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THE MECHANICAL PERFORMANCE OF THE HIDDEN IN-LINE CONNECTION FOR HOLLOW SECTION

Ph.D. Thesis – Abstract

for obtaining the scientific title of doctor at

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in the doctoral field of Mechanical Engineering

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1. INTRODUCTION

Free form structures lead to challenging development of details such as hidden connections to obtain the continuous aspect of the elements and thus the structure. If strength is the predominant consideration, the likelihood of using HSS (Hollow Structural Sections) increases, providing a significant advantage in the form of reduced self-weight for the final product. This lower self-weight brings about additional advantages, including reduced transportation and handling costs, smaller weld metal volumes resulting from thinner steel plates, and consequently, enhanced production speed. The use of HSS also enables the automation of the welding process, thanks to smaller and simpler welds.

To acquire the leading factors that influence the behavior of the connections in free form structures, the current thesis studies, experimentally and numerically, the response of a bolted connection whose aspect does not reflect the industrial view of a steel construction but more like a web of continuous elements. Due to technological reasons, this web hides the continuity connections. At high level of stress especially caused by bending, the finishing applied to the connection can deteriorate and the structure can lose the continuity aspect of elements. With limited design code provisions for such connections and no serviceability criteria, the thesis exhibits the response of in-line, invisible connections and the effect of the parameters on the capacity and separation of the connected elements.

In the entire frame assembly, the joints between elements and the connection of the pole to the foundation play crucial roles. Improper detailing of the latter can result in additional loads on the foundation, leading to uneconomical dimensioning. Currently, many structural designers limit joint verification to the maximum tensile stress in the screw. However, this approach has proven insufficient because the overall behavior of the joint is significantly influenced by other component elements, including the end plate, stiffeners, heart panel, and soles of the components.

The behavior of joints in thin-walled elements, especially the heart panel of portal metal frames, was investigated by Vayas et al., [1]. The study involved an experimental program with joints made by welding and subjected to static and dynamic loading. Joint strength was monitored through three failure mechanisms: shear failure of the heart panel, resistance of the stress field influenced by the ratio between the size of the heart panel and the soles of the components, and resistance of the soles of the components.

Previous studies by Lim and Nethercot, [2], Chung and Lau, [3], on nodes of portal metal frames made of cold-formed elements indicated that bolted joints in these frames exhibit semi-rigid behavior and are partially resistant.

The component method, a well-known calculation procedure for evaluating the properties of structural joints, is used as a reference in EN 1993, [4], and EN 1994, [5], for the dimensioning of joints in metallic and mixed structures. This method characterizes the properties of a node in three steps: identifying basic components, evaluating the mechanical properties of these components, and assembling the components.

To investigate the behavior of bolted joints under the coupled effect of bending moment and axial force, an extensive experimental and analytical study was conducted at the University of Liege. The developed mechanical model was used in a joint calculation program based on the component method. This program, conducted by Cerfontaine and Jaspart, [6], enables the numerical determination of the response of the rod-column joint subjected to bending with axial force.

The nonlinear behavior of portal metal frames with semi-rigid joints was studied by Nogueiro and Silva, [7], from the University of Coimbra, Portugal. The study concluded that the joist-to-joist joint from the ridge and the joist-post from the eaves have minimal influence on the non-linear behavior of the frame, with the fixing of the pillar in the foundation being the most significant contributor.

The thesis aims to highlight the adequate use of in-line connection by understanding the behavior of the connection components. In order to complete the thesis, the following objectives were proposed:

- Extract the existing studies on bolted connections in a literature review and state-of-the-art of the knowledge regarding the thesis subject.
- Present the theories for beam deflection.
- Perform experimental tests to obtain reference results for in-line connection subjected to pure bending.
- Validate a numerical model based on experimental results.
- Perform numerical parametric analysis to establish the influence of the access hole, end plate thickness, bolt preload, position of the access hole, the axial force.
- Propose design guidelines for the in-line connection.

2. BEHAVIOUR OF BOLTED CONNECTIONS SUBJECTED TO BENDING

This chapter presents the Architectural Steel Classes, the State of the Art, different in-line connections and a case study.

The architectural practice of revealing the structural system finds its origins in Gothic architecture. During this period, stone structures, columns, and buttress support systems were extremely detailed, becoming the defining aesthetic of the style. The development of iron and steel systems in the 1700s and 1800s continued this approach. The Structural Rationalist movement of the 19th century introduced a methodology of framed, elemental construction, accompanied by a specific language of connections.

Architecturally Exposed Structural Steel (AESS) denotes a specialized category of steel that must simultaneously satisfy two essential criteria. Firstly, it must be engineered to provide the structural integrity required to support the core functions of the building, including its primary structural elements, canopies, or supplementary structures. Secondly, AESS plays a pivotal role in the visual aesthetics of the structure, being prominently visible and serving as a significant component of the architectural language employed in the building's design. Any structural steel that remains uncovered and exposed to view falls within the ambit of architecturally exposed steel. It is important to note that AESS necessitates design, detailing, and finishing requirements that generally surpass those of conventional structural steel, which is typically concealed by other materials or finishes. Consequently, the planning, detailing, fabrication, assembly, and finishing of AESS systems invariably entail increased time and cost expenditures.

While the AESS categories 1 to 4 have been tailored specifically for the Canadian and Australasian systems, the concept of differentiated AESS categories remains broadly applicable, as they account for variations in finish, detailing, viewing distance, functionality, and cost.

In their research, Tobias Mähr and his team address a critical aspect often overlooked in structural analysis—the influence of joint behaviour on the distribution of internal forces, moments, and overall deformations in a structure, [13]. While this aspect is commonly disregarded, it becomes significant in cases where its effects are substantial, warranting thorough consideration. Eurocode 3, Part 1-8 [4], provides specific provisions for the analysis, classification, and modelling of joints concerning rotational degrees of freedom,[13]. However, it lacks provisions for translational degrees of freedom, [14].

One specific concern arises in shear connections, where a slip can occur if the slip resistance is smaller than the design load. This situation applies to shear connections falling under categories A and B according to Eurocode 3, Part 1-8, [4]. In structures predominantly subjected to normal forces, the accumulation of slip effects can significantly impact internal forces and deflections. Incorporating slip effects into structural modelling and calculations poses a complex challenge in structural mechanics.

Mähr and his team present concepts for effectively considering slip effects in joints, particularly within the context of large space frame structures. They outline "stick-slip" models and demonstrate their implementation in 3D structural models. Their paper delves into calculation techniques, particularly concerning statically over-determined systems and non-linear behavior.

The significance and advantages of this integrated analysis approach, where joint effects are integrated into the global modeling, are exemplified through the case study of the Louvre Abu Dhabi Dome. This remarkable dome, with a diameter of 185 meters and comprising approximately 11,000 steel members, predominantly utilizes shear connections. The research results underscore the imperative of not neglecting joint slip effects in global structural analysis. Furthermore, the study rigorously validates the quality and precision of the computed outcomes through on-site deflection measurements.

In their approach, the parameters required for the stick-slip model are derived from a combination of design codes and comprehensive site surveys, facilitating a practical solution. The results obtained through this heuristic method are deemed satisfactory for the Louvre Abu Dhabi Dome. However, the researchers acknowledge the need for further research endeavours aimed at establishing more comprehensive and universally applicable rules and methodologies to address the intricacies of this problem.

As K. Knebel and his colleagues S. Stephan and J. Sánchez-Alvarez present in their work “Reticulated structures on free-form surfaces”, [15], the design of reticulated structures on free-form surfaces comes with a lot of challenges for the structural engineers. Their paper is a brief review of the geometrical and the corresponding structural problems related to this topic, presenting Node Connectors for Single Layer Free-Form Structures and Double Layer Free-Form Structures.

H. Falter et al. [16] concluded in their paper “Beijing Airport Terminal3” that the architectural designs of Stansted Airport and Chek Lap Kok were profoundly shaped by the technical possibilities available in the 1980s for constructing expansive spatial structures using steel. Subsequently, advancements in fabrication technology played a pivotal role in the design of both the Airside Centre roof in Zurich and the Terminal 3 roof in Beijing, which were conceived as spatial structures composed of space trusses.

S. Stephan and C. Stutzki, [17], introduce a universal design methodology applicable to both single and multi-bolt connections of beams with arbitrarily thin-walled cross-sections. This methodology is designed for integration into computer programs. The approach draws inspiration from the classical strain iteration algorithm for cross-sections detailed in [18]. It involves iteratively determining the ultimate capacity of bolted connections through numerical calculations that capture the elastic-plastic stress distribution within the connection elements. The numerical method consists of two steps: first, determining the stress distribution within the connection for a given combination of internal forces, and second, calculating the connection's ultimate capacity.

Additionally, the paper formulates analytical design equations specifically tailored for multi-bolt tube connections. The research culminates in a comparative analysis, where results obtained from numerical and analytical calculations are contrasted with corresponding experimental test results.

In-line connections are commonly made through bolting methods, utilizing a flange plate with typically 4, 6, or 8 bolts, a splice plate, or an end plate. The end plate can be concealed behind a cover plate.

Additionally, a profiled cover plate may be incorporated to ensure the connection remains unseen.

In numerous cases, it is necessary to streamline connections or enhance efficiency, particularly through bolted methods, to adhere to erection schedules. Given the costly nature of crane time, lifts are often scheduled during nighttime, especially if closing major streets or highways is required to facilitate project access. In such scenarios, opting for a welded connection may not be feasible due to time constraints or limited access.

The Musée de Confluence, an architectural masterpiece in Lyon, France, designed by the architectural office Coop Himmelb(l)au, showcases an avant-garde fusion of computational design, biomimetic principles, and advanced engineering techniques. While the architect imagined the outer skin of the building to be a cloud, the engineers from the german company Josef-Gartner GmbH had the difficult task to transform this façade shape into reality. Situated at the confluence of the Rhône and Saône rivers, the museum's dynamic and fluid form challenges conventional norms through parametric modeling and computational algorithms, enabling intricate non-linear geometries that harmonize with the urban context.

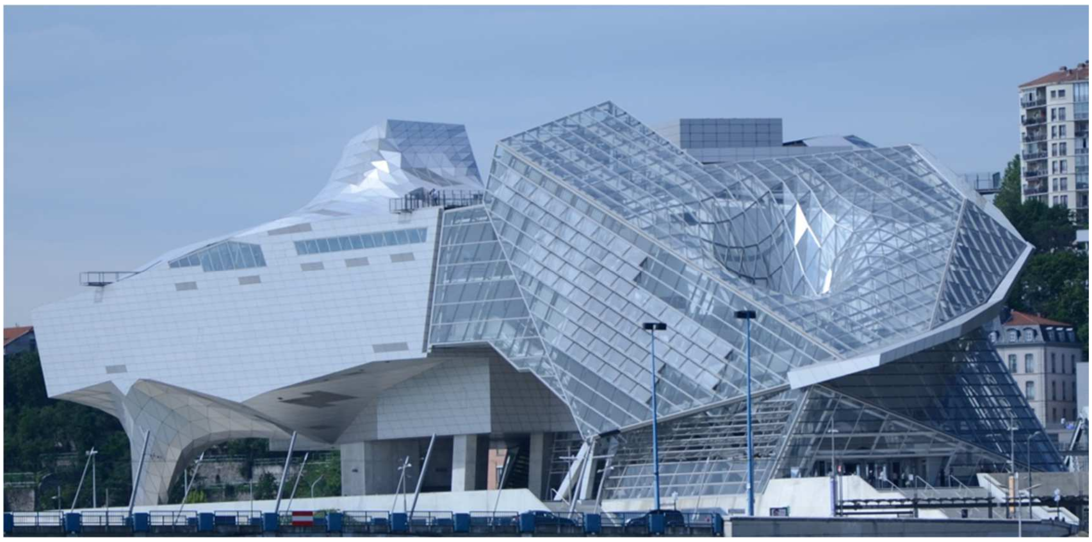


Fig.2.1 Musée de Confluence – Isometric view (Front)

The facade of the museum is an innovative composition of materials, primarily glass and metal. The extensive use of high-performance glazing ensures optimal daylighting and views, while the metal cladding acts as both a design element and a functional shading system, mitigating solar heat gain and enhancing thermal performance.

As an example of such an “invisible connection”, two RHS 450x250x20 steel profiles are presented bellow. The end-plates are welded inside of the profile with a 2 mm offset to the edge, having a minimum gap of 4 mm when connected together in order to assure the flow of stresses only through the cross-section of the profiles. There were used 8 pretensioned bolts and 2 hand holes, one on each side of the connection, covered by 2 plates fixed with countersunk screws. The RHS profiles got a 1,5x1,5mm chamfer all around the perimeter of the cross-section. These will form a V-shape that after pretensioning process will be top up with a filler, grinded and painted. The same will happen also with the gaps between the hand-holes cover plates and the profiles, and the head of the countersunk screws.

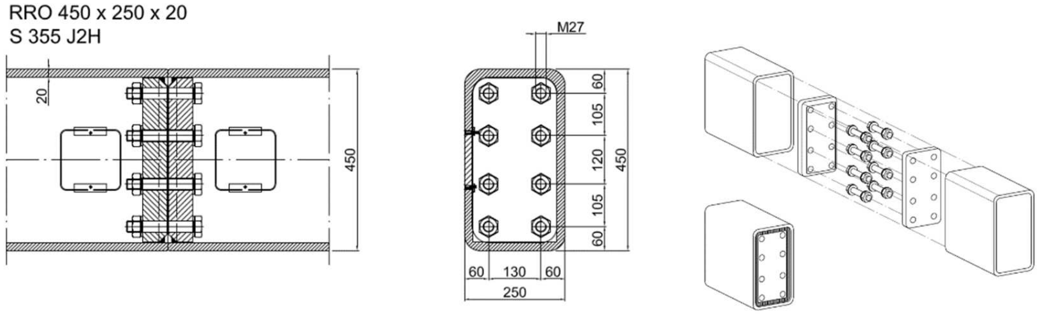


Fig.2.2 “Non-visible” connection - RHS 450x250x20

In order to achieve the precision required for a perfect fit of the components of the hidden connections, top-of-the-game advanced fabrication technologies were employed, including laser cutting, CNC machining, and robotic welding.

Despite their concealed nature, the hidden steel connections play a crucial role in providing robust structural support. Rigorous analysis and testing were conducted to meet stringent safety standards, delivering a building that is not only visually striking but also safe and stable.

3. STRUCTURAL RESPONSE OF ELEMENTS WITH IN-LINE CONNECTIONS

The 3rd chapter delves into the deflection analysis methods and the elasto-plastic of bent elements.

Structures, similar to other physical entities, undergo deformation and alter their shape when exposed to forces. In the case of elastic deformations—those that revert to the original shape once the structure is no longer under stress—various techniques for calculating deformations are recognized. These methods are categorized into geometric methods and energy methods.

The explanation of the differential equation considers a straight elastic beam exposed to random loading, acting perpendicular to its centroidal axis and within the plane of symmetry of its cross-section. The deformed state's neutral surface is named the elastic curve.

The direct integration method involves expressing the ratio of bending moment to the flexural rigidity of the beam (M/EI) as a function of the distance x along the beam axis. This expression is then successively integrated to derive equations for the slope and deflection of the elastic curve. The constants of integration are determined by considering the boundary conditions. This method is particularly effective for calculating slopes and deflections of beams when M/EI can be represented as a continuous function of x across the entire beam length. However, the direct integration method becomes less straightforward when applied to structures where the M/EI function is not continuous. This complexity arises because each discontinuity, resulting from changes in loading and/or flexural rigidity (EI), introduces two additional constants of integration in the analysis. These constants need to be evaluated by enforcing the conditions of continuity of the elastic curve, a process that can be laborious. To address this challenge, the use of singularity functions, as defined in many mechanics of materials textbooks, can provide a workaround and simplify the analysis.

Charles E. Greene introduced the moment-area method for calculating slopes and deflections of beams in 1873. This approach relies on two theorems, known as the moment-area theorems, which establish a connection between the elastic curve's geometry and its M/EI diagram. The latter is created by dividing the ordinates of the bending moment diagram by the flexural rigidity EI. The moment-area method employs graphical interpretations of integrals involved in solving the deflection differential equation, utilizing areas and moments of areas from the M/EI diagram. This makes it particularly advantageous for beams with loading discontinuities and variable EI compared to the previously described direct integration method.

The conjugate-beam method, introduced by Otto Mohr in 1868, generally provides a more convenient way to calculate the slopes and deflections of beams compared to the moment-area method. Although both methods require similar computational effort, the conjugate-beam method is preferred by many engineers due to its systematic sign convention and ease of use. Unlike the moment-area method, it does not require sketching the structure's elastic curve.

The conjugate-beam method is based on the analogy between the relationships of load, shear, and bending moment with those of M/EI, slope, and deflection.

The principle of virtual displacements for rigid bodies can be stated as follows:

"If a rigid body is in equilibrium under a system of forces and if it is subjected to any small virtual rigid-body displacement, the virtual work done by the external forces is zero", [50].

In this section, we explore another energy method for calculating structural deflections. This method, applicable only to linearly elastic structures, was first introduced by Alberto Castigliano in 1873 and is widely known as Castigliano's second theorem. (Castigliano's first theorem, which helps establish the equilibrium equations of structures, is not covered in this text.) Castigliano's second theorem is articulated as follows:

"For linearly elastic structures, the partial derivative of the strain energy with respect to an applied force (or couple) is equal to the displacement (or rotation) of the force (or couple) along its line of action".

The stage of the elasto-plastic bending represents the moment when more than one fibre reach the yield limit. For the idealized stress-strain curve (elastic-perfect plastic) the material cannot sustain a stress greater than yield stress and the fibers at the yield stress progress inwards towards the center of the beam. An elastic region and a plastic region exist over the cross-section. The ratio of the depth of the elastic core to the plastic region can take values from 1 to 0. Since an extra bending moment is being applied and no stress is bigger than the yield stress, extra rotation of the section occurs: the moment-rotation curve loses its linearity and curves, giving more rotation per unit moment which is qualitatively expressed as the loss of stiffness.

When all the fibres in the cross-section reached the yield limit the section reached the stage of Plastic Moment Capacity. At this moment the element increases its rotation without any increase of the bearing capacity.

If the material is able to harden, an increase in the bearing capacity is observed otherwise, once the plastic moment capacity is reached, the section can rotate freely similar to a hinge but having a concentrated moment in that section.

When analyzing a section for the plastic response, two characteristics are of interest, i.e.

plastic section modulus, Wpl, and the shape factor f.

Considering design codes, plastic analysis is discussed in the European steel structures design code EN 1993-1-1 [13] from the global analysis point of view. If the elastic critical buckling load for global instability mode based on initial elastic stiffnesses (Fcr) is 15 times bigger than the design loading on the structure (FEd) a plastic analysis may be performed.

Global plastic analysis can be applied exclusively when the structure exhibits adequate rotation capacity at the specific positions of the plastic hinges, whether they are within the members or at the joints.

As a simplified approach for a constrained plastic redistribution of moments in continuous beams, where peak moments exceed the 15% maximum plastic bending resistance following an elastic analysis, the excess portions beyond these peak moments can be redistributed within any member.

One of the conditions that must be fulfilled by the element is to be of class 1 or class 2. The classification of cross sections aims to determine the degree to which the local buckling resistance constrains the resistance and rotational capacity of the cross sections. Eurocod 3 [13] classifies the cross-section in four classes:

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
- Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

4. EXPERIMENTAL PROGRAM

In the preliminary phase, the study started from the use of the GAS_WIN program developed by Knebel based on the design method of spatial frames proposed by Stephan and Stutzki [17]which makes an elastic analysis of the continuity joints, embedded, of the tubular profiles. This method does not provide information, however, about the behaviour of the joint in terms of deformations, in particular about the separation of the joint profiles.

The use of finite element programs can be a solution in obtaining these results, but the correctness of the initial data and the analysis model can only be validated based on experimental tests.

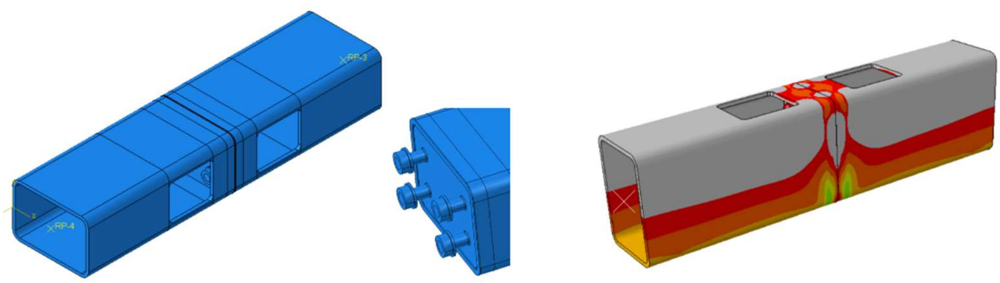


Fig.4.1 a) FEA model of the joint, b) Stress distribution (negative moment)

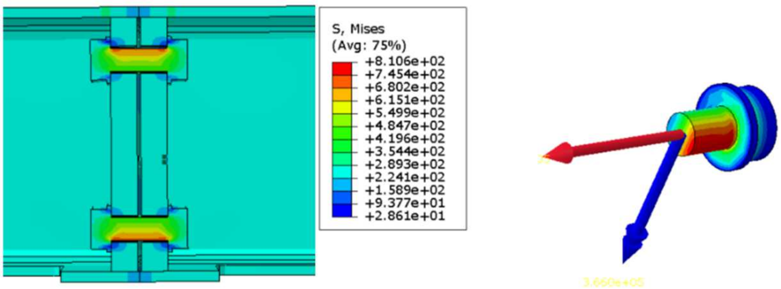


Fig.4.2 a) Stresses in the joint, b) Internal forces in the bolt

The results obtained highlighted the decrease in stiffness both due to the joint but also due to the reduced cross-section in the area of the access holes for the joint.

The purpose of the experimental tests is to obtain records that are the basis for the validation of the numerical models in the finite element analysis. With the validation of the numerical models, a parametric study of these joints can be carried out in which the separation of the elements, the pretensioning of the screws, the deformations of the joined elements and the axial force-bending moment relationship can be studied.

In practice, both rectangular and circular tubular profiles are used so that the experimental tests include both types of profiles. Two quasi-static tests, One monotonic and one cyclic, were performed on each type of profile in the configuration shown in Fig.4.4.

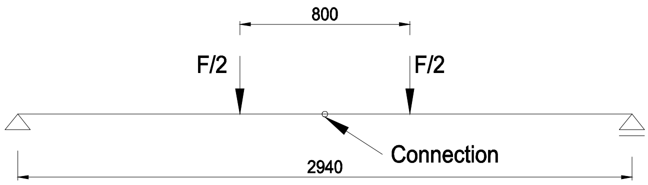
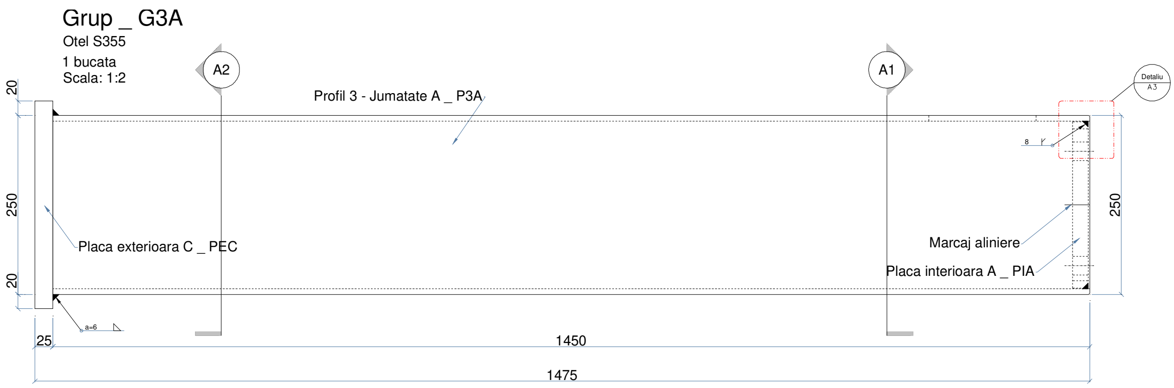


Fig.4.3 Testing configuration

The tested specimens were made by joining two elements with the same section, having the profiles RHS 250x150x8, respectively CHS 114.3x10. Their name was chosen according to the type of profile RHS or CHS followed by the number of the test.

The execution drawings are shown in Fig.4.4 and Fig.4.5. Each specimen has access holes for tightening the joints with high-strength bolts. Due to the lower capacity, the top position of the access holes was chosen.

Before testing the assemblies, measurements were made to determine the actual dimensions of the profiles.



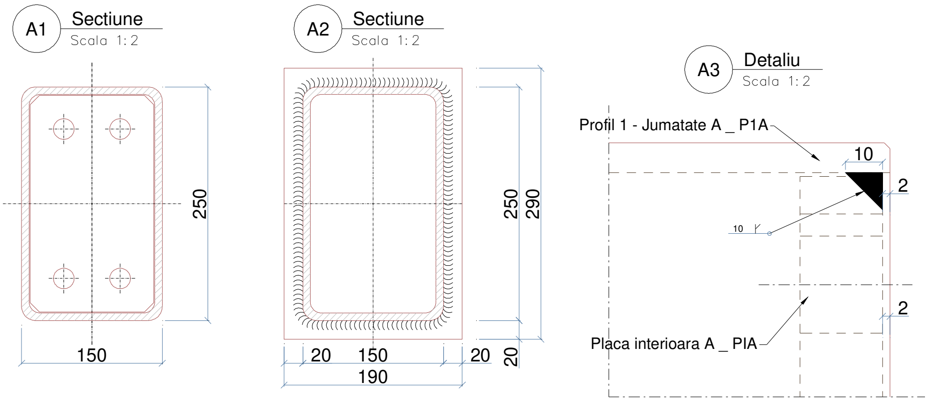


Fig.4.4 Execution drawings for RHS250x150x8

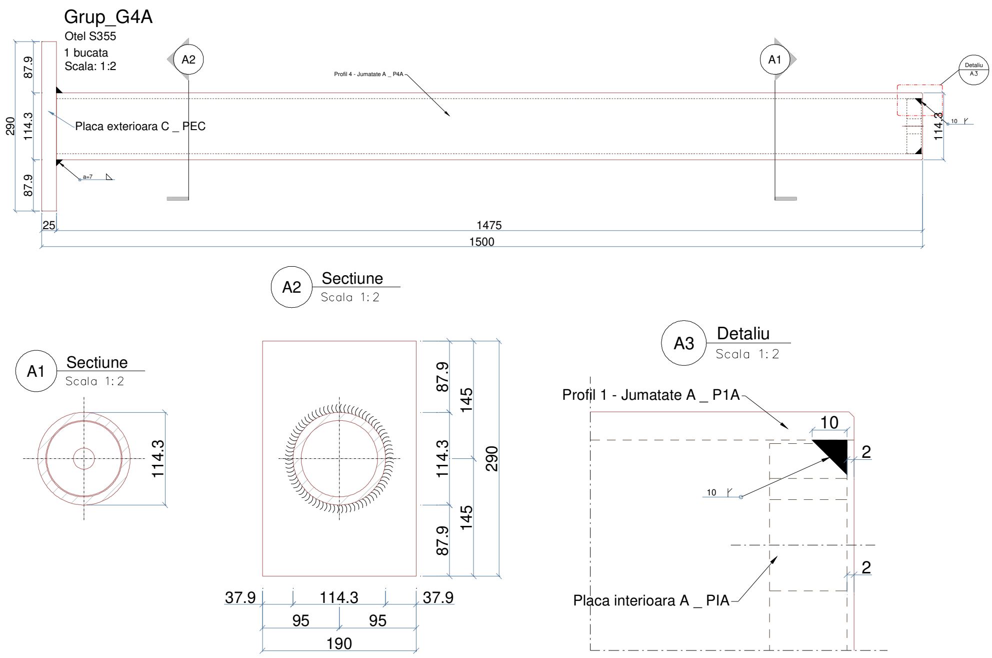


Fig.4.5 Execution drawings for CHS114.3x10

The bolts used in the joints were M20 gr 10.9 for the RHS and M24 gr 10.9 for the CHS, EN 14399.

The pairing of the specimens is a compulsory condition for a constant pressure in the joint and to obtain perfect match, the parts were noted with A and B, presented in Fig.4.6 and considered for the final assembly of the specimen.

Fig.4.6 Marking of the parts to obtain paired specimens

From the profiles to be tested, specimens were extracted for tensile tests according to ISO 6892-1 [20].

The bolt preload was evaluated separately to verify the torque wrench. Five bolts were preloaded and measured the force in the bolts by a universal testing machine load cell

The tests on the bolted assemblies were carried out in the Laboratory of the Department of Steel Structures and Structural Mechanics, in the framework of 2D tests.
In the experimental framework, a 500kN actuator of vertical direction was mounted and the supports of the assembly were positioned at the correct distance. To avoid out of plane displacements of the actuator, an independent structure was built, Fig.4.7.

The supports of the specimens obtained from steel parts and allowed rotation and horizontal displacement so that the static scheme can be considered as a beam with pinned and roller support.

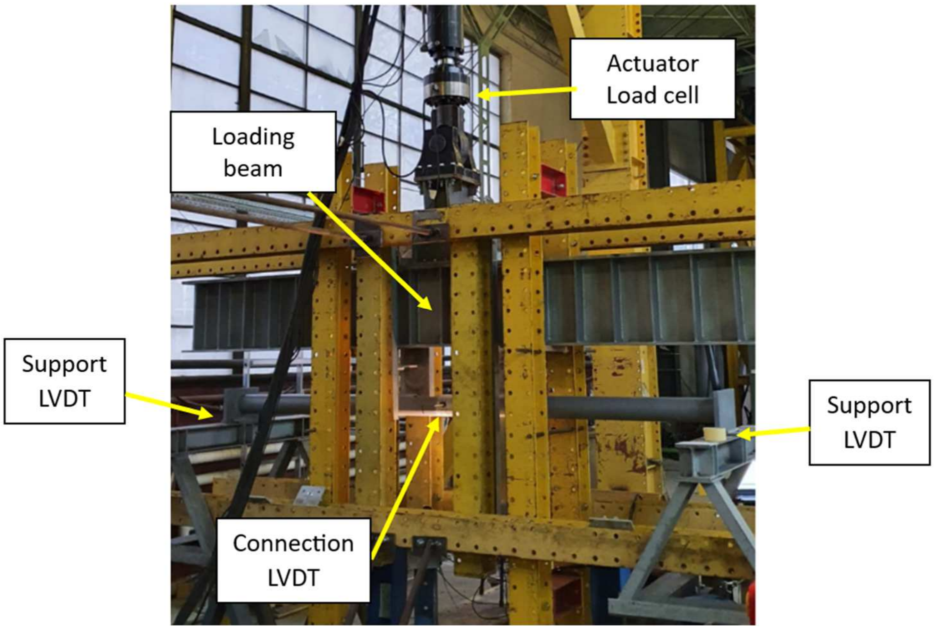


Fig.4.7 Test setup

The transmission of loading forces was achieved by means of devices that allowed the loading points to rotate freely. In the case of RHS profiles, the device contains a roller and 8mm steel plates to distribute the load over the entire width of the upper flange), while for the CHS profiles, a steel part was cut out to transmit the loads to the upper contour of the profile.

The recorded data consisted of monitoring the loading force and displacement of the actuator head, data obtained from the actuator load cell, and displacements from the LVDT (linear variable displacement transducers) positioned next to the supports, respectively.
In addition, an optical monitoring system based on the Digital Image Correlation (DIC) technique was used. With its help, the strains that appear in the monitored area, but also displacements between the set points, are monitored. Monitoring was carried out on the extended area of the bolted joint at the lower sole. A surface with surface contrast is required for monitoring, so a layer of matte white paint was applied over which the speckle patterns were created.

The current results contain both qualitative information, behavior and failure mode, as well as quantitative results, displacement force curves recorded by the actuator and the LVDTs.

During loading, no deformations of the cross-section walls were observed near the access holes provided for tightening the joint bolts. The failure occurred through the thread stripping, without a brittle failure.

The qualitative results consist of the force-displacement curves based on which the numerical model will be calibrated. The displacement consists of the difference between the vertical displacement of the joint and the average of the displacements of the supports. For the RHS profiles the F-D curve is.

The separations between the joint of the two profiles, were obtained from the DIC recordings. It is observed that the two specimens show very close values of the joint separation. Also, the optical monitoring system can provide information about the distribution of specific strains in the joint area.
Images were programmed to be recorded every 2 seconds and only each 100th image is shown here.

It was observed that after the failure of the thread, the monitored area presents positive tensile stresses. The explanation for such a response can be found either in plastic deformation or, more likely, compressive strains caused by bolt pretensioning, not considered by DIC, turn into positive strains after failure. Unfortunately, the amplitude of these deformations is not known since the failure of the thread is still subjected to force. If these deformations are to be quantified, the reference DIC image must be taken before prestressing, but since this process involves displacements of the specimen, the technique is almost impossible. A method for evaluating these deformations can be found in the numerical analysis solution, which will be presented in the following chapters.

The cyclic tests were performed to observe the behaviour of the connection, the joint separation, after several loading cycles. A pulsating tests protocol was considered with an amplitude of approximately 50% of the monotonic tests. The pulsating protocol represents loading the structure to a certain force and unloading the specimen to a null stress. For each force direction (loading and unloading), a 10 s interval was chosen resulting in a loading rate of 0.05Hz per cycle. This allowed the DIC monitoring system to record the joint separation, dL. For each specimen, a total number of 500 cycles were performed.

5. NUMERICAL STUDY

A finite element model was defined to observe the response of the joint and for parametric investigations. The analysis was performed using Abaqus [25]. 3D solid elements were defined as parts of the assembly for the RHS profile, connection end plate and bolt, respectively, Fig.5.1, [22].

Although the testing setup presents a symmetry plane, the entire experimental specimen was modeled, Fig.5.2. [22].

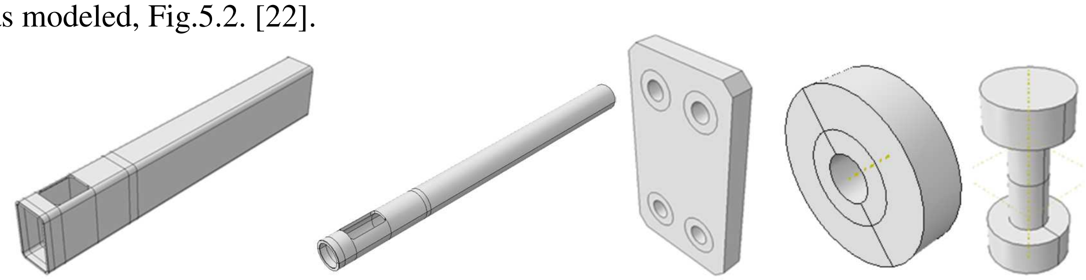


Fig.5.1 Defined parts of the RHS and CHS FEM model, [22]

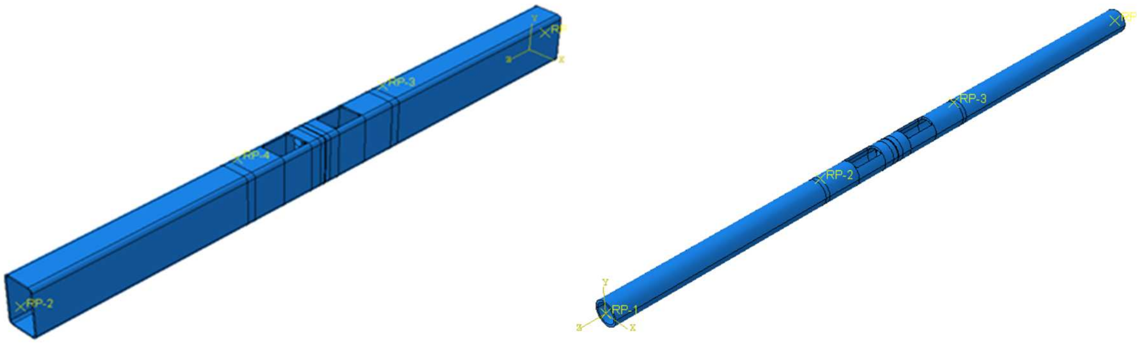


Fig.5.2 RHS and CHS assembly of the finite element model, [22]

The material properties were considered from the tensile tests performed on specimen extracted from the RHS profile and from the bolts material. In the FEM model, the plastic model was used considering a bilinear curve. The elastic part is defined by the elasticity modulus, 210000N/mm2, while the plastic part is considered to be between the yield point and the tensile strength, 441-553N/mm2 , 396-516 N/mm2 and 1104-1177N/mm2 for the RHS, end plate and bolt, respectively. These values were transformed into true stress using the relation provided by Eurocode 1993-1-5 [21], leading to a tensile strength of 605 N/mm2 , 557 N/mm2 and 1231 N/mm2, for the RHS, end plate and bolt, respectively, [22].

A pinned and a roller support were defined at the assembly ends, while a displacement of 30 mm was defined using Reference Points in the point load positions.

These four reference points were connected to the assembly by Kinematic Coupling for the RHS cross-section and loading area.

The connection end plate was connected to the RHS profile using the Tie constraint. Supplementary, contact interaction are necessary to be defined between the bolts and the end plate, and between the two RHS profile which are in contact.

The contact interaction was defined for normal and tangential behaviour, allowing separation after contact. The tangential contact was defined with a 0.1 friction coefficient.

Each part was meshed with C3D8R (an 8-node linear brick, reduced integration, hourglass control) finite element type. The size of the element was approximately 5mm, allowing 2 elements on the thickness of the RHS element. In order to reduce the computation time, the length of the finite element in the longitudinal direction was increased, [22].

For the bolts, a preload of 130kN was defined in a separate step before the applied load, analyzed in a Static step. The preload propagation was modified to "Fix at cur-rent length" for the following analyzing steps, to allow the development of internal stress in the bolts due to bending of the specimen. The value of the preload is close to the value determined in the experimental program for the torque wrench, Table 4.2

By performing the finite element analysis, the force-displacement curve is compared to the one obtained from the experiment.

A very good correspondence is obtained for the initial rigidity of the assembly. Although, the maximum force is in the range of 10% less caused by a decrease in the rigidity of the FEM model, the model is considered to be valid for further studies on the influence of preload, end plate thickness or the access hole.

An important phenomenon shown by the model is the non-uniform distribution of the pressure between the RHS profiles as well as the bending in the bolts already developed in the preload stage.

Although the endplate is very thick, 25 mm, the bending of the plate lead to non-uniform pressure of the bolt head, thus, the bending of the bolt shank.

By extracting the longitudinal displacements of the nodes close to the connection and adding the absolute value of the displacements of the two nodes, the joint separation was obtained. It must be mentioned that the preload introduced an initial deformation of 0.045 mm. This value was subtracted from the final joint separation as the deformations in the experiment were not included in the recordings of the DIC system.

A parametric study was performed taking into consideration the connection effect, the hand hole effect, the bolt preload effect, the end plate thickness effect, position of the acces hole effect and the axial force effect.

6. DESIGN RECOMMENDATIONS

Qualitative recommendations

Connection preparation of the in-line connection represents an important role for the aesthetics of the connection. Due to the possibility to obtain non perpendicular cuts to the element axis, the connection must use the elements of the same cut which must be marked such that the assembly is connected tip to tip.

Moreover, due to the bolt hole tolerance, the connection might have slippages, thus a clamp that touches both connected elements must be used until the bolt preload stabilizes the connection through friction.

Regarding the access hole, in order to avoid high stress concentration, a fillet radius between the edges of the access hole is compulsory.

The dimensions of the access hole have to be designed in accordance with the tools used for the bolt preload, i.e. torque wrench or hydraulic bolts prestress equipment.

As the access hole reduces the capacity of the profile, the designed dimensions must be ensured. Also, function of the shape of the profile and utilization factor of the profile, local buckling can occur (mostly for rectangular hollow sections).

The end plate position is considered inside the hollow section, 2 mm away from the cut plane. Nevertheless, this distance should be a carefully chosen function of the welding type (fillet weld or chamfered plate) between the end plate and the inner face of the tube. If the distance is too small for a fillet weld, the weld throat can extend to the other side of the cut plane.

As discussed, the wall thickness may deform if its thickness is too small or if the end plate is too thin and deforms under the bolt load. The capacity of the element depends on the profile height and the wall thickness. For an economical design, this bending moment capacity should have a value close to the capacity of the bolted connection. In case of a thick endplate, the bending of the element wall will be neglectable.

The failure of the tested connection showed two types of failure e.g. thread stripping and bolt shank fracture. EN 14399-1-2015 [23], describes two types of bolting assemblies, i.e. HR and HV. Type HR assembly is designed to obtain ductility predominantly by plastic elongation of the bolt having a minimum nut height ≥ 0.9 D and long thread lengths, with specifications according to EN14399-3, [26]. Type HV is designed to obtain ductility predominantly by plastic deformation of the engaged threads having the nut height at approximately 0.8 D. The HV assembly generally fails by thread stripping all the shank thread length allowing to maintain a certain amount of resistance even after failure while the HR has a more fragile failure result. Nevertheless, as shown in the current experimental test section, although HV assemblies were used for the connection, the specimen with one bolt in the connection failed in an explosive manner, without thread stripping signs. This failure of the HV assembly in the current connection is associated to the large bending effect which caused the initiation of a stress concentration in the thread.

Thus, to avoid sudden failure of the connection it is recommended to use bolts of type

HV according to EN14399-4, [24].

Quantitative recommendations

For practical purposes, the response of a connection is commonly established by using the component method. This analytical method offers reliable results for establishing force-displacement curves of semi-rigid joints.

Eurocode 3, part 1-8, [4] presents 20 components which can be calculated for their resistance and rigidity.

Starting from the connection geometry, for the current RHS connection, the following components are considered: bolt in tension (bt), end-plate in bending (epb) second row of bolts (bt'), the lower bending moment of the end-plate (epb') will be considered. For the CHS connection only bolt in tension (bt), end-plate in bending (epb) and the beam wall in tension (bwt) are considered.

In this subchapter the following are calculated: connection resistance, connection stiffness for both RHS and CHS.

7. CONCLUSIONS OF THE THESIS

Unobservable stress development and deformations of the structural parts involved in an assembly represent the in-depth behaviour of a connection. The aim of this research is to highlight the response of an in-line connection of the tubular section with the interior end plate.

The research work is focused on three main parts:

•Experimental tests on the in-line connection of rectangular and circular hollow sections
•Numerical analyses (validation of a numerical model and a parametric study)
•Design guidelines for the in-line connection.

Starting from the literature review regarding the "invisible" connections which can transmit combined internal forces, the research highlighted the possible configurations of continuous connections. The position of such connection in a structure is important and the technology of using "invisible" connections, with bolts situated inside the tubular sections involves certain assembling restrictions and limited flexural rigidity as most of such connections are used for axial force transmission.

Information on the design regulations for architectural steel and how connections should be treated and processed depending on the visual impact on the observer are iterated by the author. This is followed by an explanation of the in-line connection types and a practical case study on how the in-line connection was used in a structure.
Since an important part of the connection is represented by its deformation, a comprehensive list of deformation computation methods is presented and used to show a comparison between the analytical relations and the experimental results. The comparison shows the importance of the in-line connections in determining the real behaviour of structures.

The experimental program results offered the base of the numerical study, for the model validation and parametric analyses. Nevertheless, the assembling of the specimen showed that even if the elements are perfectly cut, the alignment of the connected elements depends on the tolerance of the bolt holes.

Due to the position of the bolts, inside the section perimeter, for a rational design, all the connections depend on the bolt capacity. HR bolt type are recommended to be used as the thread stripping of the bolts maintains a certain amount of resistance.

The numerical analyses showed a small influence of the bolt preload and the position of the hand hole, while a great importance in the capacity of the joint are the bolt resistance and the end plate thickness.

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