EVALUATION OF THE FIRE RESISTANCE CALCULATION METHODOLOGIES FOR STEEL FRAMED STRUCTURES ACCORDING TO EUROCODES

Apostolos Koukouselis
Dr. Civil Engineer, MSc, Laboratory of Structural Analysis and Design,
Department of Civil Engineering, University of Thessaly, Volos, Greece
e-mail: akoukouselis@gmail.com

Daphne Pantousa
Dr. Civil Engineer, MSc, Laboratory of Structural Analysis and Design,
Department of Civil Engineering, University of Thessaly, Volos, Greece
e-mail: dpantousa@gmail.com

Euripidis Mistakidis
Professor, Laboratory of Structural Analysis and Design,
Department of Civil Engineering, University of Thessaly, Volos, Greece
e-mail: emistaki@uth.gr

1 ABSTRACT

The paper studies the effect of the simulation methodology on the calculation of fire resistance of steel frame structures. Two different methodologies are examined, as they are proposed by EN 1993-1-2 [1], the simplified and the advanced calculation models. For this purpose, an existing two-storey storage building is selected. First, the structure is simulated through ETABS [2], using beam finite elements. The fire resistance of the structure is calculated using an ad-hoc software that has been developed by the authors, that utilizes the results of structural analysis and calculates the fire-resistance of the structural system in both time and temperature domain. The algorithm is based on the simplified calculation models, as proposed in EN 1993-1-2 [1]. Next, the problem is solved using advanced numerical models. To this end, a two-dimensional numerical model is developed using the non-linear finite element code MSC Marc [3] that takes into account both the geometric and the material non-linearity. The time-temperature history of structural members is calculated according to the guidelines of EN 1993-1-2 [1]. The comparison of the results indicates the advantages and disadvantages of the studied calculation methods.

2 INTRODUCTION

It is generally accepted that fire should be treated as a load, like earthquake or wind. The research concerning the fire-behaviour of both isolated structural members and frame structures using numerical methods, is extensive (e.g. [4], [5], [6], [7]). Moreover, numerous fire tests have been conducted during the last decades. The behaviour of steel beams is

usually studied through standard fire tests. The first full scale fire tests were carried out in the Fire Research Station in U.K.[8], while, the most significant full scale tests were undertaken by the Building Research Establishment (BRE) in Cardington, U.K. The observation and the data from the fire tests on the multistory composite steel buildings provided useful information for the behaviour of structural and non-structural components under natural fire conditions. The enhanced research activity was followed by the development of several finite element codes, specialized to the fire analysis of structures, such as Vulcan and Safir, which include both the heat transfer and the structural modeling.
A recently published extensive review for the structures in fire [9] summarizes the state-of-the-art and identifies the research and training needs for improved fire safety. In this study, it is underlined that there is a need for more experimental data. Moreover, it is recognized that the numerical modeling tools are underdeveloped and few specifications for performance-based structural fire safety design exist.
The present paper focuses on the methods that are currently available for designing structures against fire. Three different approaches are followed and the fire-resistance is evaluated in both the temperature and strength domains. Specifically, a composite steel-concrete two-storey structure, is studied. The building is used for storage and the requirement for structural fire-resistance is R60, according to the national fire-protection regulations. The goal is to determine the most efficient method for the calculation of fire-resistance of this structure.

3 GEOMETRY AND LOADING OF THE STRUCTURE

The structure at hand is a two-story storage area consisting of 13 bays. The ground floor is 6.8 meters high from the ground to the top of the steel beams that support the 20cm thick concrete floor of the upper story, which has a maximum height of 6.745m. The roof of the structure consists of steel I profile beams that support the secondary ones and the roof panels. The distance between frames along the *x*-axis is 6.05m while the total opening of each frame is 15.80m. Fig. 1**Error! Reference source not found.** summarizes the basic geometric characteristics of the building and presents a typical frame and its cross-sections.

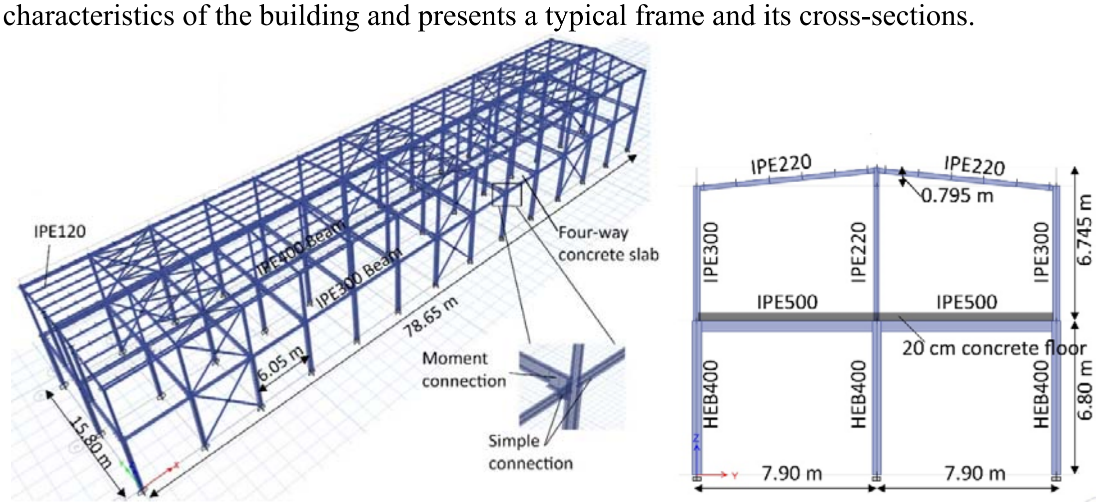


Fig. 1 The structure and the typical frame.

The connection of the IPE500 and IPE220 beams along the *y*-axis to the columns are moment resisting, while the ones along the *x*-axis (IPE400 and IPE300 beams) are simple ones (nominally pinned). The base connections of the columns to the foundation system were considered as fully clamped. As far as the loads are concerned, besides the self-weight of the structure, the first floor slab was considered to have a live load of 5 kN/m^2, the roof an additional dead load equal to 0.15 kN/m^2 due to its cladding, while the side panels and support beams that go around the building had a total weight of 0.3 kN/m^2.

4 FIRE DESIGN PROCEDURE AND ASSUMPTIONS

4.1 Validation in the temperature domain

The validation in the temperature domain was based on the methodology proposed by Eurocode 3 part 1-2, combined with the provisions of BS-EN-1993-1-2. In more detail, given the utilization ratio μ_0 of a structural member for which stability failure may not occur, the critical temperature θ_{cr} is calculated as:

$$\theta_{cr} = 39.19 \ln\left[\frac{1}{0.9674{\mu_0}^{3.833}} - 1\right] + 482 \quad (1)$$

with $\mu_0 \geq 0.013$

In eq. (1) the utilization ratio corresponds to the fire design combination (*G+0.8Q)* at time zero (temperature = 20°), calculated following the procedure of EN-1993-1-2[1]. However, to facilitate the procedure, it was chosen to use directly the utilization ratios calculated by ETABS according to EN-1993-1-1 [10]. To further simplify the study of the structure, as well as to focus on the failure phenomena that may arise due to the structural instabilities of the columns, buckling of both the IPE500 beams of the lower floor as well as the IPE220 beams of the roof was disregarded (both flexural and lateral torsional). The above assumptions are not far from reality as the concrete floor as well as the secondary beams with the roof panels can be considered to offer adequate protection against buckling. Thus, for both the above mentioned structural members, the critical temperatures were calculated according to (1). For the columns of the typical frame, which are prone to buckling, two approaches may be followed. The first approach is to follow an iterative procedure as proposed by Franssen and Vila Real [11]. The second approach is to use the critical temperature values proposed by Table NA-1 of BS-EN-1993-1-2 [12], which takes into consideration the slenderness of the member. In the present study, it was chosen to follow the latter approach. Fig. 2 shows the calculated critical temperatures per member of a typical-internal frame, grouped per 50°. In more detail, as far as the roof beams are concerned, the most critical members are located near the roof bracing and have a critical temperature of 750°. Concerning the IPE500 floor beams, the most critical temperature corresponds to the internal frames and is equal to 650°. Finally, the columns of the internal frames have a critical temperature of 650°, except the external IPE300 columns of the upper floor that have a critical temperature of 600°. The critical temperatures are summarized in Fig. 2.

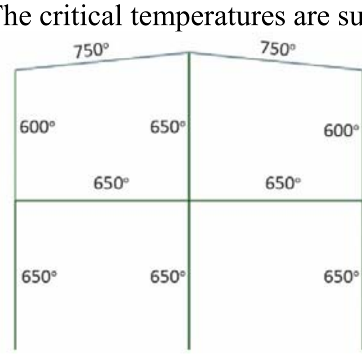
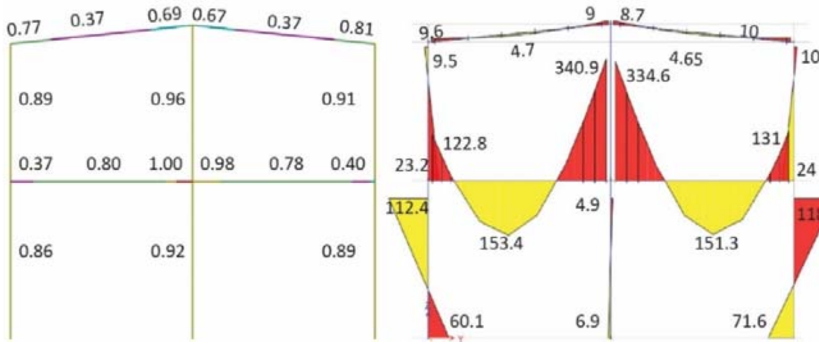


Fig. 2 Critical temperatures of typical frame members.

4.2 Validation by section forces

After the determination of the critical temperature of each member, their capacities were calculated at the specified temperature and compared to the member forces under the fire design combination. As a simplification, the temperature was considered uniform across the cross section and along the length of the member. The utilization ratios of the various



members is shown in Fig. 3. The outer columns have a utilization ratio between 86% and 91% (combined bending and axial compression), the internal ones have a ratio between 92% and 96% (mainly axial compression) while the floor beams reach their failure limit near the internal supports. The latter failure seems at a first glance rather unexpected, as the design methodology is slightly conservative. In more detail, the critical temperature of this frame according to (1) is 665° and the validation by section forces was done with a critical temperature of 650°, due to the grouping of members into temperature categories. Thus, one would expect that the utilization ratio of the member would be well below unity. However, this particular member changes section class and from Class 2 in ambient temperature drops to Class 3 in the elevated one, leading to the aforementioned failure. The extended reference to the above is done to highlight the caution needed by the designer in such phenomena.

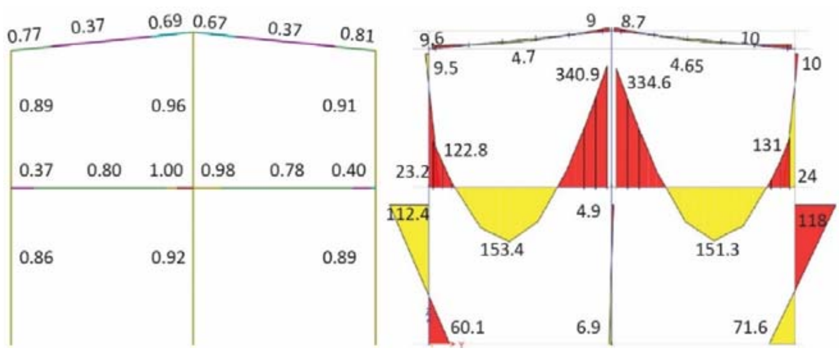
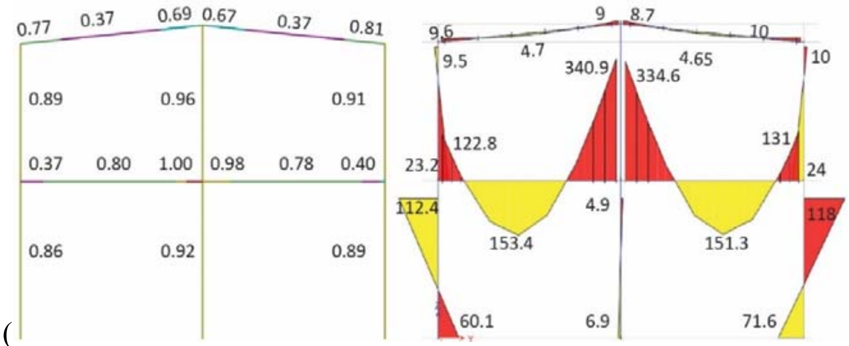


Fig. 3 Utilization ratios of typical frame members at elevated temperature.

Apart from the drop in strength due to the elevated temperature, structural steel also experiences a drop in the modulus of elasticity. Thus, the change in stiffness of the structural members or, to be more precise, the change of relative stiffness between them, can lead to stress redistribution and the section forces will differ from the ones calculated based on properties at 20°. To examine the impact of the above phenomenon on the overall design, the properties of the structural members were appropriately modified, based on their critical-design temperature, and the section forces were recalculated. By performing the validation checks using the updated section forces, the utilization ratios shown in Fig. 4

were calculated. By comparing the ratios prior and after the application of the stiffness



reduction factors (Fig. 3 and Fig. 4) it seems that the major impact of the change of the steel properties is localized at the outer columns of the upper floor due to the increase in bending moment.

Finally, apart from the change in material properties, the elevated temperature causes the elongation of the various members. As the structural members are not free to expand, this elongation causes a stress state that also changes the section forces of the members. Thus, an updated numerical model was analyzed, that had material properties as well as temperature loading based on the design temperature of each member. To simplify the study as well as for the results to be comparable with the ones produced by the advanced numerical models of Section 4, only temperature loading along the *y*-axis of the building was considered. This way the contribution of minor axis bending in the increase of the utilization ratios is neglected. **Error! Reference source not found.** displays the utilization ratios using the section forces of the latter model. Again, a significant discrepancy with the previous analysis results is noticed at the outer columns. In more detail, the upper floor columns experience a drop in their utilization ratio due to the decrease of the major axis bending, in contrast to the lower floor ones that have an increased bending and as a result their utilization ratio increases. Finally, the temperature loading also affects the floor beams, by increasing the bending moment at the outer supports and decreasing it at the internal ones.

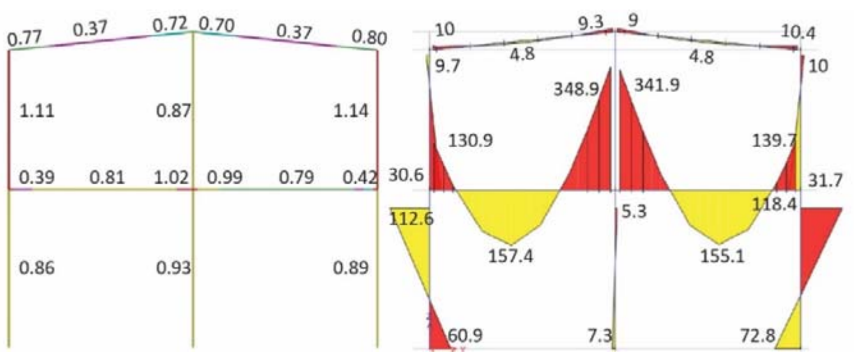


Fig. 4 Utilization ratios of typical frame members at elevated temperature after recalculation of section forces.

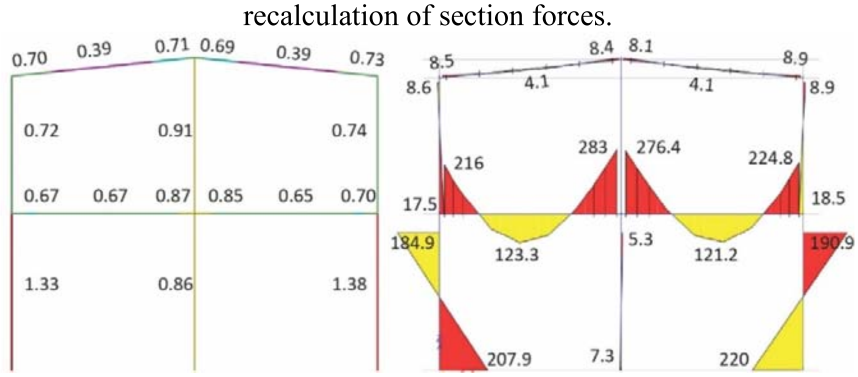


Fig. 5 Utilization ratios of typical frame members at elevated temperature with temperature loading along the *y*-direction.

5 ADVANCED CALCULATION METHOD

A two-dimensional model of a typical frame of the structure, is developed through the non-linear finite element code MSC-Marc [3]. The element type 98 of the library of the software is chosen for the simulation. This is a beam element and the cross-section that is used is a user-defined numerically integrated one. Four different branches are assigned for the simulation of the sections of structural members: the upper flange, the web (which is divided into two parts for more accurate results) and the lower flange branch. For every branch 25 different integration points are defined. The stress-strain law is integrated using the Newton-Cotes rule. Finally, the results are exported for the different integration points. Rigid offsets are used in order to model the connections between the beams and the columns. The haunched beam is simulated using gradually reduced sections. All the material properties are temperature dependent according to EN 1993-1-2 [1]. The yield stress of the structural steel is assumed to be equal to 275MPa at room temperature. It is noted that the strain hardening of steel for the temperature range of 20ºC-400ºC is neglected in order to simplify the problem. The fire is assumed to take place in both floors of the building and follows the ISO fire curve, for rational comparison with the simplified models. As indicated in Section 1, the fire-resistance of the structure should satisfy the R60 criterion and the structural members need to be protected. Vermiculite cement sprayed coating is assumed for the protection of structural members and its thickness is first calculated in order to satisfy the resistance requirement (R60). According to the selected coating thickness, the structural member reaches the corresponding critical temperature after 60 mins of fire exposure. Table 1, summarizes the coating thickness used, for all members. The temperature-time histories of the structural members are presented in Fig. 6. It should be emphasized that no thermal analysis is conducted and the temperature profile for the structural members is calculated according to the guidelines of Eurocode 3 [1], depending on the cross-section dimensions and the conditions of fire exposure (i.e. three or four side exposure). Moreover, it is assumed that the temperature is uniform along the member and the thermal gradient in the cross-section is not considered. The thermal loading is actually imposed as fixed nodal temperature. The problem is solved through geometric non-linear analysis.

Cross section	IPE 300	IPE 220	IPE 220	HEB 400	HEB 400	IPE 500
Fire exposure	*3-side*	*4 side*	*3 side*	*4 side*	*3 side*	*4 side*
Critical temp.	600	650	750	650	650	650
Coating thickness	12.2 mm	13 mm	6.4 mm	5.69 mm	4.82 mm	8.45 mm

Table 1. Calculated vermiculite cement coating thickness for the structural members.

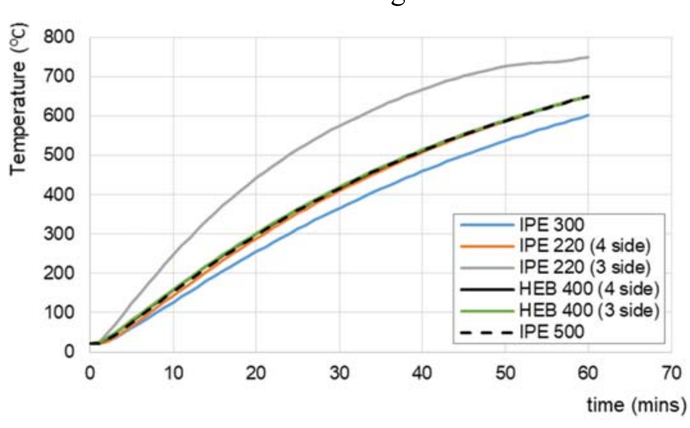


Fig. 6. Time – temperature history of the structural members.

Two different analyses are conducted. The first one does not include the thermal expansion effects, while in the second the thermal expansion coefficient is incorporated in the numerical model. The results of the analyses indicate the stress resultants for the structural members, during fire exposure. Then, individual stability checks of equivalent members, i.e. using appropriate buckling lengths, are conducted through hand calculations according to EN 1993-1-2 [1]. The utilization factors that arise are compared to the corresponding results that are presented in Section 3. It is found that the utilization factors that arise from this process are very close to the ones of Section 3.2. If the thermal expansion coefficient is ignored, the external columns of the upper floor fail and the utilization factor calculated here is 1.17 i.e almost the same with the one calculated using ETABS when the stress redistribution is considered (see Fig. 4). On the other hand, the forces that arise due to restrained thermal elongation of structural members seem to play an important role and the results of the analysis are different. In this case, the exterior columns of the lower floor fail and the maximum utilization factor is equal to 1.2 (the corresponding value that results from Fig. 5 is 1.38). Thus, it is concluded that the fire-design can be based on the simplified models (validation in the strength domain) since they are proved to give reasonable predictions.

6 SUMMARY – CONCLUSIONS

The paper examines two different methodologies for the calculation of the fire-resistance of a two-storey building. First, the critical temperatures of the structural members are calculated using EN 1993-1-2 [1] guidelines and the supplementary provisions of BS EN-1993-1-2 [12], for cases where stability should be considered (pre-design check). Then, the capacity of each member was calculated, based on its critical temperature, and, was compared to the member forces under the fire design combination (simplified models). The results of the validation in the strength domain, indicate that the floor beams reach their failure limit near the internal supports. Thus, at a first glance, the simplified design using the validation in the temperature domain seems to lead to good results. Only in certain cases, such as change of section class due to elevated temperature, the results may not be conservative. More realistic results are obtained when the appropriate properties of the structural members (i.e. based on their critical-design temperature) are used. The validation checks in this case, indicate that the outer columns of the upper floor fail. Next, the effect of thermal expansion coefficient is considered and the process is repeated. In this case, the results give failure of the columns of the lower floor. Finally, a more sophisticated model is developed in MSC Marc [3] for a two-dimensional frame of the structure (advanced model). The results confirm the utilization factors that arise from the previous simplified models. It is concluded, that the validation in temperature domain (pre-design check) should be carefully used. The models that take account both the modified material properties due to temperature effects and the thermal forces that arise from the restrained thermal expansion, indicate failure for structural members that are safe when applying the pre-design check. This is also validated by the advanced calculation models.

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ΑΞΙΟΛΟΓΗΣΗ ΤΩΝ ΜΕΘΟΔΩΝ ΥΠΟΛΟΓΙΣΜΟΥ ΤΗΣ ΠΥΡΑΝΤΟΧΗΣ ΠΛΑΙΣΙΩΤΩΝ ΜΕΤΑΛΛΙΚΩΝ ΚΑΤΑΣΚΕΥΩΝ ΣΥΜΦΩΝΑ ΜΕ ΤΟΥΣ ΕΥΡΩΚΩΔΙΚΕΣ

Απόστολος Κουκουσέλης
Δρ. Πολιτικός Μηχανικός, MSc., Εργαστήριο Ανάλυσης και Σχεδιασμού Κατασκευών
Τμήμα Πολ. Μηχ. Π.Θ., Βόλος, Ελλάδα
e-mail: akoukouselis@gmail.com

Δάφνη Παντούσα
Δρ. Πολιτικός Μηχανικός, MSc., Εργαστήριο Ανάλυσης και Σχεδιασμού Κατασκευών
Τμήμα Πολ. Μηχ. Π.Θ., Βόλος, Ελλάδα
e-mail: dpantousa@gmail.com

Ευριπίδης Μυστακίδης
Καθηγητής, Εργαστήριο Ανάλυσης και Σχεδιασμού Κατασκευών
Τμήμα Πολ. Μηχ. Π.Θ., Βόλος, Ελλάδα
e-mail: emistaki@uth.gr

ΠΕΡΙΛΗΨΗ

Η παρούσα εργασία μελετά την επιρροή της μεθόδου προσομοίωσης στον υπολογισμό της πυραντοχής μεταλλικών πλαισιωτών κατασκευών. Συγκεκριμένα, εξετάζονται οι δύο μεθοδολογίες που προτείνονται στον Ευρωκώδικα 3 – Μέρος 1-2, που βασίζονται σε απλά και προχωρημένα υπολογιστικά μοντέλα. Η εργασία επικεντρώνεται σε ένα υπάρχον διώροφο σύμμικτο κτίριο με χρήση αποθήκης. Αρχικά το δομικό σύστημα προσομοιώνεται μέσω ραβδωτών πεπερασμένων στοιχείων στο λογισμικό ETABS. Εφόσον η διαστασιολόγηση στις υψηλές θερμοκρασίες δεν γίνεται αυτόματα από το προαναφερθέν λογισμικό, αναπτύχθηκε νέο λογισμικό που αξιοποιεί τα αποτελέσματα της ανάλυσης και έτσι υπολογίζεται η πυραντοχή των δομικών μελών, σε όρους θερμοκρασίας. Το λογισμικό βασίζεται στα απλοποιημένα μοντέλα που προτείνονται στον Ευρωκώδικα 3 – Μέρος 1-2. Στη συνέχεια, το πρόβλημα επιλύεται μέσω των προχωρημένων υπολογιστικών μοντέλων. Για τον σκοπό αυτό, αναπτύσσεται ένα δισδιάστατο μοντέλο πεπερασμένων στοιχείων στο κώδικα μη-γραμμικής ανάλυσης MSC Marc με χρήση στοιχείων δοκού. Σε αυτό το στάδιο, η ανάλυση λαμβάνει υπόψιν τόσο τη γεωμετρική μη-γραμμικότητα αλλά και τη μη-γραμμικότητα του υλικού καθώς και την μεταβολή όλων των ιδιοτήτων του χάλυβα στις υψηλές θερμοκρασίες. Η χρονοϊστορία της θερμοκρασίας των δομικών μελών υπολογίζεται βάσει του Ευρωκώδικα 3 και επιβάλλεται σαν θερμικό φορτίο στους κόμβους. Η σύγκριση των αποτελεσμάτων που προκύπτουν από τις δύο διαφορετικές μεθοδολογίες, αποκαλύπτει τα πλεονεκτήματα και τα μειονεκτήματα τους και οδηγεί σε σαφή συμπεράσματα για την πιο σωστή αξιολόγηση της πυραντοχής της κατασκευής.