STUDY ON SANDWICH STEEL REINFORCED MASONRY

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1. ABSTRACT

In the current study, an innovative way of upgrading the lateral behavior of existent concrete buildings is researched, through the application of sandwich steel strips along the diagonals of a masonry infill wall. For this reason, a Finite Element model of Sandwich Steel Reinforced Masonry (SSRM) in short-scale was analyzed both in tension and compression as if it would behave under horizontal forces. The outcomes unveiled the theoretical contribution of masonry-steel interface, a large stress concentration around the drilled-hole and a diminishing ultimate tensile capacity as steel-plate thickness declines. Furthermore, a rational model was introduced for analyzing this kind of retrofit, based on the findings of this paper as well as the approach of design codes.

2. INTRODUCTION

Aging of structures as well as the obligation to diminish seismic risk in buildings of great importance such as hospitals, gave rise to the need of upgrading their behavior in alternating lateral loads. In this context, several retrofitting techniques, upgrading methods and assessments were proposed for masonry structures. Finite Element (FE) analysis offered a great assistance in this direction by providing a rational model approach to the problem. The current study attempts to simulate the lateral effect of a Sandwich Steel Reinforced Masonry (SSRM) as a means of structure seismic behavior upgrading, through evaluating its axial performance under monotonic and alternating seismic loads based on a FE model.

3. THEORETICAL MODEL

There are numerous models that have been proposed for the evaluation of masonry in-plane lateral behavior in concrete buildings. However, the model of diagonal masonry compressive strip (Fig. 1) expresses in a satisfying manner the seismic behavior of masonry infill without openings and is widely used by many seismic upgrade regulations [1] and researchers [2,3].

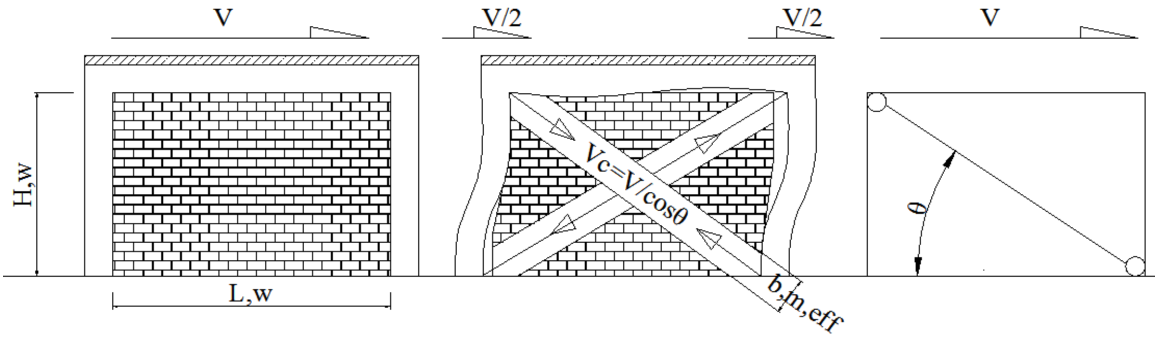


Fig. 1: Compressive strip diagonal model for masonry wall

Moreover, suggestions for the masonry diagonal effective width and thickness are described, based on the material quality, wall aspect ratio and other characteristics. An assumption of masonry effective width, $b_{m,eff}$, may be derived by eq. (1) [1]:

$$E_m \cdot b_{m,eff} = \frac{0.175 \cdot L_w \cdot (\lambda \cdot H_w)^{-0.4}}{\cos\theta}; \text{ where } \lambda = \left[\frac{E_m \cdot t_w \cdot \sin 2\theta}{4 \cdot E_c \cdot I_c \cdot H_w}\right]^{0.25} \quad (1)$$

where: L_w	is wall length,	θ	is diagonal compression angle,
H_w	is wall height,	E_m, E_c	are Young's Moduli for masonry and concrete,
t_w	is wall thickness,	I_c	is Moment Inertia for concrete column.

For usual concrete structures with outer infill walls of t_w=20-40 cm and clear storey height less or equal to 3,0 m, a value of $b_{m,eff} \approx 0{,}15L_{,w}$ could be well applied. For this reason, the application of structural steel strips of equivalent width along the masonry would upgrade the diagonal behavior and enhance the overall lateral characteristics of the structure.

4. FINITE ELEMENT APPROACH

This study is an attempt to evaluate the diagonal behavior of SSRM by assessing its load carrying capacity. The basic assumption is that the total lateral force is transferred through the steel strips, which presupposes their anchorage to the concrete frame mechanism. This assumption is close to the actual lateral behavior of the SSRM mechanism only at the beginning and at the ultimate load application step, while a stress distribution along the SSRM section is more probable at the middle load steps, but still with the steel stresses governing. In this way, the FE approach is focused on steel behavior and the interaction between the different materials.

4.1 Finite Element Model Formation

Let us consider a masonry wall that is composed by an inner and an outer masonry shell with insulation between them. Several types of FE specimens have been created for this study in order to simulate the behavior of SSRM under seismic loads. The masonry panels under consideration have a nominal thickness of 25 cm (10 + 5 + 10) where 5 cm is the middle gap that accounts for possible insulation. The panels are reinforced with diagonal structural steel (S235) strips of thickness ranging between 3 and 8 mm on the outer masonry faces (one internal and one external), connected with M12 studs in distances equal to the steel plate width ($b_{,w,eff}$=25cm). The inherent complexity of the problem leads us to apply simple models using commercial finite element software (ANSYSv12). Due to better addressing volumetric locking in thin solids and simplicity in calculations, 180 Series element type and specifically brick-8node SOLID185 was preferred.

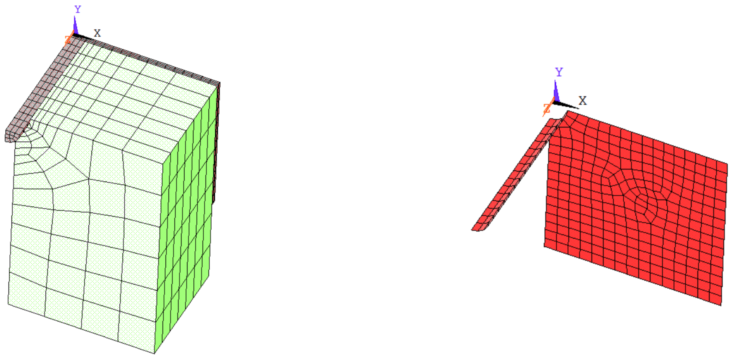


Fig. 2: 3D SSRM FE model, sectional preview and contact interfaces

Initially, a sectional preview of SSRM under axial loads is monitored and a symmetrical model of only 1/8th of the original section (Fig. 3) was analyzed. The axial loads were applied to the steel area assuming that the shear force would be primary transferred through the steel plates. This offers the possibility to monitor the material stress-strain conditions during this ultimate load state. In order to also observe the influence of the contact to this mechanism, two models with different interaction between the two materials were constructed; one with a gap of 0.1 cm, ignoring friction between plate and masonry, but without omitting the M12 stud contact, and the other, with a fully attached plate. The contact interfaces are illustrated in Fig. 2 and Fig. 5 and their attributes in Table 1.

	Masonry	**Steel**	**Contact interface**	
E (kN/cm^2)	940	20000	μ	0.15
v	0.15	0.3	FKN	0.1
Model	Elastic	Bi-linear Inelastic	Flexible between Areas	
Element	SOLID185	SOLID185	CONTA174	

Table 1: FE model properties per material

From detailed approaches to coarser simplified isotropic, orthotropic or homogenized ones [2], masonry due to its bulk, defines the effectiveness of a model. In the proposed model, due to the huge discrepancy between the Young's Moduli (E) of steel and masonry and for time effective solutions, a linear isotropic model was adopted for unreinforced masonry material properties, based on the most commonly used constituent elements (b: brick, mo: mortar) thickness height (H) and Young's Modulus (E) by eq. (2). Moreover, Table 1 illustrates the mechanical characteristics adopted for the masonry.

$$\frac{(H_b + H_{mo})}{E_m} = \left(\frac{H_{mo}}{E_{mo}}\right) + \left(\frac{H_b}{E_b}\right) \tag{2}$$

There are two steel parts in this model. The steel plates of thickness 3, 5 and 8 mm and the steel stud connectors of 12 mm diameter, placed in distances equal to the steel plate width.

4.2 FE Loading and Results

The FE model was both tested in compressive and tensile loading, gradually loaded with a load step of 1 kN/cm^2, until failure occurs. The Von Mises stress, the out-of-plain deformation and the contact pressures are some data of particular interest that are depicted in the following figures (Fig. 3 to Fig. 5).

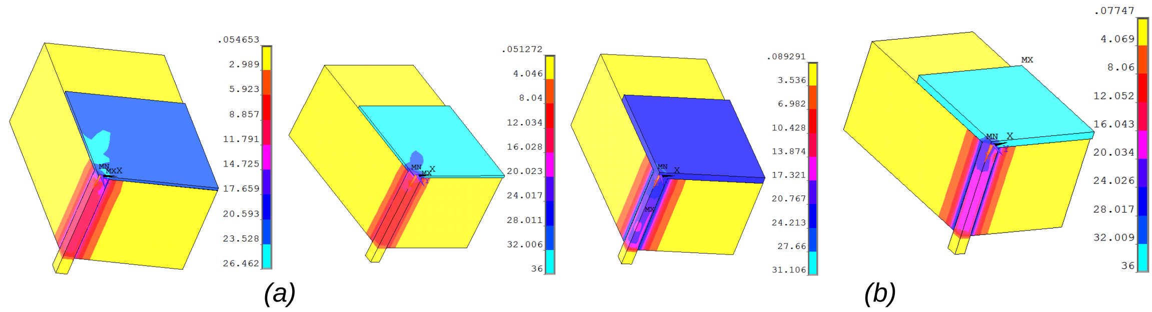


Fig. 3: Von Mises total stress in compression and in tension for: (a) t=3 mm, (b) t=8 mm.

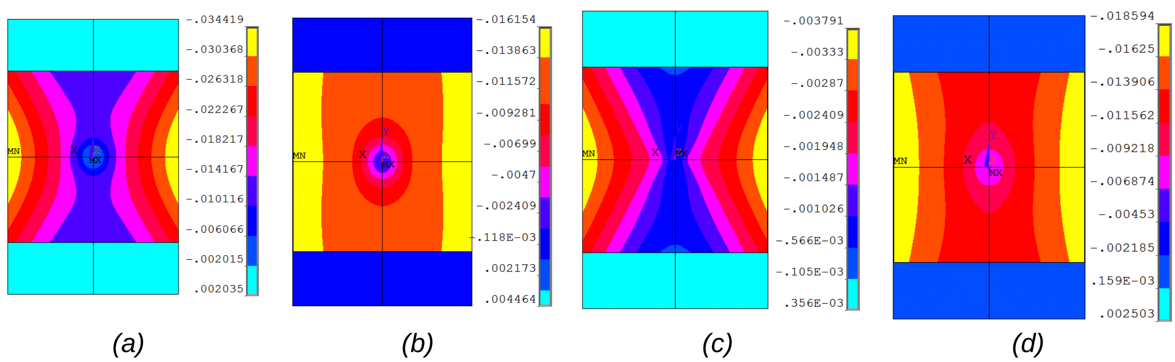


Fig. 4: Out-of plane plate deformation during compression for t equal to: (a) t=3 mm, (b) t=5 mm, (c) t=5 mm without plate contact and (d) t=8 mm

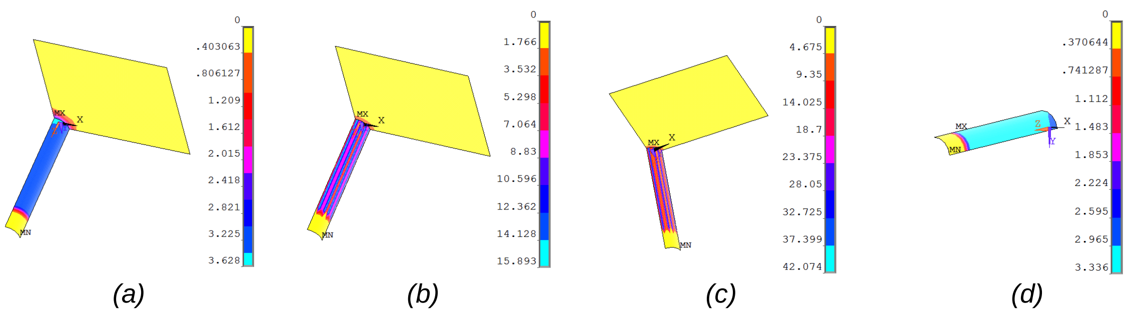


Fig. 5: Contact pressures and friction at interfaces at ultimate compressive load state for: (a) t=3 mm, (b) t=5 mm, (c) t=8 mm and (d) t=5 mm without plate contact

Moreover, the out-of-plain deformation during compression, on the distinct points ***2, 3, 4*** and ***5*** (Fig. 6) of the structure are depicted in the graphs of Fig. 7.

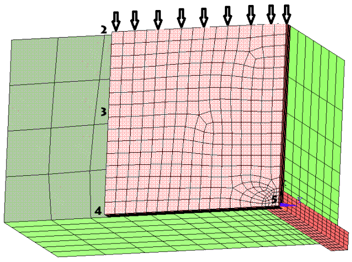
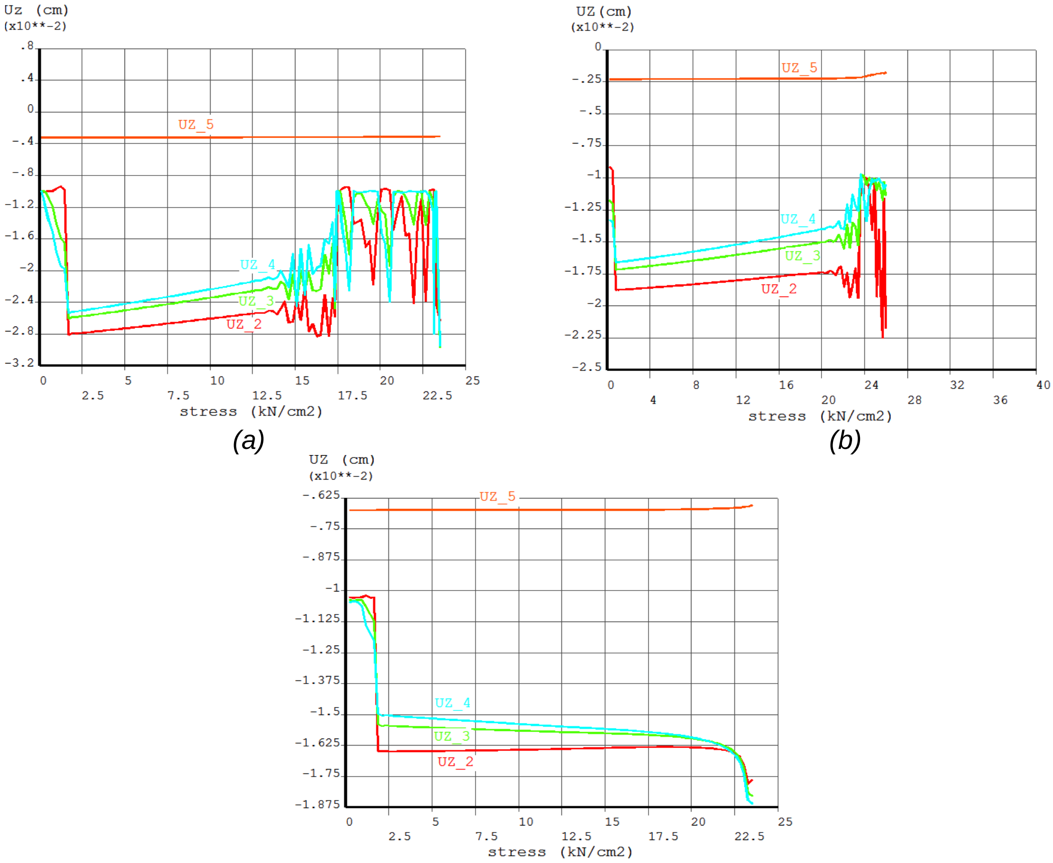


Fig. 6: Steel plate Point definition



(a) (b)

(c)

Fig. 7: Out-of-plane displacement curves for: (a) t=3 mm, (b) t=5 mm, (c) t=8 mm

5. COMMENTS

As it is illustrated in the figures, the higher the thickness, the more severe the stress concentration on studs and on masonry holes (Fig. 3). Moreover, the steel plates in compression behave elastically until the yield stress is reached, followed by the onset of plastic strains around the stud area (Fig. 3). If the load continues to rise, the plastic strains magnitude escalates, hindering the steel plate from reaching the ultimate load. On the other hand, in tension, the ultimate load can be partially reached. This might be explained by the stress variation along the section of the steel plate caused by stud connector support. This can be expressed by the induction of an "η" factor that can be seen in Fig. 8. This factor expresses the ratio of the tensile load stress reached at the last convergent solution to the ultimate tensile stress of steel plates f_u. As can be seen, the influence of stud connectors diminishes as the plate thickness becomes greater. It should be mentioned that the short scale of the analysis data, as well as the lack of experimental data on this sector to verify the results, hinders the certain definition of “η” factor. The contact stress pressure is transferred to masonry locally, posing potential threat for loosening of the hole with the appearance of diagonal cracks. The out-of-plane displacements in compression and the buckling behavior, in general are highly affected, as expected, from the differentiated plate boundary constraints applied and from the plate thickness. For thin plates (Fig. 4(a)), this variation can lead to a lower critical buckling load and to less effective steel area. This behavior is depicted by the curves in Fig 7(a), where the fluctuation of out-of-plain deformation becomes greater at an earlier load step and for a lower plate thickness. Given the sectional properties of SSRM, the lateral behavior of the global model relies on two diagonal strips, one in compression and the other in tension, with different ultimate mechanical properties as shown in Fig. 9 and Table 2. The bearing capacity of SSRM, as far as lateral loads are concerned, may be regarded as an aggregate of the bearing capacity of the constituent materials.

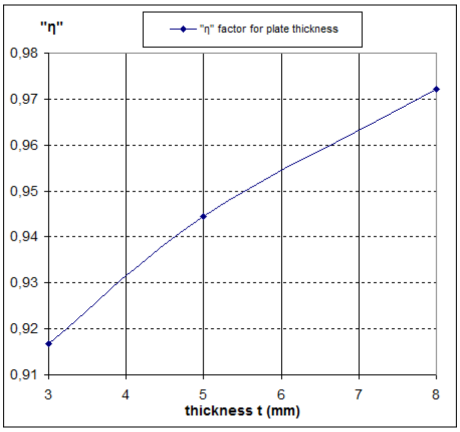


Fig. 8: Tensile stress reduction factor induced “η” along thickness

Following the European regulations [4,5], Masonry Shear strength and Steel Plate Axial strength are given by eqs. (3) and (4):

$$f_{vk} = 0.5 \cdot f_{vko} + 0.4 \cdot \sigma_d \quad \text{, but not greater than } 0{,}045 f_{b,} \text{ for “low quality” masonry} \tag{3}$$

$$f_y \quad \begin{cases} \leq \sigma_{cr} = K \cdot \dfrac{\pi^2 \cdot E_s \cdot t_s^{\,2}}{12 \cdot b^2 \cdot (1 - v^2)}, & \text{for steel plates in compression} \\ \eta \cdot f_u, & \text{for steel plates in tension} \end{cases} \tag{4}$$

where:

f_{vk}	is the characteristic shear strength,	K	is Euler buckling co-efficient,
f_{vk0}	is the characteristic initial shear strength,	E_s	is steel Young Modulus,
		t_s	is steel plate thickness,
σ_d	is the design compressive stress,	b	is steel plate width,
f_b	is the normalized unit compressive strength,	v	is steel Poisson’s ratio,
		f_y, f_u	are steel yield & ultimate strength stress.

It should be mentioned that the shear strength of masonry is purely based on its compressive strength. In the calculations, the “low quality” estimations have been applied in eq. (3). Masonry tension strength is considered negligible and thus omitted from any calculations. On the other hand, steel plate compression strength is highly related to the

Euler buckling coefficient “K”. This leads to higher plate thicknesses, but also to high local pressures at masonry holes. The equilibrium between those two states governs the application of SSRM through the model in Fig. 9. In this mechanism two sections, one in compression “S,c” and the other in tension “S,t” are responsible for shear transfer.

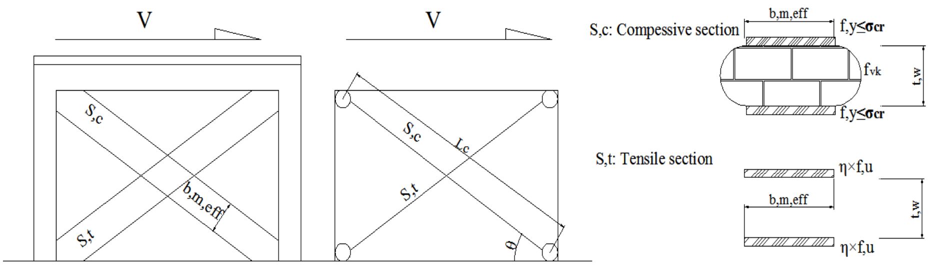


Fig. 9: Sandwich Steel Reinforced masonry global model

Sectional branch	Masonry	Steel Plates
Compressive	f_{vk} $(L_{,c} \cdot t_{,w})$	$2 \cdot f_y\,(t_s \cdot b_{,m,eff})$ with $f_y \leq \sigma_{cr}$
Tensile	-	$2 \cdot \eta \cdot f_u\,(t_s \cdot b_{,m,eff})$

Table 2: Theoretical shear contributions via diagonal mechanism for each material

With the SSRM, while the compressive wall strut is reinforced, the structure benefits also from the tensile strength of steel at the tensile strut. Thus, the struts have different sectional properties for each load state and their theoretical target value is shown in Table 2. There are still issues, as in every retrofitting method, in order for this mechanism to be applied, such as the plate anchorage and foundation enlargement. However, the proposed method appears to have many advantages. Through further research, this could be a competitive method of concrete frame building retrofit.

6. REFERENCES

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ΜΕΛΕΤΗ ΤΗΣ ΧΡΗΣΗΣ ΔΙΧΑΛΥΒΔΙΝΩΝ ΕΝΙΣΧΥΣΕΩΝ ΣΕ ΤΟΙΧΟΠΟΙΙΑ

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ΠΕΡΙΛΗΨΗ

Η ανάγκη για αναβάθμιση κτηρίων μεγάλης σπουδαιότητας (Νοσοκομεία, Μουσειακοί Χώροι) σε σεισμογενείς περιοχές αποτέλεσε κίνητρο για την εφαρμογή ποικίλων μεθόδων ενίσχυσης του φέροντα οργανισμού τους. Η εργασία εξετάζει μια εναλλακτική μέθοδο ενίσχυσης των τοιχοπληρώσεων, μέσω της εφαρμογής πλακών χάλυβα σε μορφή σάντουιτς. Με τον τρόπο αυτό μελετάται η ευεργετική συμπεριφορά της συνεργασίας των δύο υλικών στο λυγισμό (τοπικό, μέλους) των χαλύβδινων πλακών και στη διατμητική δυσκαμψία της τοιχοπλήρωσης. Για τον λόγο αυτό, δημιουργήθηκαν μοντέλα πεπερασμένων στοιχείων κατάλληλα διαμορφωμένα και φορτισμένα υπό στατική φόρτιση θλίψης και εφελκυσμού. Η δι-χαλύβδινη διατομή μικρού πάχους εφαρμόζεται αμφίπλευρα στην τοιχοπλήρωση με την βοήθεια κυλινδρικών συνδετήριων ήλων (κοχλιωμένων και συγκολλητών ανά πλευρά). Η δι-χαλύβδινη ενίσχυση διατρέχει σε μορφή χιαστί ράβδων το φάτνωμα με αποτέλεσμα η φόρτισή της να είναι κυρίως αξονική. Για τους σκοπούς της έρευνας δημιουργήθηκαν κατάλληλα διαμορφωμένα μοντέλα ανάλυσης, μικρής κλίμακας, στο πρόγραμμα πεπερασμένων στοιχείων ANSYS v12.1 έτσι ώστε να αναδειχθούν τα μηχανικά χαρακτηριστικά αυτής της μορφής ενίσχυσης.

Τα αποτελέσματα της έρευνας συνοψίζονται στην συγκέντρωση τάσεων στη διεπιφάνεια τοιχοπλήρωσης-συνδετήριου ήλου και στην αστοχία σε εφελκυσμό προτού το σύνολο της χαλύβδινης διατομής-πλάκας φθάσει στο όριο θραύσης. Το ποσοστό παραλαβής οριακής τάσης εφελκυσμού φαίνεται να έχει σαφή επιρροή από το πάχος της χαλύβδινης πλάκας και συγκριτικά εκφράζεται με την εισαγωγή συντελεστή «η». Τέλος, με βάση τα συμπεράσματα από την παρούσα έρευνα καθώς και την συνεισφορά σύγχρονων κανονισμών, μελετάται η θεωρητική αντοχή της δι-χαλύβδινης ενίσχυσης της τοιχοποιίας, με βάση το μοντέλο του Χιαστί θλιπτήρα - ελκυστήρα.