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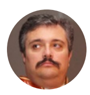

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3D Finite element analysis of end - plate steel joints

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Abstract. This paper presents a numerical investigation of the mechanical behaviour of extended end - plate steel connections including comparison with full size experiments. Contact and friction laws have been taken into account with nonlinear, three dimensional finite element analysis. Material and geometric nonlinearities have been implemented to the model, as well. Results are then compared with experimental tests conducted at the Jordan University of Science and Technology. According to the most significant observation of the analysis, a separation of the column flange from the extended end - plate occurs. Other important structural parameters of the connection, like the impact of some column stiffeners on the overall response, local buckling of the column and friction of the beam to column interface, have been examined as well.

Keywords: bolted steel connections; contact - friction; finite element analysis; end - plate; 3 dimensional analysis.

1. Introduction

In the present work the behaviour of extended end - plate, full scale steel connections is examined. Eight high strength bolts are used to connect the extended end - plate to the column flange. A nonlinear three dimensional finite element model has been created, including contact - friction laws for the simulation of the nonlinear interaction along contacting parts.

Steel joints were usually designed as fully rigid or pinned ones in the past; on the other hand, it has been recognized that most forms of them actually exhibit a semi - rigid behaviour and a finite rotational stiffness, as well. The investigation of steel connections can be generally divided into three modelling types: (a) analytical or empirical models, (b) mechanical models and (c) advanced finite element models. Analytical or empirical models (Kishi and Chen (1990), Wu and Chen (1990), Attiogbe and Morris (1991), Abolmaali *et al.* (2005)) usually consider simple expressions and they could be used for a quick, preliminary assessment. Mechanical models (Silva and Coelho (2001), Yee and Melchers (1986), Pucinotti (2001), Lemonis and Gantes (2009)) use springs and rigid elements; they are more complex and accurate than the analytical or empirical ones, but they have to make some simplifications regarding the mechanical behaviour.

Finite element models can be more accurate than the aforementioned modelling techniques and represent in detail the semi rigid behaviour of steel joints, but they exhibit an increased computational

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cost. Krishnamurthy (Krishnamurthy (1975), Krishnamurthy and Graddy (1976), Krishnamurthy (1978)) was one of the first researchers who studied the implementation of the FEM analyses to the extended end - plate connections, initially by using elastic material behaviour. Moreover, he developed a two dimensional plane stress FEA model of a bolted connection and incorporated in this model nonlinear material properties and the pretension of the bolts. Choi and Chung (1996) presented a three dimensional finite element model for the description of the behaviour of end - plate connections. They included in their model elasto - plastic solid elements in order to model the bolt's shank, head, and nut. Gantes and Lemonis (2003) proposed a three dimensional finite element model for the simulation of simple T-stub steel connections. They implemented material and geometric nonlinearities as well as contact and friction laws. Other researchers who studied the structural behaviour of similar types of steel connections, are referred to Abdalla and Stavroulakis (1989), Bursi and Jaspart (1997), Maggi *et al.* (2005), Shi *et al.* (2008), Saravanan *et al.* (2009), Kiymaz (2009).

From another point of view, experimental investigation of different types of steel connections (Baniotopoulos (1995), Mistakidis *et al.* (1996), Gebbeken *et al.* (1994), Meng (1996), Girao Coelho *et al.* (2004), Girao Coelho *et al.* (2004)) fulfils the study of these structural elements and it is used for the assessment of the above mentioned models. Meng (1996) tested extended end - plate connections under cyclic loading. Hot - rolled beam sections were used to establish the end - plates tests. Girao Coelho *et al.* (2004) conducted an experimental study in eight statically loaded end - plate moment connections. The specimens were designed to develop failure in the end - plate or bolts and prevent the development of full plastic moment on the beam. According to the results, an increase in the end - plate thickness caused an increase in the connections flexural strength and stiffness and decrease in rotation capacity, as well. In another study, Girao Coelho *et al.* (2004) performed thirty two tests on isolated T-stub connections composed of welded plates. Their primary goal was to investigate the behaviour of this type of connection, it's failure modes and deformation capacity, as well. Abdalla *et al.* (2007) performed six tests on extended end - plates to evaluate the force distribution on bolts and to obtain the moment - rotation curves. In addition, Pedro Nogueiro *et al.* (2009) developed a model reproducing the cyclic response of composite (steel - concrete) connections. Experimental research on composite joints subjected to cyclic loading took place, for the calibration of the proposed model.

A three dimensional nonlinear finite element analysis model is developed in this study. The model simulates the behaviour of full scale, extended end - plate steel connections which had been subjected to static loading at the Structural Laboratory of the Jordan University of Science and Technology. In such a structural joint characteristic zones of tension, compression and shear forces can be developed, resulting in the opening and/or sliding of the connected structural parts. To depict this behaviour, the finite element model incorporates principals taken from nonsmooth mechanics, in particular unilateral contact law for the opening and friction law for the sliding, between the column flange and the beam's end - plate. The comparison between the numerical model and the experimental data shows a satisfactory convergence.

The model developed in this article has been used for the examination of significant structural parameters of the steel joint, such as the influence of some stiffeners placed in the column, as well as the areas of the beam to column interface where contact opening or sliding occurs. According to this, separation of the column flange from the extended end - plate occured, almost until the third row of bolts at the lower part of the connection; below this level the two parts remained in full contact. Consequently, the neutral axis of the interface seems to be close to the line connecting the third pair of bolts. Finally, a parametric investigation concerning the influence of some contact parameters on the behaviour of the connection has been performed.

2. Experimental data

According to the experimental study conducted in Abdalla *et al.* (2007), the behaviour of six full scale, three stiffened and three unstiffened, cantilever connections of HEA- and IPE- sections had been investigated. In Fig. 1 the geometry of the steel connection and some characteristic dimensions (in m) are shown, as well.

Eight high strength M20 bolts grade-8.8 ($F_y = 640$ N/mm^2 and $F_u = 830$ N/mm^2 nominal values) having nominal diameter of 20 mm (21mm hole diameter) were used for the connection of the extended end - plate with the column flange, see Fig. 2. For each test, an IPE-360 beam section was attached to an HEA-220 column section (Fig. 3) through an extended end - plate. Beams, columns, and end - plates were made of steel having average yield and ultimate stresses $F_y = 314$ N/mm^2 and $F_u = 450$ N/mm^2, respectively, obtained from three different uniaxial coupons tests. In fact, experimentally obtained stress - strain laws for both the steel parts (column, beam, end - plate) and the bolts, are shown in paragraph 4 of this article. Frames were categorized as: columns with stiffeners (KGS-2S, KGS-4S, and KGS-5S) and without stiffeners (KGS-1, KGS-3, and KGS-6) both having identical mechanical properties, see Figs. 4, and 5, respectively. In Fig. 3 the geometry of the beam and the column sections is represented. The system was supported against side movements, with three beams tied together with the column, as shown in Fig. 1. Finally, in Fig. 6 are shown the basic measurement points, which were used to measure the beam deflection at three locations. The external load is applied at point 3, thus 1.158 m away from the end - plate of the beam.

3. The nonlinear model including unilateral contact and friction laws

To study the nonlinear behaviour of the aforementioned steel joint, a three dimensional nonlinear finite element model has been prepared. Unilateral contact law in the normal direction of the interface between the extended end - plate and the column flange, simulates the possible opening of the connected parts. Coulomb friction law is adopted to depict the contribution of the friction to the shear strength of the interface, together with the one obtained by the bolts.

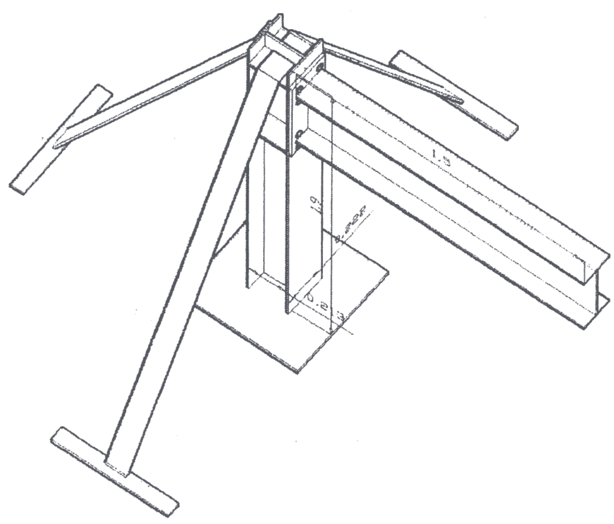
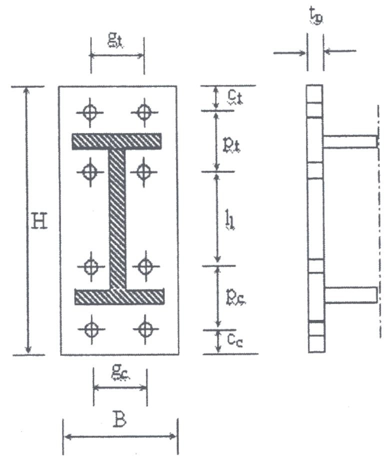
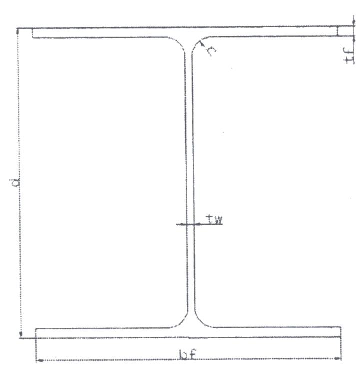


Fig. 1 Geometry of the steel connection (stiffened)



Thickness t_p (mm)	B (mm)	H (mm)	c_t (mm)	p_t (mm)	l_l (mm)	p_c (mm)	c_c (mm)	g_t (mm)	g_c (mm)
15	218	528	43	90	262	90	43	140	140

Fig. 2 Geometry of the end - plate and positioning of bolts



Beam section IPE360									
Depth d (mm)	b_f mm	t_w mm	t_f mm	A mm^2	I_x mm^4	I_y mm^4	Wt kg/m	L mm	r mm
360	170	8	12	6768	1.4886×10^8	9.8403×10^6	53.1965	1500	18

Column section HEA220									
Depth d (mm)	b_f mm	t_w mm	t_f mm	A mm^2	I_x mm^4	I_y mm^4	Wt kg/m	L mm	r mm
213	222	7	10	5791	4.9973×10^7	1.8241×10^7	45.5173	1900	18

Fig. 3 Geometry of beam and column sections

Here a short description of the unilateral contact and Coulomb friction laws and the way we have treated them numerically will be discussed. At each point of an interface the basic unilateral contact mechanism can be described by the following set of relations:

· The no - penetration inequality

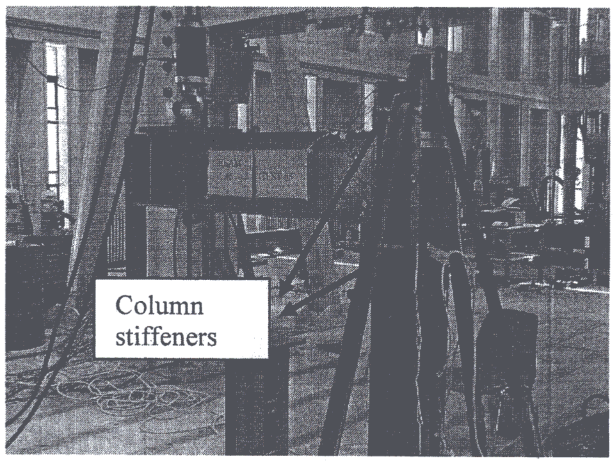


Fig. 4 Specimen with column stiffeners

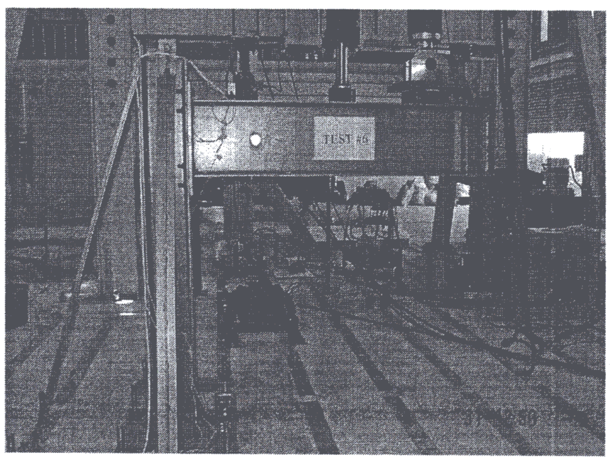


Fig. 5 Specimen without column stiffeners

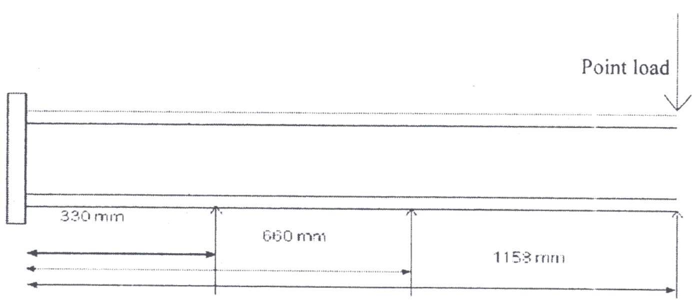


Fig. 6 Measurement points

- The no - tension inequality, as well as
- A complementarity, either - or relation, indicating that either separation with zero contact force or compressive contact force with zero gap appears.

For a discretized structure the previous relations are written for every point of a unilateral boundary or interface by using appropriate vectors. The arising nonsmooth structural analysis problem has the form of a nonlinear complementarity problem. More details can be found, among others, in, Panagiotopoulos (1985), Stavroulaki and Stavroulakis (2002).

The behaviour in the tangential direction is defined by a static version of the Coulomb friction model that exhibits a similar either - or, variable structure, unilateral behaviour:

- No - slip is enforced, if the frictional force is within limits, and
- Slip with constant friction force is allowed at the limits.

Similarly with the contact problem, a complementarity problem arises for friction.

The one - sided (unilateral) behaviour, due to contact, and the stick - slip mechanism of friction transform the usual variational equality formulation of the mechanical problem into a variational inequality (Panagiotopoulos (1985)). From the area of nonsmooth mechanics we have chosen suitable, stable techniques for the numerical solution. In particular the contact constraints have been enforced with Lagrange multipliers. The friction problem can, in principle, be solved by similar methods. For reasons that are related to the physical instability of friction and to the numerical stability of the solution, this problem has been solved here by means of a smoothing technique (see Stavroulaki and Stavroulakis (2002), Christensen *et al.* (1998), Stavroulakis and Antes (2000)). Finally the coupled problem has been solved by the Newton – Raphson incremental iterative procedure.

4. The finite element model

Among the goals of this article, is the study of steel joints with common computational tools. For this reason, commercial finite element analysis packages (Abaqus and Marc) have been used by the authors.

For the proposed finite element model three dimensional 8 - node brick elements have been used. In order to avoid the shear locking effect, reduced integration brick elements have been used. From another point of view, the depth of each steel part (especially the column web, column flange and the beam's end - plate) has been simulated with two finite elements. For the satisfactory convergence of the analysis, a quite small average element size has been applied, resulting in a large number of finite elements (107326). The mesh which has been applied in the model is denser around the area of the connection while becomes less dense away from it. In Fig. 7 the mesh of the structure is shown.

The contact and friction laws have been implemented to the interface between the end - plate of the beam and the column flange. Thus, opening or sliding of the interface can be depicted. The friction coefficient for the beam - column interface is equal to 0.4. The two surfaces of this interface have been considered to be in contact in the start of the analysis. For the sake of simplicity, the interface between the bolt shank and the plate has not been considered, while interfaces between the bolts' head and nut and the plate are simulated as tie connections which do not permit either sliding or opening. In Fig. 8 the structural parts of the proposed model, e.g., the column (a), the beam (b) and the bolts (c) are shown.

For the representation of the steel yielding the von Mises plasticity model has been adopted. The average yield and ultimate stresses for the beam - column are $F_y = 314$ MPa and $F_u = 450$ MPa and for the bolts $F_y = 600$ MPa, $F_u = 800$ MPa, respectively. For the calibration of the model with experimental results no pretension has been applied to the bolts. However pretension has indeed been considered in the parametric investigation of the structure. In general, pretension forces in commercial packages are applied by implementing tensile forces in bolt shank. Starting from one section of the shank of the bolt, which remains steady, tensile forces are applied to the whole shank, enforcing the bolts head and nut to come closer. Moreover, a vertical concentrated load is applied to the beam, 1.158 m away from the face of the end - plate of the beam.

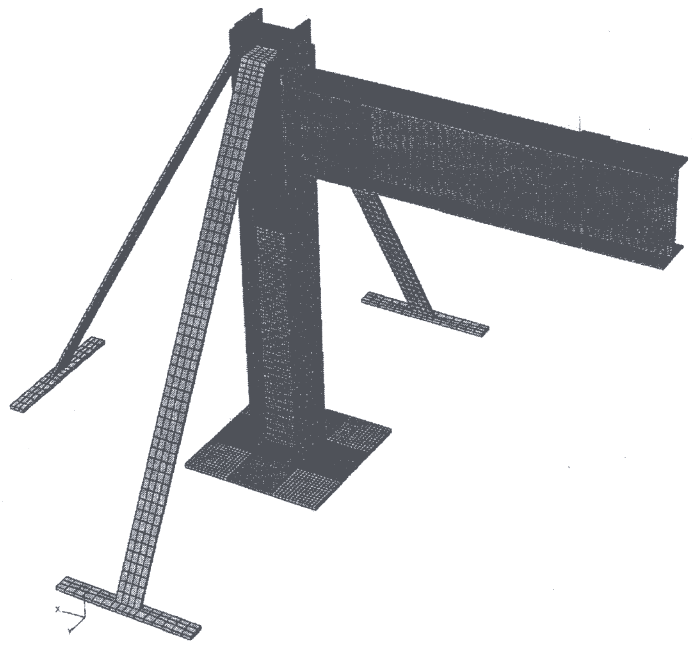


Fig. 7 Mesh of hexahedral (brick) 8 - noded finite elements

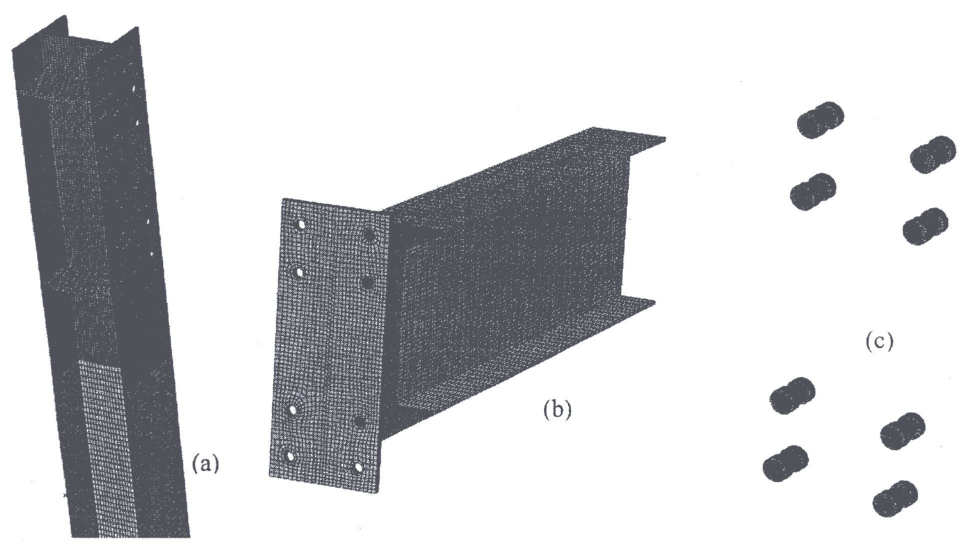


Fig. 8 Structural parts of the connection (a) column, (b) beam and (c) bolts

The Young's modulus for the beam - column and the bolts is approximately equal to 120 GPa, as it is shown in coupon tests represented in Figs that follow. This value seems to be relatively small, may be due to an error in the measurements that could not be fixed earlier. Nevertheless, the results depend more on other mainly nonlinear effects and are relatively insensitive with respect to the Young's modulus, thus, in case a slightly higher value was adopted, the corresponding results are not significantly influenced. The Poisson's ratio for the whole structure is taken equal to 0.3. Finally, geometrical nonlinearity (large displacement analysis) has been considered for the structure, which has been proved to be a reasonable assumption, according to the results.

The above mentioned mechanical properties of the materials used in the connection, have been

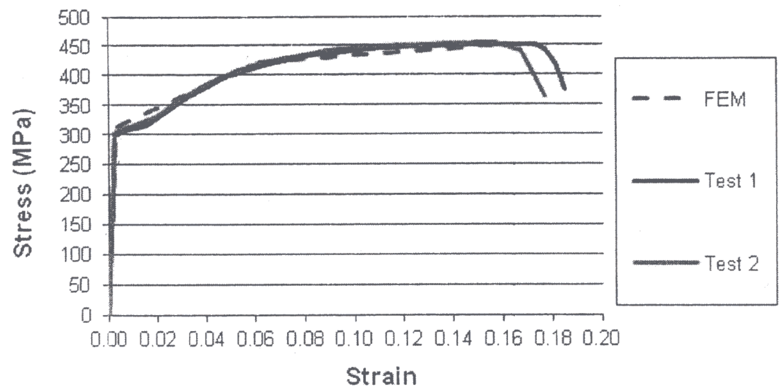


Fig. 9 Average Stress - strain results obtained from column, beam and end - plate coupon tests - Stress - strain law adopted for the numerical analysis

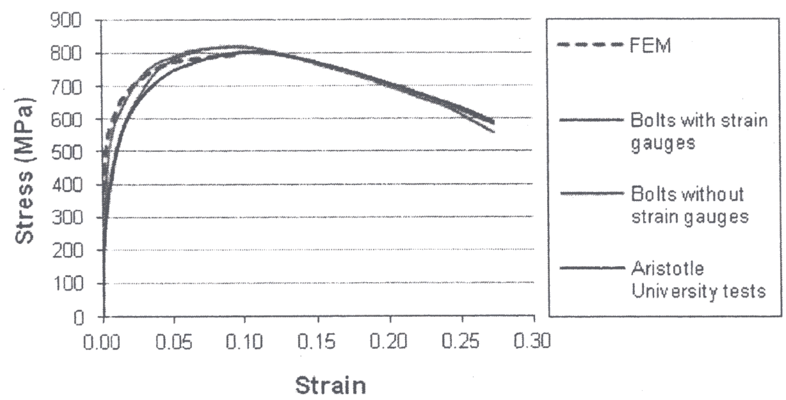


Fig. 10 Stress - strain results obtained from three bolts, tested with strain gauges, without strain gauges and in Aristotle University - Stress - strain law adopted for the numerical analysis

adopted from coupon tests conducted to the steel parts (column, beam, end - plate) and the bolts, as well. Figs. 9 and 10 represent the experimental data as well as the stress - strain laws which have been adopted for the numerical analysis.

5. Results

The most important observation in the results obtained from the finite element analysis, is the appearance of separation along parts of the interface between the column flange and the end - plate of the beam, together with a small sliding of the interface, for higher levels of load near the ultimate (limit) or collapse loading. This separation takes place approximately until the third row of bolts, low in the connection (there are four rows of bolts which connect the two steel parts, see Fig. 2). Moreover, the sliding in the interface occurs around each bolt in a small area, where the two parts remain in contact due to the presence of bolts.

Another important comment is related to the influence of the column stiffeners on the structural behaviour of the connection. The model with stiffeners shows a smaller opening of the interface, a higher limit load and a higher inclination of the load - displacement curve (indicating higher total stiffness), in comparison with the one without stiffeners.

Finally, in both models local buckling of the column flange and column web were observed, as well. Similar results are obtained from the experimental research on the same specimens.

5.1 A general look to the connection's mechanical behaviour

In Fig. 11(a) the deformed shape of the structure with stiffeners obtained from the finite element analysis, subjected to the ultimate, limit load is shown. Opening of the beam to column interface, as well as yielding of the steel (light colour) are shown. A similar image is received from the experiment (Fig. 11(b)). When the experimental connection is close to failure, for a small increase of the point loading which is applied to the beam, we get a significantly increased vertical displacement of the beam. The structure then reaches it's ultimate strength and the experiment is terminated. Similarly, in the framework of finite element analysis as the point load is increased, opening of the interface takes place, yielding of the steel parts occurs together with the increase of the vertical displacement of the beam and the force - displacement diagram tends to become horizontal, indicating that the structure is close to collapse. This point, which is characterized with at least one zero eigenvalue of the tangential stiffness matrix, is the end of a path of stable mechanical equilibrium which in fact has been obtained with a monotonic application of the loading in the laboratory. The numerical tools are more general and can proceed further, following branches of unstable solutions in analogy to post - buckling effects, using for example tools like arc - length techniques. We have not done this investigation, which would require application of displacements instead of forces in the experiment, because we do not have experimental results of this type.

The limit load obtained from the proposed method for the model with stiffeners is equal to 127.7 KN which is close with the experimental one (112.5 KN). In Fig. 12 force - displacement diagrams from both the experimental research and the finite element models, are represented. From these it can be commented that there is a satisfactory convergence between the proposed model with stiffeners and the experiment. Moreover, the behaviour of the contact model without stiffeners is, as expected, different. In particular, the stiffness of this model is smaller in comparison with the model with stiffeners. According to Fig. 13(a) and 13(b) the opening of the interface is quite bigger in case stiffeners are not present, in contrast with Fig. 11 where stiffeners are present. It is mentioned that an identical image from the

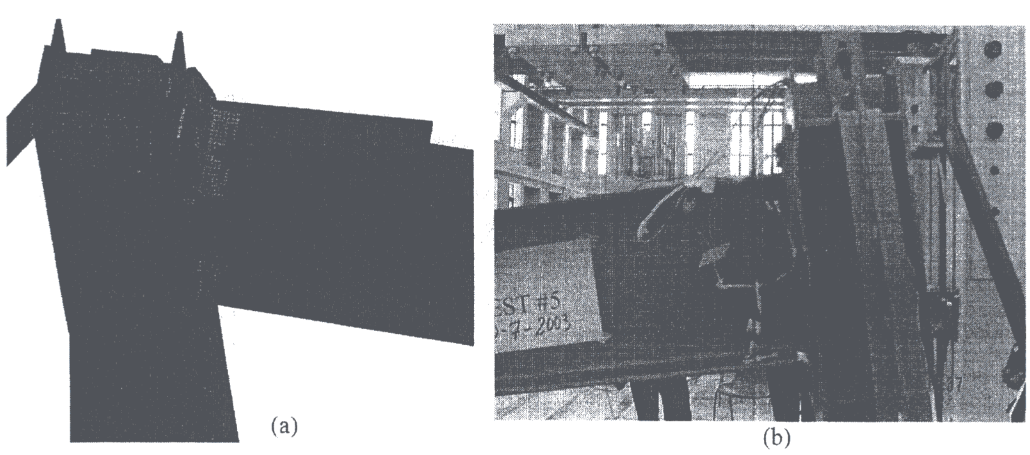


Fig. 11 Failure of the model with column stiffeners (a) finite element analysis (b) experimental research

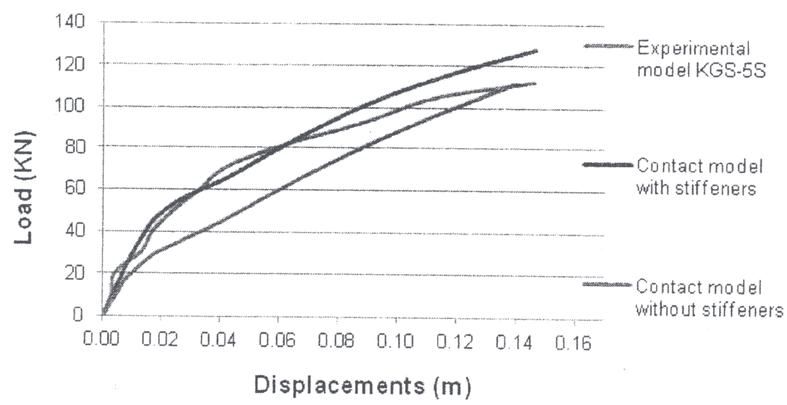


Fig. 12 Force - Displacement diagrams

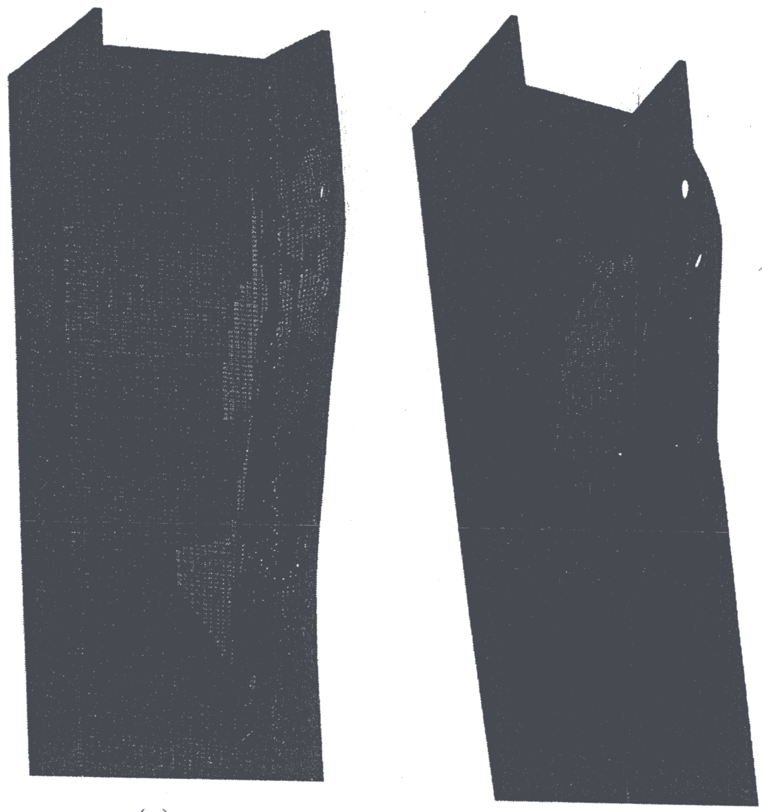
(a) (b)

Fig. 13 (a) Finite element model without stiffeners at failure (b) Experimental model without stiffeners at failure

experimental specimen without stiffeners is obtained (Fig. 13(b)).

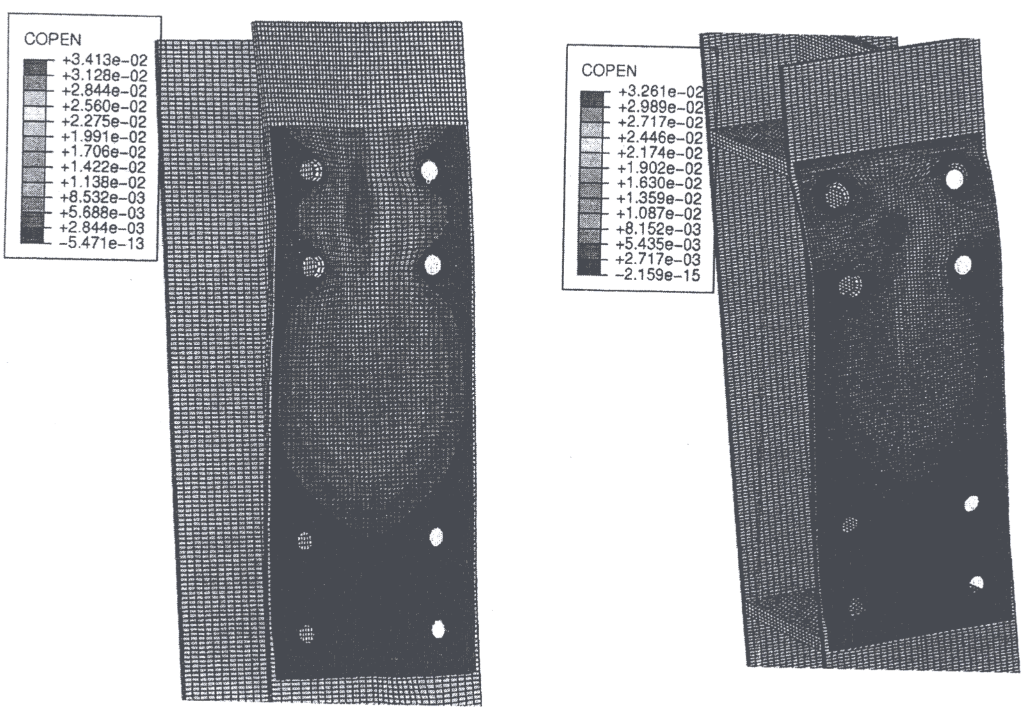
5.2 An insight to the connection's behaviour

Figs. 14 and 15 show some other differences between the models without and with stiffeners. In Fig. 14 yielding of the column near the final loading step of the analysis can be shown. In the model without the stiffeners (Fig. 14 (a)) yielding (light colour) is accompanied with local buckling of the flange, expanded until the top of the column. On the other hand, buckling of the flange for the model with stiffeners seems to be less severe and restrained to the area between the top and the bottom stiffener. Moreover, values of the opening of the interface of the connection are shown for both models at failure, in Fig. 15. For the finite element model without stiffeners (Fig. 15(a)), the opening for a region between the top of the connection and the first row of the bolts, takes values between 2.9 cm and 3.1 cm, respectively. In the centre of the upper 4 bolts, opening becomes maximum and equal to 3.4 cm; the experimental opening of the specimen without stiffeners (test KGS-6) has a maximum value equal to



(a) (b)

Fig. 14 Yielding and local buckling at failure for the model (a) without stiffeners (b) with stiffeners



(a) (b)

Fig. 15 Opening of the interface at failure for the model (a) without stiffeners (b) with stiffeners

2 cm. However, for the finite element model with stiffeners (Fig. 15(b)), the opening in the top of the connection is approximately 5 mm; according to the experimental data (test KGS-5S) there is an average value around 5 mm, at this region. For a specific region between the upper four bolts, opening of the finite element model becomes maximum and equal to 3.2 cm, which is compared with the experimental value of 2 cm. From these Figs it can be noticed that almost until the third row of bolts at the lower part of the connection, an opening between the column flange and the end - plate of the beam takes place, for both the model without and with stiffeners. Thus, the neutral axis of the interface is close to the line connecting the third pair of bolts, as it will be further explained in the next paragraphs.

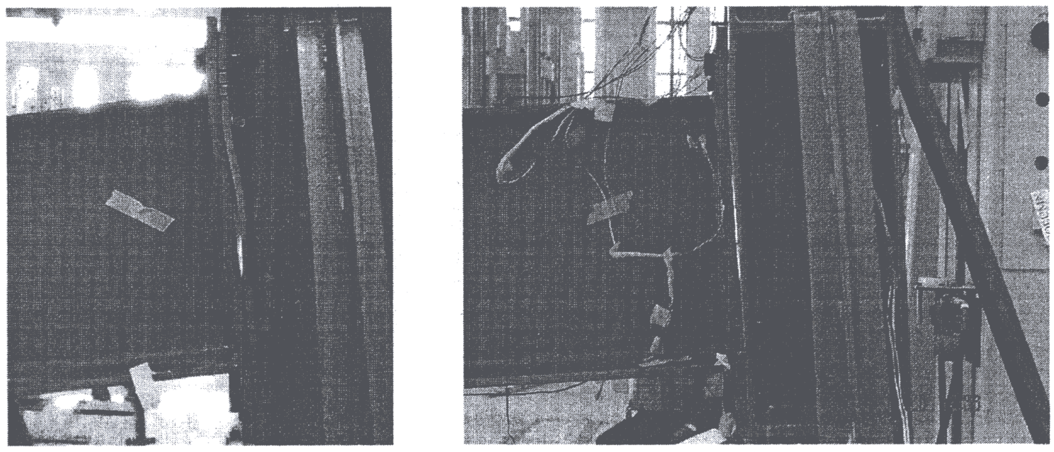
According to Fig. 15, another opening takes place in both lateral sides of the connection. It is attributed to the local buckling of the column flange, see Fig. 14. For the model without stiffeners, the opening has a maximum value of 3 mm and a length approximately equal to 16 cm (in the direction of the column's length). On the other hand, for the model with stiffeners, the maximum value is 2.4 mm and the length is not significantly changed. A similar observation is recorded from the experimental research. In Figs. 16 and 17, are represented analysis as well as experimental results, concerning the above mentioned comment. Thus, it can be shown that the lateral opening is minimized, in case stiffeners are present (Figs. 16(b) and 17(b)).

In Fig. 18 sliding along the interface is shown, for both the model without and with stiffeners. According to this, sliding occurs mainly in a small area surrounding each hole of the extended end - plate of the beam. It is worth noticing that sliding can be depicted only in areas where contact, thus no opening of the interface occurs, see Fig. 15.

In Fig. 19(a) and 19(b), deformations of the column flange obtained from both the FEA and the experimental models with stiffeners, are shown. These displacements have been measured by the metrological department of the laboratory. In both images, the surface of the column flange is deformed in a similar way, in the ultimate (limit) load of the joint. However, it seems that the finite element model overestimates the deformation of the area around the top of the connection, in comparison with the experiment.

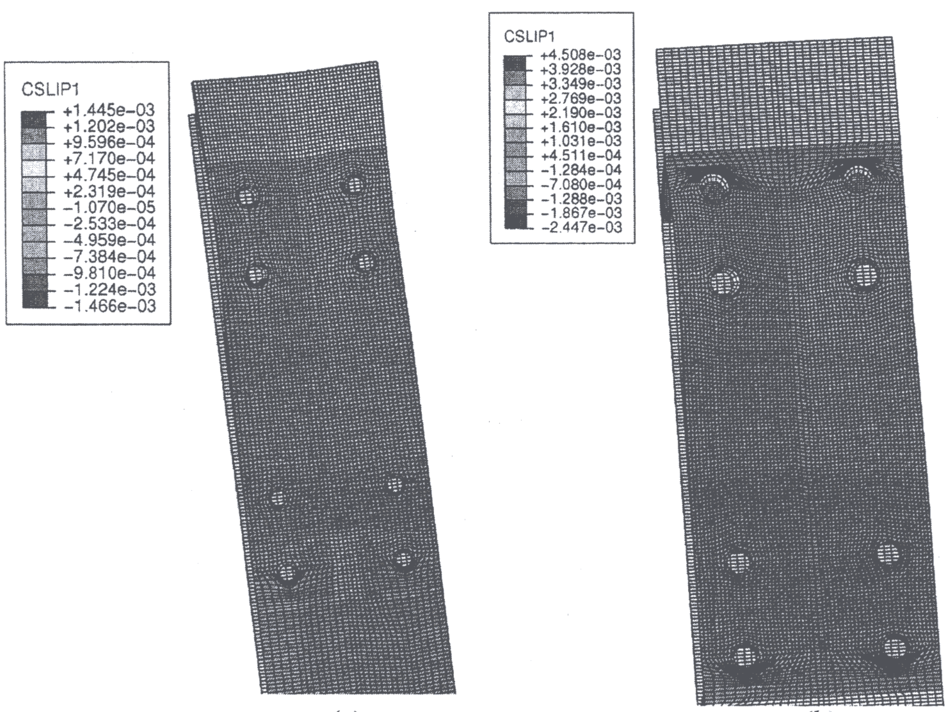
(a) (b)

Fig. 16 Opening of the end - plate in the lateral side of the beam at failure, for the model (a) without stiffeners (b) with stiffeners - Finite element contact/friction model



(a) (b)

Fig. 17 Opening of the end - plate in the lateral side of the beam at failure, for the model (a) without stiffeners (b) with stiffeners - Experimental research



(a) (b)

Fig. 18 Sliding at failure for the model (a) without stiffeners (b) with stiffeners

Models without stiffeners lead to similar observations (Fig. 20(a) and 20(b)). In this case it is also noticed an increase in the deformation of both the experimental and the numerical model.

5.3. Comparison of the shear forces in the bolts with the friction forces

An interesting aspect of the present study is related to the proportion of shear forces transmitted through the interface by friction and through the bolts. For the sake of simplicity, the following procedures

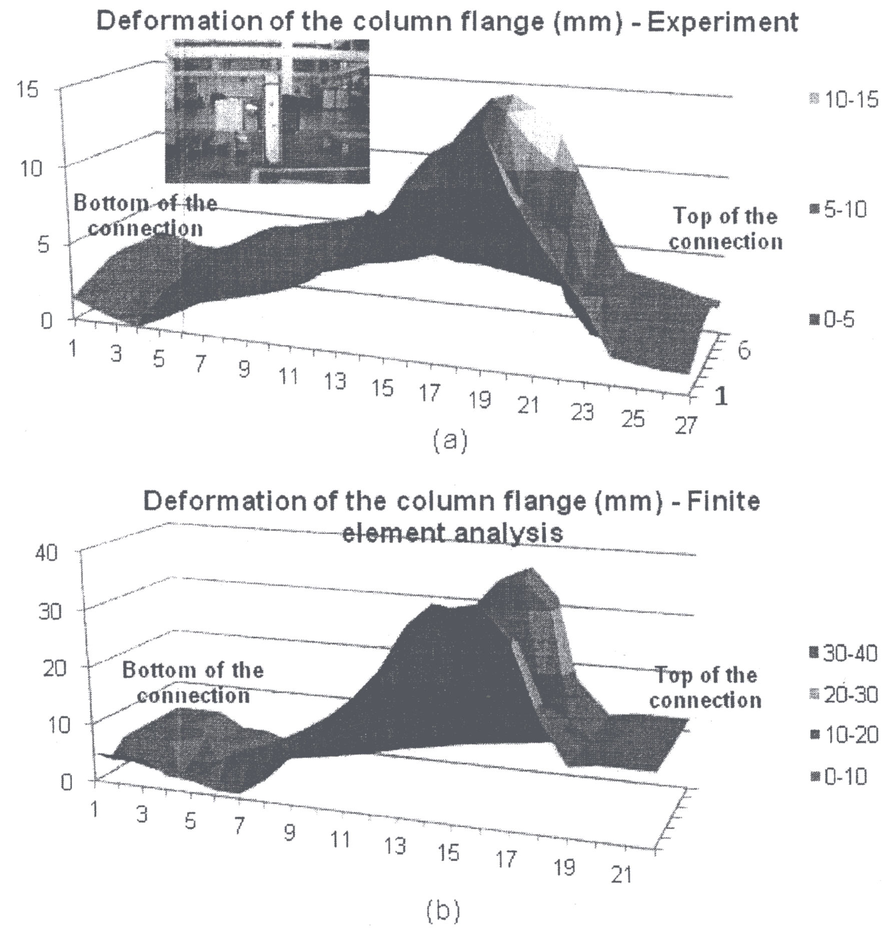


Fig. 19 Deformation of the column flange at failure, for the models with stiffeners (a) experimental research (b) finite element analysis

have been applied to the model with stiffeners, while no pretension is initially applied to the bolts.

In the framework of the finite element analysis packages, the output of the total friction force of the contact interface throughout the loading is calculated as the integration of frictional stresses in the interface. Consequently, shear forces transmitted through bolts should be found. The integration of shear stresses can be used in this case as well. Another, straightforward and simpler way is related to the study of the free body diagram of the beam. In particular, according to the free body diagram of the beam, the vertical, concentrated load applied in the beam, P, is equal with the total shear force of the connection. For the equilibrium of the connection, a part of this force is transmitted to the joint as friction force, F_f, and another part as shear force obtained by the total number of bolts, F_b, according to the following equation: $P = F_f + F_b$ (1). The forces P and F_f are given by the finite element package, consequently the shear force obtained by the total number of bolts is given by the equation (1): $F_b = P - F_f$.

Fig. 21 shows the proportion of shear forces transmitted through the bolts and through friction forces throughout the loading. For a friction coefficient equal to "μ" = 0.4, both friction and bolt shear forces have similar values until the total loading gets approximately equal to 70 KN. As the total loading is further increased, shear forces of bolts are significantly increased while friction forces remain almost constant. Bolts' shear forces are greater than the friction forces during the analysis, with a small exception (near the value 65 KN of the total force).

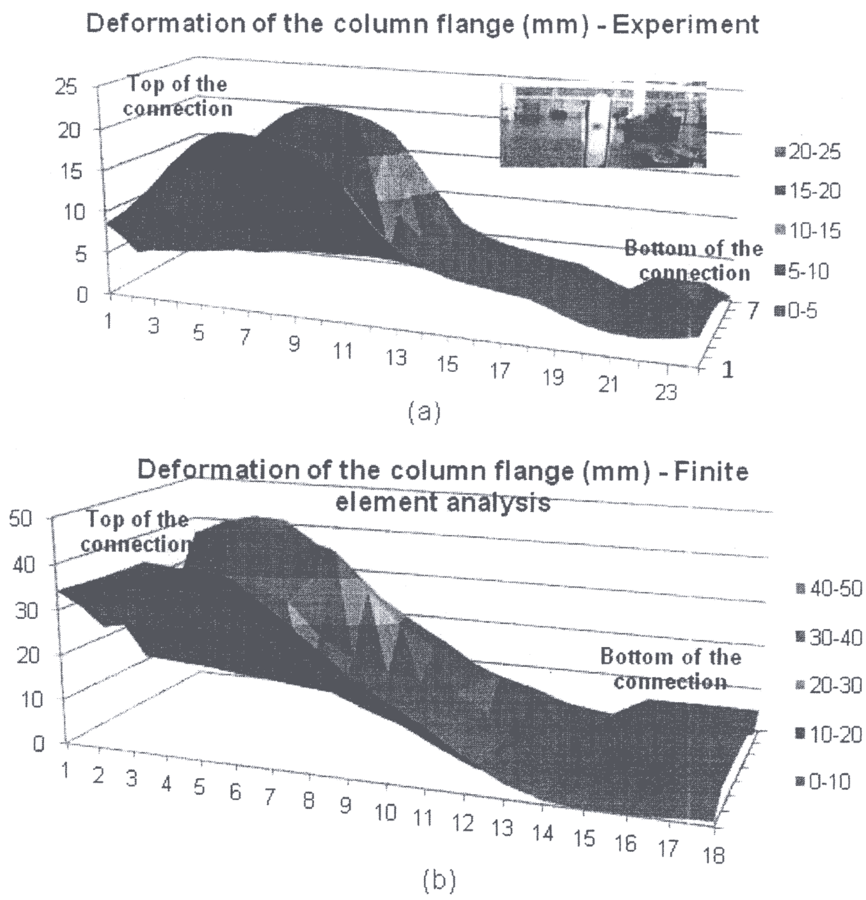


Fig. 20 Deformation of the column flange at failure, for the models without stiffeners (a) experimental research (b) finite element analysis

In Fig. 22 friction forces and shear forces of bolts in the interface are related to the slip of the interface. ı the beginning of analysis, in relatively small total shear forces, both friction forces and bolts' shear forces have almost vertical branches in the diagram. It is noted that according to the computational scheme which has been chosen for the present study, the friction constraint is enforced with the penalty method. Within this method a small sliding (approximately 0.5 mm) is permitted until shear stress t^t becomes critical and equal to $\mu x t^n$. This is the reason shear forces transmitted through bolts are not equal to zero in the beginning of the analysis. Instead, if sticking condition would be chosen for the enforcement of the friction constraint, for instance with the Lagrange multiplier method, then no slip would be permitted in the beginning, indicating that only static friction forces would be developed. As the total force is increased, slip of the interface is also increased. When slip becomes approximately equal to 1.5 mm and the total force to 70 KN, shear forces of bolts are vertically increased, while slip and friction forces remain constant. When the total force becomes equal to 110 KN slip begins to increase again, but the bolts' shear forces have become significantly greater than the friction forces of the interface.

5.4 Parametric investigation of contact parameters

In order to investigate the influence of some contact parameters on the mechanical behaviour of the connection with stiffeners, a parametric investigation has been performed. In particular, the friction

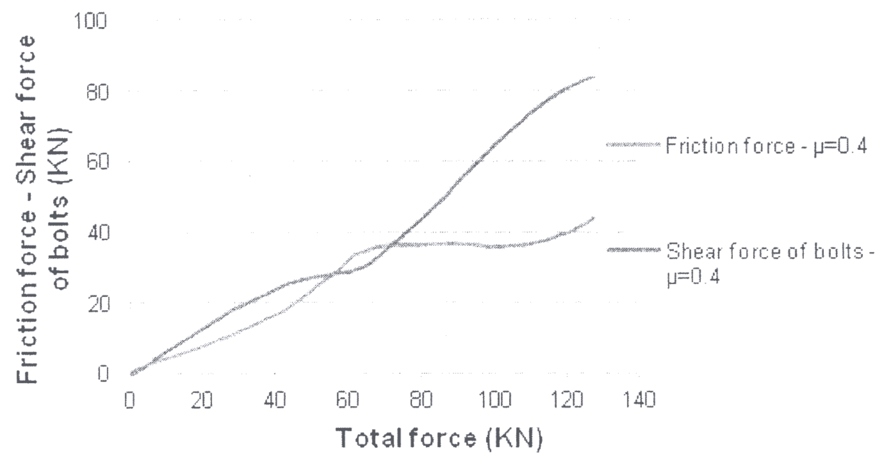


Fig. 21 Proportion of shear forces transferred through the interface by friction and through the bolts

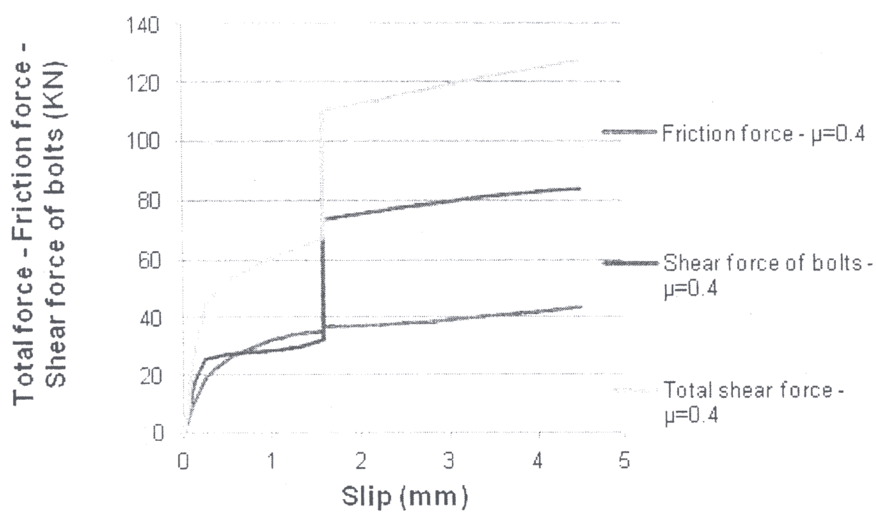


Fig. 22 Relation of shear forces transferred through the interface with slip, for friction coefficient equal to 0.4

coefficient (μ) was considered equal to 0.2, 0.4, 0.6. On the other hand, an initial opening was applied to the interface and μ was kept equal to 0.4.

Modifications of the value of friction coefficient did not change the force - displacement diagram, thus the joint had identical limit load and collapse mechanism with those obtained from the initial analysis ($\mu = 0.4$). However, some alterations in the way forces transmitted through the interface have been observed. In Fig. 23 is indicated that for a friction coefficient μ equal to 0.6 the friction forces have been increased and the bolts' shear forces reduced, in comparison with Fig. 22, where $\mu = 0.4$. In addition, for values of slip between 0.25 mm and 1.20 mm friction forces are greater than shear forces of bolts.

Considering the fact that increasing of the friction force as well as reducing of the shear force of bolts does not lead to the improvement of the overall response of the structure (e.g., a higher force - displacement diagram), it could be concluded that the connection's failure is not attributed to the shear failure of bolts, but to the failure of the steel parts (mainly to the column's failure). For further investigation somebody could perform computational tests by changing the dimensions of steel parts. For instance, it

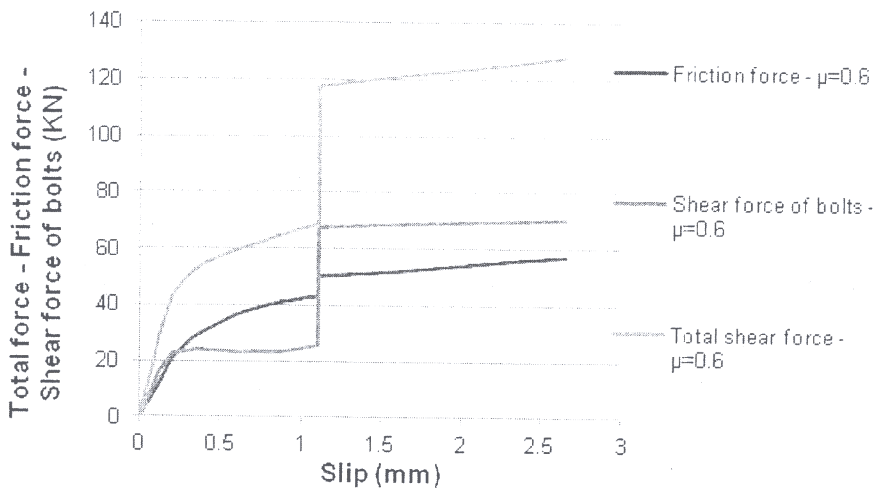


Fig. 23 Relation of shear forces transferred through the interface with slip, for friction coefficient equal to 0.6

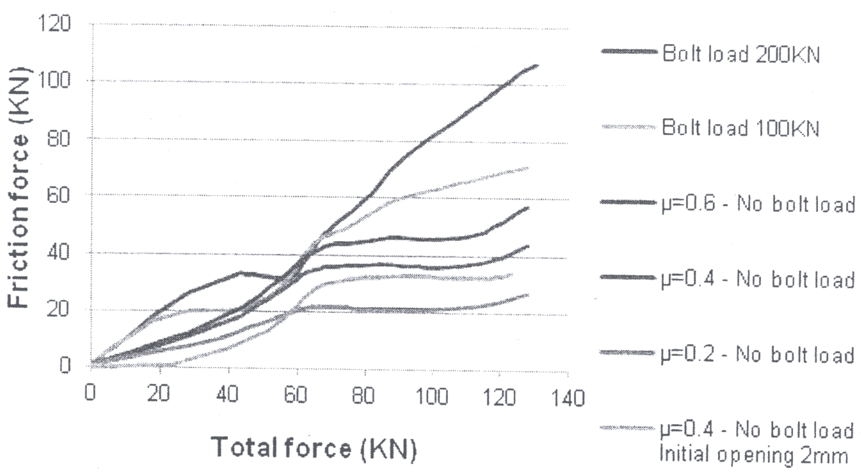


Fig. 24 Parametric investigation of friction forces on the contact interface

could be expected that if column would become stronger, then failure mode would possibly change and bolts' shear failure may become critical. However this investigation lies beyond the scope of the present study.

In order to investigate the impact of the alteration of the proportion between friction forces and bolts' shear forces of the connection, models inducing pretension bolt forces equal to 100 KN and 200 KN have been developed. In this case the friction coefficient is kept constant and equal to 0.4. Similarly with the previous comments, the force - displacement diagram does not significantly change, in comparison with the models with no pretension forces (only a small increase in the corresponding diagram is obtained for the case of a high pretension force, equal to 200 KN). At Fig. 24 is shown that the friction force of the interface is increased, as the friction coefficient and the bolt pretension load is also increased.

At Fig. 25 is shown, that for a pretension bolt load equal to 100 KN, the friction forces are greater than the shear forces of the bolts, almost for the whole range of the analysis.

Finally, an initial opening equal to 2 mm has been considered to the contact interface, as represented in Figs. 26, 27, 28. In Fig. 27 the corresponding force - displacement diagram is shown. According to this, a small reduction in the overall behaviour is obtained, in the case that an initial opening exists. A

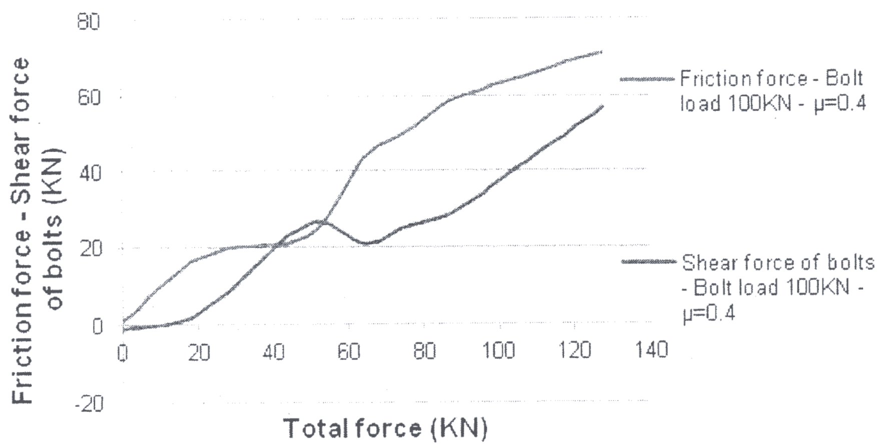


Fig. 25 Proportion of shear forces transferred through the interface by friction and through pretensioned bolts

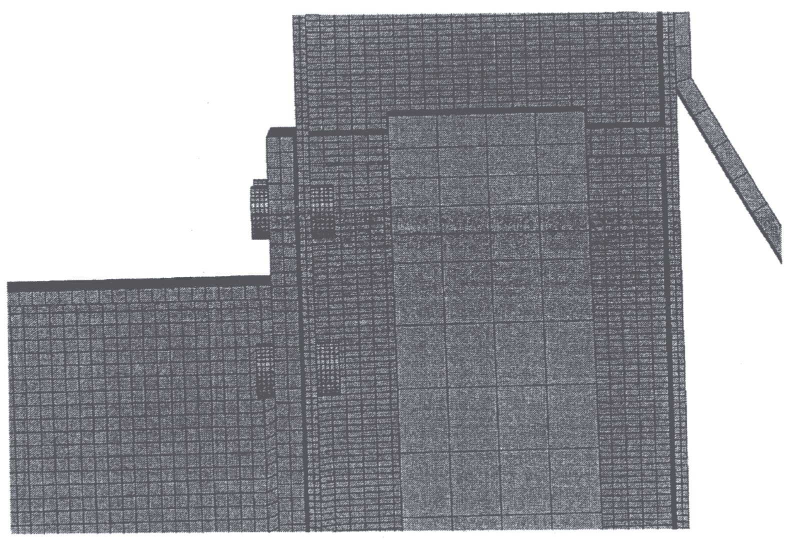


Fig. 26 An initial opening in the contact interface

possible explanation for this could be obtained from Fig. 28, where it is shown that the shear forces of bolts are greater than the friction forces and the bolts' shear forces received from Fig. 21, where no initial opening is considered.

5.5 Investigation of the position of the neutral axis

The point where the opening stops and the two parts of the connection remain in full contact, will approximately show the position of the neutral axis. It is worth noticing that for a simplified theoretical treatment of this issue a cross - section can be considered to remain plane, which is not the case for the contact interface arising from the three dimensional model in this study.

At Fig. 29 the opening of the contact interface at different load increments, as well as information about the initiation and expansion of plastic forces at the bolts, are shown. According to this Figure, it seems that the opening approximately stops at the line which connects the third row of bolts. In particular, until the beginning of the first row of bolts yielding ($F = 43.56$), the neutral axis seems to be slightly above the line connecting the third row of bolts. When the first row of bolts starts yielding, the

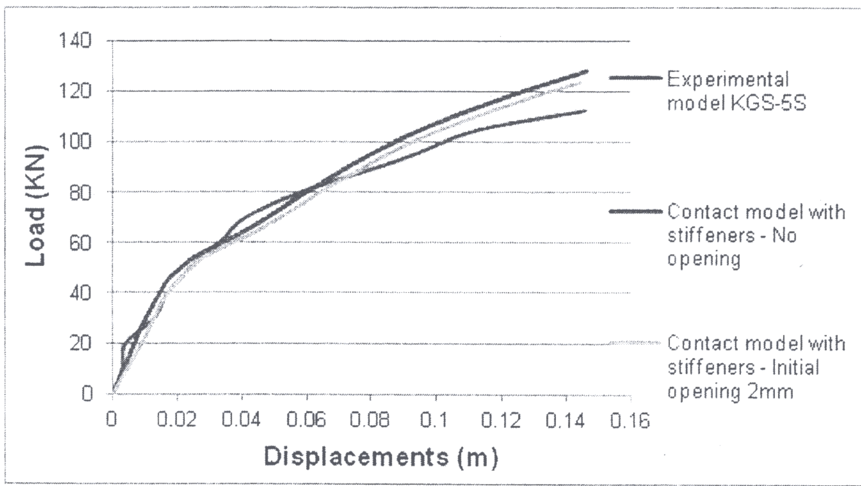


Fig. 27 Force - Displacement diagrams obtained for an initial opening of the interface

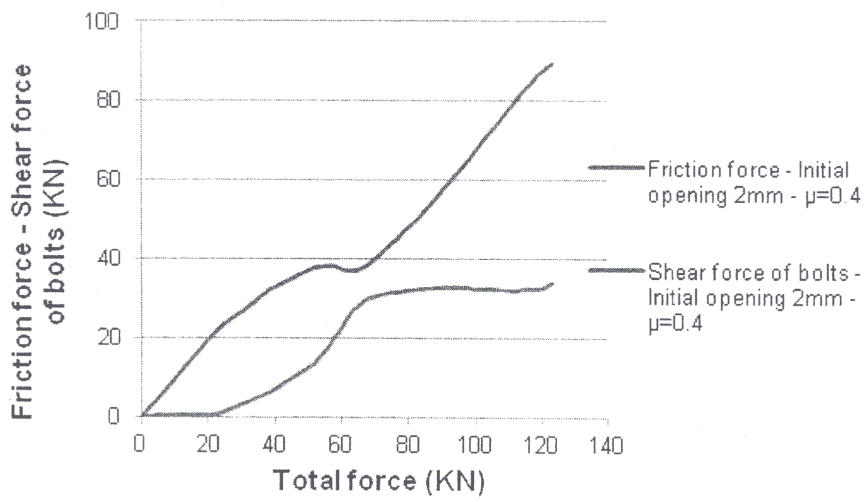


Fig. 28 Proportion of shear forces transferred through the interface by friction and through the bolts, in case an initial opening of the interface exists

neutral axis (and the opening) moves slightly lower. This position of the neutral axis remains constant, until the third and the fourth rows of bolts start yielding (F = 120.98 KN); at this point the neutral axis slightly moves up again.

6. Conclusions

A three dimensional nonlinear finite element model has been developed for the study of a steel joint, which had been previously subjected to experimental research at the Jordan University of Science and Technology. An IPE-360 beam section was attached to an HEA-220 column section through an extended end - plate. Eight high strength M20 bolts were also used for the connection of the extended end - plate with the column flange.

The work presented herein focuses on the detailed modelling of given bolted steel connections, with emphasis on contact, elastoplastic and limit analysis, as well as on comparisons with experimental results obtained in one of the collaborating research groups.

The proposed model incorporates principles taken from nonsmooth mechanics. In particular, the interface between the end - plate of the beam and the column flange is simulated with unilateral contact and friction laws. They permit the representation of a semi - rigid behaviour, where opening and sliding of the connected parts can be depicted. Material nonlinearity following the von Mises plastic yielding criterion, as well as geometrical nonlinearity have been included to the proposed model. Although the ultimate failure is mainly caused by the plastic yielding of the material the influence of contact and friction effects is significant and makes the numerical solution much more complicated.

The model developed herein, can give an insight of the mechanical behaviour of steel joints. According to the most significant observation of the analysis, a separation of the column flange from the extended

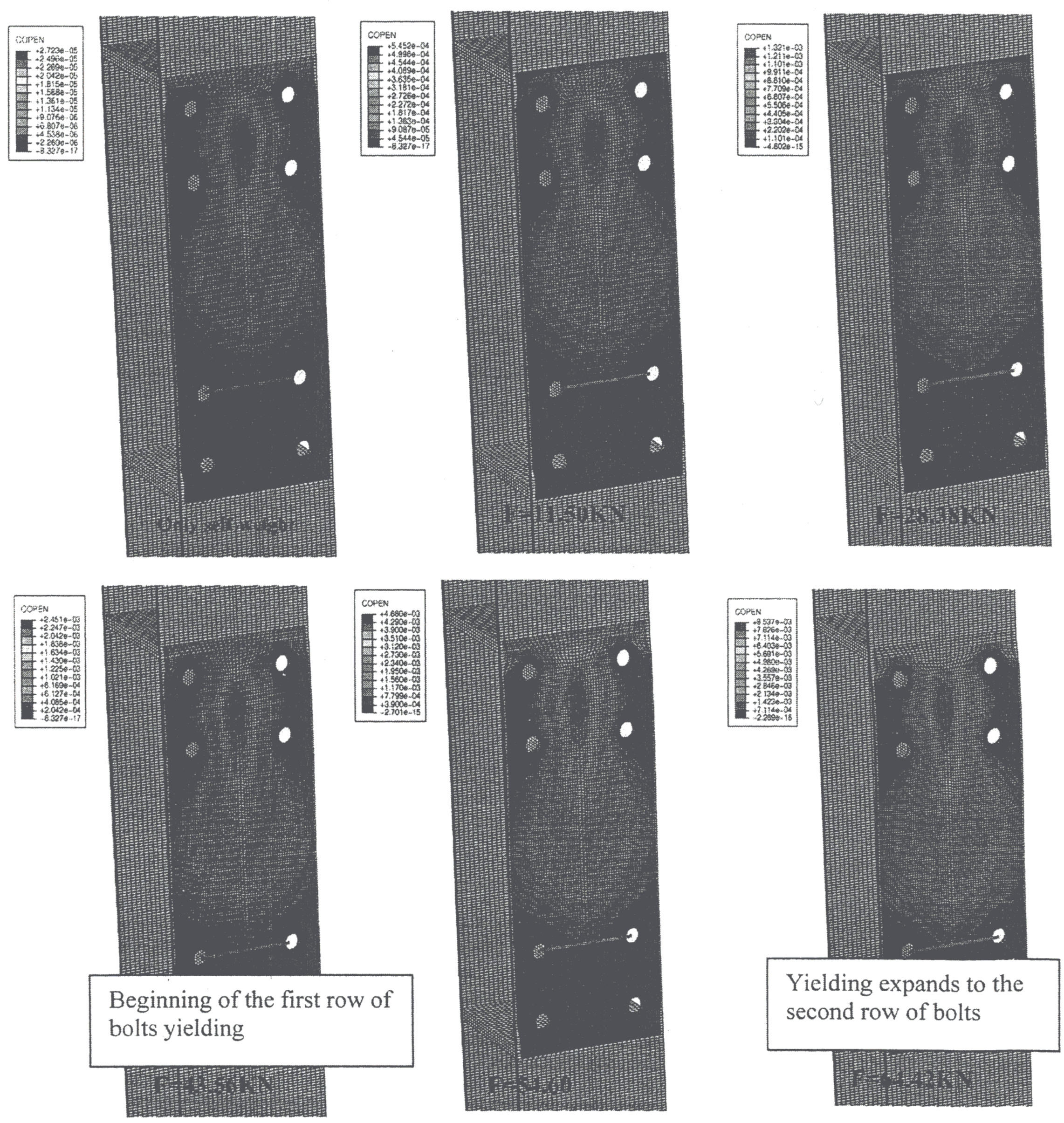


Fig. 29 Investigation of the position of the neutral axis of the contact interface throughout the loading

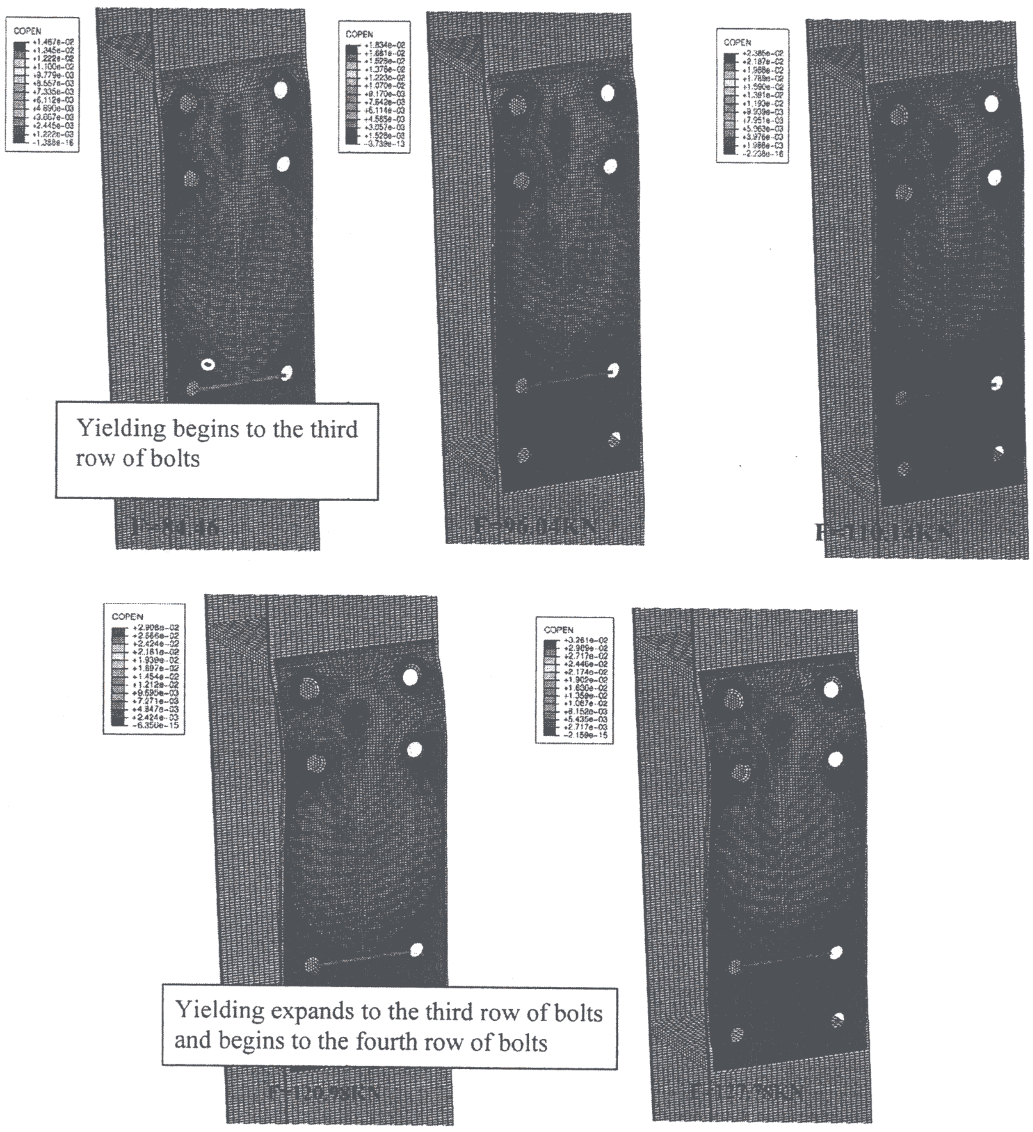


Fig. 29 Continued.

end - plate occurs, almost up to the line connecting the third pair of bolts (there are four pairs of bolts, starting from the top of the connection). Below this line the two parts remain in full contact. This means that the neutral axis is very close to the line joining these bolts.

The influence of column stiffeners on the overall response is also significant. In particular, when they are present the ultimate load is higher and the force - displacement diagram demonstrates a higher total stiffness. Consequently, opening is less expanded along the interface and reaches lower values. This indicates the effect of stiffeners on the distribution of the force among the rows of bolts.

A small sliding between the connected parts is also noticed. It takes place in a small area surrounding each hole of the end - plate of the beam. Moreover, local buckling of the column was observed in the finite element model.

An investigation of the proportion of shear forces transmitted through the interface by friction and through the bolts was also considered. For the model with a friction coefficient equal to 0.4, the shear forces of the bolts are greater than the friction forces for almost the whole range of the loading path. However, friction forces are significantly increased and become even greater than the shear forces of the bolts, in case the friction coefficient is increased or a pretension force is applied to the bolts.

In case an initial opening of the contact interface is considered, an overall degradation of the behaviour of the connection is observed. In addition, friction forces are significantly reduced contrary to the shear forces of bolts, which are increased.

Experimental results show a satisfactory convergence with the output obtained by the proposed finite element model. Thus, most of the main characteristics of the analytical model's output, like the opening of the interface connecting the two steel parts, the local buckling of the column flange - web and the force - displacement response, are also demonstrated by the conducted experiments.

Further investigation need to be done, especially for the simulation of the contact between the bolt surface and the hole around it, as well as the influence of bolts' threads on the overall behaviour of the connection. Finally, it would be interesting a comparison between the type of connection that is studied herein, with other types of connections like the top, seat and web angle connections.

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References

Abdalla, K.M., Abu-Farsakh, G.A.R. and Barakat, S.A., (2007), "Experimental investigation of force - distribution in high - strength bolts in extended end - plate connections", *Steel Compos. Struct., An Int'l J.*, 7(2).

Abdalla, K.M. and Stavroulakis, G.E. (1989), "Zur rationalen Berechnung des 'Prying Actions' phaenomens in Schraubenverbindungen", *Stahlbau*, **58**(8), 233-238.

Abolmaali, A., Matthys, J.H., Farooqi, M. and Choi, Y. (2005), "Development of moment-rotation model equations for flush end - plate connections", *J. Constr. Steel Res. Int.*, **61**(12), 1595-1612.

Attiogbe, G. and Morris, G. (1991), "Moment_rotation functions for steel connections", *J. Struct. Eng ASCE*, **117**(6), 1703-1718.

Baniotopoulos, C.C. (1995), "On the separation problem in bolted steel splice plates", *J. Construct. Steel Res.* **32**(1), 15-33.

Bursi, O.S. and Jaspart, J.P. (1997), "Calibration of a finite element model for isolated bolted end - plate steel connections", *J. Constr. Steel Res. Int.*, **44**(3), 225-262.

Choi, C.K. and Chung, G.T. (1996), "A gap element for three-dimensional elasto-plastic contact problems", *Comput. Struct.*, **61**(6), 1155-1167.

Christensen, P.W., Klarbring, A., Pang, J.S. and Stromberg, N. (1998), "Formulation and comparison of algorithms for frictional contact problems", *Int. J. Number Meth. Eng.* **42**(1), 145-173.

Gantes, C.J. and Lemonis, M.E. (2003), "Influence of equivalent bolt length in finite element modeling of T-stub steel connections", *Comput. Struct.*, **81**(8-11), 595-604.

Gebbeken, N., Rothert, H. and Binder, B. (1994), "On the numerical analysis of end - plate connections", *J. Construct. Steel Res. Int.*, **30**(2), 177-196.

Girao Coelho, A.M., Bijlaard, F., Gresnigt, N. and Simoes da Sliva, L. (2004), "Experimental assessment of the behavior of bolted T-stub connections made up of welded plates", *J. Constr. Steel Res. Int.*, **60**(2), 269-311.

Girao Coelho, A.M., Bijlaard, F. and Simoes da Sliva, L. (2004), "Experimental assessment of ductility of extended end - plate connections", *Eng. Struct.*, **26**(9), 1185-1206.

Kishi, N. and Chen, W.F. (1990), "Moment-rotation relations of semi rigid connections with angles", *J. Struct. Eng-ASCE*, **116**(7), 1813-1834.

Kiymaz, G. (2009), "Investigations on the bearing strength of stainless steel bolted plates under in-plane tension",

Steel Compos. Struct., An Int'l J., **9**(2).
Krishnamurthy, N. (1975), *Two dimensional finite element analysis of extended and flush end - plate connections with multiple rows of bolts*, Report to AISC and MBMA, no. CE-AISC-MBMA-6, Dept. of Civil Engineering, Auburn University.
Krishnamurthy, N (1978), "Fresh look at bolted end - plate behavior and design", *Eng. J., AISC.*
Krishnamurthy, N. and Graddy, D.E. (1976), "Correlation between 2 and 3 dimensional finite element analysis of steel bolted end - plate connections", *Comput. Struct. Int.*, **6**(4-5), 381-389.
Lemonis, M.E. and Gantes, C.J. (2009), "Mechanical modeling of the nonlinear response of beam - to - column joints", *J. Constr. Steel Res. Int.*, **65**, 879-890.
Maggi, Y.I., Goncalves, R.M., Leon, R.T and Ribeiro, L.F.L. (2005), "Parametric analysis of steel bolted end - plate connections using finite element modelling", *J. Constr. Steel Res. Int.*, **61**(5), 689-708.
Meng, R. (1996), *Design of moment end - plate connections for seismic loading*, Doctoral Dissertation, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.
Mistakidis, E.S., Baniotopoulos, C.C., Bisbos, C.D. and Panagiotopoulos, P.C. (1996), "Steel T-Stub connection under static loading: An effective 2-D numerical model", *J. Construct. Steel Res. Int.*, **44**(1-2), 51-67.
Nogueiro, P., Simoes da Silva, L., Bento, R. and Simoes, R. (2009), "Calibration of model parameters for the cyclic response of end-plate beam-to-column steel-concrete composite joints", *Steel Compos. Struct., An Int'l J.*, **9**(1).
Panagiotopoulos, P.D. (1985), *Inequality problems in mechanics and applications. Convex and nonconvex energy functions*, Birkhäuser Verlag, Boston, Basel, Stuttgart.
Pucinotti, R. (2001), "Top-and-seat and web angle connections: Prediction via mechanical model", *J. Construct. Steel Res. Int.*, **57**(6), 661-694.
Saravanan, M., Arul Jayachandran, S., Marimuthu, V. and Prabha, P. (2009), "Advanced analysis of cyclic behaviour of plane steel frames with semi - rigid connections", *Steel Compos. Struct., An Int'l J.*, **9**(4).
Shi, G., Shi, Y., Wang, Y. and Bradford, M.A. (2008), "Numerical simulation of steel pretensioned bolted end - plate connections of different types and details", *Eng. Struct.*, **30**(10), 2677-2686.
Silva, L.S. and Coelho, A.M.G. (2001), "A ductility model for steel connections", *J. Const. Steel Res. Int.*, **57**(1), 45-70.
Stavroulaki, M.E. and Stavroulakis, G.E. (2002), "Unilateral contact applications using FEM software", *Int. J. Appl. Math. Comput. Sci.*, **12**, 101-111.
Stavroulakis, G.E. and Antes H. (2000), "Nonlinear equation approach for inequality elastostatics: a two-dimensional BEM implementation", *Comput. Struct*, **75** (6), 631-646.
Wu, F.H. and Chen, W.F. (1990), "A design model for semi-rigid connections", *Eng. Struct.*, **12**, 88-97.
Yee, Y.L. and Melchers, R.E. (1986), "Moment - rotation curves for bolted connections", *J. Struct. Eng-ASCE*, **112**(3), 615-635.

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