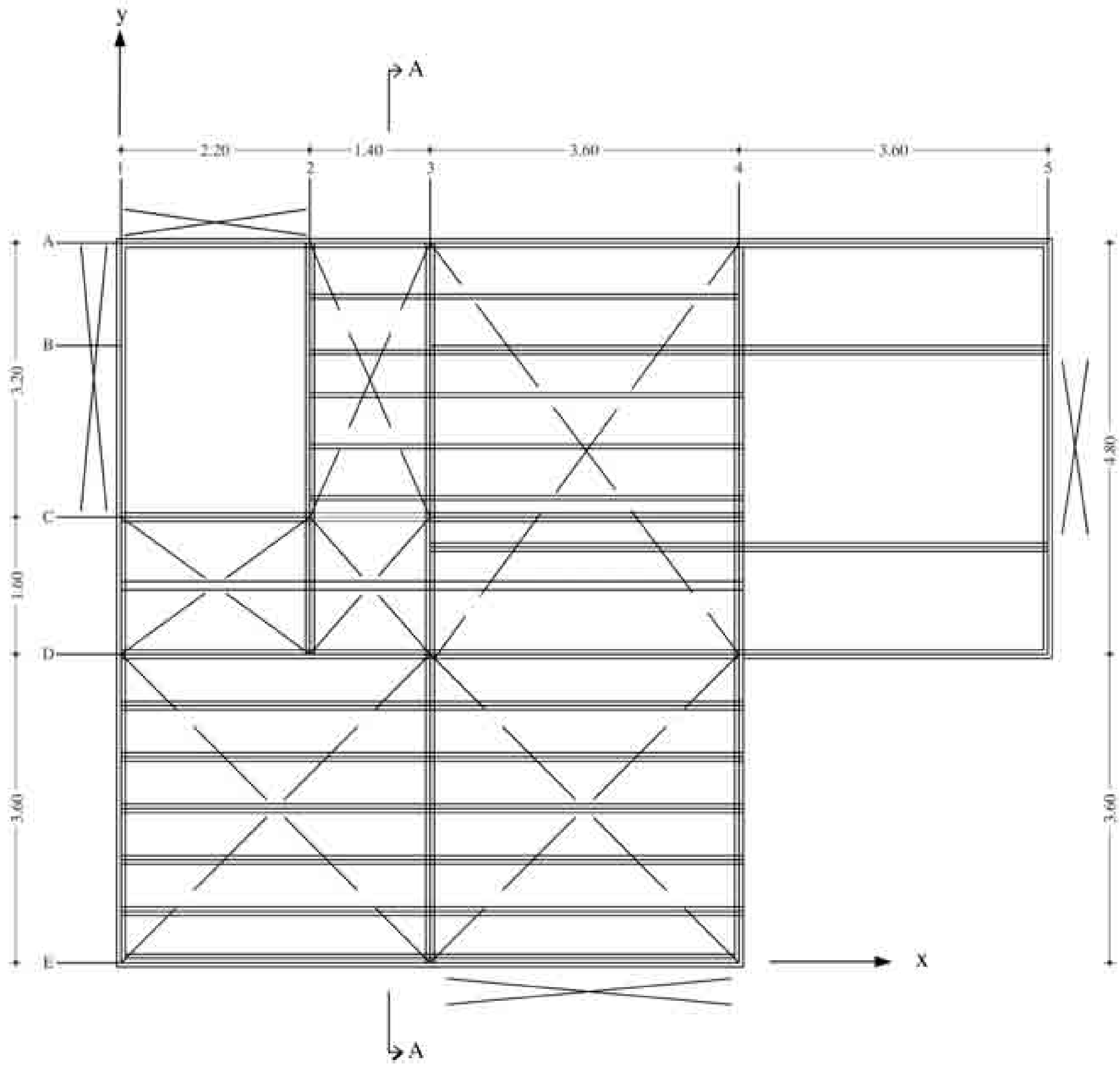

Fig. 2. Plan view of the first floor of building.

bracings [11]:

$$\bar{\lambda} = \sqrt{Af_y/N_{cr}} \leq 1.50. \qquad (1)$$

Functional and architectural requirements combined with code requirements determined the loads and material properties that are listed in Tables 1 and 2, respectively [12,16–18].

In order to design the building for seismic loads, modal analysis is performed to determine mode shapes and corresponding frequency eigenvalues. Based on the analysis, the two translational natural periods in the x and y axes are equal to $T_1 = 0.15$ s and $T_2 = 0.60$ s, respectively. The site is characterized by spectral design acceleration for weathered rock that lies in the constant acceleration region of the spectrum [11,12].

3.1. *Design of structural members*

The procedure to calculate the design capacity of aluminum members based on EC 9 [10] is similar to the procedure for steel members according to EC 3 [17] in terms of both the acceptance criteria and the material strength modification factors. However, unlike steel sections, there is a great variety of aluminum sections with complex geometry, coatings and variable thickness attributed to the availability of advanced fabrication methods. Thus, classification of the aluminum sections should be done very carefully.

Design is based on limit state and performance requirements. Regarding limit state, the maximum internal forces experienced by the components of the structure according to the load combinations are used for the analysis.

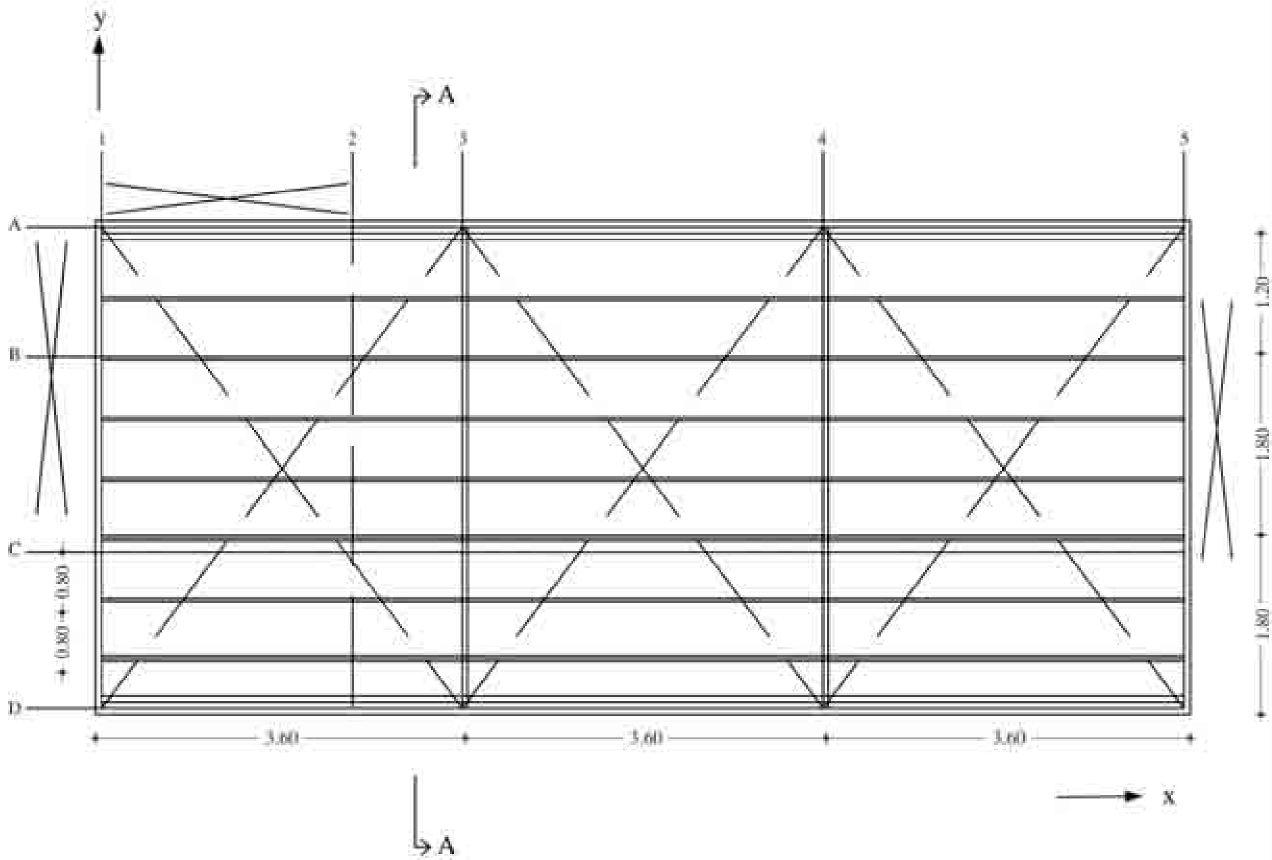

Fig. 3. Plan view of the second floor of building.

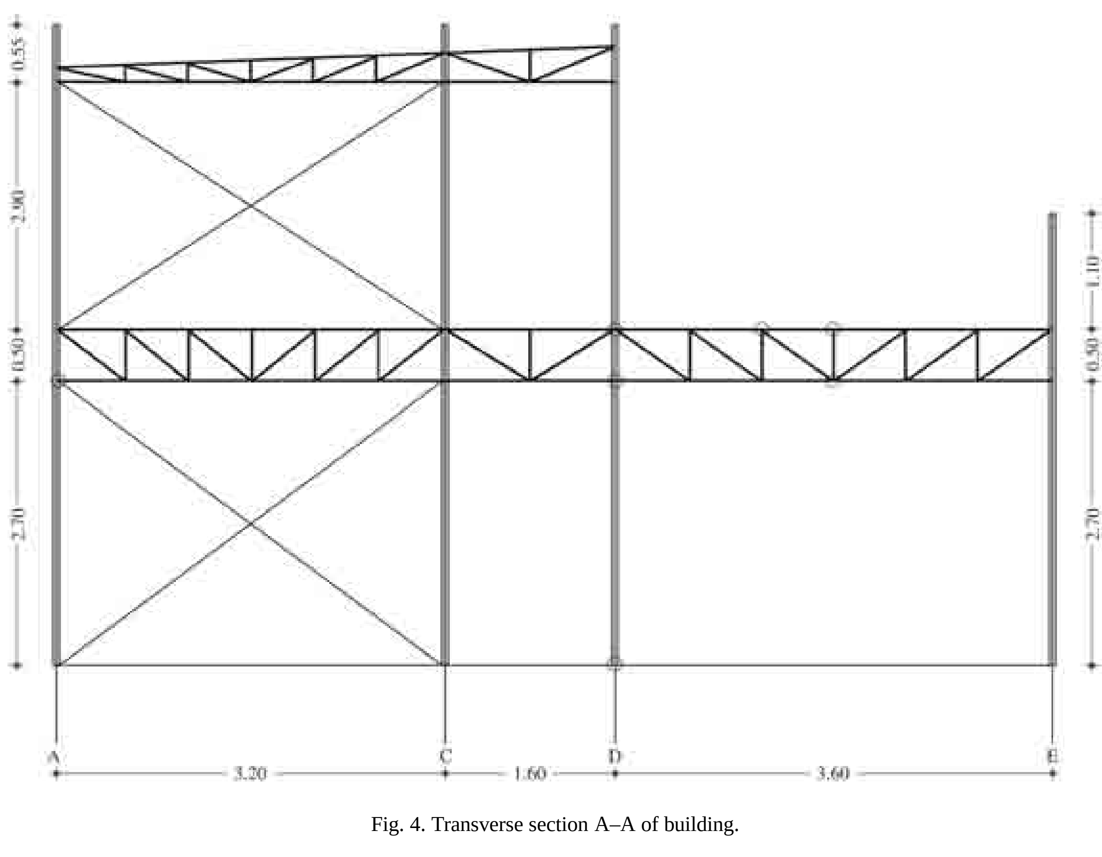

Fig. 4. Transverse section A–A of building.

Fig. 5. Diagonal steel bracing.

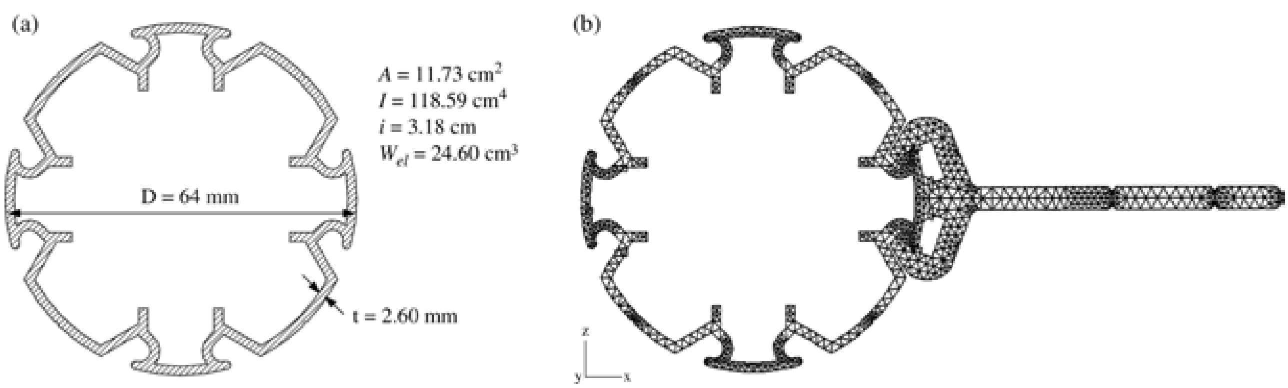

Fig. 6. (a) Column section. (b) Plane strain model of connection.

A detailed presentation of the design procedure according to EC9 [10] for the aluminum members and EC3 [17] for the steel members can be found in Ref. [12]. As elaborated in Ref. [12] the governing biaxial combination for the aluminum columns corresponds to the simultaneous action of biaxial bending and axial compression. According to EC9 [10] the following criterion should be satisfied:

$$\left(\frac{N_{\mathrm{Ed}}}{\chi_{\min}\cdot\omega_x\cdot N_{\mathrm{Rd}}}\right)^{\psi_c}+\frac{1}{\omega_0}\left(\frac{M_{\mathrm{Ed}}}{M_{\mathrm{Rd}}}\right)^{1.02}\le 1 \tag{2}$$

where ψ_c is the interaction component, and ω_x and ω_0 are heat affected zone (HAZ) softening factors. Since no welded connection exists, both ω_x and ω_0 can be taken equal to one.

Regarding the upper and the lower chords of the aluminum trusses the governing design combination corresponds to the maximum axial load (either tensile or compressive), that is

$$N_{\mathrm{Ed}}\le (N_{t,\mathrm{Rd}}\text{ or }N_{u,\mathrm{Rd}}). \tag{3}$$

The SHS steel diagonals were designed according to EC3 [17] requirements for members subjected to either axial tension or compression, i.e.:

$$N_{\mathrm{sd}}\le (N_{t,\mathrm{Rd}}\text{ or }N_{c,\mathrm{Rd}}). \tag{4}$$

In order to secure a conservative design, the tensile force acting on the diagonals was assumed to be twice as much as the value obtained from the analysis [12].

Table 1
Loads acting on the structure

Design loads	
Live load: 2.00 kN/m^2	Snow load: 0.75 kN/m^2
Wind load and temperature design parameters	
Velocity: $v = 30$ m/s Soil type: II k_T: 0.19	
Regional configuration coefficient $c_t = 1$	Temperature change: ±20 °C
Seismic load design parameters	
Soil type: B	Ground acceleration: $A = 0.24g$
Importance factor: $\gamma_I = 1.0$	Foundation factor: $\theta = 1.0$
q-factor: $q = 1.0$	Critical damping ratio: $\zeta = 4\%$

Table 2
Material properties

Aluminum: EN AW 6082(T6)	Yield limit: $f_y = 26$ kN/cm^2 Yield strain: $\varepsilon_y = 0.00376$ Ultimate strength: $f_u = 31$ kN/cm^2 Ultimate strain: $\varepsilon_u = 0.0298$	
Steel bolts: M12 8.8 and M16 8.8	Girders and girts: S355	Tension ties: S355
Foundation: C20/25	Foundation reinforcement: S500s	

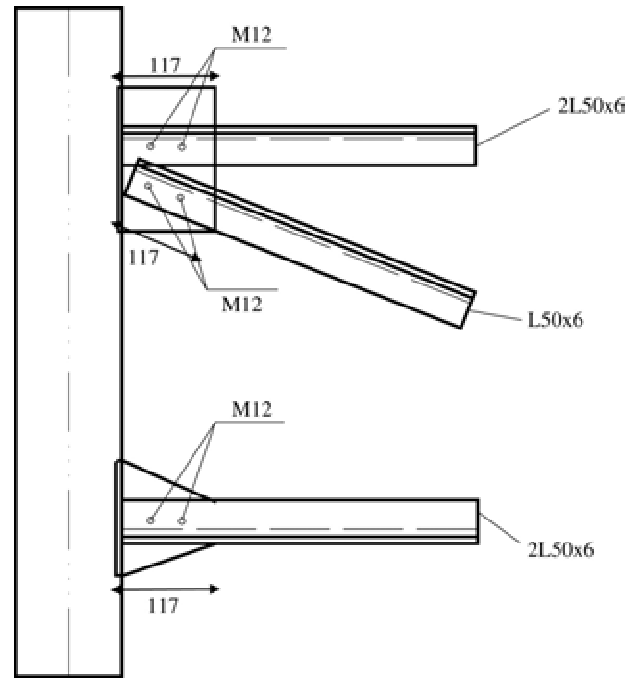

Fig. 7. Connection types between the columns and the trusses.

4. Structural analysis and design of column connections

4.1. Connection layout

The design of the structural components according to EC 9 [10] and EC 3 [17] for the aluminum and the steel members, respectively, is followed by the design of joint connections. Assuming linear elastic behavior, i.e., behavior factor $q = 1$, the seismic load combinations provided the design loads for the majority of the members.

Figs. 7 and 8 depict the two types of connections between the columns and the bays used for the structural system. The first type is the connection of the column to the lower L-shape 2L50 × 6 chord of the truss. In the second type, the column is further connected to an L50 × 6 diagonal member. Steel bolts M12 8.8 and M16 8.8 are used for the member connections. The bolts have a yielding strength $f_{yb} = 64$ kN/cm^2 and an ultimate strength $f_{ub} = 80$ kN/cm^2. Fig. 9 shows the connection between the foundation steel plate and the column. In order to avoid galvanic corrosion of aluminum, direct contact between aluminum and steel is not permitted. Therefore, the bolts are encased in a synthetic sheath that prevents corrosion.

4.2. Connection structural analysis model

The general-purpose finite element analysis program MSC/Nastran [19] is used to study the behavior of the joint formed by the column and the connection plate. In order to increase the reliability of the analysis, the connection has been modeled using two types of models, i.e., a plane strain model and a 3-D model, respectively. Based on data provided by the manufacturer of the profiles, no slip would occur between the column and the connection plate for the design loads. For this reason, complete bond has been assumed between the column and the connection plate for all loads and analysis. Use of the plane strain model also serves as a means to examine whether such model type could give results accurate enough for a preliminary analysis of the column–plate connection. Fig. 6(b) shows the 1448 plane strain finite element mesh used to model the column–plate connection. Notice that the load is transferred only through the contact areas of the two members, i.e., through the notches and the extrusions that are common on both the column and the Y-shape connection plate that is used to connect the truss members to the columns. The elements that are used to model plane strain systems are either triangular or quadrilateral with two translational degrees of freedom per node [15].

The 3-D model that is used to simulate the column has a total length of 1700 mm. The 300 mm long Y-shape connection plate is placed symmetrically in the middle of the column model. The finite element column model is pin-supported at both ends, while the connection plate is rigidly attached to the column in order to enforce displacement compatibility at the common nodes. In the 3-D model a total number of 29 790 plate elements are used for the connection plate and the column. Since the column section is very thin, i.e., $t = 2.60$ mm, use of this type of elements produces output files with the internal moments, displacements, stresses, etc., that are quite manageable in size. Care has been taken in developing the model mesh so that the dimensions of each element do not exceed

Fig. 8. Beam–column connection.

Fig. 9. Column–bracing–foundation connection.

the aspect ratio, λ, as recommended in the literature, that is $\lambda \leq 3$ [12,15].

Nonlinear analysis has been performed in order to account for nonlinear material behaviour exhibited under strong earthquake loads. In order to simplify the analysis, the uni-axial stress–strain relationship of the aluminum is replaced by the bilinear curve expressed by

$$f(\varepsilon) = \begin{pmatrix} E\varepsilon \\ f_y + E_t\varepsilon - \varepsilon_y \end{pmatrix} \text{ if } \begin{matrix} \varepsilon \leq \varepsilon_y \\ \varepsilon > \varepsilon_y \end{matrix} \text{ with } E_t = \frac{f_u - f_y}{\varepsilon_u - \varepsilon_y}$$
$$\text{and } E = \frac{f_y}{f_u}. \quad (5)$$

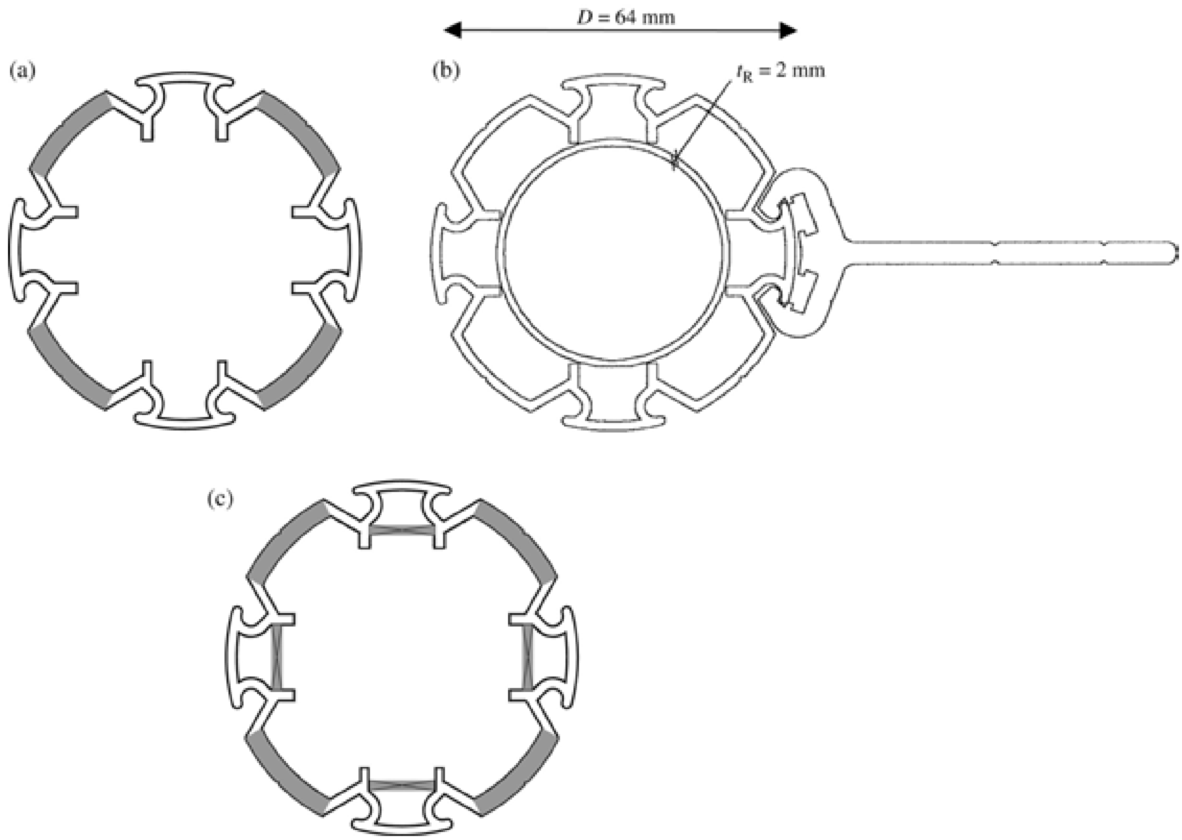

Fig. 10. (a) First strengthening scenario. (b) Third strengthening scenario. (c) Fourth strengthening scenario.

The first branch of the bilinear curve is characterized by the Young's modulus $E = 70$ GPa. The second line segment represents the idealized post-yield strain-hardening region with a slope characterized by the hardening modulus $E_t = 2$ GPa. The Poisson's ratio is taken as 0.3 for both branches. The material properties of aluminum are listed in Table 2.

4.3. Column section seismic design

In order to determine the maximum load that can be safely carried by the column–plate connection, the section has been subjected to an incrementally increasing compressive load exerted from the plate to the column. The analysis has shown that the static ultimate load that the section can resist is 93 kN.

Global seismic structural analysis has been performed based on the space frame model for the building site located in seismic zone I with a maximum ground acceleration $A = 0.16g$, where g is the acceleration of gravity [11,12]. The compressive load that develops at the connection is 50.96 kN. For the seismic zone II with a maximum ground acceleration $A = 0.24g$ and for the seismic zone III with a maximum ground acceleration $A = 0.36g$ [11], the analysis has shown that the corresponding compressive load is 76.44 and 114.66 kN, respectively. Comparing the ultimate compressive load the column section can resist with the compressive force that develops for the seismic load combination, the conclusion is drawn that the section is adequate for the seismic loads corresponding to zones I and II, but inadequate for the seismic loads in zone III.

4.4. Column section strengthening for higher design loads

In order to increase the compressive load capacity, four alternative strengthening scenarios are investigated. For all the alternative solutions, nonlinear finite element analysis was conducted using plane strain and 3-D models. In the first scenario, the section is strengthened at specific locations by increasing its thickness, as shown in Fig. 10(a). Shading has been used to indicate the portions of the cross-section with increased thickness. Analysis of the strengthened section has shown that the capacity of the section is increased to 108 kN, which is close to the demand corresponding to seismic zone III. Further increase of thickness at the chosen specific locations has an insignificant effect on the strength of the section [12].

It should be noted that the second scenario; that is the selection of an aluminum alloy with higher strength, instead of increasing the thickness of the section, does not suffice to increase the capacity of the section to 114.66 kN, that is the seismic demand for zone III.

The third alternative solution to strengthen the section for compression by adding an internal circular ring made

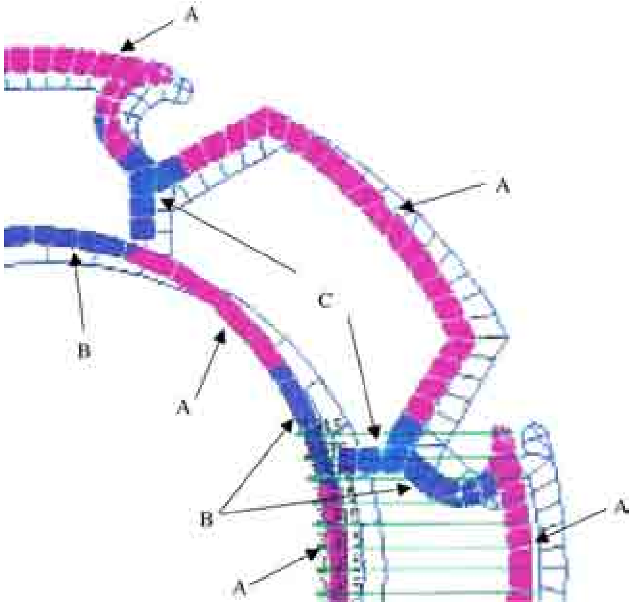

Fig. 11. Von Mises stresses variation corresponding to the third strengthening scenario. Variation of von Mises stress in: (i) region A: 0–35 MPa, (ii) region B: 36–195 MPa and (iii) region C: 196–260 MPa.

of aluminum is shown in Fig. 10(b). The thickness of the interior ring is $t_R = 2$ mm. The dimensions have been selected so that the ring can be placed inside the column leaving no gap between the ring surface and the interior venations of the column.

The von Mises stress for the third alternative, for the most heavily stressed section shown in Fig. 11, is well below the yielding strength of the aluminum. At a few regions in the interior ring between the venations of the column section depicted with C in Fig. 11, the stresses are almost equal to the von Mises stress. However, there is no element in the section where yielding strength is exceeded, and the deformations are much smaller compared to the deformations developed without the interior ring.

Finally, a fourth alternative solution to strengthen the joint has been tried by increasing the thickness at several locations combined with the placement of transverse shims at four locations, see shaded parts in Fig. 10(c). The modification of the section amounts to a total increase of the section area by 37.7%, while the ultimate section strength increases by a similar amount to approximately 126 kN, compared to 93 kN for the original column section [12].

5. Conclusion

The analysis and design of the two story residential building according to Eurocodes and the highly demanding Greek Seismic Code demonstrate the capability of an aluminum alloy load-carrying space frame to effectively accommodate all requirements.

The most critical elements of the system are the connections of the columns with the bracing and the horizontal truss girders, as well as the profile of the columns. With the aid of a 3-D finite element analysis accounting for nonlinear aluminum alloy behavior, several column profiles have been proposed. The profiles can successfully resist all relevant design load combinations including the combinations for the seismic loads that correspond to the highest seismic region specified by the current Greek Seismic Code.

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