BEHAVIOUR AND MODELLING OF SEMI-RIGID STRUCTURAL FRAME SYSTEMS

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A computational finite element model is developed to predict the load-deflection characteristic of portal frames with semi-rigid welded and bolted joints. Plate components of frame members and joints are modelled by thin shell finite elements while bolt connectors by springs with the force-deformation characteristic evaluated from test results. The computer software ABAQUS is used. Experimental investigations were designed in order to evaluate frame load-deflection characteristics, joint local rotations, failure modes, and the contribution of end-plates and bolt connectors to the joint rotations. Theoretical results of frame load-deflection characteristics predicted with use of ABAQUS program are compared with the experimental results. The modelling of friction effect between the contact surfaces of bolted joints is verified to be the most important factor. The comparative analysis leads to the conclusion that the proposed advanced shell model can be very useful in the verification of simplified models being developed for practical applications.

Keywords: Finite element, portal frame, semi-rigid joint, welding, structural frame

1 INTRODUCTION

Among several joint types, steel end-plate connections have been widely studied in the literature Krishnamurthy and Graddy [9], Kuktreti et al. [11], Rothert et al. [12], Nemati and Le Houedec [10]. Finite Elemet Method (FEM) has been proved to be the most suitable tool for conducting an extensive numerical study validating different geometrical and mechanical parameters that influence the joint behaviour. Recent research has mainly concentrated on joints with relatively thick end-plates compared to the thickness of flange and web components of connecting members. A number of finite element models have been developed and verified by means of experimental studies. The discussion of all those models could be found in the paper written by Nemati and Le Houedec [10].

The joint behaviour and frame response have also been investigated at the Warsaw University of Technology, taking into account the connection local effects of bolts, gaps and friction on the global behaviour of the structure, Gizejowski et al. [3], Gizejowski et al. [4], Gizejowski et al. [5]. It has been concluded that further research is needed to study the end-plate joint behaviour and its mode of failure depending on the plate thickness, the bolt diameter and the connector arrangement. The study presented herein aims to develop an experimentally verified finite element model of portal frames with semi-rigid beam-to-column connections. The focus is on systems with relatively thin end-plate joints in order to investigate the possibility of their practical applications in more economical, lightweight structural systems of building structures. This type of structural system is referred here as to a semi-rigid system. The summary of an extensive experimental study conducted for the purpose of verification of the developed FEM model is also presented. Theoretical frame load-deflection characteristics are compared with experimental ones. Concluding remarks are drawn with respect to both the FEM modelling and the verification procedure based on experimental evidence.

2 FINITE ELEMENT MODEL

The details of frame specimens considered in numerical simulations and experimental investigations are given in Figure 1. The general purpose ABAQUS computer program is used. The shell finite element model is adopted. Since relatively thin plate components of the frame members and beam-to-column connections were used, they are fully discretised with thin shell finite elements what serves the amount of degrees of freedom when compared with solid finite element models.

The column section IPE 240, the beam section IPE 120 and the end-plate thickness of 6 mm were applied. The column height (from the level of the load application point to the beam longitudinal axis) was equal to 508 mm, except one specimen for which the said distance was taken as 558 mm. The beam length was 3180 mm for all the specimens. The steel grade corresponding to S235 according to the Eurocode 3 [2] was used, except for the connectors. The bolts M12 of grade 10.9(10) were used. The welding material was selected to conform the grade of the parent material of the connecting elements (beams, columns and end-plates).

The parameters of the stress-strain characteristic of steel elements are evaluated from tests. Bolts are modelled by springs with the force-deformation characteristic based also on the experimental investigations. The frame non-linear behaviour is

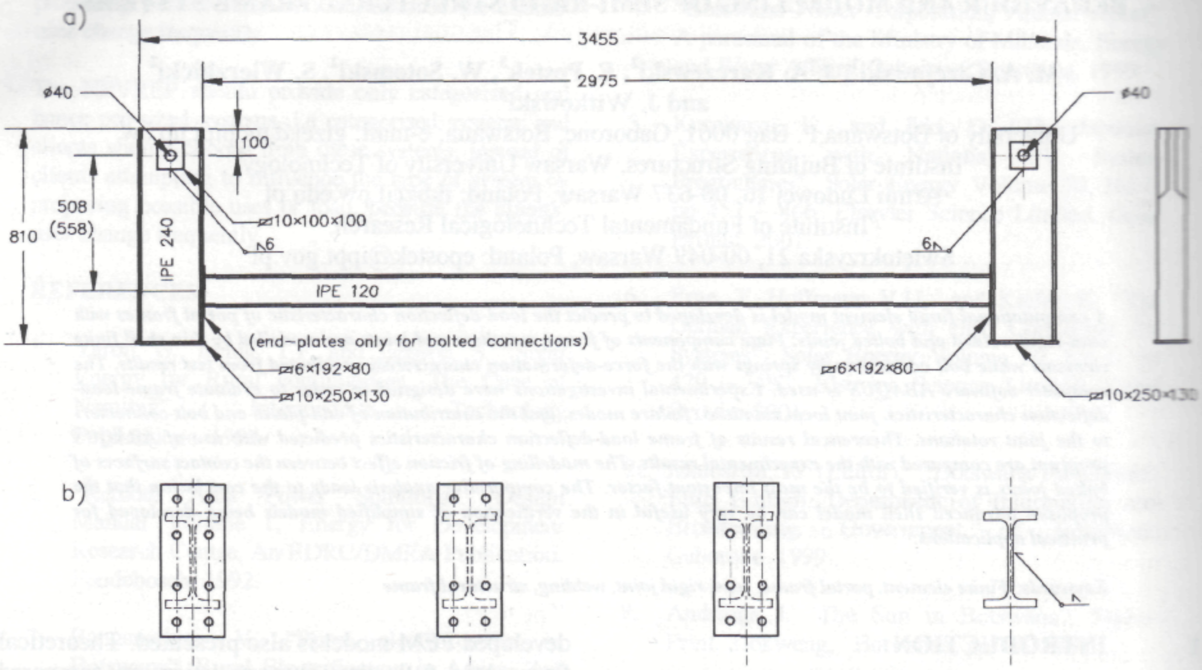


Figure 1: Details of considered frame specimens; a - specimen details, b - connection details

examined by applying a bilinear elastic - strain hardening approach to the modelling of properties of beam cross section, end-plate components and HS bolt connectors.

To include the friction effect, the contact surfaces are defined between the mid surface of the column flange and the mid surface of the beam end-plate. The coefficient of friction is taken as for the plain steel surfaces being in contact. Numerical simulations are performed for the frames with welded connections (a nominally rigid joint frame) and a range of frames with bolted connections (semi-rigid frames).

3 EXPERIMENTAL INVESTIGATIONS

3.1 Introductory Remarks

Experimental investigations on semi-rigid joints have mostly been performed on isolated joint specimens. Test results were collected and stored in data banks, such as the SERICON bank used for the verification purposes of the M-ϕ prediction model adopted in the Eurocode 3 [2] (see also Jaspart [7]). In tests conducted for isolated joints, the following assumptions were usually adopted:

1. Joint specimens were loaded in such a way that forces applied on the joint specimen were increasing monotonically, and changing in a quasi-static manner.
2. Load increments were applied up to the joint failure or they were stopped quite arbitrarily at the point of excessive joint rotation, not necessarily associated with the joint failure.
3. Joint rotations were calculated from the measurements recorded by inductive displacement transducers that were placed on the beam at the distance L_o from the column face. The measurements recorded by transducers were able to reproduce the rotation ϕ_r being a sum of the connection rotation ϕ and a contribution from the beam length L_o curvature rotation.

In general, such a testing procedure is not an accurate enough procedure in order to model the real connection behaviour in steel frame buildings. A more accurate procedure was therefore suggested for the experimental investigations reported herein:

1. The connection behaviour was examined on the basis of tests conducted upside-down for portal frame specimens (see Figure 1).
2. Frame specimens were loaded in such a way that the force applied on the specimen was increasing incrementally in a quasi-static manner, but at each incremental load step the load cycles were repeated up to the stabilisation of the deflection state. The moment amplitude in the connections was selected as 0.4 times the actual connection moment.
3. The measurement set-up was devised in such a way that the overall joint rotation ϕ and the rotation ϕ_r of the joint plus the contribution of the beam length L_0 could be examined.
4. Tests were continued up to the point of the local failure of the connection or the global failure of the frame system due to instability.

The procedure suggested for the experimental investigations of semi-rigid joint behaviour is a component-based procedure used by Bernuzzi et al. [1] for isolated column-to-foundation joints. It has however a more general aspect since it includes the testing of frame specimens instead the testing of isolated joints. In this way, the interaction of failure modes could be examined and the influence of joint semi-rigidity on the frame ultimate strength evaluated. With the reference to the joint moment-rotation curve, operational definitions of the rotation of the whole connection ϕ, the rotations associated with the various connection components (ϕ_p, ϕ_b) and the rotation ϕ_r associated with the connection rotation and the beam length L_o were adopted. The definitions of ϕ, ϕ_p and ϕ_b are consistent with Bernuzzi et al.[1]. The experiments were guided on-line by a Personal Computer (486 processor) and equipped with the GENIE software developed by Advantech. All the results were stored on the computer hard disc in the format compatible with the EXCEL MICROSOFT OFFICE 97. This allowed for a convenient post-processing in order to investigate detailed response of joint components, including the bolt elongations and bolt bending deformations to be evaluated. These in turn were used for the development of a semi-empirical load-deformation characteristic of bolts in the thin shell - spring connector finite element representation of structural joints.

3.2 Frame Specimens Tested

A total of 17 portal frame specimens were tested. The description is given in Table 1, Gizejowski [6] (see also Figure 1a). Two of them were tested just to check the adequacy of the stand arrangement and the measurement set-up. The results of the pilot testing were not considered in the final evaluation of test results. The remaining 15 specimens were used for the project purposes. Frame specimens were tested upside-down, i.e. the beam-to-column connections were placed at the bottom while the column free ends were heading up. One column end was immovable and simply supported. The other one was moveable and loaded with a horizontal force. The point of load application was allowed to rotate freely in the frame plane. The following forms of the connection were investigated (see Figure 1b):

a) Extended end-plate bolted connections with 8 bolts; 4 bolts were placed in each flange connection (for both tension zone and compression zone) and they were distributed symmetrically with a reference to the beam section flange and web axes.
b) Extended end-plate connections with 6 bolts; 4 bolts were placed in the beam compression flange connection and 2 bolts - in the beam tension flange connection, below the tension flange, and all of them were distributed symmetrically with reference to the beam section web axis.
c) Flush end-plate connections with 6 bolts; 4 bolts were placed in the beam compression flange connection and 2 bolts - in the beam tension flange connection, above the tension flange, and all of them were distributed symmetrically with reference to the beam section web axis.
d) Welded face-to-face connections.

The specimens were divided into 4 series (see Table 1). The first two were arranged to cover the situations in which the frame beam was subjected to tension and bending, and comprised of 11 specimens, 7 of which were laterally restrained (2 specimens for each bolted connection type), and 4 - laterally unrestrained. The remaining two series comprised of 2 frames with the beam subjected to compression and bending (1 specimen for both the end-plate connection with 8 bolts and the welded connection). The butt welds were used everywhere. In order to eliminate a possible weld fracture under a variable loading programme considered in testing, the butt welds were over-strengthened by the fillet welds. The welding process was carried out in several runs to minimise the local effects of residual stresses and thermal shrinkage deformations.

3.3 Measurement Set-up

The testing stand is shown in Figure 2. The horizontal load was applied to the free end of the left column by means of a hydraulic jack. The power jack, which had a 400 kN capacity and a 160 mm stroke, was attached to a specific frame, designed to accommodate the backward movement. The horizontal force was transferred onto the frame beam through the columns, causing its bending and tension or compression, depending upon the direction of the load applied on the specimen. The amount of bending moment and axial force transferred onto the beam depended upon the properties of the beam-to-column connections. The failure mode of the tested frame specimen depended on the connection ability to transfer the forces from the column to the beam. In the case of a strong connection (such as welded and bolted with 8 bolts), laterally unrestrained specimens were more sensitive to the failure in a global instability mode. In the case of weak connections (bolted with 6 bolts), specimens were more sensitive to a local mode of failure (end-plate fracture or its excessive plastic deformations). The fracture mode resulted from the end-plate rupture in the Heat Affected Zone (HAZ) while the excessive plastic deformations resulted from the formation of end-plate yield lines.

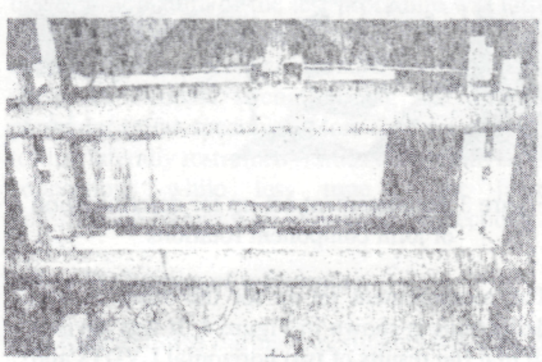
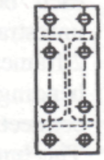
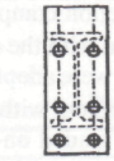
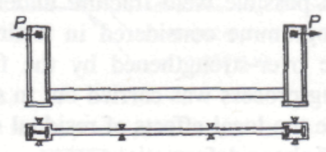
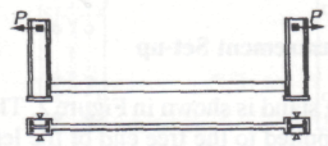
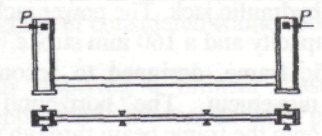
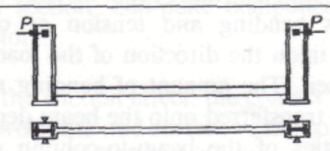


Figure 2: View of frame specimen installed in testing stand

Table 1: Specification of frame specimens tested experimentally

Description of specimen		Specimen number in accordance with the joint type			
		Bolted joints with reference to the bolt arrangement			Welded
Series no.	Restraining and loading conditions				
1		2*, 3, 4	7, 8	5, 6	9
2		10	11	12	13
3		14	-	-	15
4		1*, 16	-	-	17

No* specimens tested in the pilot investigations

The measurement set-up for the development of the connection rotation is shown in Figure 3. Inductive displacement transducers (LVDTs shown in Figure 3) enable determination of both the rotation of the connection as a whole and the contributions of the various joint components to be obtained.

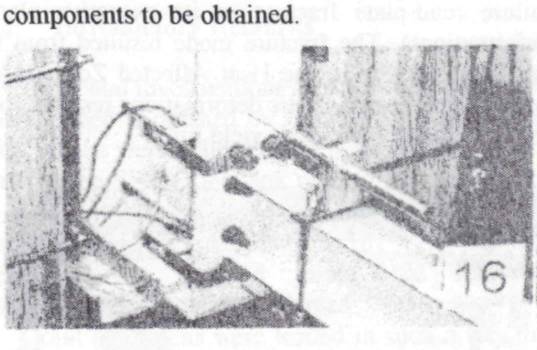


Figure 3: Measurement set-up for determination of joint component rotations

The measurement set-up for the beam section deflections is shown in Figure 4. It was devised so that the results of two horizontal LVDTs and two vertical LVDTs allowed the spatial displacement state to be reproduced from the recorded measurements. The frame column lateral stiffness of length being approximately 6 times less than the beam length is much greater than the beam stiffness. Hence testing conditions approximated quite closely the case of beam-to-column joints with negligible column deformability and the case of beam instability with appropriate out-of-plane boundary conditions, resulting from the connection properties and the beam in-span lateral restraints.

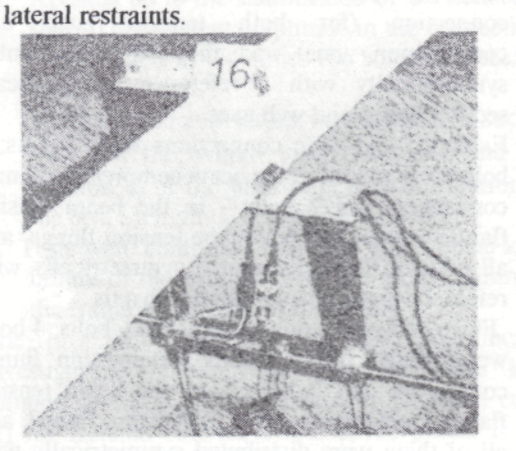


Figure 4: Measurement set-up for determination of beam mid-span deflections

The presented experimental set-up allowed for the instability mechanism to be monitored. The failure mechanism could be detected trough the examination of the beam mid-span section lateral displacement and its rotation about the beam longitudinal axis. For tested frame specimens, the above displacements were expected to be relatively small in the initial stage of testing and to increase rather slowly in course of the loading programme. In the final stage, close to the frame specimen ultimate strength, the beam lateral displacement and the twist rotation were expected to remain small in the case of specimens insensitive to lateral torsional instability. This is indicative for the following modes of failure to occur:

1. Local joint failure due to:
   a) a large rotation without fracture (for plastic joints),
   b) a relatively large rotation with localised fracture (for plastic or semi-plastic joints),
   c) a relatively small rotation with overall fracture of the end-plate expected above the weld line or in the welding material (for non-plastic or brittle joints).
2. Global in-plane failure of the frame due to:
   a) excessive plastic deformations (for plastic beam sections),
   b) local instability mechanism (for non-plastic beam sections).

In the case of specimens sensitive to lateral torsional instability, an increase in the beam section lateral displacement and twist rotation was expected, indicative for the overall instability mode of failure to occur.

3.4 Testing procedure

The loading history was chosen in such a way that it reproduced on average the time-dependent connection behaviour in building steel frameworks. It is schematically shown in Figure 5. A step-by-step loading-monitoring procedure was adopted. All the specimens were loaded in the same way by a force applied horizontally on the top of the left column of each specimen (see Figures 1 and 2). The loading system of building steel structures consists of dead and live load components. The analysis of typical design situations points out that the amplitude of live load components may be assumed as $\Delta P = P_{max} - P_{min} = 0.4\ P_{max}$. Thus, the loading program was arranged in such a way that after each incremental load step, a sequence of load cycles was applied with the amplitude of 0.4 times the load magnitude of every particular load increment. The load repetitions were automatically stopped when the overall response of the tested specimen stabilized. This is referred to as the shakedown of the structural system, i.e. to the situation when it shakes down to the repeated loading conditions (deflections adapted within the tolerance limit prescribed to the repetition of load amplitude of the current incremental load). A tolerance limit of 0.01 times the deflection of the first cycle at each incremental step was adopted. The load was applied incrementally assuming the load increment $\Delta P = 2,0$ kN in the elastic range while it was reduced by half in the inelastic region (i.e. $\Delta P = 1,0$ kN). Passing the new load increment, the „overloading" of 1% of the total load P_i (at particular incremental step) was assumed and then, the loading amplitude ΔP_i was applied cyclically in the range P_i and $0,6\ P_i$, till the stabilisation of the structural response (shakedown). Considering two respective load cycles, the following criterion was used as the shakedown condition: $\Delta u_i = |u_{i,j} - u_{i,\,j-1}| \leq 0,01\ u_i$, where u_i is the displacement „u" at the beginning of the cycle „j" at the increment „i".

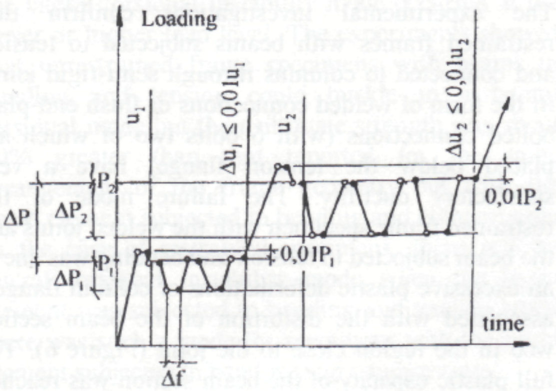


Figure 5: Loading program

Usually 2-4 cycles were needed for the deflections to stabilise at the lower load level while the number of cycles to the adaptation was increasing up to maximum 8 at the failure point for frame specimens with bolted joints. The automatic readings from the displacement transducers and strain gauges were taken at each change of loading level, either due to the new load step, i.e. the load level $1,01P_i$, or due to load cycles at load levels P_i and $(P_i - \Delta P_i)$. The results were automatically stored on the computer hard disc. At each reading point, an allowance was made for the so-called hydraulic system relaxation effect. The quasi-static nature of the test procedure was ensured by a low rate of load application. The duration of each test ranged between 1 and 3 hours, depending upon the ultimate strength being reached by the particular frame specimen. More time was needed for testing laterally restrained frames with relatively rigid connections while less time - for laterally unrestrained frames with semi-rigid connections.

3.5 Evaluation of test results

The data recorded during experiments was stored on the computer hard disc for graphical post-processing in the format compatible with the Microsoft

WINDOWS 95 EXCEL. The following graphs were reproduced from the test results:

1. Load-deflection characteristics of frame specimens in terms of the applied load and horizontal displacement corresponding to the point and the direction of this load.
2. Moment-rotation (M-ϕ) characteristics of frame joints.
3. Moment-rotation (M-ϕ_r) characteristics of frame joints with the beam length L_0 contribution.
4. Contributions of the end-plate and the bolts to the connection rotation as a whole.
5. Bolt strain-load characteristics evaluated for the most stressed bolt in tension.
6. Finally, the beam section deflection-load characteristics evaluated for the spatial displacement state.

The experimental investigations confirm that restrained frames with beams subjected to tension and connected to columns through semi-rigid joints in the form of welded connections or flush end-plate bolted connections (with 6 bolts two of which are placed below the tension flange) have a very satisfactory ductility. The failure mode of the restrained frame specimen with the welded joints and the beam subjected to tension and bending was due to an excessive plastic deformations of column flanges, associated with the distortion of the beam section web in the region close to the joint (Figure 6). The full plastic capacity of the beam section was reached at the ultimate strength of the frame specimen. There was no sign of weld fracture or any other local mode of joint failure.

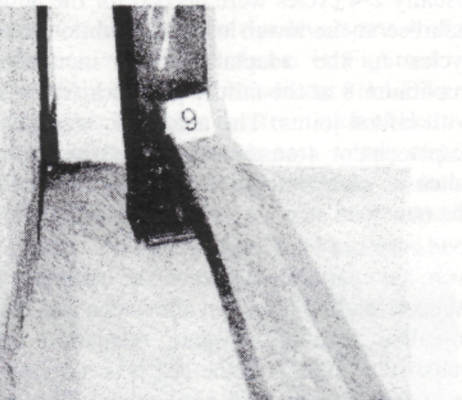


Figure 6: Detail of permanent deformations of distorted beam web

The restrained frame specimens with joints in the form of flush end-plate bolted connections (with 6 bolts two of which are placed below the tension flange) exhibited an excessive plastic deformation of the end-plates (the yield line pattern was developed), with no plastic deformations in beam sections. That was a result of the connection low stiffness and strength. No cracking of the end-plate was observed but the deflected profile of the end-plate was distinguished by a rigid-like rotation of its cantilever part (Figure 7).

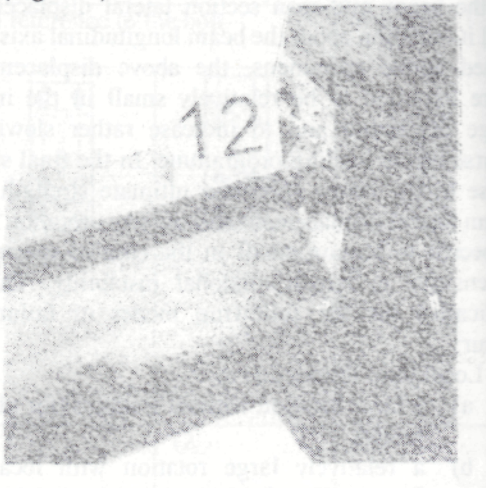


Figure 7: Details of joint flush end-plate permanent deformations

The restrained specimens with extended end-plate connections and with 8 bolts exhibited a better balance between the stiffness and rotational ductility. They were practically equivalent (in terms of stiffness and strength) to the welded connections. This allowed the same joint ultimate strength to be reached for welded connection specimens and extended end-plate connection specimens, provided that there are 8 bolt connectors applied. Although the rotation attained by such bolted connections is much lower than that of welded joints, it was yet sufficient for the plastic stress redistribution to take place in the frame members. The lower rotation ability of this type of joint is associated with the localised fracture occurring in the end-plates, in the area of bolt holes (Figure 8).

Figure 8: Detail of joint extended end-plate permanent deformations and partial fracture

Only the end-plate connections with 6 bolts two of which were placed above the beam tension flange showed a very low ductility, insufficient for the plastic moment redistribution to take place in real frame structural systems. These connections in all the tested frame specimens failed in a brittle mode by rupture of the end-plate in the Heat Affected Zone (HAZ), above the tension flange (just above the outer flange weld, see Figure 9). This type of failure mode is undesirable, and because of that joints with thin end-plates and bolts located only on the outer side of the tension flange must be avoided in practice. If such a joint type needs to be applied, a relatively thick end-plate has to be used in order to ensure that stresses in the end-plate, close to the tension flange welds, are in the elastic region. This increases the joint strength and stiffness, resulting in the plastic moment redistribution to take place in structural members.

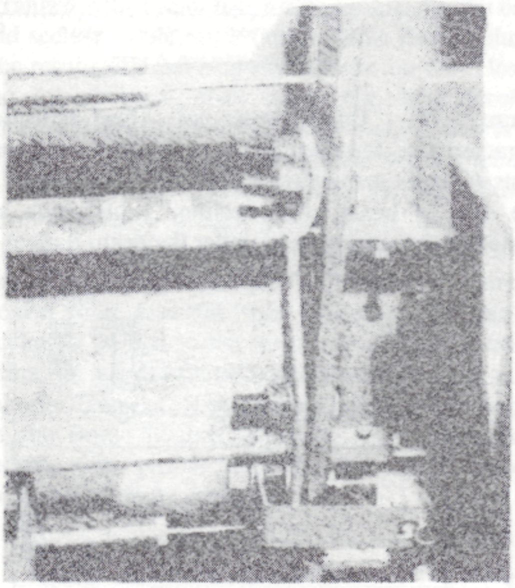


Figure 9: Detail of joint extended end-plate permanent deformations and complete fracture mechanism

The experimental results showed that the bolt contribution to the connection rotation ϕ can be generally neglected in all the bolted connections tested experimentally. The connection deformability is controlled by the ability of the end-plate to plastic deformations. When the end-plate most stressed regions are not associated with HAZ, the yield lines can be developed and the connection exhibits a good ductility. Otherwise, the end-plate fracture (overall or localised) is most likely to occur. As a result of the end-plate flexural deformations, the bolt shanks are subjected to combined tension and bending. In spite of an asymmetrical strain state in the bolts, the average strain associated with the bolt shank elongation is the dominant factor.

Experimental results presented for laterally restrained frame specimens are typical for these cases. They show that such specimens exhibit the increase of lateral displacements and twist rotations practically from the beginning of the loading process. This is because of geometrical imperfections existing in the specimen members. These initial out-of-plane displacements were subjected to amplifications in the course of loading. The amplified quantities remained rather small with comparison to in-plane displacements in the first stage of loading programme, up to a certain load level. Starting from this point, the beam lateral displacement and twist rotation increased more rapidly with the load incrementation. This is a sign that the specimen was going to fail in a global instability mode (due to lateral torsional buckling). Depending on the stress state in the frame beam in the pre-critical phase (bending and tension or bending and compression), the lateral torsional instability mode occurred at the lower or higher load level. The experiments showed that unrestrained frame specimens with beams in bending and tension could buckle in a lateral torsional mode but their ultimate strength was about 70% greater than that reported for the same arrangement of the frame geometry but with the beam element subjected to bending and compression. In the case of restrained specimens, there was no lateral torsional instability mode when the beam element was subjected to bending and tension while there was such a mode for specimens with the beam element subjected to bending and compression.

4. VERIFICATION OF FINITE ELEMENT MODEL

The verification of the developed thin shell finite element model is presented only for the frame specimens with the beam restrained at distances of 1/3 and 2/3 of its length and at both ends. The specimens loaded only in tension and bending are considered. These arrangements were proven by experiments to be the most appropriate for the verification of the semi-rigid joint action and the effect of joint semi-rigidity on the frame behaviour. Figure 10 shows the average experimental load-deflection characteristic obtained for frame specimens with 8 bolts and its comparison with the results obtained for two cases by the ABAQUS code. In the first case the contact and friction effects were considered in the analysis while in the second – the contact effect was neglected.

From the comparison it is clear that below the load level corresponding to approximately 2/3 of the ultimate load P_u the experimental load-deflection characteristic is below those obtained from the computer simulations. Above this level the frame equilibrium path based on computer simulations with the contact and friction effects included approaches that obtained from experimental investigations. The experimental ultimate load is however lower from that evaluated numerically. The joint failure due to a partial end-plate fracture below the beam tension flange could not be detected in the computer analysis since the material non-homogeneity and anisotropy in HAZ resulting from the welding process as well as the effects of residual stresses and end-plate lack-of-fit were not considered in the FEM modelling. It is seen that the effect of contact-friction phenomena should not be neglected. It appears that the behaviour of bolt connectors is linear even in the ultimate limit state of the frame. The plastic zones are localised in the end-plates.

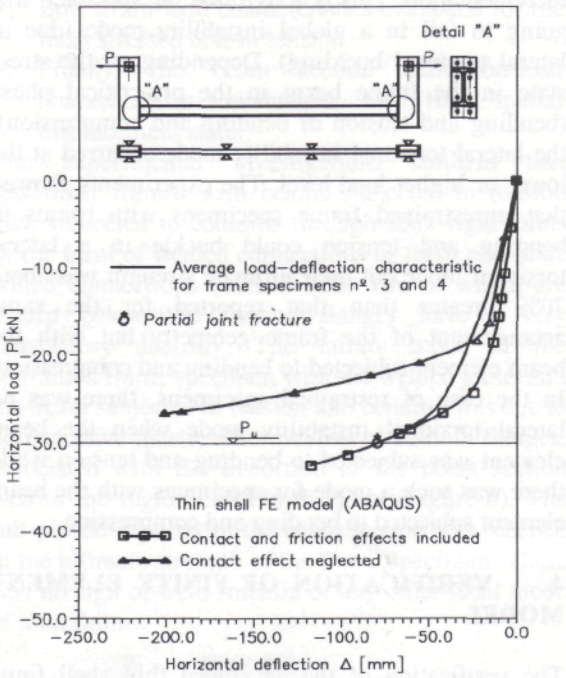


Figure 10: Comparison of load-deflection characteristics for specimens with extended end-plate joints (8 bolts symmetrically placed with respect to both beam cross section axes)

Figure 11 shows the similar comparison made for the frame specimens with 6 bolts two of which were placed in the tension zone below the beam flange. The experimental load-deflection characteristic coincides with those obtained by ABAQUS code in the region below the load level of approximately ½ of the ultimate load P_u. The shell finite element models underestimate the frame ultimate load but again the results obtained for the case in which the effects of contact and friction were considered are closer to the experimental ones. The frame specimen experimental characteristic ends earlier than those obtained numerically. Excessive plastic deformations of the beam system could not be examined experimentally due the existing limitation of the testing equipment. No joint failure is detected in the computer simulations, however it is proven that the end-plate is subjected to large bending deformations causing large permanent end-plate deflections observed after unloading of the tested frame specimens.

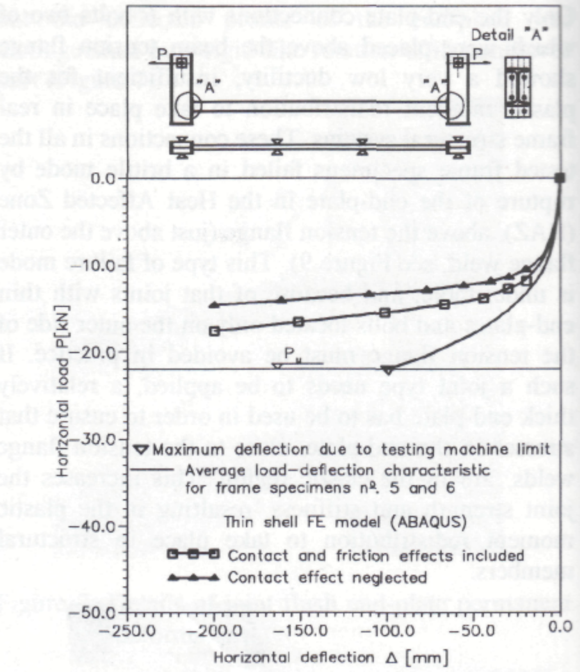


Figure 11: Comparison of load-deflection characteristics for specimens with flush end-plate joints (6 bolts symmetrically placed with respect to web axis of beam cross section, 2 of which are located in the tension zone under the beam flange)

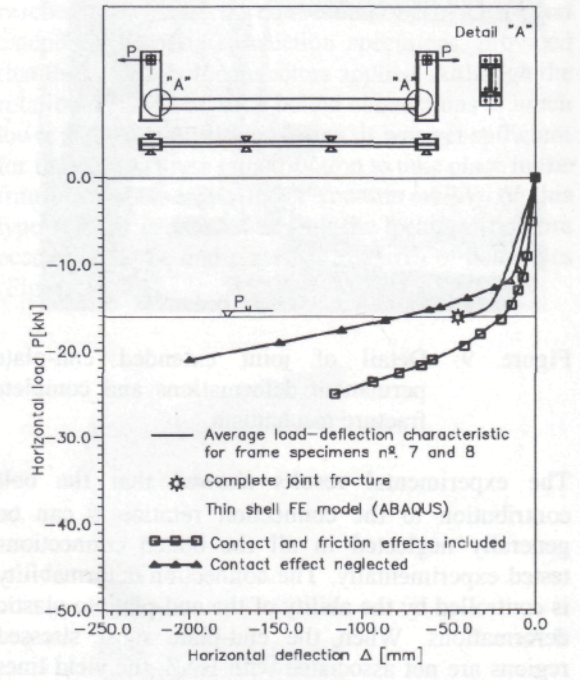


Figure 12: Comparison of load-deflection characteristics for specimens with flush end-plate joints (6 bolts symmetrically placed with respect to web axis of beam cross section, 2 of which are located in the tension zone above the beam flange)

Figure 12 shows the results obtained for specimens with 6 bolts two of which are placed in the tension zone but above the beam tension flange. The shell finite element model overestimates the frame ultimate load P_u. The shape of the load-deflection curve obtained numerically for the case in which the contact-friction phenomenon was included reproduces more accurately the curve obtained from experimental investigations. The joint failure mode resulting from the end-plate complete fracture could not however be detected in numerical simulations. This is due to the fact that the welding degradation effect on steel ductility properties is not considered and the assumption of homogeneous and isotropic material is used. Thus the shell finite element model underestimates frame deflections what can be attributed to the effect of end-plate residual stresses causing early yielding of a large volume of end-plates in the region of beam tension flange welds.

Finally the results obtained for the frame with welded joints are presented in Figure 13. Joints were arranged with use of butt welds connecting the beam end section to the inner flange of the frame column. The results of numerical simulations are very close to the experimental ones below the load level of approximately 2/3 of the ultimate load P_u. This is due to the fact that the effect of residual stresses induced by welding is not so important in the case of direct beam-to-column welded joints unlike in thin end-plate bolted joints. The thick column flange of 17 mm in thickness is less affected by the beam directly welded to it than the thin end-plate of 6 mm in thickness welded to the beam end section in bolted joints. As a result the residual stresses induced in the column flange of welded joints have a lesser impact on the frame equilibrium path. Their importance become more visible in an inelastic region, close to the frame ultimate limit state.

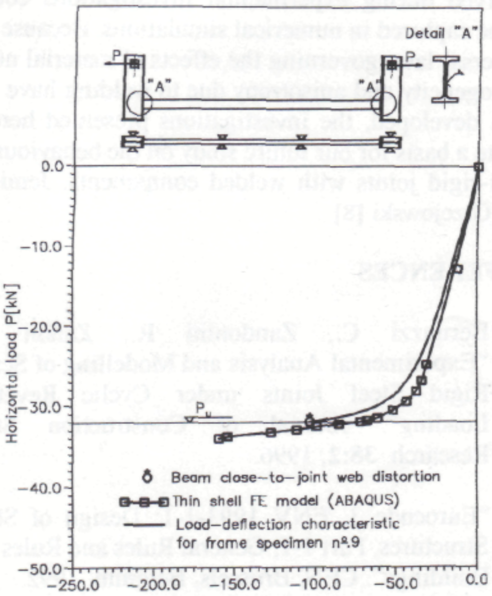


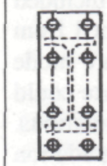
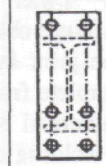
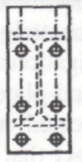
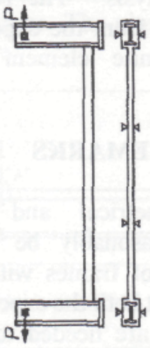
Figure 13: Comparison of load-deflection characteristics for specimens with welded joints

Because the effect of welding residual stresses is not considered in the developed finite element model, the experimental ultimate load P_u is lower than that obtained from the ABAQUS code. The experimental evidence showed that there was no joint failure. The ultimate limit state was associated with an excessive inelastic distortion of the beam web in the region between the joint and the first beam span restraint. The same mode of failure is detected in the computer stability eigen-mode analysis. The results of the frame ultimate load both from the experiments and from the developed finite element model are presented in Table 2.

5. CONCLUDING REMARKS

The number of geometrical and mechanical parameters that can reasonably be expected to influence the behaviour of frames with semi-rigid joints is significant, so that both the experimental and numerical investigations are needed to provide an efficiency in engineering design practice. In the paper a finite element model is presented as a part of the wider research program carried out at the University of Botswana and the Warsaw University of Technology on the quasi-cyclic and quasi-static behaviour of steel semi-rigid joints and frame specimens. Theoretical and experimental investigations were concerned with portal frame specimens composed of I section members and of welded or bolted joints. At this stage of research 6 mm end-plates were used. The thin shell finite element model is developed to represent physical frame models used in experimental investigations. Bolt connectors are modelled by springs with the force-deformation characteristic evaluated on the basis of average bolt stress-strain response monitored during testing. The member parent steel constitutive relationship for homogeneous and isotropic material with strain hardening is used. No allowance is made for the ultimate stress increase and ductility reduction effects resulting from welding. In experiments an average time-dependent joint behaviour in typical steel building frameworks was reproduced. The study presented herein was purposefully limited to the connections with a relatively small thickness of the end-plate (if compared with real joints). Experimental investigations were designed in order to evaluate frame load-deflection characteristics, joint local rotations, the contribution of end-plates and bolt connectors to the joint rotations. Testing stand was equipped with the computer on-line monitoring system. The experimental set up developed allowed for detailed investigations of 15 frame specimens in two series. The first one aimed at investigations of 9 frames laterally supported at distance of 1/3 of the beam length, and the second - at investigations of 6 frames with beams laterally unsupported. This allowed the investigation of different modes of connection and frame failure.

Table 2. Comparison of frame ultimate loads

Description of specimen		Modelling type		Ultimate load P_u [kN] for the joint type*			
Series No.	Restraining and loading conditions						
1		Experiment		30,0	16,0	21,5	32,0
		Finite element model	Effects of contact and friction included	30,0 (1,00)	20,0 (1,25)	15,0 (0,70)	33,5 (1,05)
			Contact effect neglected	22,0 (0,75)	15,5 (0,90)	13,0 (0,60)	

* Rounding not greater than ±0,5 kN.
In the brackets, the ratio of the load from the finite element model to the experimental load with rounding not greater than ±0,05.

The application of thin end-plates and the bolt rows placed only outside of the tension flange resulted in a brittle mode of joint failure, even below the load level corresponding to a global failure mode due to the out-of-plane instability of laterally unrestrained frame specimens. It indicates, in particular, that a proper selection of the strength and stiffness of the end-plate relative to the bolt group can enable to optimise the strength, stiffness and ductility of the whole joint and the structural system accordingly. More ductile connections are those in which their end-plates were not subjected to excessive bending deformations in the regions subjected to the HAZ effect. Depending on the connection strength, the arrangement of beam in-span lateral restraints and the sign of axial force in the beam, the local joint modes of failure or the global instability modes due to lateral torsional buckling can occur. The verification procedure shows that the accurate modelling of the contact-friction phenomenon is highly important in finite element modelling of end-plate connections. Any simplification would affect seriously the results. The shape of experimental frame load-deflection characteristics can only be reproduced by the finite element modelling scheme when the mentioned above effect is taken into account. The present investigations show that for relatively thin end-plates the effect of welding on mechanical properties of steel in HAZ regions is significant and it has to be included in the FE modelling. The increase of the values of the yield stress and the ultimate stress as well as the reduction of ductility ratio depend on the steel chemical composition, heat input, cooling rate, etc. An increase in the value of the yield stress has a positive effect slowing down the onset of plastification but the reduction of ductility increases the sensitivity to cracking due to the accumulation of plastic deformations and due to the low cycle fatigue, even in the case of quasi-cyclic loading. Because the presented finite element model is based on the conventional constitutive relationship used for parent steel, joint failures due to the end-plate fracture observed during experimental investigations could not be captured in numerical simulations. Because no universal laws governing the effects of material non-homogeneity and anisotropy due to welding have yet been developed, the investigations presented herein create a basis for our future study on the behaviour of semi-rigid joints with welded components, Jemiolo and Gizejowski [8].

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