High Strength Long Span Structures (HILONG)
European Commission

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High Strength Long Span Structures (HILONG)

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1 FINAL SUMMARY

Objectives

The use of high strength steel (HSS) can lead to a significant reduction in the weight of a steel structure. A lighter structure requires smaller foundations, shorter transportation and construction times, and leads to lower CO$_2$ emissions and energy use (both directly in less materials used and also indirectly due to lower transportation costs). Although HSS have found application in machinery and automotives, they are not widely used in construction because the benefit of reduced weight struggles to outweigh the disadvantages of higher price/tonne, reduced availability and different weld procedures.

HILONG investigated innovative structural arrangements, design methods and cross-sections which enabled the benefit of high strength to be maximised by suppressing buckling and reducing deflection. The study focussed on long span applications such as stadia, auditoria, exhibition halls etc. The strengths of HSS studied were S460 and S690.

The technical objectives of the work were:

- To develop more cost-effective design methods which suit the specific material characteristics of HSS,
- To develop design methods for HSS prestressed cable-stayed columns and trusses which enable a greater proportion of the higher strength to be utilised by suppressing buckling and limiting deflection,
- To investigate the structural performance of innovative U-shaped and semi-closed polygonal cross-sections which enable joints to be fabricated more easily,
- To develop comparative designs for two types of functionally equivalent long span structures which demonstrate the savings possible using HSS in terms of weight, cost and environmental indicators,
- To prepare a series of design examples for members and joints which demonstrate the design methods developed,
- To hold a seminar to disseminate the results of the project to designers and other interested parties.

The idea for the HILONG project came from Sweco, the designer of the new Friends Arena stadium in Stockholm, which uses four grades of HSS: S460, S550, S690 and S900 in various structural forms for the main roof truss. The use of HSS led to a reduction in the weight of steel and environmental impact of between 13-17%. Sweco expressed an interest in studying how further cost savings could be made if HSS was used more extensively and more efficiently in the design of the stadium roof, and this motivation led to the definition of the HILONG research objectives.

WP1 Case studies

The objective of WP1 was to take two different structural configurations, a long span roof truss and a prestressed cable stayed column, and prepare two functionally equivalent designs in conventional strength steel and HSS to compare the savings in weight, cost and environmental indicators.

The first case study was a roof truss based on the design of the Friends Arena main truss. A number of design configurations were considered which included studying the benefit of prestressing the bottom chord, different cross-sections for the top and bottom chords (tubes, U-shaped profiles, polygonal sections) and the impact of using higher strength steels. The environmental impact of using HSS in all aforementioned scenarios was also assessed through a Life Cycle Environmental Analysis in terms of global warming potential, acidification potential, eutrophication potential and primary energy demand.

The second case study was based on the masts supporting the roof at Manchester City’s Stadium and a range of different configurations were studied which involved varying the cross-section of the column and using higher strength steels. The advantages of adding a prestressed cable stayed system to the column with varying levels of prestress were also quantified.

Within the scope of these studies, the potential advantages of using higher strength steels in terms of cost and environmental impact were clearly demonstrated. The size of the savings depended on a wide number of parameters including the governing load case, the cross-sectional and global slenderness of the sections, the structural configuration, level of prestress, the assumed cost premium for HSS material and weld consumables, the assumed cost premium for fabricating HSS etc. Other factors may also influence the results of a cost analysis, for example fire protection costs may be higher because HSS structural members will be lighter.
A public seminar was held in Sweden at the start of the project with practitioners and researchers to discuss the opportunities and challenges to the use of HSS in structures. HSS producers presented information on their production processes and structural section ranges, illustrated with examples of previous structural applications. A survey of three large UK fabricators was also undertaken to understand their experiences of fabricating HSS structures in recent projects.

**WP2 Materials and section design**

The main objective of this Work Package was to perform full-scale tests on HSS structural material and cross-sections in order to address the current lack of experimental data which is restrictive for the development of design guidance. A programme of tests was carried out on hot finished square and rectangular hollow sections in S460 and S690. The sections were subject to bending moments, and axial compression, both concentric and eccentric. Tensile and compressive coupon tests were also carried out to determine the basic stress-strain response of the material.

The tests were numerically modelled. Close agreement was achieved between the measured and predicted response. The sensitivity of the models to geometric imperfections was also assessed and a geometric imperfection of $t/50$ was found to best replicate the cross-section response. A parametric study extended the scope of the test programme to study a wider range of cross-sectional slenderness, aspect ratio and loading scenarios. In these models, the material properties were based on the averaged flat coupon stress-strain curves.

The experimental and numerical results enabled an assessment of current design guidance for HSS structures, codified in EN 1993-1-1\(^1\) and EN 1993-1-12\(^2\) to be carried out. It was shown that S460 to S690 satisfy both the more relaxed requirements for material ductility given in EN 1993-1-12 as well as the stricter limits of EN 1993-1-1 related to the ultimate to yield strength ratio $f_u/f_y$, the strain at fracture $\varepsilon_f$ and the ultimate to yield strain ratio $\varepsilon_u/\varepsilon_y$. Moreover, it was found that the EN 1993-1-12 relaxed $f_u/f_y$ limits and $\varepsilon_f$ limits are also generally satisfied by the higher strength materials with yield strengths above 700 MPa; this was however not the case for the ultimate to yield strain ratios $\varepsilon_u/\varepsilon_y$.

Regarding section classification, the stricter limits for internal compression elements which will be introduced into the next revision of EN 1993-1-1 ($c/t_e = 28$ for Class 1; $c/t_e = 34$ for Class 2 and $c/t_e = 38$ for Class 3) better reflected the behaviour of HSS internal compression elements than the existing limits ($c/t_e = 33$ for Class 1; $c/t_e = 38$ for Class 2 and $c/t_e = 42$ for Class 3).

The effective width equation for Class 4 cross-sections and the interaction curves for cross-sections under combined axial load and bending gave conservative predictions for HSS cross-sections.

An alternative design method for stocky HSS was developed called the Continuous Strength Method (CSM) which takes advantage of the beneficial effects of strain hardening. The method has previously been successfully applied to stainless steel cross-sections. The CSM offers enhanced capacities for stocky HSS cross-sections of up to about 10% compared to the Eurocode 3 predictions.

A reliability analysis was carried out to assess the reliability of the slenderness limits and the effective width equation for HSS. As part of this analysis, material strength data were collected from producers in order to determine the mean and standard deviation values for the yield and tensile strengths. A partial safety factor greater than unity was determined for the effective width formula for Class 4 HSS internal elements. However, a similar observation has also been made for conventional strength steel, so this is clearly not an issue just relating to the higher strength.

**WP3 Joints**

This Work Package studied the performance of different types of truss joints between tubular, U-shaped and polygonal cross-sections through tests and numerical analysis. Polygonal cross-sections give freedom to optimise the cross-section (external dimension, thickness) and ability to fabricate the cross-section wherever a press-braking facility is available, with plates delivered in sizes required for optimal fabrication.

**Crocodile nose connections (CN)**

A CN connection is a new type of tube to gusset plate connection, suitable for the diagonal members in a truss, which avoids the classical abrupt and unattractive right-angle termination of a CHS. In a CN connection, semi-elliptical nose plates welded to the tube provide a smoother transition with many structural advantages by continuing beyond the termination and then being bolted to the gusset plate. A programme of strength and long-term tests were carried out to investigate the structural behaviour of this type of connection. For the test specimens, the tubes were S690 159 x
8.8 mm seamless CHS. The inflected plates were 4 mm thick S650 steel. There were six pretensioned 10.9 M26 x 70 bolts in each connection.

Four strength tests studied the effect of the angle of inflection of the nose plates and the presence (or absence) of the connection piece on the strength of the connection. Both specimens with a connecting piece failed suddenly in a brittle way, one by facture of the full width of the inclined plate and the other in the weld. The specimens with the shallower angle of inflection failed at a 15% higher load than the specimen with the steeper angle. In comparison, both specimens without connecting pieces failed at significantly lower loads, experiencing progressive fracture of the weld, demonstrating a hardening post-failure behaviour. As the crack propagated along the weld length, the inflected plates started to straighten, at the same time bending below the lower bolt row and separating from the gusset plate.

Four long-term tests studied the loss of pretension in the bolts to the gusset plate over a four month period. A logarithmic curve of the form \( \text{pretens} = \ln(\cdot) + \) was fitted to model the asymptotic loss of pretension due to relaxation induced by creep.

FE models of the CN connection consisting of the inflected plate, CHS member and weldment were developed and validated against the strength test data; the models predicted the tensile resistance of the four test connections reasonably well. A parametric study was carried out to determine the effect of varying the steel strength, plate thickness and inflected angle. The parametric study confirmed that:

- A more ductile failure of the weld material will occur when the connecting plate is not used. Inclusion of the connecting plate results in a more brittle failure of the weld material (though at a significantly higher load). Indicatively, the connecting plate reduced the failure displacement by approximately half.
- Higher strength steel S690 and S460 provide 62% and 13.4% greater load resistance with 13.3% and 7.6% smaller failure displacement, respectively, compared to S355.
- In general, a smaller angle of inflection results in higher resistance due to the larger interface between the inflected plate and the CHS member and therefore the greater length of weld. Without the connecting plate, the ultimate strength of the connection increases by 38% and 60% when the angle of inflection is reduced from 35° to 25° and from 35° to 21° respectively. Without the connecting plate, the strength is increased by 45% when the angle is reduced from 35° to 25°.
- Increasing the inflected plate thickness and the thickness of the CHS member will significantly increase the strength of the connection. This is primarily due to the size of the weld material which is physically limited by the thickness of the inflected plate and the CHS member. Also, components of greater size/area can accommodate more weld material and therefore result in a higher load resistance.

A design equation was subsequently developed to calculate the resistance of the weld in the CN connection. The equation can be used to determine a transition angle at which failure shifts from the weld to the inflected plate. This minimum angle can be used in design to ensure that the weld in the connection will not be the weakest part under tension.

**Other truss connections**

Tests were carried out on two 14 m long trusses to study the response of different types of joints between the chord and bracing diagonals. One of the trusses comprised innovative cross-sections: polygonal sections for the top chords and a U-shaped profile for the bottom chord. The other truss was made from tubular members. Polygonal and U-shaped cross-sections facilitate easier connections with minimal welding. The polygonal cross-section is also less susceptible to local buckling than a conventional tubular cross-section.

**Innovative truss**

The polygonal compression chord was made from 4 mm S650 plate formed from 3 sections of folded plate into an octahedral cross-section. The U-shaped tension chord was made from 8 mm thick S650 plate. Four different configurations for the joint between the U-shaped profile and tubular diagonal were tested which enabled the effect of increasing the local chord thicknesses from 8 mm to 12 mm and inserting an axial stiffener under the gusset plates to be studied.

The tests demonstrated that varying the tensile chord thickness did not significantly affect the ultimate load and the axial stiffener provided substantial additional resistance. It was not possible to determine the exact increase in the joint resistance due to the presence of the axial stiffener because when the stiffener was present, the failure mechanism changed from failure in the gusset plate to...
U-shaped profile weld to failure in the diagonal. The basic requirement for the design of the axial stiffener is that the fillet welds connecting it to the tensile chord should be able to undertake the axial load variation of the chord due to the vector sum of the diagonal forces.

**Tubular truss**

The tubular truss comprised two top chords and two bottom chords, all made from S690 seamless CHS. The joints were identical throughout the truss. Two joints were tested and they both failed at a similar load and displacement. In the first test, failure was in the weld between the tension diagonal and the tension chord. In the second test, failure was in the weld between the tension diagonal and the compression chord.

The performance of the HSS tubular truss was analysed using ABAQUS to assess the accuracy of the hand calculation design methods in Eurocode 3 and CIDECT guides for HSS joints.

**WP4 Prestressed HSS structures**

This WP studied the performance of two different prestressed structural systems and demonstrated how prestressing enables a greater proportion of the higher strength to be exploited.

**Prestressed cable stayed columns (PSSCs)**

The resistance of slender columns is limited by global instability, but through the addition of cross-arms and external prestressed cables, buckling displacements can be inhibited and the resistance of the column increased. WP4.1 investigated the application of this technology in conjunction with HSS columns.

121 full-scale tests were carried out on PSSCs, varying the length (12 m and 18 m), cross-section and steel strength (S355 and S690) of the column. The number of cross-arms (0, 1, 2) and the diameter and level of prestress in the stays (cables) was also varied. The benefit of including a stay system with cross-arms for long columns was clearly demonstrated; the benefit increased as the slenderness increased. In stockier columns, however, the benefit of including a stay system and cross-arms is less clear. Columns with two cross-arm systems showed greater resistance than columns with a single cross-arm system because the angle between the cables and the core column was greater, increasing the perpendicular component of the tension in the stays at bracing points which stabilises the column better. For the same reason, shorter columns benefit more from a stay system than longer columns. The test programme also demonstrated that PSSCs are highly sensitive to geometric imperfections. Although limitations in the capacity of laboratory equipment meant that the columns tested were highly slender, and hence the benefit of using HSS was not apparent, the programme was useful for characterising the general behaviour of these systems and for identifying imperfection profiles. Higher strength will become beneficial in these systems as the cross-section of the column increases.

Each of the tests was modelled (a total of 132 linear buckling analyses and 396 geometrically and material nonlinear analyses with imperfections included). Different assumptions regarding the geometric imperfections were tested out. In the majority of cases, the numerical model predicted the experimental behaviour reasonably well. In some cases the numerical model predicted higher ultimate loads, which may be because unpredicted deviations and lack of verticality of the load application system led to a premature loss of stiffness in the tests.

A new design method for PSSCs based on the Ayrton-Perry formulation was developed. The results were calibrated against the experimental and numerical results, as well as previously published simplified design methods. Worked examples for two cable stayed columns with different geometrical and material properties were prepared to illustrate the design procedure.

The natural frequencies and respective vibration modes of the PSSCs were obtained for different levels of pretension in the cables. Experimental modal analysis was applied using output-only measurement techniques based on ambient vibration of the structure. FE models were developed and calibrated with experimental results. The same operational modal analysis techniques were used to evaluate the level of pretension in the cables. A simple methodology was proposed to control the cable pretension during construction.

**Prestressed trusses**

For conventional structural steels, the need to maintain deflections under service loads within acceptable limits for ever-increasing spans has led to the emergence of prestressed steel tubular trusses for long-span structures, in which prestressed cables inside the bottom chord are utilised. In
WP4.2, the feasibility of prestressing as a means of increasing the stiffness of HSS trusses was investigated, on the basis of 22 individual truss member tests and 6 tests on long trusses. The individual truss member tests included tests in tension and compression. The key variables were the steel strength, initial prestress level and the presence or absence of grout. The level of initial prestress studied were $P_{\text{nom}}$, $0.5P_{\text{opt}}$ and $P_{\text{opt}}$, where $P_{\text{opt}}$ is the optimum prestress force that causes the cable and the tube to yield simultaneously. The addition of a prestressing cable increased the tensile capacity of the system by about 30%. Increasing the prestress in the cable delays yielding of the tube and brings it closer to the yield point of the cable. This increase in the elastic response range reduces the axial deflection required to attain the ultimate load. The presence of the grout had minimal effect. The S460 specimens gained more benefit from prestressing than the S690 specimens.

Four arched steel trusses of 11 m length, made up from S460 steel SHS, were tested under vertical loading. Each truss was identical apart from the level of prestressing applied to the cable located in the bottom chord (none, $P_{\text{nom}}$, $0.5P_{\text{opt}}$ and $P_{\text{opt}}$). Prestressing delayed yielding of the bottom chord and led to an increase in the failure load of the truss from around 35 kN (no prestressing) to 54 kN (cable prestressed to $P_{\text{opt}}$). To investigate the effects of the addition of prestressing cables and increasing prestress levels on the vibration behaviour on the trusses, measurements of their dynamic properties were carried out using a modified impact hammer modal testing method. The recorded acceleration versus time responses were used to extract the natural frequencies of each of the trusses by means of the Fast Fourier Transform (FFT), which translated the measured acceleration from the time domain into the frequency domain. The results showed that the addition of the cable and increasing prestress levels had an insignificant effect on the modal properties of prestressed HSS arched trusses.

In parallel with the experimental programme, numerical models were generated and validated against the experimental results. Ten different combinations of out-of-plane and in-plane initial geometric imperfections were investigated. The ultimate load and the initial stiffness were very well replicated in most of the cases, whilst, as anticipated, the displacement at the maximum load appeared to be more sensitive to the initial geometric imperfections. It was concluded that the model with out-of-plane imperfection L/750 and in-plane imperfection L/1500 gave best agreement with the test results. These imperfection amplitudes were therefore introduced as initial geometric imperfections in the subsequent parametric studies which studied the applied prestress level, the span-to-depth ratio, the steel strength of the truss members, the truss shape and the section sizes for the top and bottom chord. The increase in resistance and the reduction of the midpoint displacement at failure load that arise from prestressing were also studied for a wide range of structural configurations likely to occur in practice.

A design process for trusses with and without prestress was developed and three worked examples illustrated its application for trusses of various spans. The results of the worked examples were verified against predictions using ABAQUS. It was observed that the insertion of a nominally prestressed cable in the bottom chord of a truss substantially increased its ultimate load resistance. The ultimate resistance is however attained at very large displacements. By applying the optimal prestress force $P_{\text{opt}}$, the midspan deflection at failure reduces to about 50% of the respective value when only a nominal prestress force $P_{\text{nom}}$ is applied. In addition to this benefit, the introduction of an optimally prestressed cable leads to approximately 10-15% material savings, compared to a conventionally designed truss without any cable in the bottom chord.

In order to benefit from prestressing, premature buckling of the top chord should be prevented as it can significantly reduce the ultimate load of the truss. Attention should also be paid to the material response of the cable, whose effective stiffness reduces for increasing span and stress levels. In order to mitigate prestress losses due to relaxation, the initial prestress of the cable should not be higher than 70% of the yield strength of the cable.

**WP5 Exploitation of results**

The purpose of this WP was to present the results of the research in an appropriate way for the intended audiences.

**Design examples**

A series of eight design examples was developed showing the step-by-step design procedure for HSS members and joints. The examples comprise:

- Example 1: Square hollow tension member (SHS) in S460 welded to a connection plate.
- Example 2: As Example 1, except the tension member is S690 steel.
• Example 3: Internal I shape column in multi-storey building in S460 steel.
• Example 4: Top chord member in tubular truss, subject to a high compression force and external loads.
• Example 5: Rectangular hollow section (S460) in compression demonstrating the Continuous Strength Method (CSM).
• Example 6: Rectangular hollow section (S460) in bending demonstrating the CSM.
• Example 7: SHS (S460) under combined loading demonstrating the CSM.
• Example 8: Crocodile Nose connection.

Whilst the first four examples do not involve the technologies specifically studied in HILONG, they nevertheless demonstrate the weight saving of using HSS. It should be noted that the limited range of sections available in HSS compared to conventional strength steels can limit the weight savings achievable. Also, for tension members, the design of the connection will often be governing, especially as fabricators tend to use S355 steel for connection details, which can again limit the achievable weight savings by using a higher strength member.

As mentioned previously, design examples were also prepared to demonstrate the new design methods developed for HSS PSSCs and prestressed arches; these are presented in WP4.

**Contribution to European standardisation**

The HILONG research has made a useful contribution to the understanding of HSS members, connections and structural systems. In order to carry out Eurocode reliability analyses, material strength data were collected from steel producers in Europe. Strength data were also extracted from numerous mill certificates and research papers. This enabled a mean ratio of measured to minimum specified strength, accompanied by standard deviations, to be derived. Under WP2, EN 1993-1-1 and EN 1993-1-12 design rules were confirmed. Concerns were raised over the level of reliability of the effective width formulation for Class 4 cross-sections. These results were presented to the Working Group (WG) for EN 1993-1-12 and the relevant deliverables will be forwarded to the WG once the final report is approved. Under WP3, the application of the design rules for tubular joints made from HSS was confirmed. This is significant because data on the performance of HSS tubular joints are very sparse. WP4 studied structural systems and developed design methods for PSSCs and prestressed trusses, aligned where appropriate to the provisions in Eurocode 3.

**Public seminar**

A public seminar was held in July 2015 to which more than 100 designers, steelwork manufacturers, contractors, etc. attended. The contents of the seminar was designed so that the information presented was varied, giving a ‘complete’ picture of the use of HSS in structures, and not just the topics studied in HILONG. Presentations included guidance on specification, design and fabrication as well as a comprehensive presentation of the results of the research carried out in HILONG. Case study information about HSS structures was also included. Tata Steel, Vallourec and ArcelorMittal presented details of their HSS product range. Some very encouraging feedback was given by delegates. The presentations given can each be downloaded from the SCI News site [http://news-sci.com/high-strength-steels-in-long-span-structures-30th-june-2015/](http://news-sci.com/high-strength-steels-in-long-span-structures-30th-june-2015/), which extends the impact and reach of the seminar to a much larger audience.

**Conclusions**

The HILONG project has sought to overcome some of the obstacles relating to design limitations due to buckling and deflections by studying innovative structural arrangements and cross-sections which suppress buckling and reduce deflection. The project has shown that the use of HSS presents many opportunities for lighter structures which comply with serviceability requirements. The data generated in the project will supplement the body of data on HSS cross-sections and contribute to the next revision of Eurocode 3. The database of material strength properties collected for the reliability analysis will also be useful in the future for reliability assessments for HSS design rules, and has already been shared with HILONG’s sister RFCS project RUOSTE.

The work completed in WP3 sets the foundation for promoting the use of innovative cross-sections in HSS rather than just continuing to use the standard rectangular and circular hollow and I-sections designers are familiar with. The tests on the HSS trusses are valuable since there are very few data on the performance of any types of HSS joints. The work on the CN connection will be very useful for trusses made from conventional as well as high strength steel members.
The outcomes of the research into prestressing in WP4 are applicable to any strength steel and will help to promote the use of prestressing in general. Prestressing can significantly increase the load-carrying resistance of a truss or column without increasing its structural weight. The combination of HSS as a structural material and prestressing technology can potentially lead to material savings and lighter and more elegant structures.

The results of the HILONG project have shown there is much potential for designing more efficient higher strength steel structures and the potential scope of application of the results of this project is extensive. Further dissemination activities are required to enable designers to make use of the recommendations and techniques arising from the HILONG research.
2 INTRODUCTION

2.1 The production of high strength structural steels

Steel properties like strength and toughness depend both on the chemical composition and processing procedures; steel producers use a wide range of different concepts to achieve the required balance of properties. Although the easiest way to improve the strength of steel is to increase its carbon content, this reduces other important properties like weldability, toughness and formability. Microalloying with elements like niobium, vanadium, molybdenum or titanium in amounts below 0.1 wt % (1000 grams/tonne) is a cost-effective method of achieving a balanced combination of properties and is widely adopted by steel producers around the world.

Table 2.1 summarises the steel grades and qualities covered in the European Standards which apply to the products tested in HILONG.

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<tr>
<td>EN 10025-2</td>
<td>Non-alloy structural steels: S275, S355</td>
</tr>
<tr>
<td></td>
<td>JR, J0, J2, K2</td>
</tr>
<tr>
<td>EN 10025-3</td>
<td>Normalised/normalised rolled weldable fine</td>
</tr>
<tr>
<td></td>
<td>grain structural steels: S275, S355, S420,</td>
</tr>
<tr>
<td></td>
<td>S460 N, NL</td>
</tr>
<tr>
<td>EN 10025-4</td>
<td>Thermomechanical rolled weldable fine grain</td>
</tr>
<tr>
<td></td>
<td>structural steels: S275, S355, S420, S460</td>
</tr>
<tr>
<td></td>
<td>M, ML</td>
</tr>
<tr>
<td>EN 10025-6</td>
<td>Flat products of high yield strength structural steels in the quenched and tempered condition: S460, 500, 550, 620, 690, 890, 960</td>
</tr>
<tr>
<td></td>
<td>Q, QL, QL1</td>
</tr>
<tr>
<td>EN 10210-1</td>
<td>Hot finished structural hollow sections of</td>
</tr>
<tr>
<td></td>
<td>non-alloy and fine grain steel: Non alloy:</td>
</tr>
<tr>
<td></td>
<td>S275, S355 JRH, J0H, J2H, K2H</td>
</tr>
<tr>
<td></td>
<td>Fine grain: S275, 355, 420, 460 NH, NLH</td>
</tr>
</tbody>
</table>

The next revision of EN 10210 will include steels up to S960.

2.2 HSS in long span applications

There appears to be a gradual trend towards the use of higher strength steel. Twenty years ago, S275 was the norm, and S355 the exception. Now, S355 is the norm, and higher strength steels are available. The case for increased use of high strength steels in structures is compelling: less material, lower carbon emissions, lighter supporting structures and foundations, easier installation etc. Yet despite these perceived advantages, steels of strength S460 and above make up less than 5% of the tonnage of structural steel worldwide.

One of the most common applications of HSS is for columns in high rise buildings, particularly in Japan where structures are required to resist large seismic loads. In order to utilise the high strength, the columns tend to be relatively stocky steel I-sections or hollow sections. Alternatively, they may be filled with, or encased in, concrete to suppress buckling. HSS are also used for bridge girders, for example in trusses where self-weight is the dominant load condition.

It is usually the case that deflections restrict the use of HSS in buildings or carparks for members in bending. However, for long span structures like arenas, less stringent deflection limits apply because the overall height is so large. In addition, the structure deadweight is a considerable proportion of the design load and thus a reduction in the deadweight is of great value. There are many examples of HSS being used for long span roof trusses, for example the retractable roof truss at the Reliant Stadium, Houston[3] and the Airbus Hangar in Frankfurt Airport[4]. The main roof truss of the new Friends Arena in Stockholm uses S460, S690 and S900 steels in various structural forms for the chords and diagonals[5]; the use of HSS led to a reduction in the weight of steel of between 13-17%. The idea for the HILONG project came from Sweco initially, the designer of the Friends Arena, who expressed interest in studying how further cost savings could have been made if HSS was used more extensively and more efficiently in the stadium roof truss.
The European design code containing general rules for steel structures is Eurocode 3: Part 1.1\textsuperscript{(1)}; this provides structural design guidance for material strengths up to 460 MPa. Eurocode 3: Part 1.1\textsuperscript{(2)} provides additional rules for extension of Eurocode 3 to S700 material, but with a number of restrictions. These special rules, together with the limitation on strength to S700 material, are partly related to insufficient test data on HSS structural elements. Although the most common obstacles to the more widespread use of HSS relate to those phenomena that are controlled by stiffness rather than strength, specifically instability and serviceability, reduced material ductility also creates some limitations. Additionally, connecting high resistance members with relatively small cross-sections is more difficult as the forces being transferred are far greater than when joining conventional strength steel members.

\section*{2.3 Prestressing technologies in long span applications}

The term prestressing is used to describe the process of introducing internal forces (or stress) into a structural element during the construction process in order to counteract the external loads that will be applied on the structure during its life\textsuperscript{(6)}. Prestressing enhances the efficiency of structures by increasing their load-bearing capacity or stiffness, or both, without using more material. In the case of steel structures, the technology has not been widely used as the costs involved have been considered too high compared with the benefits, coupled with stability problems from compressing slender elements.

The use of prestress can effectively reduce the weight and cost of steel structures in long spans. Previous applications of prestress revealed reductions in steel quantities of between 10-30 \% (single span beams and trusses) and reductions in cost up to between 8-20 \%\textsuperscript{(6,7)}. More recent prestressed steel structures for long span roofs provided up to 50\% savings in steel consumption\textsuperscript{(8,9)}. Application of prestress within trusses is typically performed by the pretensioning of tendons anchored to elements of the truss. It is important to point out that the effectiveness of prestress may be lost under loads that are not acting downwards. Under lateral loading, a situation which is not common for latticed roof girders, frames have inversion of bending moments. This is an undesirable situation when dealing with some types of prestressing techniques.

An alternative system using prestressing is the addition of cross-arms and external prestressed cables to slender structural elements in order to inhibit buckling displacements and thus enhance load-carrying capacity. Cable stayed columns may be executed in several different configurations by varying the number of cross-arms per bracing point, the number of bracing points, and by using different stay arrangements.

The application of prestressed technologies in conjunction with HSS is studied in WP4 of HILONG.
3 WP1 CASE STUDIES

3.1 Objectives of WP1
The objective of WP1 was to take two different structural configurations, a long span roof truss and a prestressed cable stayed column, and prepare two functionally equivalent designs in conventional strength steel and HSS to compare the savings in weight, cost and environmental indicators arising from the technologies studied in HILONG.

3.2 Task 1.1 Workshop
A workshop entitled “The use of high strength steels in long span structures – a structural and sustainability perspective” took place at the offices of Sweco Structures AB, on September 4th 2012. Practitioners were invited and the benefits of using HSS were discussed. Table 3.1 gives the programme. The Workshop concluded with a guided tour of the construction site of the Friends Arena. Minutes were taken on the topics discussed by the delegates at the Workshop. A copy of these Minutes, along with a summary of the presentations given at the Workshop is available as deliverable D1.1 [10].

Issues discussed at the Workshop included why HSS is more expensive than conventional strength steel. Although adding alloying elements to steel has a modest impact on cost, the primary impact on cost is the process by which the steel is produced. HSS require more working during production and additional forms of heat treatment. The scales of production also have an effect on cost because HSS are not produced in the same quantities as conventional strengths of steel.

For some projects, short lead times are critically important and lead to designers being forced into choosing conventional steel strengths where supply is guaranteed within a known timeframe.

Photographs from the site visit to the Friends Arena, illustrating the structural arrangement of the roof structure, are given in Figure 3.1.
Table 3.1  HILONG Workshop Agenda (4 September 2012, Stockholm)

<table>
<thead>
<tr>
<th>WORKSHOP AGENDA ITEM</th>
<th>PRESENTED BY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welcome and Opening of the workshop</td>
<td>Sweco Managing Director</td>
</tr>
<tr>
<td><strong>Contributions from HILONG partners:</strong></td>
<td></td>
</tr>
<tr>
<td>Brief introduction to the HILONG (High Strength Long Span Structures) project:</td>
<td>Nancy Baddoo</td>
</tr>
<tr>
<td>objectives, scope of work, partners</td>
<td><em>SCI</em></td>
</tr>
<tr>
<td>Design of long span structures (recent trends, key influences affecting design and</td>
<td>Fergus McCormick</td>
</tr>
<tr>
<td>construction for long span structures and future drivers)</td>
<td><em>Buro Happold</em></td>
</tr>
<tr>
<td>Prestressed long span steel structures</td>
<td>Murray Ellen</td>
</tr>
<tr>
<td><strong>Contributions from invited guests:</strong></td>
<td></td>
</tr>
<tr>
<td>Thirty years’ experience of working with Advanced High Strength Steel for light</td>
<td>Jan-Olof Sperle</td>
</tr>
<tr>
<td>weight design. Product development, structural design, fabrication and application</td>
<td><em>Former senior researcher at SSAB</em></td>
</tr>
<tr>
<td>examples.</td>
<td></td>
</tr>
<tr>
<td>Production of HSS-structures for the Friends Arena at Ruukki Construction</td>
<td>Alanko Timo</td>
</tr>
<tr>
<td>Steel Construction – Ruukki’s focus on HSS</td>
<td>Petri Ongelin</td>
</tr>
<tr>
<td>RFCS Project Rules on High Strength Steel (RUOSTE), a project to improve rules for</td>
<td>Prof. Markus Feldman,</td>
</tr>
<tr>
<td>HSS</td>
<td>Nicole Schillo</td>
</tr>
<tr>
<td>ECCS databases and tools for LCA</td>
<td>Prof. Luis Simoes da Silva</td>
</tr>
<tr>
<td>Discussion:</td>
<td></td>
</tr>
<tr>
<td>Opportunities for HSS in long span structures</td>
<td>Milan Veljkovic</td>
</tr>
<tr>
<td>Challenges to maximising the benefits of HSS in long span case studies</td>
<td><em>Luleå University of Technology</em></td>
</tr>
<tr>
<td>What types of structure are more likely to benefit from HSS?</td>
<td></td>
</tr>
<tr>
<td>Are HSS a more sustainable option?</td>
<td></td>
</tr>
</tbody>
</table>
3.3 **Tasks 1.2 and 1.3 Case studies**

Two case studies were selected to study the impact of using HSS and the technologies studied in HILONG on the weight of the structure. Full details of this study are given in D1.2.1[^11] and D1.2.2[^12].

### 3.3.1 **Case Study 1: Friends Arena**

**Introduction**

Case Study 1 is based on the Friends Arena, a multipurpose stadium built in Solna, Sweden, which has a movable steel roof structure supported on four 3D trusses, spanning 162 m. Due to the size and complexity of the Friends Arena roof, it was decided to focus just on the main trusses, as they are the most important structural element of the roof and a significant portion of the framework total weight (about 35%) (Figure 3.2).

HSS was used in the Friends Arena main roof trusses for the top and bottom chords and the outer diagonal member closest to the support points. The top chords consisted of steel tubes with a diameter of just over a metre, using S460M steel. Higher strengths were used in the bottom chords and the outer diagonals (S690QL and S900 respectively) as these were subject to predominantly tensile loading. The bottom chord was a U profile to simplify welding. The outer diagonal members were flat plates.

[^11]: D1.2.1
[^12]: D1.2.2
The use of HSS in the as-built roof truss led to savings in the roof weight compared to a roof made from conventional S355 structural steel\footnote{13}. When it came to cost, although HSS are slightly more expensive than conventional structural steels, the cost of fabrication was lower, mainly due to the reduced welding required. It was estimated that a 14.5\% saving in the cost of the roof was achieved through the use of HSS\footnote{13}.

**Effect of prestressing the bottom chord on truss weight**

Although the HILONG case study mainly investigated the effect of using HSS more extensively in the roof truss, the addition of prestress into the bottom chord was also studied to mitigate unwanted effects from the loss of stiffness due to the reduction of members’ cross-section sizes. The prestress was introduced by inserting a cable in the lower chord (Figure 3.4) which led to compression in the lower chord of the same value as the tension in the cable (in the absence of transverse loading). The total load that the chord is able to carry is the amplitude between the initial compression value and its tensile resistance.
The following cases were studied:

- 1.a: As-built truss - Truss using top chords from S460 steel, bottom chords from S690, web diagonals from S355 and end-diagonals of S900 steel;
- 1.b: As-built truss but using only S355 steel, without prestress;
- 1.d: As-built truss, prestressed by a cable placed in the bottom chord;
- 1.e: As-built truss, prestressed by a cable placed in the bottom chord and only using S355 steel (exception for end-diagonals);

The roof truss was designed to EN 1993-1-1, using different load arrangements according to the requirements for the as-built design. Figure 3.5 summarises the weight of the options, expressed as a Global Utilisation Ratio (GUR), the weighted average of the utilisation ratio of all the members.

The application of prestress (cases 1.d and 1.e) had more impact on the weight of the conventional steel solution than for the as-built truss. This may be due to the fact that the bottom chord is an open section, designed to facilitate butt-welding of HSS plates (by welding on both sides) and unable to resist significant compression forces. The S355 solution had greater deadweight and so was able to sustain higher prestress forces. Although the as-built truss was not designed to carry compression in the bottom chord, it may be useful to apply a level of prestress which overcomes the action of permanent loads and allows hogging of the truss. Such an action should, in theory, balance the envelope of bending moments of the truss \( (M_{\text{max}} = M_{\text{min}}) \) and result in a lighter structure.

The observed vertical displacements show that the prestress had no or little effect (Figure 3.6). Again, this is probably because the limited compression resistance of the bottom chord inhibited higher levels of prestress. Varying the position of the cables could have enabled a higher imposed upward deformation to be obtained. In the case of changing the steel strength, as expected, vertical displacements were substantially reduced by increasing member sizes (cases 1.b and 1.e).

An additional reason why prestressing had little effect on the weight or deflection of the roof truss was that the Friends Arena was designed for heavy snow loading. However, prestress seeks to neutralise the internal forces generated by permanent loading. For the Friends Arena, the permanent load was small compared to the snow load.

![Figure 3.5 Case Study 1 – final weight of truss](image)

<table>
<thead>
<tr>
<th>OPTION CASE</th>
<th>GUR x TRUSS WEIGHT (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.a</td>
<td>156.50 274.27 24.47</td>
</tr>
<tr>
<td>1.b</td>
<td>52.29 65.17 359.48</td>
</tr>
<tr>
<td>1.d</td>
<td>113.56 66.00 213.01</td>
</tr>
<tr>
<td>1.e</td>
<td>46.90 63.92 189.89</td>
</tr>
</tbody>
</table>

1.a: as-built
1.b: S355
1.d: as-built, with prestress
1.e: S355, with prestress
GUR: Global Utilisation Ratio
Effect of steel strength, snow load and bottom chord cross-section on truss weight

Additional cases were also analysed which involved:

- Using S690, with and without prestressing (1.c and 1.f),
- Removing the snow load, with and without prestressing (1.a.s and 1.d.s),
- Changing the bottom chord from a U profile to a CHS, and varying the position of the prestressing cable (1.d.t1, 1.d.t2, 1.d.t3).

The weights of these options are illustrated in Figure 3.7. Using S690 led to a 26.7% reduction in the weight of the truss compared to the as-built case (1.c compared to 1.a). For the pre-stressed options, the use of S690 led to a 31.1% reduction (1.f compared to 1.d). This is a good indicator that it is possible to reduce weight significantly using HSS, despite the challenges related to stability.

When the snow load was removed (1.a.s), the weight of the truss was reduced by 42% but prestress did not enable any substantial reduction in weight. For alternative solutions 1.d.t1, 1.d.t2, and 1.d.t3, where the bottom chord cross-section was changed to a CHS (keeping the snow load) and the position of the cable varied, the weight of the truss was reduced by about 10%. This is a small but interesting reduction, although the cost of having a closed cross-section will probably overcome the savings arising from the steel weight reduction, due to the added cost of fabricating the joints. The maximum absolute deflection results did not change significantly.

The analyses showed that using a CHS bottom chord (in order to allow higher levels of prestress) only reduced the truss weight by about 9%, which was lower than expected. The reason was thought to be that the truss was not originally designed to be prestressed, so compressing the bottom chord generates forces that quickly start to govern the design, and so result in heavier solutions. Because of this, the post-tension force in case 1.d.t1 was just 35% greater than initially applied for case 1.d. Differences in cable position did not significantly affect the weight nor deflection.
Effect of varying the top chord cross-section on truss weight

The purpose of these studies was to determine the benefit of using HSS novel cross-sections with enhanced resistance to local buckling. For cases 1.g, 1.h and 1.i, the upper chords were replaced by closed polygonal cross-sections in S355, S460 and S690 respectively (Figure 3.8 and Figure 3.9). As well as giving enhanced compression resistance by increasing resistance to local buckling, polygonal sections also allow the use of bolted connections, known as crocodile nose (CN) connections. The structural performance of polygonal cross-sections and CN connections are studied in WP3. For these cases, the bottom chord was a U profile and the diagonals were circular hollow sections.

Figure 3.8 Main truss section with closed polygonal solution in the top chords
The weights and maximum deflections of these options are given in Figure 3.10 and Figure 3.11. The modification of the top chords to a novel polygonal closed cross-section had an insignificant effect on truss weight and displacement. The weight difference between case 1.b and case 1.g, where the only difference is the cross-section shape of the top chords, is not significant (5.38%), and the same occurs for case 1.c and case 1.i where the difference is 6.62%. However, the use of these cross-sections will lead to cost savings because connections are easier.
Effect of using two CHS for the bottom chord

For cases 1.k, 1.l and 1.m, the U profile for the bottom chord was replaced by two CHS (Figure 3.12) and the diagonals and top chords were also CHS. The height of the truss was 12 m, as compared to the height of the previous cases studied (16 m). This new configuration was also tested with three different steel strengths. The weights and maximum deflections of these options are also given in Figure 3.10 and Figure 3.11.

Figure 3.12 Triangular main truss configuration (left: case a and g to i) and new truss configuration with two bottom chords (right: cases k to m)

The new configuration using two bottom chords did not demonstrate any benefit when compared to the as-built truss configuration. In fact, the weight of the truss and its displacements were higher for the new configuration. Comparing cases using the same steel strength, the truss configuration with two bottom chords was heavier than the original configuration by 28.4%, 22.2% and 23.5%, respectively for cases 1.g and 1.k, cases 1.h and 1.l, cases 1.i and 1.m. The same is true for maximum displacements which were respectively, 34.4%, 36.4% and 38.2% higher. This can be explained by: i) the decreased structural height (from 16 m to 12 m) and ii) the fact that bottom chords were in tension, not being susceptible to any buckling phenomenon. Adding a new bottom chord could have some influence if the suction effect caused by wind action was more significant and the bottom chords were in compression, and susceptible to the buckling.

Effect of using HSS on truss weight

All the truss configurations using S690 revealed a significant reduction in the final weight. The weight decrease was around 15% when comparing the truss configuration using S690 (case 1.i) with the original configuration (case 1.a). The difference was higher if the comparison was performed for the truss configuration using S690 (case 1.i) and the truss using only S355 (case 1.g) (33.3%). For the configuration with two bottom chords the weight difference between the S690 truss (case 1.m) and S355 truss (case 1.k) was even more significant (37.4%).

3.3.2 Case Study 2: City of Manchester Stadium

Case Study 2 is based on the 12 masts supporting the roof of the City of Manchester Stadium, home to the English team Manchester City (Figure 3.13). Full details of this study are given in D1.2.1 [11] and D1.2.2 [12]. The base case for this study is a tapered mast, with a diameter ranging from 0.70 m to 1.58 m. Alternative designs were initially studied:

- 2.a: Tapered with varying thickness, S355 steel;
- 2.c: Constant diameter column, S355 steel;
- 2.e: Prestressed stayed column (PSSC), constant diameter, S355 steel.

For case 2.a, the diameter varied from 0.70 m to 1.5 m, with the thickness varying from 25 mm to 30 mm. For case 2.c, the diameter was 1.5 m and the wall thickness 24 mm. For case 2.e, the core wall diameter was 1.3 m with wall thickness of 25 mm.
Figure 3.13 Manchester City Stadium, UK

Figure 3.14 shows case 2.a and 2.e. The prestressed solution was designed with 3 sets of cross-arms to enable the column to have similar compressive strength to the others, or else the required load would not be attained while reducing the weight of the system. Although not presented in this report, increasing the number of cross-arms not only increased the ultimate load, but also the amount of applicable pretension in the stays from around 50 kN to 1000 kN (a factor of 20!). This is important because it shows the optimum prestress increases dramatically as its contribution to buckling is balanced by the greater number of bracing points. Nevertheless, and similarly to what was observed in the first case study, the cross-section diameter was limited by local buckling.

A 3D shell FE model was used for the analysis and design of the masts, except for the prestressed option, where, for the sake of simplicity, beam fibre elements were used to capture plasticity. Instead of using load combinations, a direct comparison of resistance between the different options was performed. The numerical analysis of the original configuration dictated the target resistance to be achieved by the other cases. The results from the analysis were expressed in terms of corrected weight (final weight multiplied by the global utilisation ratio) to enable comparisons. Notice that solutions obtained were the lightest weight solution found by a manual optimisation (no numerical algorithm was used).

The results are shown in Figure 3.15 and Figure 3.16 in terms of axial force versus compression force and final corrected weights.
The corrected weights show that the constant section mast (2.c) is the heaviest, consuming 1.5% more material than 2.a. The advantages of using a tapered mast with so little reduction of steel consumption is probably the simplification of end connections and, mostly, for improved appearance. The lightest solution was the PSSC weighing 2.1 tonnes less than case 2.a (-6.2%), even accounting for additional cross-arms and stays.

**Effect of HSS on mast weight**

The effect of HSS on the weight of the mast was determined by re-designing the three mast options in S690 steel:

- 2.b: Tapered with varying thickness, S690 steel;
- 2.d: Constant diameter column, S690 steel;
- 2.e: PSSC, constant diameter, S690 steel

For case 2.b, the diameter varied from 0.70 m to 1.2 m, with the thickness varying from 25 mm to 38 mm. For case 2.d, the diameter was 1.1 m and the wall thickness 35 mm. For case 2.f, the core wall diameter was 0.96 m with wall thickness of 32 mm.

The final corrected weights are illustrated in Figure 3.17. The results show that using HSS only led to a reduction in weight for the PSSC. As for case study 1, the wall slenderness governs the design for large diameter sections which meant that less economical compact sections had to be used, instead. In the case of the constant section mast, the weight increased by 12.3% for the S690 solution.
2.a: Tapered with varying thickness, S355 steel
2.c: Constant diameter column, S355 steel
2.e: PSSC, constant diameter, S355 steel
2.b: Tapered with varying thickness, S690 steel
2.d: Constant diameter column, S690 steel
2.f: PSSC, constant diameter, S690

Figure 3.17 Case Study 2 – Weight of the mast and weight reductions compared to base case 2.a

3.4 Task 1.4 Environmental impact and whole life cost comparison

3.4.1 Case Study 1: Friends Arena Truss

Cost and Life Cycle Environmental Assessments (LCEA) were compared for a range of truss designs.

Cost Analysis

A cost estimate for the production of the members in the truss was prepared based on a bill of quantities submitted prepared by Ruukki (Annex A, D1.4 [14]). Table 3.2 gives the assumed material costs. Costs for the end and secondary connections, as well as the cost of transport and erection on site, were excluded.

Table 3.2 Cost estimate for each type of material

<table>
<thead>
<tr>
<th>Material</th>
<th>€/kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>S355</td>
<td>0.76</td>
</tr>
<tr>
<td>S460</td>
<td>0.90</td>
</tr>
<tr>
<td>S690</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Crocodile nose connections were designed using one fork end and two free plates at the other end in order to simplify the erection process. More details on their geometry and fabrication steps can be found in Annex B of D1.4 [14]. The cost per connection was estimated based on information provided by Sweco to be €6,469.

The cost of the as-built truss solution (case 1.a) was compared against six cases based on the above stated reference cost values. Figure 3.18 summarises the estimated cost for each truss member for all seven configurations.
It is concluded that configurations using two bottom chords (cases 1.k, 1.l, and 1.m) lead to an increase in the total cost. On the other hand, configurations using the polygonal cross-section and the CN connection (cases 1.g, 1.h and 1.i) show a consistent decrease in the total cost of the truss. It is interesting to note that using the closed polygonal cross-section together with S690 high strength steel (case i) minimises the cost (5.47% decrease compared to the cost of the original solution).

**Life Cycle Environmental Assessment**

**Framework and main steps of LCEA**

The scope of a life cycle analysis for a structure usually covers all stages over the service life of the structure, from the material production stage to the end-of-life stage, as illustrated in Figure 3.19. However, due to lack of data, the stages of construction and operation were not taken into account. Nevertheless, it was assumed that such stages would not influence the results since the structures being compared have similar performance over their respective service lives.

**Figure 3.19 Scope of a LCEA**

The framework for the LCEA follows ISO 14040\(^{[15]}\) and 14044\(^{[16]}\), and includes the following main steps: (i) definition of goal and scope; (ii) inventory analysis; (iii) impact assessment; (iv) normalisation and weighting and (v) interpretation of results. The step of normalisation and weighting is considered to be optional in ISO standards and was not addressed in this study.

According to EN 15978\(^{[17]}\), the scope of the life cycle of the building is represented by a modular concept as illustrated in Figure 3.20. The system boundaries adopted in this study limit the scope of the LCA to the material production (modules A1 to A3) and recycling (module D) stages.
The environmental data needed to perform the impact assessment were provided by the WorldSteel Association\(^{18}\). However, these datasets do not differentiate between steel strengths, although the latter vary in their alloying constituents and heat treatment processes\(^{19}\). For this reason, three different scenarios were considered in this study. In scenario 1, no difference was assumed between the environmental data of steel strengths S355, S460 and S690. In scenarios 2 and 3, the environmental impact of higher strength steels was increased by 10% and 20% for S460 and 20% and 40% for S690 respectively.

The impact assessment stage of an LCEA aims to associate inventory data with specific environmental impact categories and is made of two parts: (i) mandatory elements, such as selection of environmental indicators and classification and (ii) optional elements, such as normalisation, ranking, grouping and weighting. The classification implies a previous selection of appropriate impact categories, according to the goal of the study and the assignment of inventory results in the chosen impact categories. Characterisation factors (indicators) are then used to represent the relative contribution of an inventory result \((m)\) to the impact category indicator result, as expressed by Equation (5-1). Brief definitions of the most common indicators are presented in the following paragraphs, while more details can be found in D1.4\(^{14}\).

\[
\text{impact}_{\text{cat}} = \sum_i m_i \times \text{charact}_{\text{factor}}_{\text{cat}}
\]  

\(5-1\)

**Indicators of environmental performance**

*Global Warming Potential (GWP)* measures the impact of human emissions on the radiative forcing of the atmosphere. GWP of substance \(i\) is defined as the ratio between the increased infrared absorption due to the instantaneous emission of 1 kg of the substance and that due to an equal emission of carbon dioxide (\(\text{CO}_2\)), both integrated over time\(^{20}\). In the adopted approach, GWPs were calculated for a 100 year time horizon.

*Ozone Depletion Potential (ODP)* represents the total quantity of ozone depleting gases, such as bromine and chlorine, released into the stratosphere and is expressed in kg of CFC-11 equivalents\(^{21}\). ODPs are divided into steady-state, in which the time span is eternity, and time-dependent. In the adopted approach, only indicators corresponding to the steady state were considered.

*Photochemical Ozone Creation Potential (POCP)* assesses various emission scenarios for Volatile Organic Compounds (VOCs). VOCs together with carbon monoxide (CO) in the presence of nitrogen oxides (\(\text{NO}_x\)) oxidise under the influence of ultraviolet light to form photo-oxidants in the troposphere\(^{21}\). POCP is given by the ratio between the change in ozone concentration due to a change in the emission of that VOC and the change in the ozone concentration due to a change in the emission of ethylene (\(\text{C}_2\text{H}_4\))\(^{21}\). The determination of POCPs depends on the level of background \(\text{NO}_x\) concentration. In the adopted approach, only indicators corresponding to high concentration levels were considered.

*Acidification Potential (AP)*, expressed in kg \(\text{SO}_2\), is based on the RAINS-LCA model\(^{22}\). Based on this model, Huijbregts developed characterisation factors for 44 regions in Europe and average European
factors by a weighted summation of the regional factors for each acidifying emission. In the adopted approach, only the latter were considered.

_Eutrophication Potential (EP)_ of substance $i$ reflects its potential contribution to biomass formation and is defined as the aggregation of the potential contribution of emissions of N, P and C (given in terms of Chemical Oxygen Demand, COD) to biomass formation expressed in kg $PO_4^{3-}$ equivalents.

_Abiotic Depletion Potential (ADP)_ aims to evaluate the environmental problem related to the decreasing availability of natural resources, such as minerals and materials found in the earth, sea or atmosphere and biota that have not yet been industrially processed[23]. ADP of resource $i$ is defined as the ratio between the quantity of resource extracted and the recoverable reserves of that resource expressed in kg of a reference resource. The reference resource is antimony.

**LCEA of the original truss configuration and re-design configurations**

In this case study, the original truss solution (case 1.a) was compared with the re-designed configurations described in Section 3.4.1 (cases 1.g to 1.m) in terms of the amount of steel used (Table 3.3) and the environmental impact. The selected environmental categories for the LCEA were: Global Warming Potential (GWP); Acidification Potential (AP); Eutrophication Potential (EP) and Primary Energy Demand (PED).

**Table 3.3  Weight of steel (in tonnes) used in each solution**

<table>
<thead>
<tr>
<th>Case</th>
<th>Total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.a</td>
<td>As-built</td>
</tr>
<tr>
<td></td>
<td>S355: 80.7, S460: 171.5, S690: 60.8 = Total: 313</td>
</tr>
<tr>
<td>1.g</td>
<td>All sections in S355: closed polygonal cross-section for top chords with CN connections;</td>
</tr>
<tr>
<td>1.h</td>
<td>All sections in S460: closed polygonal cross-section for top chords with CN connections;</td>
</tr>
<tr>
<td>1.i</td>
<td>All sections in S690: closed polygonal cross-section for top chords with CN connections</td>
</tr>
<tr>
<td>1.k</td>
<td>All sections in S355 with two bottoms chords (CHS)</td>
</tr>
<tr>
<td>1.l</td>
<td>All sections in S460 with two bottoms chords (CHS)</td>
</tr>
<tr>
<td>1.m</td>
<td>All sections in S690 with two bottoms chords (CHS)</td>
</tr>
</tbody>
</table>

The different solutions are compared in Figure 3.21 to Figure 3.23 for the three scenarios studied. In all three scenarios, the most favourable solution is case 1.i with lower environmental impacts in all selected environmental categories. In general, cases 1.a, 1.g, and 1.h have a better environmental performance than cases 1.k, 1.l and 1.m. In scenario 2 and 3, there is little difference between the environmental performance of 1.a, 1.1.g and 1.h because the benefit of lower weight achieved by using higher strength is balanced by the penalty of greater environmental impact of the higher strength.
Figure 3.21 Results of the LCEA for scenario 1: (a) Global Warming Potential; (b) Eutrophication; (c) Acidification and (d) Primary Energy Demand
Scenario 1: Environmental impact of S355, S460 and S690 assumed to be equal

Figure 3.22 Results of the LCEA for scenario 2: (a) Global Warming Potential; (b) Eutrophication; (c) Acidification and (d) Primary Energy Demand
Scenario 2: Environmental impact of S460 is 10 % greater than S355, and S690 is 20% greater than S355
Figure 3.23 Results of the LCEA for scenario 3: (a) Global Warming Potential; (b) Eutrophication; (c) Acidification and (d) Primary Energy Demand

Scenario 3: Environmental impact of S460 is 20% greater than S355, and S690 is 40% greater than S355

3.4.2 Case Study 2: Prestressed stayed column

The tests in WP4 showed that the use of PSSC can lead to weight savings compared to unstayed columns and these savings are enhanced by using high strength steels. In order to quantify these savings, data from the tests presented in D4.1.1[45] were used to perform a cost comparison.

PSSCs of S355 and S690 were tested in WP4. As the slenderness of the tested columns was very high, the benefit of using HSS was small and the failure of the columns was mainly driven by stiffness. Nevertheless, some improvement was observed. Figure 3.24 compares specimens with the same cross-section and length but with different steel strength (i.e. C06 with C07 and C09 with C10). Columns C06 (S355) and C07 (S690) have a small profile and very high slenderness; the effect of using HSS is negligible. However, for columns C09 (S355) and C10 (S690), the beneficial effect of HSS is more significant. For the comparison between C10 and C09, for the 10 mm diameter stays and an initial tension of 2 kN, using S690 increased the ultimate resistance of the column by a factor of 1.2. When the initial tension was 9 kN, using S690 increased the ultimate resistance of the column by a factor of 1.19. By increasing the initial tension in the stayed systems, the slenderness of the column decreases and the influence of the strength becomes more significant. Hence, focusing only on the direct costs of the material of the core columns (i.e. not taking into account the cost of acquiring and assembling the stay system) and assuming that for hot rolled circular hollow sections S690 is on average 30% more expensive than S355[24], the benefits of HSS can be quantified. Figure 3.25 shows the material and cost savings which can be achieved by using the S690 PSSC C07 compared to a functionally equivalent S355 unstayed column. Note that for the cost estimate in Case Study 1, the cost differential between S690 and S355 plate was assumed to be closer to 60% (Table 3.2). As both estimates were based on data from steel producers, it is assumed that the difference is mainly due to the different product forms, size of components and the size of the order.

This study has only focused on the direct material cost, excluding the cost of transportation, labour and erection of the stay system. In order to have a more realistic cost comparison, a more detailed analysis should be performed.
3.5 A survey of fabricators’ experiences using HSS

A survey was conducted in order to understand existing barriers to the use of HSS in construction. Three major UK fabricators gave their views on their experiences using HSS. A detailed account of the survey can be found in D1.5[25].

3.5.1 General

All three fabricators had previous experience with HSS in the past 25 years, although it was in most cases limited to S460. The majority of applications, including the Sony Centre in Berlin and the external megaframe for the Leadenhall Building in London, involved members such as gravity columns, plate girders and tie bars.

All fabricators agreed that the barriers to the use of HSS were the cost premium of the material, which could not always be justified by reduced weight, the restricted stock availability, welding and fabrication complexity. Certain structural requirements concerning ductility, potential loss of overall stiffness and member buckling resistance were also identified as barriers. No quality control concerns were raised about the potential for mixing up steels of different strength since most current quality assurance procedures followed by fabricators in the workshop ensure full traceability.

3.5.2 Supply

As far as procurement of HSS is concerned, all three fabricators described difficulties in procurement, especially for strengths over S550 and thicker sections. They had purchased HSS from Thyssen-Krupp, Salzgitter, Dillinger, Tata Steel, Spartan and ArcelorMittal. Only ArcelorMittal
supplied open hot rolled sections in S460. S460N/NL and ML plate products were available from several certified EU mills, with each mill having their own length, width, thickness and weight restraints.

Rough cost comparisons between S355 and S460 cited by the fabricators showed that the price of HSS varied inconsistently with changes in the market and the source company. However, some characteristic values suggest an increase of 20% for thickness \( t < 60 \text{ mm} \), 40% for \( 60 \text{ mm} < t < 80 \text{ mm} \) and up to 100% for \( t > 80 \text{ mm} \), while another fabricator rounded the price rise up to 30% between S355JR and S460.

Minimum order requirements were also mentioned by the fabricators. Although some rolling mills were able to work to a minimum order of 10 t-20 t for plate, some of them impose a minimum order of up to 100 t. ArcelorMittal offered a range of rolled sections with a minimum order size as low as 5 t, other producers quoted higher minimum orders, e.g. 32 t.

3.5.3 Fabrication and weldability

No special weldability issues with S420 and S460 were reported by any of the fabricators. However, the number of companies possessing a welding certificate for S460 and particularly for S690, complying with EN ISO 3834\(^{[26]}\), was relatively small. Normally, the fabricators reported that welding required no special treatment for the lower HSSs.

The fabricators said that the main difference in welding HSSs in comparison with conventional steels lay in the selected consumables, which depend on the strength, toughness and grain structure achieved during the original manufacturing process.

Preheating was carried out in line with EN 1011\(^{[27]}\) and became a necessity as the carbon equivalent value increased: for \( C_{eq} < 0.4 \), preheating can be minimised. The necessity for pre-heating also depended on product thickness and steel strength.
4 WP2 MATERIALS AND SECTION DESIGN

4.1 Objectives of WP2

The main objective of this Work Package was to perform full-scale tests on HSS structural material and cross-sections to address the current lack of experimental data which is restrictive for the development of design guidance. To this end, a series of experimental tests on material, cross-sectional response, complemented by numerical modelling was completed, which enabled the assessment of current design guidance for HSS structures, codified in EN 1993-1-1[1] and EN 1993-1-12[2] to be carried out. Complementary to the assessment of current design guidance, a deformation-based design method was developed for HSS cross-sections to enable the full benefits of the material to be exploited for stocky cross-sections, thus encouraging more widespread usage of HSS in construction.

4.2 Task 2.1 Laboratory testing

4.2.1 Overview

In this deliverable, an experimental study on HSS hollow sections comprising tensile and compressive coupon tests, stub column tests, full cross-section tensile tests, cross-section combined axial load and bending tests and beam tests was carried out. The tested structural hollow sections were S460 (NH) and S690 (QH). Table 4.1 reports the type and number of tests performed. A detailed account of the test programme is given in D2.1[28].

Table 4.1 Testing programme for D2.1

<table>
<thead>
<tr>
<th>Strength</th>
<th>Cross-section</th>
<th>Material tests</th>
<th>Stub column tests</th>
<th>Beam test</th>
<th>Combined loading test</th>
<th>Full section tensile test</th>
</tr>
</thead>
<tbody>
<tr>
<td>S460 (NH)</td>
<td>SHS 50×50×5</td>
<td>4(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>SHS 50×50×4</td>
<td>5(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>SHS 100×100×5</td>
<td>3(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>SHS 90×90×3.6</td>
<td>3(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>RHS 100×50×6.3</td>
<td>5(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>RHS 100×50×4.5</td>
<td>3(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>S690 (QH)</td>
<td>SHS 50×50×5</td>
<td>5(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>SHS 100×100×5.6</td>
<td>3(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>SHS 90×90×5.6</td>
<td>3(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>RHS 100×50×6.3</td>
<td>5(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>RHS 100×50×5.6</td>
<td>3(T)+1(C)</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>1</td>
</tr>
</tbody>
</table>

T = tension, C = compression

4.2.2 Material tests

Material tensile and compressive coupon tests were performed to determine the basic engineering stress-strain response of the flat and corner material for each of the tested sections. For S460 SHS 50×50×5, 50×50×4 and RHS 100×50×6.3 and S690 SHS 50×50×5 and RHS 100×50×6.3 specimens, four tensile flat coupons were extracted from each of the four faces of the section, while for the remaining sections, two parallel flat coupons were machined, to confirm the uniformity of properties around the sections.

All tensile flat and corner coupons were cut out in the longitudinal (rolling) direction, and dimensioned in accordance with EN ISO 6892-1[29]. One compressive flat coupon was also tested for each of the section sizes to assess the difference in the stress-strain behaviour of HSS in tension and compression. The following material parameters were extracted from each of the recorded tensile stress-strain responses:

- Young’s Modulus $E$
- Upper yield strength $f_y$
- Ultimate tensile strength $f_u$
- Plastic strain at fracture $\varepsilon_f$ (based on an original gauge length of $L_0 = \sqrt{5.65 A_c}$, where $A_c$ is the cross-section area of the coupon),
• Cross-sectional area reduction at fracture Z%,
• Tensile to yield stress ratio \( f_u / f_y \).

The results of the compressive coupon tests, including the measured Young’s Modulus \( E \) and the yield strength \( f_y \), were also extracted.

### 4.2.3 Stub column tests

A total of eleven stub columns were tested under pure axial compression - one test for each of the S460 and S690 sections. The compression tests were carried out in an Instron 3500 kN hydraulic testing machine. Accurate measurements of the geometric dimensions of the tested specimens were made before testing. From the measured axial load \( N \) and end shortening \( \delta \), graphs of normalised load-end shortening \( N/(Af_y) \) versus \( \delta/L \) for all the stub columns were produced, where \( A \) is the cross-sectional area, \( f_y \) is the yield strength and \( L \) is the initial measured length of the specimen. The observed failure modes included local buckling, elephant-foot buckling and global buckling triggered by initial local buckling as shown in Figure 4.1.

![Figure 4.1 Stub column failure modes](image)

#### 4.2.4 Full section tensile tests

While tensile coupon tests provide a measure of the local engineering stress-strain response of structural sections, a full insight into the tensile performance of the whole cross-section, allowing for the influence of the distribution of material and geometric properties, may be obtained from full cross-section tensile tests. One full cross-section tensile test was performed on each of the S460 and S690 hollow sections. Accurate measurements of the geometric dimensions of the specimens were made before testing. All tests were carried out in an Instron 2000 kN hydraulic testing machine in accordance with the procedures of EN ISO 6892-1. Typical failure modes of the specimens included fracture at the specimen ends and ductile fracture, where fracture occurred at or near the mid-height of the specimen as shown in Figure 4.2. Key results of the full cross-section tensile tests, including the yield strength \( f_y \), ultimate tensile strength \( f_u \) and strain at fracture \( \varepsilon_f \) were extracted from the recorded stress-strain curves.

![Figure 4.2 Full section in tension test failure modes](image)
4.2.5 Combined axial load and bending tests

A total of 12 eccentric stub column tests was carried out to investigate the cross-section response of HSS cross-sections under combined axial load and bending. The stub column specimens were compressed eccentrically, with different amounts of eccentricity $e_0$ to generate different ratios of axial load to bending moment. The combined loading tests were conducted in a 2000 kN Instron hydraulic testing machine. The load and end-rotation relationships of the tested specimens were plotted, from which the following key results of the tests were obtained:

- Failure load $N_u$.
- Applied eccentricity, referred to as the first order eccentricity $e_0$ based on strain gauge readings.
- Generated lateral deflection at failure load, referred to as the second order eccentricity $e'$.
- Failure moment $M_u = N_u (e_0 + e')$, with the corresponding failure end rotation $\varphi_u$
- End-shortening $\delta_u$

All the SHS 50×50×5 specimens failed by material yielding, while the SHS 90×90×5.6 displayed a local buckling failure mode, as illustrated in Figure 4.3.

![SHS 50×50×5, eccentricity of 5 mm](image1)

![SHS 90×90×5.6, eccentricity of 5 mm](image2)

**Figure 4.3** Eccentric stub column buckling modes

4.2.6 Beam tests

Three-point bending tests and four-point bending tests were conducted to establish a relationship between cross-section slenderness and the moment resistance and rotation capacity of HSS hollow sections. One three-point bending test and one four-point bending test were performed on each of the S460 and S690 cross-section sizes considered. The tested beams were loaded symmetrically, in an Instron 2000 kN hydraulic testing machine, at mid-span and at third-points for the three-point bending and four-point bending arrangements, respectively. Non-dimensional moment-rotation and moment-curvature responses from the three-point and four-point bending test, respectively, were therefore plotted. Key results from the tests, including the ultimate test bending moment $M_{max}$ and the cross-section rotation capacity $R$ were extracted. All test specimens, apart from those which did not reach their ultimate moment resistance, failed by yielding and local buckling of the compression flange and the compressive part of the web. Typical failure modes of the three-point and four-point beam tests are shown in Figure 4.4 and Figure 4.5 respectively.

![Three-point bending failure mode](image3)

**Figure 4.4** Typical three-point bending failure mode (S460 SHS 100x50x4.5)

![Four-point bending failure mode](image4)

**Figure 4.5** Typical four-point bending failure mode (S690 SHS 90x90x5.6)

4.3 Task 2.2 Numerical modelling

A numerical study on the structural response of HSS (S460 and S690) hot-rolled square and rectangular hollow sections subject to pure compression, bending and combined loading was carried
out and a detailed account of the numerical modelling is given in D2.2. Nonlinear numerical models using the general-purpose FE software ABAQUS were developed and their accuracy validated against the laboratory tests.

4.3.1 Development of the FE models

For the development of the FE models, the 4-noded general-purpose element S4R with reduced integration and finite membrane strains was used. For modelling convenience, only the specimen was modelled and the effect of the supports and the loading jack were introduced through appropriate boundary conditions and constraints. The symmetry of the test configuration in geometry and boundary conditions was also exploited, thus greatly reducing the size of the model. An initial convergence study was performed in order to choose an appropriate mesh size that would provide accurate results while keeping computational time to a minimum.

The material properties that were incorporated in the FE models were derived from the material tests. Tensile and compression coupon tests showed that both HSS (S460 & S690) exhibit a similar behaviour to conventional strength carbon steels. A linear stress-strain response up to the yielding point was observed, followed by a well-defined plateau and a strain hardening part after it, which was more prominent in the case of S460. Even though this type of material behaviour can be represented by an idealised elastic, perfect-plastic model, the necessity of accurate replication of test results dictated the use of a multi-linear approximation of the actual stress-strain curve to better capture the strain hardening response. ABAQUS requires the material properties input to be in the form of points on a true stress-logarithmic plastic strain curve. The multi-linear approximated stress-strain curve was therefore transformed to true stress and logarithmic plastic strain by the following equations:

\[
ue = ng(1 + ng)
\]

\[
\frac{pl}{ln} = \ln(1 + ng) - \frac{ue}{6 - 2}
\]

Real structural members can never be perfectly straight and this deviation from the ideal geometry needs to be accounted for, as it can greatly affect their structural behaviour. For modelling convenience, geometric imperfections were incorporated in the models in the form of an appropriate buckling mode shape. An initial linear buckling analysis using the subspace iteration method was performed to extract the buckling modes and the shape of the lowest local buckling mode was subsequently used to introduce imperfections into the nonlinear models. For the magnitude of the introduced imperfections, various values were examined: 1%, 2% and 10% of the section wall thickness, the maximum measured imperfections and the following value calculated from the predictive model of Dawson and Walker:

\[
\beta = \beta \cdot \left(\frac{cr}{0.5}\right)
\]

The coefficient \(\beta\) of the predictive model should ideally be determined through a regression analysis performed on imperfection measurement data. However, due to the limited amount of measured imperfection data for HSS (in part owing to the high roughness observed on the surface of the S690 specimens that prevented accurate imperfection measurements), it was decided to adopt the respective value for conventional carbon steel hot-rolled sections, \(\beta = 0.028\).

Nonlinear static analysis was performed using the modified RIKS procedure (a variation of the classic arc length method), which is suitable for capturing the post buckling response.

4.3.2 Validation of the FE models

The verification of the FE models was based on the experimental results of 22 bending tests, 11 concentric and 15 eccentric stub column tests. Six values for the imperfection magnitude were assumed for each test, thus allowing the assessment of the sensitivity of the models to geometric imperfections.

The validation of the beam models was based on the numerical to experimental ratio of the ultimate moment \(M_u\) and its corresponding rotation \(\theta_u\) at the plastic hinge for the 3-point bending tests (or its corresponding curvature \(k_u\) for the 4-point bending tests). The concentric stub column models were validated by comparing the experimental and numerical values of the ultimate load \(F_u\) and its corresponding end shortening \(\delta_u\), whereas for the eccentric stub columns, the experimental to numerical ratios of the ultimate load \(N_u\) and moment resistance \(M_u\) were reported. In most of the
cases, the aforementioned ratios achieved a mean value close to unity with a small coefficient of variation (COV). It was observed that the ultimate load (for concentric and eccentric stub columns) and moment resistance (for eccentric stub columns and beams) are not significantly affected by the imperfection magnitude, whereas the corresponding values of rotation (for 3-point bending tests), curvature (for 4-point bending tests) and end shortening (for concentric stub columns) present a greater sensitivity to geometric imperfections. Failure modes and full load-response curves were also examined to further verify the accuracy of the models.

Typical graphs of the validated beam, concentric stub and eccentric stub column models are shown in Figure 4.6, Figure 4.7 and Figure 4.8 respectively. As can be observed, the numerical curves closely matched the experimental ones in most cases. It was concluded that very good agreement between the experimental and numerical results was achieved overall. The sensitivity of the models to geometric imperfections was also assessed. An imperfection of $t/50$, with the average comparison ratios of Table 4.2, was found capable of replicating successfully the cross-sectional response.

Figure 4.6  Experimental versus numerical behaviour for S460 beam (SHS 50x50x4)

Figure 4.7  Experimental versus numerical behaviour for S690 concentric stub column (SHS 90x90x5.6)
Figure 4.8  Experimental versus numerical behaviour for S690 eccentric stub column (SHS 50x50x5)

Table 4.2  Comparison of FE against test results for imperfection magnitude t/50

<table>
<thead>
<tr>
<th>Section</th>
<th>Moment Distribution</th>
<th>Rotation Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>3pt bending</strong></td>
<td>$M_{u,FE}/M_{u,Exp}$</td>
<td>$\theta_{u,FE}/\theta_{u,Exp}$</td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td>1.01</td>
<td>1.16</td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td>0.03</td>
<td>0.16</td>
</tr>
</tbody>
</table>

| **4pt bending**  | $M_{u,FE}/M_{u,Exp}$ | $k_{u,FE}/k_{u,Exp}$ |
| **Mean**         | 0.99                | 1.07                  |
| **COV**          | 0.04                | 0.19                  |

| **Concentric stub** | $F_{u,FE}/F_{u,Exp}$ | $\delta_{u,FE}/\delta_{u,Exp}$ |
| **Mean**           | 0.96                | 1.37                  |
| **COV**            | 0.04                | 0.35                  |

| **Eccentric stub**  | $N_{u,FE}/N_{u,Exp}$ | $M_{u,FE}/M_{u,Exp}$ |
| **Mean**            | 0.92                | 0.98                  |
| **COV**             | 0.03                | 0.10                  |

### 4.3.3 Parametric studies

Following the successful validation of the FE beam and stub column models, a series of parametric studies was performed to obtain results over a wider range of cross-sectional slenderness and loading scenarios.

The validated beam model was investigated for three sections with aspect ratios of 1.0, 2.0 and 2.44, varying thickness to provide cross-sectional slenderness $c/t_e = 10$ to 90 and three testing configurations to examine the influence of the moment gradient on the deformation capacity of the beams for both steel strengths.

Based on the expression for the non-dimensional plate slenderness $c/t_e = \left( \frac{c}{t_e} \right)^{0.5}$ and on the different stress distributions within the flange and the web through the Eurocode values for the buckling coefficient $k_e$ ($k_e = 4$ for flanges of SHS in compression, $k_e = 23.9$ for webs under bending), 2.44 is the limit in aspect ratio value that considers equal slenderness for the compression flange and the web of a hollow section. Hence 2.44 is the most unfavourable aspect ratio value.
For the concentric stub column models, two sections with aspect ratios of 1.0 and 2.0 and varying thickness to provide cross-sectional slenderness \( c/t \varepsilon = 10 \) to 90 were studied for both S460 and S690.

For the eccentric stub columns, varying thickness to provide cross-sectional slenderness \( c/t \varepsilon = 20 \) to 60 and varying eccentricities to provide initial stress ratio \( \psi = \sigma_2/\sigma_1 = -0.75 \) to 1.00 were investigated for sections with cross-sectional aspect ratios 1.0 and 2.0 in both steel strengths.

In total, 216 beams, 48 concentric stub columns and 336 eccentric stub columns were modelled. The material properties were based on the averaged flat coupon stress-strain curves, whilst a magnitude of \( t/50 \), which was found to best replicate the test results, was used for the geometric imperfection. Typical numerically predicted failure modes are depicted in Figure 4.9.

![Numerically predicted failure modes](image)

**Figure 4.9 Numerically predicted failure modes**

### 4.3.4 Discussion

After the execution of the parametric studies, the bending test results (both experimental and numerical) were normalised by the moment resistance and plotted against the cross-sectional slenderness, allowing the evaluation of the influence of key parameters (i.e. material properties, aspect ratio, cross-sectional slenderness) on the flexural response of S460 and S690 SHS and RHS.
beams. Figure 4.10 presents the results of S460 beams, whilst similar graphs were developed for S690 beams. The ultimate compression loads as obtained by experimental and numerical concentric stub column tests were normalised by the compression resistance and plotted against the cross sectional slenderness, as depicted in Figure 4.11. Finally, the eccentric stub column results were plotted in N-M interaction curves, separately for Class 1 and 2, Class 3 and Class 4 cross sections. The results of SHS eccentric stub column tests are shown in Figure 4.12, Figure 4.13 and Figure 4.14. Similar graphs were created for RHS sections in bending around the major and around the minor axis.

**Figure 4.10**  Beam results (normalised ultimate moment – slenderness, S460)

**Figure 4.11**  Concentric stub column results (normalised ultimate load – slenderness)
Figure 4.12 Eccentric stub column results for Classes 1 & 2 (SHS, S460 & S690)

Figure 4.13 Eccentric stub column results for Class 3 (SHS, S460 & S690)
As was observed both in the bending and the stub column tests, S460 stocky sections outperform their S690 counterparts due to superior strain-hardening characteristics. This effect decreases with increased slenderness and becomes insignificant in the slender region, where failure is governed by local buckling at strains within or below the yield plateau. As far as the cross-sectional aspect ratio effect is concerned, it was found that slender SHS have lower compression resistance and improved flexural performance than their RHS counterparts. This can be attributed to the fact that the onset of local buckling is delayed for the RHS compression models, where the stiffer corner region provides restraint to the web and for the SHS beam models, where the less slender webs provide greater degree of restraint to the compression flanges. Finally, the moment gradient effect was clearly observed for the stocky beam models, where the rotation capacity increases with steeper moment gradient.

### 4.4 Task 2.3 Development of design rules

A detailed description of the development of design rules is given in D2.3\[34\].

The results of the experiments and numerical analyses, supplemented with other test results collected from the literature, were used to assess the current HSS design guidelines set out in EN 1993-1-12\[2\], with a focus on:

- Material ductility requirements.
- Cross-section classification limits for internal elements in compression.
- Effective width equation for Class 4 internal elements in compression.
- Interaction equation for cross-sections under combined axial load and bending.

#### 4.4.1 Assessments of EN 1993-1-12 material ductility requirements

For steel structures designed to Eurocode 3, a series of ductility requirements are set out:

For material strengths up to and including S460, the ductility requirements given in EN 1993-1-1 are:

- \(\epsilon / \geq 1.10\)
- \(\epsilon / \geq 15\%\)
- \(\epsilon / \geq 15\)
For higher strength steels, up to S700, the following, more relaxed ductility requirements, are set out in EN 1993-1-12:

- \( f_u / f_y \geq 1.05 \)
- \( \varepsilon_f \geq 10\% \)
- \( f_u / f_y \geq 15 \)

In Figure 4.15 to Figure 4.17, the results from the tensile coupon tests on both the flat (F) and corner (C) material, have been combined with those collated from the literature, and used to assess the variation of the \( f_u / f_y \) ratio, \( \varepsilon_f \) and ultimate-to-yield strain \( \varepsilon_u / \varepsilon_y \) with yield stress \( f_y \). In summary, it was shown that HSS with yield strengths in the range of 460-700 MPa, satisfy both the existing more relaxed limits of EN 1993-1-12 as well as the stricter limits of EN 1993-1-1 related to the ultimate to yield strength ratio \( f_u / f_y \), the strain at fracture \( \varepsilon_f \) and the ultimate to yield strain ratio \( \varepsilon_u / \varepsilon_y \). Moreover, it was found that the EN 1993-1-12 relaxed limits for the ultimate to yield strength ratio \( f_u / f_y \) and the strain at fracture \( \varepsilon_f \) are also generally satisfied by the higher strength materials with yield strengths above 700 MPa; this was however not the case for the ultimate to yield strain ratios \( \varepsilon_u / \varepsilon_y \).
4.4.2 Assessment of EN 1993-1-12 cross-section classification limits and effective width equation

Eurocode 3 adopts the concept of cross-section classification for the treatment of local buckling. EN 1993-1-12 adopts the same classification limits for HSS as are set out for conventional strength steel in EN 1993-1-1, but with the restriction that plastic design is not permitted for HSS structures. The current slenderness limits for internal elements in compression were assessed against the assembled experimental and numerical data for HSS structural hollow sections. Stricter slenderness limits that have been provisionally accepted for inclusion in the next revision of EN 1993-1-1 for conventional strength steel were also considered.

- In Figure 4.18, the rotation capacity $R$ from the beam tests and the FE models is plotted against the local slenderness $c/t\varepsilon$ of the flange, allowing an assessment of the Class 1 limit for internal elements in compression.
- In Figure 4.19, the ultimate moment resistance $M_{\text{max}}$ from the beam tests and the FE models has been normalised by the plastic moment resistance $M_{\text{pl}}$ and plotted against the $c/t\varepsilon$ ratio of the flange, allowing an assessment of the Class 2 limit for internal elements in compression.
- In Figure 4.20, the ultimate moment resistance $M_{\text{max}}$ obtained from the test and FE models is normalised by the elastic moment resistance $M_{\text{el}}$ and plotted against the $c/t\varepsilon$ ratio of the compression flange, allowing the assessment of the Class 3 limits based on the beam test results.
- In Figure 4.21, the ultimate load $N_u$ reached in the stub column tests has been normalised by the cross-section yield load and plotted against the local slenderness parameter $c/t\varepsilon$ of the compression flange, allowing the assessment of the Class 3 limits based on the stub column test results.
- Based on the same collection of test data presented in Figure 4.21, the effective width equation for slender Class 4 internal elements in compression is examined in Figure 4.22.

In all cases, the stricter limits ($c/t\varepsilon = 28$ for Class 1; $c/t\varepsilon = 34$ for Class 2 and $c/t\varepsilon = 38$ for Class 3) better reflected the behaviour of HSS internal compression elements than the existing limits ($c/t\varepsilon = 33$ for Class 1; $c/t\varepsilon = 38$ for Class 2 and $c/t\varepsilon = 42$ for Class 3) and were proposed to be adopted for HSS.
Figure 4.18 Assessment of Class 1 slenderness limit for internal elements in compression

Figure 4.19 Assessment of Class 2 slenderness limit for internal elements in compression (beam tests)

Figure 4.20 Assessment of Class 3 slenderness limit for internal elements in compression (beam tests)
4.4.3 Assessment of EN 1993-1-12 interaction equations for combined axial load and bending

The suitability of the Eurocode interaction curves for the design of HSS cross-sections under combined axial load and bending were examined using the relevant test and FE results reported in D2.1 [28] and D2.2 [30].

- Figure 4.23 show interaction curves for determining the resistance of Class 1 and Class 2 cross-sections under combined axial load and bending with the HSS data also depicted.

- Figure 4.24 presents the results of the numerical parametric studies on Class 3 sections with the linear interaction curve of Eurocode 3. The design values for Class 3 sections under compression and uniaxial bending appear to be conservatively predicted.

- The ultimate moment and load resistance achieved for the slender (Class 4) eccentrically loaded stub columns of the parametric study have been normalised by their respective Eurocode predictions and plotted in Figure 4.25. The data lie above the Eurocode linear interaction equation, demonstrating that the Eurocode effective width equation, used in predicting the cross-section combined axial load and bending moment resistance, leads to safe design values for both steel strengths S460 and S690.
Figure 4.23 Assessment of Class 1 and 2 interaction curve for SHS and RHS
4.4.4 The Continuous Strength Method

The continuous strength method (CSM) is a deformation based design approach for structures of metallic elements that allows for the beneficial effects of strain-hardening in predicting their load carrying capacities. Within the CSM design framework, treatment of cross-section local buckling is based on a continuous relationship between cross-section slenderness and cross-section deformation capacity; this replaces the classic cross-section classification approach. Strain hardening is explicitly allowed for in CSM design resistance expressions through adopting a linear hardening material model; this is in place of the commonly used idealised elastic perfectly plastic material model. The CSM, originally developed for the design of stainless steel structures, has also been extended for the design of conventional strength carbon steel and aluminium alloy structures. The CSM has been extended in this project to also cover the design of HSS hot-finished hollow sections up to S690, supported by experimental and numerical comparisons, and HSS I-sections. The CSM may also be applied to cold-formed HSS sections, but with a modified material model, as described in D2.3. The method involves first determining the limiting strain that the cross-section can endure $\varepsilon_{cs}$ from the 'base curve' given by Equation (6-4), where $\bar{s}$ is the slenderness of the cross-section, taken as the square root of the ratio of the yield stress to the elastic buckling stress of the cross-section (or conservatively of its most slender element).

$$\frac{cs}{cs} = \frac{0.25}{3.6} \quad ut \quad \frac{cs}{cs} \leq \min(15, 1)$$  \hspace{1cm} (6-4)

where $1 = 0.40$ for cold-formed sections

and $1 = \frac{h + 0.25(1)}{h}$ for hot-finished sections

in which:

$\varepsilon_{t} = 0.6(1 - h)$ is the predicted strain at the ultimate tensile stress

$\varepsilon_{h} = (h)$ is the strain at the onset of strain hardening.
The cross-section capacity is then obtained with reference to the material model for hot-finished sections shown in Figure 4.26. Hence, cross-section resistance in compression $N_{csm,Rd}$ for sections with $\lambda_p \leq 0.68$, is given by Equation (6.5), where $A$ is the gross cross-sectional area, $\sigma_{csm}$ is the limiting stress determined from the proposed material models and $\gamma_{M0}$ is the material partial safety factor as recommended in EN 1993-1-1.

\[ \frac{N_{csm,Rd}}{A} = \frac{\sigma_{csm}}{\gamma_{M0}} \]  

(6-5)
where \( cs = + h \left( \frac{cs}{h} - 1 \right) \)

The cross-section in-plane bending resistance \( M_{cs,Rd} \) for hot-finished sections with \( \bar{\lambda}_p \leq 0.68 \) that are capable of reaching the strain hardening region (i.e. those with \( \varepsilon_{csm} > \varepsilon_{sh} \)) is given by Equations (6-6) and (6-7) for major axis bending and minor axis bending, respectively, where \( W_{pl} \) is the plastic section modulus, \( W_{el} \) is the elastic section modulus and \( \alpha \) and \( \beta \) parameters are given in Table 4.3 for box-sections and I-sections.

\[
M_{cs,Rd} = \frac{1}{0} \left[ 1 - \left( 1 - \frac{\varepsilon_{pl}}{\varepsilon_{sh}} \right) / \left( \frac{cs}{h} \right) + \left( \frac{cs}{h} - 1 \right) \right] \quad (6.6)
\]

\[
M_{cs,Rd} = \frac{1}{0} \left[ 1 - \left( 1 - \frac{\varepsilon_{pl}}{\varepsilon_{sh}} \right) / \left( \frac{cs}{h} \right) + \left( \frac{cs}{h} - 1 \right) \right] \quad (6.7)
\]

Figure 4.26 CSM elastic, linear hardening material model for hot-finished material

For hot-finished sections with \( \bar{\lambda}_p \leq 0.68 \), which have not advanced into the strain hardening region (i.e. those with \( \varepsilon_{csm} < \varepsilon_{sh} \)), the cross-section bending resistance \( M_{cs,Rd} \) is given by Equations (6-8) and (6-9) for major axis bending and minor axis bending, respectively.

\[
M_{cs,Rd} = \frac{1}{0} \left[ 1 - \left( 1 - \frac{\varepsilon_{pl}}{\varepsilon_{sh}} \right) / \left( \frac{cs}{h} \right) \right] \quad (6-8)
\]

\[
M_{cs,Rd} = \frac{1}{0} \left[ 1 - \left( 1 - \frac{\varepsilon_{pl}}{\varepsilon_{sh}} \right) / \left( \frac{cs}{h} \right) \right] \quad (6-9)
\]
Table 4.3  CSM coefficients $\alpha$ and $\beta$ for bending

<table>
<thead>
<tr>
<th></th>
<th>(\alpha)</th>
<th>(\beta)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Major axis</td>
<td>Minor axis</td>
</tr>
<tr>
<td>I-sections</td>
<td>2</td>
<td>1.2</td>
</tr>
<tr>
<td>Box-sections</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

Under combined loading, the existing Eurocode 3 interaction curves, but with the CSM end points, are proposed. Comparing with the capacity predictions from Eurocode 3 reveals that the CSM offers enhanced capacities (of up to about 10%) beyond the Eurocode, while retaining safe side results. The level of enhancement in resistance is greater for cross-sections with higher deformation capacities (i.e. stocky cross-sections); it is therefore recommended that the use of the CSM be considered for stocky cross-sections and where optimal design efficiency is sought.

4.4.5 Reliability Analysis

Statistical analyses in accordance with Annex D of EN 1990\(^{[35]}\) were performed to assess the reliability of the proposed Class 2 and Class 3 slenderness limits for HSS internal elements in compression as well as the EN 1993-1-5\(^{[36]}\) effective width equation for Class 4 internal elements in compression. Owing to the high scatter associated with measured rotation capacity data, a reliability assessment was not been performed for the Class 1 limit. As part of this analysis, material strength data were collected from producers in order to determine the mean and standard deviation values for the yield and tensile strengths. This database of material strength properties will also be useful in the future for reliability assessments for HSS design rules and has already been shared with HILONG’s sister high strength RFCS project RUOSTE. The test data reported in D2.1 and, where appropriate, the numerical results from the parametric studies performed in D2.2 were used in the statistical analyses.

Overall, it was found that the revised slenderness limits of conventional strength steel internal elements limits (\(c/\epsilon = 28\) for Class 1; \(c/\epsilon = 34\) for Class 2 and \(c/\epsilon = 38\) for Class 3) are equally applicable to high strength steel. The applicability of the effective width formula to HSS Class 4 internal elements in compression was also investigated, where it was revealed that while the HSS data follow the general trend of this formula, a partial safety factor greater than unity is needed. These findings were reported to the EN 1993-1-12 Working Group who replied that a similar observation has also been made for conventional strength steel, so this is not an issue confined to higher strength steels.
5  WP3 JOINTS

5.1  Objectives of WP3

The objective of this WP was to study the performance of joints in HSS trusses, in particular joints involving U-shaped chord profiles and also a new type of joint called a crocodile nose connection.

After some preliminary work, it was decided to re-organise the activities proposed in the Technical Annex into a more logical arrangement of tasks with corresponding deliverables given in Table 5.1.

Table 5.1  Re-organisation of activities in WP3

<table>
<thead>
<tr>
<th>Original Sub-task</th>
<th>Task description in Technical Annex</th>
<th>New task description</th>
<th>Deliverable</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Crocodile Nose (CN) Connection</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.1.2</td>
<td>Behaviour of pre-tensioned bolts in tubular chord to gusset plate</td>
<td>CN long term test</td>
<td>D3.1 CN joints in HSS trusses: Tests, numerical analysis and design guidance [37]</td>
</tr>
<tr>
<td>3.2.1</td>
<td>Tubular diagonal to gusset plate detail</td>
<td>CN strength tests</td>
<td></td>
</tr>
<tr>
<td>3.1.3</td>
<td>Numerical analysis and design guidance</td>
<td>CN modelling and design examples</td>
<td></td>
</tr>
<tr>
<td><strong>Joints in HSS trusses (reference truss made from hollow sections, and innovative truss using U-shaped and polygonal cross-sections)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Design of HSS tubular truss (reference)</td>
<td></td>
<td>D3.2.1 Design of tubular truss for tests [38]</td>
</tr>
<tr>
<td>3.2.2</td>
<td>HSS truss (hollow sections)</td>
<td>Reference test</td>
<td></td>
</tr>
<tr>
<td>3.2.3</td>
<td>Polygonal compression chord to gusset plate detail</td>
<td>Second part of test on innovative truss</td>
<td>D3.2.2 Joints in HSS tubular and innovative trusses: Tests and analysis [39]</td>
</tr>
<tr>
<td>3.2.4</td>
<td>Numerical analysis and design guidance</td>
<td>Polygonal and U-shape chord modelling and design examples</td>
<td></td>
</tr>
</tbody>
</table>

5.2  Investigation of crocodile nose (CN) connection

A detailed account of the test programme, numerical analysis and development of design guidance is given in D3.1[37].

5.2.1  Introduction to CN connections

A CN connection is a new type of connection which avoids the classical abrupt and architecturally unappealing right-angle termination of a CHS. In addition to aesthetics, a smoother transition from the CHS to a gusset plate has several structural advantages. The left hand side of Figure 5.1 shows the classic knife-plate welded into a slotted CHS. The detail shown in the middle of the figure is a “hidden knife-plate” connection: the circular member has been appropriately bevelled so that the CHS converges smoothly to the forward extension of the knife-plate, however the load transfer mechanism is similar to the classic detail as the elliptical plates covering the bevels are not designed to carry any load. The CN connection is shown on the right hand side of the figure: the load is transferred to the welded semi-elliptical plates that continue after the termination of the member and are bolted to the gusset plate. Figure 5.2 shows the individual components of a CN connection.
Figure 5.1  The evolution of the crocodile nose connection

From left to right, typical knife-plate connection, bevelled connection, crocodile nose connection.

Figure 5.2  Parts of the crocodile nose connection

There are numerous parameters defining the behaviour of a CN connection, including:

- Angle of inflection of the semi-elliptical plates.
- Gap form: either parallel or wedge-shape gap.
- Gap size.
- Presence (or absence) of the connecting plate.

The geometrical characteristics of the tube, the design load, the steel strength and the bolt characteristics are also important parameters.

5.2.2 Tests on CN connections

A programme of tests on CN connections was carried out to investigate two issues: strength tests studied failure of the inflected plate and long-term tests measured the loss of pretension force of the bolts over time.

The initial test matrix consisted of 8 strength test and 8 long term specimens. Due to unexpectedly high fabrication costs, the number of the specimens was reduced to a total of 8. This was achieved by removing the gap parameter from the test matrix, which was considered of minor influence to the behaviour of the joint. Instead of testing 2 gap sizes, all specimens had the same gap, thus reducing the number of specimens required to half: 4 strength test specimens (Figure 5.3) and 4 long-term tests (Figure 5.4). Figure 5.5 shows a photograph of the strength tests specimens.
The tubes were S690 159 x 8.8 mm seamless CHS. The inflected plates were 4 mm thick S650 steel. There were six pretensioned 10.9 M26 x 70 bolts in each connection.

Figure 5.3  Assembled CN strength test specimen with attachment parts

Figure 5.4  Assembled CN long-term specimen

Figure 5.5  Strength test specimens

To perform the two series of tests, a number of ancillary parts were designed and fabricated, including the gusset plates (S650 8 mm thick for the strength test, S355 16 mm thick for the long-term test), attachment grapples with holes to facilitate a pin connection, pins and securing plates for the pins, etc.
**Specimen preparation**

Once the specimens were fabricated, the following preparatory work was carried out:

- Sandblasting and painting of the specimens and the faying surfaces.
- Marking the positions and attaching strain gauges to the specimen.
- Drilling the strain gauge boreholes, instrumentation and calibration of the bolts.
- Assembling the specimens and applying the pretension force.
- Mounting the specimen on the rig and testing.

First, the specimens were prepared according to Table 18 of EN 1090-2 for class B surface treatment aiming to achieve a friction coefficient of 0.4 between the faying surfaces. This preparation included washing with hot steam and shot blasting with steel grit (G17 grade). Afterwards, they were painted with two component ethyl silicate zinc-rich paint. The dry film thickness of the paint was 60 μm. Next, the strain gauges were attached, a task that proved to be rather challenging due to the difficulty of precisely measuring the specimens’ complex geometry. In total, 13 biaxial strain gauges were attached. Monitoring of the pretension for the long-term tests was achieved by instrumenting the bolts with strain gauges. Holes of 2 mm diameter were drilled from the head of the bolts and along the length of the shank. The holes were cleaned with acetone to remove any remaining lubricants from the mechanical process. The boreholes were filled with epoxy glue and a special bolt strain gauge with a 3 mm grid length was inserted into each one. Next, after a curing for a day at room temperature, they were cured at 70°C for 5 hours.

In order to calibrate the bolts, they were loaded three times up to 300 kN; the third time the load was sustained for 300 sec to monitor the integrity of the adhesion. The $R^2$ correlation coefficient between the load and the strain gauge readings was calculated to evaluate the adhesion.

**CN strength tests**

Having completed the preparatory works, the connections were assembled and tightened up to the pretension load. Tightening was completed in 2 rounds, first aiming for 70% of the pretension force, and second 110% to account for the instant relaxation, as described in EN 1090-2. A torque wrench was calibrated to 850 N/m which was the required torque for the first tightening round (torque method). The bolts were tightened in a descending order of stiffness, starting from the two bolts closer to the tube and working outwards. In addition to using the calibrated torque wrench, two of the bolts were instrumented and monitored while tightening (direct tension indicator method).

After the first tightening round, the bolts’ relative position to the specimen was marked. The instrumented bolts were then tightened up to the desired level of 0.7 $f_{ub} A_t$ with the help of the strain gauge readings. The rotation angles were noted and the four remaining bolts were tightened according to that angle. The testing rig had to be calibrated to ensure the precision of the jack and the load cell for tensile testing. The two grapples prepared to hold the specimens were mounted on the test rig and the load cell was bound with belts in order to measure the applied tensile force and compare it with the force measured by the test rig.

After calibrating the test rig up to a 1.5 MN tensile force, the specimens were ready to be lifted and fitted in position with the help of the two pins. The tests were run with displacement control at a rate of 0.05 mm/sec. Figure 5.6 shows the load-displacement curves. Both specimens with the connecting piece (CN1 and CN3) failed suddenly in a brittle way, one by fracture of the full width of the inclined plate, just below the connecting piece (CN1, see Figure 5.7(a)) and the other by separating the entire weld at one end (CN3, Figure 5.7(c)). These failures occurred at 800 kN and 700 kN respectively. In comparison, both specimens without the connecting piece failed at a significantly lower load, experiencing a progressive fracture of the welds, demonstrating a hardening post-failure behaviour. As the crack propagated along the weld length, the inflected plates started to straighten, at the same time bending below the lower bolt row and separating from the gusset plate (see Figure 5.7(b) and (d)).
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Angle of inflection</th>
<th>Connecting piece</th>
<th>First crack [kN]</th>
<th>Max load [kN]</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>CN1</td>
<td>1:2 (26.6°)</td>
<td>Yes</td>
<td>810</td>
<td>810</td>
<td>Brittle</td>
</tr>
<tr>
<td>CN2</td>
<td>1:2 (26.6°)</td>
<td>No</td>
<td>160</td>
<td>510</td>
<td>Ductile</td>
</tr>
<tr>
<td>CN3</td>
<td>1:1.5 (33.7°)</td>
<td>Yes</td>
<td>711</td>
<td>711</td>
<td>Brittle</td>
</tr>
<tr>
<td>CN4</td>
<td>1:1.5 (33.7°)</td>
<td>No</td>
<td>220</td>
<td>&gt;230</td>
<td>Ductile</td>
</tr>
</tbody>
</table>

**Figure 5.6  Load - displacement curves of the CN specimens**

(a) CN1 Brittle failure of inflected plate  
(b) CN2 Ductile progressive failure of weld  
(c) CN3 Brittle failure of weld  
(d) CN4 Ductile progressive failure of weld

**Figure 5.7  Failure of the four CN connections**

**CN long term tests**

The long-term behaviour of the CN connection was studied by subjecting four pretensioned specimens to a sustained load for a period of 4 months. The clamping force of the bolts was monitored to assess the loss of pretension. The specimen matrix for the long-term tests was identical to that for the strength tests, with 4 specimens alternating the angle of inflection and the presence...
of the connecting piece. The same material was used for the same parts as the strength test specimens.

In contrast to the strength test specimens, they did not have the thick gusset plate welded to the lower end. This part was designed to facilitate the pin connection which was not used on the long-term tests. Instead, the bottom caps contained threaded holes at their centres, into which correspondingly threaded rods were mounted. The upper gusset plate was fabricated from a single S355 plate, on one end of which a second threaded rod was welded. In this way, the assembled specimens carried two threaded rods protruding from either end, which were used to apply tension (Figure 5.4).

The specimens were assembled in a similar way as the strength test specimens. All bolts were instrumented; applying the necessary pretension was monitored real-time through the strain gauges. The bolts were tightened in two rounds, aiming at 75% and 110% of the desirable pretension force on the first and second round respectively. In addition, each specimen was equipped with a pair of extensometers monitoring the relative position of the gusset plate to the inflected plates. All four specimens were fixed in a pair of specially designed frames (see Figure 5.8). Specimens CN1 and CN3 shared one frame, and CN2 and CN4 the other.

Ensuring a constant load throughout the 4 months test duration required each specimen to be mounted in series with a spring. This way, possible longitudinal deformations of the specimen would result only in minimum changes of the force. This was achieved by placing six spring-washers on each specimen in series with one another.

Tension was applied while the hydraulic jack pulled the rod. A tensile load of 300 kN was applied to the two specimens with the connecting piece and 50 kN to the two without it. The loading was monitored through the load cell and the pair of strain gauges mounted diametrically on the rod. When the target load was reached, the rod was locked by tightening the nut on top of the frame. The load applied was less than 40% of the ultimate capacities obtained in the strength tests.

Throughout the 4 month test period, all 24 bolts, the tension gauges and the extensometers were continually monitored, recording one value every 20 seconds (0.05 Hz). There was a steep initial drop in tension in the bolts almost directly after tightening and an asymptotic drop over the following months. Two days after tightening, a drop in pretension was measured in the bolts in specimens CN1 and CN3 due to the introduction of tension on the specimens (see Figure 5.9). When tightened, the clamped plates of the friction connection elongated under the load, at the same time reducing their total thickness according to Poisson’s ratio, $\nu = 0.3$. The reduced thickness relaxes the strain in the bolts. The loss in tension for the bolts with a pretension of 300 kN was between 8 kN and 10 kN (2.5 – 3 %).

A logarithmic curve was fitted to the loss of pretension with time. Data from the first 12 hours after tightening were excluded. The drop in pretension due to the introduction of tension into the connection was excluded, so that the loss of pretension was due to relaxation induced by creep only. A very good fit was achieved by linear regression (Figure 5.10).
Table 7.2 gives the coefficients and derived from the regression in the equation:  
\[ p_r n_s = ( \ ) = \ln( ) + . \]

Figure 5.9  Relaxation of the bolts in CN1 and CN3 during load application

Figure 5.10  Long-term relaxation plotted on logarithmic scale with the fitted curves  
Note: 4 months is approximately \(10^7\) seconds
Table 5.2 Coefficients of linear regression analysis of bolt pretension relaxation

<table>
<thead>
<tr>
<th>CN1</th>
<th>a</th>
<th>b</th>
<th>CN2</th>
<th>a</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-34.03</td>
<td>2634.4</td>
<td>A</td>
<td>-28.12</td>
<td>2980.5</td>
</tr>
<tr>
<td>B</td>
<td>-38.03</td>
<td>2676.4</td>
<td>B</td>
<td>-28.34</td>
<td>2082.7</td>
</tr>
<tr>
<td>C</td>
<td>-34.04</td>
<td>2989.7</td>
<td>C</td>
<td>-47.22</td>
<td>2365.5</td>
</tr>
<tr>
<td>D</td>
<td>-38.19</td>
<td>3090.8</td>
<td>D</td>
<td>-28.53</td>
<td>2389.2</td>
</tr>
<tr>
<td>E</td>
<td>-34.74</td>
<td>2716.3</td>
<td>E</td>
<td>-25.25</td>
<td>2541.2</td>
</tr>
<tr>
<td>F</td>
<td>-35.88</td>
<td>2769</td>
<td>F</td>
<td>-28.89</td>
<td>2708.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CN3</th>
<th>a</th>
<th>b</th>
<th>CN4</th>
<th>a</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-41.95</td>
<td>2481.6</td>
<td>A</td>
<td>-36.23</td>
<td>2717.1</td>
</tr>
<tr>
<td>B</td>
<td>-32.26</td>
<td>2528.4</td>
<td>B</td>
<td>-29.77</td>
<td>2751.2</td>
</tr>
<tr>
<td>C</td>
<td>-39.7</td>
<td>2946.3</td>
<td>C</td>
<td>-27.33</td>
<td>2654.9</td>
</tr>
<tr>
<td>D</td>
<td>-31.61</td>
<td>2785.2</td>
<td>D</td>
<td>-29.55</td>
<td>2651</td>
</tr>
<tr>
<td>E</td>
<td>-35.59</td>
<td>2661.8</td>
<td>E</td>
<td>-11.87</td>
<td>2534.5</td>
</tr>
<tr>
<td>F</td>
<td>3.74</td>
<td>2169.8</td>
<td>F</td>
<td>-8.4</td>
<td>2369.8</td>
</tr>
</tbody>
</table>

where $\eta_{ten} = (\ ) = \ln(\ ) +$

5.2.3 Numerical analysis and design guidance for crocodile nose connections

Numerical model description

The CN tests were modelled using nonlinear FE analysis. The numerical model was validated using the measured data, for example the strain-gauge readings of the inflected plate and the load-displacement curves of the specimens. Once the model was calibrated, a parametric study was carried out to determine the effect of the steel strength, plate thickness and the angle of inflection of the plate. Design guidelines were developed using the test and numerical results.

The general purpose nonlinear FE analysis package ABAQUS was used to model the CN connection. One quarter of the test detail was modelled as shown in Figure 5.11 with appropriate symmetric boundary constraints. The pretensioned bolts and plates were not included in the model for simplicity, as failure only occurred in the region of the weldment, and not in the bolts. Loading cells were also neglected. The bottom of the CHS was fully fixed while the displacement was applied through a pilot node on the inflected plate at the other end.

The connecting piece was simply removed for the cases where it was absent from the assembly. A single surface-to-surface contact was defined between the inflected plate and the tube. Tie constraints were used for the interface between the inflected plates and the weld, as well as the CHS and the weld.

Figure 5.11 Nonlinear FE model of the crocodile nose connection

The mechanical properties of the materials used in the components and weld were obtained through coupon tests[37]. The stress-strain curves were idealised by a bi-linear elastic-plastic model. The tube
and the connecting plate were assumed to have the same material properties as the inflected plate. A rupture strain of 0.15 was assumed for the weld material in order to model the damage of the weldment observed in the test.

**FE model validation**

The numerical model was first validated by comparing the strain gauge readings at various locations on the inflected plate and tube for CN3, as shown in Figure 5.12. Only elastic deformations were considered.

![Strain gauge positions on test specimen of CN connection](image)

**Figure 5.12 Strain gauge positions on test specimen of CN connection**

Figure 5.13 shows that the numerical elastic strains from the FE model compare reasonably well with their experimental counterparts. The difference in experimental strain measurements between mirrored locations should be noted. These discrepancies, when still in the elastic range, might be due to eccentricities in the application of loading either by slight misalignment of the specimen in the frame or fabrication tolerances, as well as inaccuracies/errors from the measuring devices. However, overall, there is good agreement and the elastic strains are reasonably close with the exception of location 6(8) where the numerical results are significantly greater than the test. This may be attributed to the complex stress/strain field around the crown of the ellipse.

The FE model was firstly solved using a general static method, but minor convergence issues were experienced due to material failure and large deformation of elements in the weldment. In order to avoid numerical instability and have a robust model for the parametric study, an explicit dynamic method was then used instead. Details of the measures taken and procedures followed to ensure the validity of the dynamic solver are explained in D3.1[37].

Figure 5.14 presents an overall comparison between the FE model and the four strength tests. The difference in the predicted initial stiffness for all four cases to that measured in the tests is thought to be mainly attributed to the elastic deformation of the loading cell and possible gap/slip between components. The general characteristics of the structural behaviour of the connection were reproduced by the numerical models reasonably well. For specimen CN1 and CN3, the predicted ultimate capacities were similar to the test measurements. The progressive damage of the weld was also reproduced by the FE model for specimen CN2 and CN4.

Table 5.3 presents a comparison of the measured and numerically predicted load at different damage stages. The corresponding failure modes are shown in Figure 5.15. For cases CN1 and CN3 with the connecting plate, the dominant failure mode is brittle and it can therefore be assumed that load at first crack is the same as the ultimate load. FE results compare well with the experiments for both cases with and without the connecting plate. The load predicted by the numerical model at first crack for CN2 (151 kN) is close to the test measurements (160 kN). However, for test specimen CN4 the numerical load level at first crack is 155.5 kN which is lower than the test value of 230 kN. This could be attributed to uncertainty in the test specimen and also the definition of first crack in the numerical model (i.e. when the first element of the weldment reaches a strain of 0.15, it is deleted).
Figure 5.13 Comparison of elastic measurements in the test against FE predictions (specimen CN3)

Figure 5.14 Validation of the CN model against all four tests
Table 5.3 Comparison of ultimate load capacities (experimental vs numerical)

<table>
<thead>
<tr>
<th>Inclination/connecting piece</th>
<th></th>
<th>First crack [kN]</th>
<th></th>
<th>Max load [kN]</th>
<th></th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>FE</td>
<td>Test</td>
<td>FE</td>
<td>Test</td>
<td>FE</td>
</tr>
<tr>
<td>CN1 1:2 / yes</td>
<td>810</td>
<td>790</td>
<td>810</td>
<td>790</td>
<td>Brittle</td>
<td>Brittle*</td>
</tr>
<tr>
<td>CN2 1:2 / no</td>
<td>160</td>
<td>151</td>
<td>510</td>
<td>384</td>
<td>Ductile</td>
<td>Ductile</td>
</tr>
<tr>
<td>CN3 1:1.5 / yes</td>
<td>711</td>
<td>653</td>
<td>711</td>
<td>653</td>
<td>Brittle</td>
<td>Brittle</td>
</tr>
<tr>
<td>CN4 1:1.5 / no</td>
<td>220</td>
<td>155.5</td>
<td>&gt;230</td>
<td>248</td>
<td>Ductile</td>
<td>Ductile</td>
</tr>
</tbody>
</table>

* Brittle failure in weld NOT the inflected plate as occurred in the test

Figure 5.15 Comparison of failure modes obtained from FE and those observed in tests

The brittle failure occurred in the weld material for CN1 rather than in the inflected plate as observed in the test. As previously mentioned, no damage model was used for the material of the inflected plate. However, the strain level in the inflected plate was checked in the analysis results. It was found that the inflected plate strain was smaller than the strain in the weld (as predicted), which led to failure occurring in the weld material first.

It was discovered later that the maximum plastic strain in the inflected plate was not sensitive to the angle of inflection. Varying the angle of inflection in the FE model did not change the plastic strain in the plate significantly throughout the loading process. It was not possible to shift the failure from weld material to inflected plate without adding a damage model for the inflected plate and setting an unrealistically small rupture strain. This observation indicated that the asymptotic nature of the stress near areas of high stress concentration and particularly discontinuities would always lead to a high level of strain in the weld material. As a result, failure always tends to start in the weld material.

Nonetheless, generally failure modes are well predicted by the numerical models and the maximum failure load of the weldment can be reasonably accurately estimated by FE analysis.

Parametric study

Having validated the FE model for the CN connection, a parametric study was carried out using the calibrated model to examine the effect of certain design parameters. Nine cases were defined (Table 5.4) in order to examine the effect of steel strengths, the presence of the connecting plate and the angle of inflection of the plates. It can be seen that the plate thickness and size of the CHS are greater than those tested.
Cases P1, P7 and P3 examined steel S690, S460 and S355. The material properties of S690 steel and its matching weld consumables were taken from coupon tests of the components of the CN connection in the strength test. For S460 and S355, the material properties were based on EN 1993-1-1 and the weld consumables were assumed to have the same properties with a rupture strain ($\varepsilon_{\text{rupture}}$) of 0.1 and 0.15 respectively. The strength of the consumables was assumed to be the same as the parent material according to Eurocode requirements.

The effect of the presence of the connecting plate was investigated through cases P1, P4 and P2, P8 at two angles of inflection (35° and 25°). The effect of the angle is examined by cases P1, P2, P6 and P4, P8 for a connection with and without the connecting plate, respectively. In addition to the above three parameters, the thickness of the inflected plate and size of CHS member were varied in cases P5 and P9.

In the parametric study, damage always occurred in the weld material first, which is also the type of failure observed in three out of the four tests. Due to the numerical stress concentration at the weld tip, however, this does not affect the ultimate resistance because the strength of the rest of the weld material was accurately predicted by the FE model. Figure 5.16 presents the result of the parametric study. The effect of steel strength is shown in Figure 5.16(a). Higher strength steel S690 and S460 provide 62% and 13.4% greater load resistance with 13.3% and 7.6% smaller failure displacement, respectively. The effect of the connecting plate is presented in Figure 5.16(b). Similarly to what was observed in the tests, a more ductile failure of the weld material can be observed when the connecting plate is not used. Inclusion of the connecting plate will result in a more brittle failure of the weld material (though at a significantly higher load). Indicatively, the connecting plate reduced the failure displacement by approximately half.
Figure 5.16 Results of parametric study of CN connection

The effect of the angle of inflection with and without the connecting plate is shown in Figure 5.16(c). In general, a smaller angle results in higher resistance due to the larger interface between the inflected plate and the CHS member and therefore the greater length of weld. It is also shown that, with the connecting plate, the ultimate strength of the connection can be increased by 38% and 60% when the angle of inflection is reduced from 35° to 25° and 21°, respectively. Without the connecting plate, the strength is increased by 45% from 35° to 25°. However, varying the angle of inflection does not alter the damage mode of the weld material. It can be observed that failure displacements are all reduced while connection strength is increased.

Results from cases P5 and P9 suggest that increasing the inflected plate thickness and the thickness of the CHS member will significantly increase the strength of the connection. This is primarily due to the size of the weld material which is physically limited by the thickness of the inflected plate and the CHS member. Also, components of greater size/area can accommodate more weld material and therefore result in a higher load resistance.

For each case, the ultimate load resistance \( F_{\text{ultimate}} \) and extension at failure \( U_{\text{failure}} \) are presented in Table 5.5.
Table 5.5 Ultimate resistance and failure displacement predictions in parametric study

<table>
<thead>
<tr>
<th></th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>P4</th>
<th>P5</th>
<th>P6</th>
<th>P7</th>
<th>P8</th>
<th>P9</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{\text{ultimate}}$ (kN)</td>
<td>752</td>
<td>1040</td>
<td>463</td>
<td>483</td>
<td>1286</td>
<td>1202</td>
<td>525</td>
<td>702</td>
<td>1259</td>
</tr>
<tr>
<td>$U_{\text{failure}}$ (mm)</td>
<td>1.36</td>
<td>1.25</td>
<td>1.57</td>
<td>2.46</td>
<td>1.21</td>
<td>1.1</td>
<td>1.45</td>
<td>2.26</td>
<td>1.36</td>
</tr>
</tbody>
</table>

**Design guidelines**

It is desirable to calculate the tensile resistance of the weld in the CN connection to ensure that other parts of the connection, i.e. inflected plate and CHS member, can be specified accordingly.

The strength of weld can be checked using the directional method in EN 1993-1-8. By reversing the directional method and calibrating against available test and numerical results, the weld resistance can be explicitly estimated using Equation (7-1). Detailed formulation and calibration are presented in D3.1.

The effective length of weld is approximated as 80% of the full length of the weld (i.e. $k_e = 0.8$). The inflected plate is assumed to extend up to the mid-thickness of the wall of the CHS member (with the remaining half thickness accommodating the weld), which is perhaps conservative. The resistance values $N_{\text{weld}}$ calculated by Equation (7-1) are lower than the FE predictions and are therefore conservative. The tensile resistance of the inflected plate and CHS member can be checked according to EN 1993-1-1. The correlation factor is set to 0.9 for S355 and 1.0 for all other HSSs.

\[
\frac{R_{\text{weld}}}{Wd} = \frac{4}{2\sqrt{\left(\frac{0.707 \sin \theta}{x}\right)^2 + 3\left(\frac{as}{x}\right)^2 + \left(0.707 \sin \theta\right)^2}}
\]

where

\[
. = \frac{1}{4}\left(\frac{1}{2} + \sqrt{\left(\frac{1}{2}\right)^2 + \left(\frac{x}{2\tan \theta}\right)^2}\right), \quad = 0.8
\]

\[
= \min\left[\frac{\text{inf}, \frac{1}{2}}{\sqrt{2}}\right]
\]

where:

- $\frac{R_{\text{weld}}}{Wd}$ is the weld design resistance
- $\sigma_{\text{ult}}$ is the ultimate stress of weld material
- $c$ is a correlation factor
- $\nu$ is the partial factor
- $w$ is the weld throat width
- $L_{\text{weld}}$ is the effective length of weldment
- $k_e$ is the effective length factor
- $\sin \theta$ is the inflected angle
- $x$ is the inner diameter of CHS
- $\text{inf}$ is the inflected plate thickness
- $\frac{1}{2}$ is the wall thickness of CHS

The procedure for designing a CN connection is as follows:

- The inflected plate, CHS member and slip-resistant bolts are designed according to EN 1993-1-1 and EN 1993-1-8.
- The weld between the CHS member and the inflected plate is designed using Equation (7-1), assuming a moderate angle of inflection.
- From Equation (7-1), an inflected angle can be selected so that the strength of weld is greater than that of the inflected plate (to ensure that failure will not occur in the weldment of the connection).
Hence Equation (7-1) can be used to check the design resistance of the weld and also determine a transition angle at which failure shifts from the weld to the inflected plate. This minimum angle can be used in design so that the weld in the connection will not be the weakest part under tension. In the future, more accurate design equations could be derived based on modelling the failure of the inflected plate which would require specialist tests to be carried out for the development of an accurate material damage model for both plate and weld material. Further work could also be done on the modelling of the geometry at the weldment to reduce the stress concentration at its tip.

5.3 Investigation of HSS trusses (innovative and tubular)

In order to design competitive and architecturally appealing HSS trusses, new structural details for long span trusses made of tubular, semi-closed polygonal and U-shaped profiles were investigated. Two 14 m long HSS trusses were tested, one using polygonal top chords and a U-profile bottom chord, and one using conventional tubular sections (Figure 5.17). Full details are given in D3.2.2 [39].

Semi-closed polygonal profiles, made from galvanised steel, facilitate easier connections with minimum welding.

![Figure 5.17 Basic members of the trusses](image)

Upper: innovative truss, lower: tubular truss

5.3.1 Experimental setup

The truss experiments studied the joints between the chords and the bracing diagonals. The ultimate resistance of these joints governs in situations where strong shear is predominant. Although the Technical Annex suggested a 4-point-bending test on a three-module truss, this was deemed inappropriate because shear transfer between the diagonals and the lower chord takes place at the extreme joints of the tension chord. In addition, although the shear force distribution of the 4-point-bending test is anti-symmetrical, the specimen is symmetrical, thus requiring double the cost for a single failure. Therefore an alternative experimental setup was developed (Figure 5.18).

In this concept, the trusses are positioned with the compression chords at the bottom and the tension chords at the top. The cantilever part of a simply supported girder is acted upon by a hydraulic actuator force. This causes shear in the cantilever that subsequently affects the connections of the main diagonals. Following failure of the first module, the specimen is repositioned one module forward and the procedure is repeated for the first four modules. The essential point of this procedure
is that every joint that is tested to failure can have a different configuration. Therefore, a large number of alternative joint designs can be tested with a single truss.

![Figure 5.18](image.png)

**Figure 5.18 Consecutive joints to be tested and forces acting during the first test**

### 5.3.2 Innovative truss

#### General

The actuators were anchored to a strong beam which was fixed onto the laboratory columns. The actuators were facing downwards in order to push down onto the spreader beam. The spreader beam was bolted to the gusset plates protruding from the polygonal chords through which the forces were transferred to the first two diagonals of the truss. The force in the diagonal was 1.764 times the force in the actuator.

Prior to testing, the following resistance checks were performed:

- Buckling of the compression diagonals.
- Lateral buckling of the built-up laced compression member (pair of compression chords and the lacing between them) according to EN 1993-1-1 clause 6.4\(^1\).
- Welds on the knife-plate connection of the main diagonals.
- Bolted connection of the protruding gusset plate to the spreader beam.
- Anchoring forces to the laboratory floor.

Hand calculations showed that the weakest point was the weld between the U-shaped profile and the gusset plates.

#### Compression chord

The cross-section and buckling resistance of HSS polygonal sections was investigated through a programme of tests.

Clause 3.2.2 of EN 1993-1-3\(^{42}\) gives a method for taking into account the influence of corners on the strength of a cold-formed section wherein an average value of the yield strength of a cross section is determined, related to the number of corners in the cross-section. In order to investigate the influence of the corners on the strength of HSS semi-closed polygonal profiles, tensile tests were carried out on flat and corner specimens of thickness 4 mm and 6 mm in S650. The corner specimens were machined from L profiles. A significant reduction in ultimate strength was observed when the angle of the corner was greater than 100 - 120\(^\circ\).

In order to investigate the influence of corners on the buckling behaviour of the section, compression tests on L profiles with different angles were performed, from 90\(^\circ\) to 170\(^\circ\). The thicknesses of the L profile specimens studied were again 4 mm and 6 mm. The lengths of the specimens were 600 mm. As a general indication, for both thicknesses, the resistance with a 170\(^\circ\) angle was about 83\% less than that with a 90\(^\circ\) angle.

Initially, three configurations for the polygonal chord were considered: hexagonal, octagonal and decagonal profiles. Out of the three alternatives, the octagonal was preferred because it provided a 45\(^\circ\) angle which was the most convenient for the plane of the diagonals. The designed octagonal profile and its properties are shown in Table 5.6. The thickness of the plates was 4 mm and the strength of steel was S650. The 3 parts forming the cross-section were bolted together at regular spacing down the length of the member so that the plates were stabilised adequately and buckling was prevented.
Table 5.6 Octagonal cross-section geometry and properties

<table>
<thead>
<tr>
<th></th>
<th>Part 1</th>
<th>Part 2</th>
<th>Part 3</th>
<th>Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td>A [mm²]</td>
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<td>1761</td>
<td>775</td>
<td></td>
</tr>
<tr>
<td>A/A_total [%]</td>
<td>47</td>
<td>37</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>Moment of inertia about principal axes [mm⁴]</td>
<td>6.80 x 10⁶</td>
<td>1.67 x 10⁶</td>
<td>0.18 x 10⁶</td>
<td></td>
</tr>
<tr>
<td></td>
<td>33.9 x 10⁶</td>
<td>20.4 x 10⁶</td>
<td>1.83 x 10⁶</td>
<td></td>
</tr>
</tbody>
</table>

The reaction at the fulcrum introduced in-plane compression into the gusset plate and would have led to serious plate buckling problems in the particular experimental set-up adopted (Figure 5.18). It was therefore decided that the vertical fulcrum reaction should be introduced to the joint through the rib reinforcements shown in Figure 5.19.

![Figure 5.19](image)

Figure 5.19 Compression chord joint detail; (a) rib plates; (b) tie gusset; (c) main gusset

**Tension chord and joint**

A U-shaped profile was designed for the tension chord, as was used in the roof trusses on the Friends Arena, Stockholm. The profile was fabricated from an 8 mm thick S650 plate by press-braking consecutive obtuse longitudinal folds. Four different joint configurations were tested in order to evaluate the effect of stiffening the joint, either by locally increasing the thickness of the U-shaped profile or by inserting a horizontal stiffener under the gusset plates, or both:

- Joint 1: 8 mm profile thickness, no stiffener
- Joint 2: 12 mm profile thickness, no stiffener
- Joint 3: 8 mm profile thickness, with 8 mm axial stiffener
- Joint 4: 12 mm profile thickness, with 8 mm axial stiffener

The joints were positioned in the truss in increasing order of strength with the least resistant joint tested first. This reduced the chance of introducing plastic strains into the remaining joints. The main diagonals were directly welded onto the gusset plate by means of a typical knife plate connection. The form of the solution that was chosen and test matrix can be seen in Figure 5.20.
U-shaped Joint | Stiffener | Chord thickness
---|---|---
1 | Without | 8 mm
2 | Without | 12 mm
3 | With | 8 mm
4 | with | 12 mm

**Figure 5.20 Tension chord detail of the innovative truss and test matrix**

A series of tests on loaded fillet and butt welded connections between S650 8 mm thick plate was carried out. Three different energy input values were used. The tests showed that the measured strengths were higher than those predicted by the directional method in EN 1993-1-8.

The tension joints were extensively strain gauged and a series of extensometers monitored vertical and lateral displacements at strategic locations along the truss.

**5.3.3 Tubular truss**

The tubular truss was made from S690 seamless circular hollow sections and was designed in accordance with Eurocode 3. The chords were CHS $159 \times 8.8$ mm and the braces CHS $114.3 \times 6.3$ mm. D3.2.1[38] gives a detailed account of the design of this truss.

The instrumentation of the truss was mainly in the vicinity of the tension joints. All strain gauges were attached to one side of the truss and 8 extensometers were positioned in a similar pattern as for the innovative truss. Figure 5.21 shows an instrumented joint prior to testing.

**Figure 5.21 Instrumented joint in tubular truss**
5.3.4 *Tests and results*

Figure 5.22 shows the innovative truss in place prior to the start of testing.

![Innovative truss in place for testing](image)

**Innovative truss results**

The innovative truss was tested first. Photographs of the failed joints are shown in Figure 5.23. Load-displacement curves are shown in Figure 5.24 and Figure 5.25.

![Joint failure in the tests on the innovative truss](image)
Joint 1 — **U-shaped profile 8 mm, no stiffener**

The first joint failed in the weld connecting the gusset plate to the inner face of the U-shaped profile. The crack occurred at a total load of 900 kN. At that load, the front tip of the truss where the load was applied had a total displacement of 57 mm. A significant wave-like deformation occurred on the triangular part of the U-shaped profile between the two pairs of diagonals. This triangular piece was under pure shear stress. This could be either a first order or second order phenomenon, an issue that would not alter the weld resistance which proved critical.

Joint 2 — **U-shaped profile 12 mm, no stiffener**

The behaviour of the second joint was very similar to the first one. Failure occurred in the weld between the gusset plate and the U-shaped profile. The weld failed at a total load of 1 MN, at which the displacement was 58 mm. The buckling effect of the sides of the U-shaped profile was significantly less than that for Joint 1. Nevertheless, little difference was observed in the stiffness and the resistance of the joint.

Joint 3 — **U-shaped profile 8 mm, with stiffener**

The third joint that was tested had a horizontal stiffener under the gusset plates. Failure occurred in the welds of the tension diagonals to the gusset plates. The diagonals were completely detached from the gusset plates. At the same time the diagonals cracked around their perimeter, with initiation at the tip of the gusset plate. Failure occurred at a total load of 1.43 MN when the displacement at the load application point reached 100 mm. The increase in resistance compared to the unstiffened joints was 43%. In addition to the failure of the tension joint, significant buckling was observed in the polygonal chords in front of the fulcrum. Moreover, the concrete stands that were fixed to support the polygonal chords started cracking under the high pressure.

Joint 4 — **U-shaped profile 12 mm, with stiffener**

This joint behaved in a very similar way to Joint 3. The welds connecting the diagonals to the gusset plates failed, accompanied by the propagation of a crack around the perimeter of the diagonal,
initiating at the end of the weld. Failure occurred at a load of 1.42 MN at which the displacement was 100 mm. The same phenomenon as before was observed in the compression chord. It buckled under the combined act of compression and bending just in front of the fulcrum.

The joint tests on the innovative truss demonstrated that (i) varying the tensile chord thickness did not significantly affect the ultimate load and (ii) the axial stiffener provided substantial additional resistance. It was not possible to determine the exact increase in the joint resistance due to the presence of the axial stiffener because as the increase in capacity was activated, the failure mechanism changed to failure in the diagonal. The basic requirement for the design of the axial stiffener is that the fillet welds connecting it to the tensile chord should be able to undertake the axial load variation of the chord due to the vector sum of the diagonal forces.

**Tubular truss results**

For the tubular truss, there was no variation between the joints; they all shared the same topology and were welded with a 6 mm throat thickness. The truss was tested in the same arrangement as the innovative truss. The weakest point of the joints was the welds. Two joints were tested. In the first test, failure occurred at the weld between the tension diagonal and the tension chord. In the second test, failure occurred at the weld between the tension diagonal and the compression chord. The first joint failed at a total load for both actuators \( F_{\text{tot,1}} = 1128 \text{ kN} \) and the downwards displacement of the compression chords where the load was applied was 66 mm. The second joint had an increased resistance with \( F_{\text{tot,2}} = 1302 \text{ kN} \) with a displacement of 75 mm. Figure 5.26 show the load-displacement curves for the joints and Figure 5.27 and Figure 5.28 show the failed joints.

![Figure 5.26 Load-displacement curves for the two joints tested in the tubular truss](image)

![Figure 5.27 Tension diagonal to tension chord weld failure (first joint test)](image)
5.3.5 Numerical modelling of tubular truss

The performance of the HSS tubular truss was analysed using ABAQUS to assess the accuracy of the hand calculation design methods in Eurocode 3 and the CIDECT guides\textsuperscript{[43],[44]} for HSS. Full details are given in D3.2.2\textsuperscript{[39]}. Good correlation was achieved between the numerical predictions and the hand calculations.

The measured bending stiffness of the trusses was compared with stiffnesses calculated by methods found in literature for planar trusses based on a beam equivalent with stiffness reduced by 1/3; reasonably close agreement was found. The polygonal truss was stiffer than the tubular truss because of the different joints configurations.

For the tubular truss, the characteristics of the measured stress distribution around the perimeter of the tension diagonals close to the weld were compared to predictions using FE methods. The same pattern of uneven strain distribution was observed in the tests, but this was not predicted in the numerical model. It is speculated that this non-uniformity was caused by defects in the welding process.
WP4 PRESTRESSED HSS STRUCTURES

6.1 Objectives of WP4

Using high strength steel in slender columns has little or no effect on the load carrying capacity, so typically the cross-section dimensions are increased. Whenever elastic buckling in compression is the critical failure mode, adding stays and cross-arms (planar or spatial) to slender columns can improve both the elastic buckling load and the load carrying capacity as they provide translational and rotational restraint along the length. WP4.1 investigates the application of this technology in conjunction with HSS columns.

For conventional structural steels, the need to maintain deflections under service loads within acceptable limits for ever-increasing spans has led to the emergence of prestressed steel tubular trusses for long-span structures, in which prestressed cables inside the bottom chord are utilised. In WP4.2, the feasibility of prestressing as a means of increasing the stiffness of HSS trusses is investigated.

6.2 Task 4.1 Prestressed stayed columns

6.2.1 Task 4.1.1 Tests on prestressed stayed columns

Overview of tests

A typical PSSC comprises a slender column (also known as core or main column), cross-arm members and pretensioned stays (i.e. cables or bars). Stayed columns usually have up to three cross-arms along their length. Cross-arms have two (plane or planar) or four (spatial or 3D) arms depending on the application. PSSCs have a wide range of applications: these structural typologies have been investigated experimentally, numerically and analytically by researchers since the 1960s. However, there is still no practical method available for design to be adopted by codes of practice and designers.

The experimental programme on PSSCs comprised 121 full-scale tests. The programme studied different column lengths, column strengths, cross-sections, number of cross-arms and level of prestress. A detailed account of the test programme is given in D4.1.1\[45]. Two different steel strengths were considered: S355 and S690. Cables of two different diameters were tested, where one was approximately twice the area of the other (Figure 6.1).

<table>
<thead>
<tr>
<th>Designation (according to supplier)</th>
<th>Nominal diameter [mm]</th>
<th>Cross Section Area [mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>6x19S+CWR</td>
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<td>47</td>
</tr>
<tr>
<td>6x19S+CWR</td>
<td>13</td>
<td>89</td>
</tr>
</tbody>
</table>

Figure 6.1 Cable cross section

The PSSCs with a single bracing point were composed of a core column with 4 cross-arms intersecting at the midpoint and with an 8 cable system, each connecting a tip of the column to the tip of a cross-arm (Figure 6.2). The PSSCs with double bracing point were composed of a core column with 4 cross-arms intersecting at the two points dividing the column into three equal parts and with a 12 cable system (see Figure 6.3). At the bracing points, the cross-arms were welded to the core column (Figure 6.4). For transport reasons, the 18 m columns were divided into two segments, one segment of approximately 12 m and the remaining segment of 6 m. The segments were fixed together with a bolted connection (the effect of the splice was negligible to the column behaviour in compression).
Table 6.1 summarises the test programme. Note that the values of slenderness for the unstayed column are based on the nominal geometric properties. For the numerical modelling and development of design guidance, slenderness values were based on the actual measured properties and hence varied slightly from those tabulated.
Table 6.1 PSSC test programme

<table>
<thead>
<tr>
<th>Code</th>
<th>Column length</th>
<th>Column cross-section</th>
<th>Number of cross-arms</th>
<th>Steel</th>
<th>Cable diameter</th>
<th>Slenderness of unstayed core column</th>
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<td></td>
<td>12m</td>
<td>18m</td>
<td>CS1</td>
<td>CS2</td>
<td>CS3</td>
<td>4</td>
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<td>X</td>
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<td>X</td>
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<tr>
<td>C01-C2</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>C02-C1</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>C02-C2</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
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<td>C03-C1</td>
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<td>X</td>
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<td>X</td>
</tr>
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<td>C03-C2</td>
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<td>C07-C2</td>
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<td>C08-C1</td>
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<td>X</td>
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<td>X</td>
</tr>
<tr>
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<td>X</td>
<td>X</td>
<td>X</td>
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<td>X</td>
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<tr>
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<td>X</td>
<td>X</td>
<td>X</td>
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<td>C09-C2</td>
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<td>X</td>
<td>X</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>C10-C2</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>C11-C1</td>
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<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

where: CS1 – CHS 101.6 x 8.0; CS2 – CHS 139.7 x 6.3; CS3 – CHS 177.8 x 6.3.

Tensile coupon tests were carried out to obtain the mechanical properties of the core and cross-arms in accordance with EN ISO 6892-1. The cables were tested to confirm their ultimate load and axial stiffness. The mechanical behaviour of cables demonstrate hysteresis upon cyclic loading. This effect is more pronounced upon the first cycle (zero-tension-zero) where the cable suffers an internal rearrangement of its wires, known as constructional adjustment. The permanent elongation upon cycle stabilisation is known as constructional stretch. Therefore 16 tensile tests on the cables were carried out, 8 tensile tests for each cable. The cables were prepared for testing according to EN ISO 377 and tested according to EN 10218-1 and EN 10002-1. As the equivalent elastic modulus was less than 150 GPa, the cables were tested at a stress rate less than 20 MPa/s, and greater than 2 MPa/s. As the anchors significantly influence the boundary conditions of the cables, the cables were tested with the anchors.

Test set up and configuration

Figure 6.5 shows the test set-up. For each column, 11 full scale tests were performed; a first test consisting of the unstayed system and then 5 tests for each cable size with different prestress levels. All columns were tested until the lateral displacement reached a value of around 200 mm at the mid point of the column (for all cases this occurred in the post-buckling range).

Due to laboratory height constraints, the columns were tested in a horizontal position and were propped so that self-weight could be disregarded. Five supports equally spaced along the column length were used, inhibiting significant deformation and stresses in the vertical plane. Nevertheless, the fact that the test was performed in a horizontal position (which prevented buckling from occurring
in a plane other than the horizontal) meant that it was necessary to perform a preliminary test for each column to assess which position (out of four possible positions corresponding to four cross-arms) presented the lowest initial stiffness.

Figure 6.5  PSSC test for column C05

Additionally, the available distance between reaction walls was not sufficient to test 18 m long columns, so reaction frames able to resist the horizontal forces were constructed.

The PSSCs were instrumented to obtain sufficient data for understanding and analysing their behaviour and to be used in future numerical and analytical studies. It was not possible to obtain all the parameters directly from experimental tests and some had to be indirectly assessed through measuring strains in the test specimens. The following parameters were recorded during the experiments:

- Externally applied force.
- Axial displacement of the column.
- Transverse displacements at several points along the column.
- Forces in the cables.
- Forces in the core column and cross-arms.

The load was applied through a 90 ton hydraulic jack ENERPAC, placed between the column and one of the reaction frames (Figure 6.6) and driven by a 700 bar hydraulic pump ENERPAC. In the other reaction frame, the external compression force was measured using a 400 kN load cell placed between the reaction frame and the universal hinge at the opposite end of the hydraulic jack. To release bending moments, hinges were provided at each end of the column.

The axial forces in the cable were measured with eight custom made load cells that read the axial forces in the cables before and during each test. The displacements and deformations during each test were measured with displacement transducers (LVDTs) and linear strain gauges (TML FLA-6-11) placed along the column length.

The initial imperfections were measured using a level laser pointer positioned at the end of the column pointed at graph paper fixed onto a magnetic base (Figure 6.7). Due to the presence of cross-arms, which would block the laser beam, the acquisition was performed at 45° projection. For each column, the profile of the initial imperfections was plotted according to measured values at equally spaced sections along the core column (2000 mm).
Generally, the imperfections do not follow the traditional half-sine shape. The effect of prestress on imperfection profile was evaluated for four of the columns (Figure 6.8 shows the results for column C08, where $T_m$ is the prestress). In some cases the shape of the imperfection was totally changed with increasing prestress.

**Figure 6.6  Test set-up showing hydraulic jack and load cell**

**Figure 6.7  Measurement of initial imperfections**

**Test results**

Stress-strain characteristics for the steel members and cables were obtained from the tensile tests. The stress-strain characteristics of the cables were not linear for small strains, so an equivalent elastic modulus was determined which did not take into account the values at low strain. The values were 117 GPa and 104 GPa for the 10 mm and 13 mm diameter cables respectively.
Figure 6.8 Measured geometric imperfections for all tests for column C08

Each test series was initiated with an unstayed column test followed by the remaining tests on the stayed columns with different prestress levels. Figure 6.9 shows the lateral displacement at mid span against the applied load for all unstayed columns. As previously stated, all columns were tested until the lateral displacement at mid span reached values of around 200 mm. For those columns with higher values of slenderness, the load protocol appeared to be inappropriate as it caused vibrations during load application. As shown in Figure 6.9, these vibrations occurred for columns C01 and C02: after the application of the maximum load, the columns oscillated and then stabilised at the experimental load value (near to the elastic critical buckling load value).

Figure 6.9 Force versus lateral displacement at mid span for unstayed columns

The PSSCs were tested in the elastic range. For this reason, failure was associated with geometric nonlinearities rather than material yield strength. In general, the results show that strains increase with the increase in prestress level, while the cable diameter seems to have a negligible influence on the strain. The structural response of the columns with the stay system and the cross-arms was measured by the axial load versus end-shortening and lateral displacement curves recorded at mid span, for example Figure 6.10 shows the behaviour for column C10.

The test results clearly showed the benefit of including a stay system with the exception of columns with higher values of slenderness (columns C01, C02 and C05), where vibrations occurred during the test affecting the post-buckling response.
Figure 6.10  Force versus lateral displacement and end shortening for C10 stayed column (18 m length, S690)

Figure 6.11 and Figure 6.12 present the measured ultimate load versus initial pretension load for the 12 m and 18 m stayed columns respectively. The normalised values of experimental ultimate loads versus initial pretension load are also presented. Table 6.2 compares the ultimate loads for the stayed and unstayed columns. The benefits of including a stay system with cross-arms for long columns are clearly demonstrated. The benefit increases as the slenderness increases. In stockier columns, the benefits of including a stay system and cross-arms is not distinct. Additionally, the benefits of including a stay system with cross-arms in long columns are also more noticeable for columns with a stay system with a double bracing point. This is because the angle between the cables and the core column is higher for double braced columns, increasing the perpendicular component of the tension in the stays at bracing points which, ultimately, results in a beneficial effect by stabilising the column. Comparing column C11 (stay system and cross-arms with double bracing point) and C09 and C10 (stay system and single cross-arms), although they all have the same slenderness, the benefits are more evident in column C11. The same is true when comparing another set of three columns that have the same slenderness: the increase in the ultimate load for column C08 (double bracing point) is more evident than in columns C06 and C07 (single bracing point). For the same reason, shorter columns present more benefits from the stay system and cross-arms than longer columns. Comparing the results of column C01 and C02 with results from the most slender column (C05), it is possible to conclude that shorter columns present more significant benefits.
Figure 6.11 Ultimate load versus initial pretension load for 12 m stayed columns

All columns presented a buckling shape close to a symmetric mode, with the exception of columns C01 and C02. The non-symmetric shape can be explained by the shape of initial geometric imperfections which forces one half of the core to start deflecting prior to the other half. Besides the obvious confirmation that a properly designed stayed column will significantly increase its ultimate resistance compared to the corresponding unstayed column, the following conclusions could be drawn from the experimental investigations:

- Adding a stay system to long and very slender columns provides effective additional restraint that increases the ultimate resistance of column. This observation confirms similar conclusions from the literature for short stayed columns.
- The main column cross-section has a direct effect on ultimate resistance as it changes the column slenderness and should be directly accounted for in a design method.
- Prestressed columns are highly sensitive to geometric imperfections.
The use of HSS showed no benefit for the tested PSSC. This was to be expected as the unstayed core columns exhibited very high slenderness (Table 6.1). Because of laboratory limitations, it was not possible to test larger cross-sections with the same 12 m and 18 m length, hence the choice of specimens C01 to C11. However, testing HSS stayed columns was very useful to identify imperfection profiles and to be able to characterise the behaviour of realistic columns for practical application going through a normal fabrication process. It is expected that the steel strength will become relevant as the cross-section increases, should these PSSC be used, for example, for applied compressive forces in the range of 500 kN.

Figure 6.12 Ultimate load vs initial pretension load for 18 m stayed columns

The use of HSS showed no benefit for the tested PSSC. This was to be expected as the unstayed core columns exhibited very high slenderness (Table 6.1). Because of laboratory limitations, it was not possible to test larger cross-sections with the same 12 m and 18 m length, hence the choice of specimens C01 to C11. However, testing HSS stayed columns was very useful to identify imperfection profiles and to be able to characterise the behaviour of realistic columns for practical application going through a normal fabrication process. It is expected that the steel strength will become relevant as the cross-section increases, should these PSSC be used, for example, for applied compressive forces in the range of 500 kN.
<table>
<thead>
<tr>
<th>Columns</th>
<th>12 m</th>
<th>4 X -arms</th>
<th>C35</th>
<th>18 m</th>
<th>4 X -arms</th>
<th>S355</th>
<th>18 m</th>
<th>4 X -arms</th>
<th>C35</th>
<th>12 m</th>
<th>4 X -arms</th>
<th>S355</th>
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Table 6.2 Comparison of the tests results for the stayed and unstayed columns

<table>
<thead>
<tr>
<th>Columns</th>
<th>12 m</th>
<th>4 X -arms</th>
<th>C35</th>
<th>18 m</th>
<th>4 X -arms</th>
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<td>1.96</td>
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<td>12.05%</td>
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C1: 10 mm diameter cable, C2: 13 mm diameter cable

CS1: CHS 101.6 x 8.0; CS2: CHS 139.7 x 6.3; CS3: CHS 177.8 x 6.3
6.2.2 Task 4.1.2 Numerical analysis of prestressed columns

A detailed account of the investigation is given in D4.1.2\textsuperscript{[50]}.

**Finite element model**

The measured values of the material and geometrical properties of the columns, cables and cross-arms were used in the numerical model. The model assumed bow imperfections with values of L/1000 and L/750. Besides deviations from the axis of the column, these imperfections also cover deviations and lack of orthogonality of the load application system as well as residual imperfections.

Four-node shell finite elements (S4R) were used to model the core columns and cross-arms. The cables were modelled using truss finite elements where an initial pre-defined field (prestress) was applied. A brief mesh discretisation study was performed and it was concluded that 25 elements were enough to achieve numerical convergence. The prestressed application was defined using the "predefined field" option in ABAQUS. This option enables a predefined field for a given parameter to be defined, in this case, a predefined field of initial stresses. These stresses are directly computed from the force applied to the cables divided by their nominal cross-sectional area.

The columns were simply supported at both ends. Rigid body constraints (tie - nodes) were applied between the central nodes and the sections at the end of the column and tie constraints were applied between the column and the cross arms. Between the column and cables and between cross-arms and cables coupling constraints were applied. An axial compression load was applied at the axially free end in the central reference point.

Linear buckling analysis (LBA) and geometrically and materially nonlinear analysis with imperfections included (GMNIA) were performed using the FE software ABAQUS (arc length method). First, an LBA (with cables, but without any applied prestress) was carried out to obtain the eigenmodes of the column. Secondly, using the *IMPERFECTION option, the shape and amplitude of the eigenmode was inserted into the model as an imperfection. Then, a first GMINA was performed (*STATIC, GENERAL) in which the prestress was applied. The column undergoes small axial displacements and, as a consequence, the cables lose a small amount of the applied prestress. After this analysis, another nonlinear GMNIA was performed (*STATIC, RIKS) where the compression load was applied.

**Comparison between numerical and experimental results**

Each of the tests was modelled (a total of 132 linear buckling analyses and 396 GMNIA). In the majority of cases, the numerical model predicted the experimental behaviour reasonably well. In some cases the numerical model predicted higher ultimate loads, which may have been caused by unpredicted deviations and lack of verticality of the load application system, leading to a premature loss of stiffness in the tests. The assumption of a bow shaped imperfection may also lead to differences in the numerical prediction compared to the measured behaviour. In order to assess the influence of the shape of geometric imperfections, a comparison using two imperfection shapes (bow imperfection and measured imperfection) was performed. In some cases using the measured imperfection led to closer agreement with the test results. However, in other cases, it made little difference. For the majority of results, using L/1000 and L/750 gave good agreement with experimental results, although not always on the safe side.

Load-displacement curves (experimental vs. numerical) for the S690 column C04 are shown in Figure 6.13 and Figure 6.14 for a prestress of 2 kN and 14 kN respectively.
Figure 6.13 Load-displacement (mid span deflection): C04-C1 ($T_{\text{ini}} = 2$ kN)

Figure 6.14 Load-displacement (mid span deflection): C04-C1 ($T_{\text{ini}} = 14$ kN)
### Table 6.3 Selected results of numerical modelling of PSSC tests

<table>
<thead>
<tr>
<th>Prestress</th>
<th>Ultimate load</th>
<th>Diff. 1</th>
<th>Diff. 2</th>
</tr>
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<tr>
<td></td>
<td>$T_i$ [kN]</td>
<td>$\sigma_i$ [MPa]</td>
<td>$P_{u,exp}$ [kN]</td>
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<td>104.66</td>
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</tr>
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<td>2000</td>
<td>22.47</td>
<td>112.17</td>
</tr>
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<td>192.89</td>
</tr>
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<td>123.35</td>
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<tr>
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<td>223.40</td>
<td>129.29</td>
</tr>
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<td>120.20</td>
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<td>103.96</td>
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<td>134.71</td>
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<tr>
<td>C11 C2</td>
<td>9000</td>
<td>101.12</td>
<td>155.85</td>
</tr>
</tbody>
</table>

Ampl 1: geometric imperfection: L/1000
Ampl 2: geometric imperfection: L/750
6.2.3 Task 4.3a Design guidance for prestressed columns

Literature review

Two methods for the calculation of the critical elastic buckling load and a simplified method for the calculation of the ultimate buckling resistance form the basis for the development of novel design guidance for PSSCs. Method 1 was developed by Smith et al\textsuperscript{[51]} for pin-ended PSSCs with one cross-arm and aims to calculate the load that triggers instability through a series of equilibria. Equilibrium in the longitudinal and transverse direction leads to the initial axial force in the column and cross-arm respectively, while kinematics under the hypothesis of small deformations lead to the tensile force in the stays. The critical load for Mode I (symmetric) and Mode II (antisymmetric) is finally obtained by equilibrium at the end of the column (Figure 6.15).

Method 2 (Hafez and Temple\textsuperscript{[52]}) is a modification of Method 1, expressing the force at the moment of buckling in terms of the load in the column (instead of the tension in the stays). The concept of optimum prestress in the cables, $T_{\text{opt}}$, as “the initial pretension in the stays that disappears completely just after the load in the column reaches its maximum buckling” is introduced. Three distinct zones associating the level of initial pretension with the corresponding critical load are defined, as shown in Figure 6.16.

The critical load is determined according to Method 3 proposed by Wadee et al\textsuperscript{[53]}, in which the ultimate capacity of PSSCs is determined through curve fitting of experimental and numerical results. Three different global geometric imperfection amplitudes were considered in the numerical models ($L_c/1000$, $L_c/400$ and $L_c/200$, where $L_c$ is the total length of the core column) and the maximum prestress level was set at $3T_{\text{opt}}$. The load carrying capacity was expressed in terms of $N_{\text{max}}/N_c$. The formulae were only derived for stayed columns presenting symmetric and antisymmetric buckling modes meeting the following conditions:

- The stayed column is completely symmetrical and concentrically loaded without any eccentricity and geometrical imperfections.
- The connections between cross-arms and core are rigid.
- The connections between the stays and the core are ideal hinges.
- The connections between the stays and the cross-arms are ideal hinges.

![Figure 6.15 Model and free body diagram for Mode I buckling \textsuperscript{[51]}](image-url)
Figure 6.16 Behavioural zones for ultimate resistance according to [52]

**Design procedure**

A flowchart summarising the proposed design method is shown in Figure 6.17. More explanation is given in D4.3.a[54].

Step 1: Obtain \( N_{cr,a} \) from LBA

Step 2: Calculate \( N_{cr,f} \) from:

\[
N_{cr,f} = \frac{N_{cr,a}}{C_2} + 4T_{ini} \cos \alpha
\]

\( C_2 = 1 + \frac{2 \cos^2 \alpha}{\frac{1}{K_y} + \frac{2 \sin^2 \alpha}{K_w}} \)

Step 3: Calculate \( \lambda \) from:

\[
\lambda = \sqrt{\frac{f_y}{N_{cr,f}}}
\]

Step 4: Calculate \( \eta_{tot} \) from:

\[
\eta_{tot} = \eta_p \times \eta_{ecc} \times C_p
\]

\[
C_p = \left( 1 - \frac{K_y \sin^2(\alpha) L}{N_{cr,f}} \right)
\]

\[
\eta_p(T_{ini}) = \frac{p_1 T_{ini}/A_1 + p_2}{(T_{ini}/A_1)^q + q_1 T_{ini}/A_1 + q_2}
\]

Step 5: Reduction factor:

\[
\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}
\]

\( \phi = 0.5(1 + \eta_{tot} + \lambda^2) \)

Step 6: Obtain the maximum applied force:

\[
N_{max} = C_2 \left( \mu N_{cr} - 4T_{ini} \cos \alpha \right)
\]

Figure 6.17 Flowchart of the proposed design method for PSSCs

where:

- \( A \) is the core column cross-sectional area
- \( A_s \) is the cable cross-section
- \( T_{ini} \) is the initial prestress in the cables
- \( \alpha \) is the angle between the core column and the stays
$K_s$, $K_z$ and $K_{ca}$ are parameters which account for the interaction of the axial stiffness of the column, stays and cross-arms respectively.

$\eta_{pr}$ is an additional imperfection factor caused by the prestress in the cables.

$C_{pr}$ is a parameter which accounts for the cable stiffness.

$\eta_{EC3}$ is the normalised geometric imperfection factor directly taken from curve a0 ($a_{EC3}$=0.13) for S690 steels and from curve a ($a_{EC3}=0.21$) for S355 steels.

$N_{pl}$ is the axial plastic resistance of the core column ($= Af_y$).

$N_{cr,a}$ is the elastic critical load, obtained from LBA (linear buckling analysis).

$N_{cr,f}$ is the elastic critical load at the instant of buckling of the prestressed column.

$N_{a,\text{max}}$ is the ultimate buckling resistance of a prestressed column.

The variation of the elastic critical stress with the increase in the pretension in the cables for different levels of slenderness ($L/i$) was studied using the numerical model developed in WP4.1.2. Bearing in mind that Mode I (symmetric) is always dominant, a consistent increase in $N_{cr}$ is observed (Figure 6.18).

The basis of the proposed design guidance is the calculation of column slenderness, so the first step involves the calculation of the elastic critical stress of the PSSC, i.e. the ultimate buckling resistance, $N_{cr,a}$, which can be obtained directly from an LBA (Linear Buckling Analysis).

The critical load at the moment of buckling, $N_{cr,f}$, is then determined (Step 2) according to Hafez and Temple[52], in terms of $N_{cr,a}$, the initial pretension in the stays, the angle between the core column and the stays, and the parameter $C_2$.

The non-dimensional slenderness, $\bar{\lambda}$, is calculated next (Step 3), as a function of the modified critical load according to EN 1993-1-1, Clause 6.3.1.2 (1).

Step 4 involves the determination of $\eta_{tot}$, a factor which takes account of the bending moment arising from the presence of the cable pretension. $\eta_{tot}$ is used in the determination of $\chi$, a modified reduction factor applied to the plastic column axial resistance (Step 5).

In the final step, $N_{a,\text{max}}$ is obtained by combining steps 2 and 5. The critical load, $N_{cr,f} = \chi N_{pl}$, is recalculated and substituted into the equation for $N_{cr,a} = N_{a,\text{max}}$, leading to the computation of the ultimate buckling resistance of a PSSC.

### Figure 6.18 Elastic critical load versus pretension in the cables

The graphs show the relationship between the critical load and the pretension in the cables for different configurations of the core column and stays. The data points for configurations C01 and C09, with 12 m and 18 m unstayed lengths, are plotted. The curves indicate the trend of the critical load increase with pretension for configurations C1 (10 mm φ cable) and C2 (13 mm φ cable).

- **C01**: 12 m unstayed = 4.8
- **C09**: 18 m unstayed = 4.1
- **C1**: 10 mm φ cable
- **C2**: 13 mm φ cable
**Statistical assessment**

Using this method, the ultimate load was calculated for all column cases (except for those with two cross-arms, i.e. columns C08 and C11) and compared against experimental and numerical results, as well as results obtained from the method proposed by Wadee et al.[53] for three different imperfection amplitudes ($L_c/1000$, $L_c/400$ and $L_c/200$). Figure 6.19 shows that a better correlation (higher $R^2$ and lower coefficient of variation values) is achieved for this new proposed method.

![Figure 6.19 Comparison of predictions using the new method (left) and Wadee’s method [53] (right) against test results](image)

6.2.4 Task 4.2.5a Dynamic response of prestressed columns

Prestressed structures allow the minimisation of mass with increasing stiffness. As a consequence, the level of dynamic actions necessary to introduce significant vibration in the structure tend to decrease and in some cases there is the need for including dissipative devices such as tuned mass dampers. The design of such devices depends very significantly on the precision of the estimated eigenfrequencies in the original structure.

In this task the natural frequencies and respective vibration modes of the PSSCs were obtained for different levels of pretension in the cables. Experimental modal analysis was applied using output-only measurement techniques based on ambient vibration of the structure. FE models were developed and calibrated with experimental results. Moreover, the same operational modal analysis techniques were used to evaluate the level of pretension in the cables. A simple methodology was proposed, which can be used to control the cable pretension during construction. A detailed account of the investigation is given in D4.2.5a[55].

**Experimental investigation**

Cables are critical force-transmitting members in PSSCs. The tension of cables influences the internal force distribution and therefore an accurate determination of cable forces plays an important role in the construction. Several approaches have been adopted in engineering practice to evaluate the cable forces, either in the construction or service stage of structures. Permanent devices such as load cells at the end of the cables, strain gauges or fibre sensors embedded inside the cable cross-section have been utilised in civil engineering. The first method is accurate but expensive, while the other two are less expensive but generally suffer deterioration in accuracy during time.

The geometry of a cable allows a simple relationship between installed tension and natural frequencies. The ambient vibration method to measure the natural frequencies provides an excellent way to estimate indirectly the installed force during the construction or service stage. This method is typically applied by first identifying the cable frequencies from the ambient vibration measurements. A pre-determined formula or numerical simulation can then be used with these identified frequencies to estimate the cable force.

Dynamic tests were carried out to obtain the tension in the cables from the natural frequency. One cable (10 mm diameter) was studied under various levels of pre-tension. The vibration measurements were carried out using one accelerometer attached to the cable in a position that was able to detect the transverse vibration of the cable in at least the three lower natural modes, as illustrated in Figure 6.20.
The natural frequencies obtained from measurements were compared to those predicted by the following equation:

\[ f_n = 4 \left( \frac{1}{T} \right)^2 \]  

where:

- \( f_n \) is the frequency of the \( n \)th mode of vibration
- \( n \) is the mode of vibration in consideration
- \( T \) is the tension in the stay
- \( m \) is the mass of the stay by unit length
- \( L \) is the length of the stay between supports

From Table 6.4 it can be concluded that the use of natural frequencies to control the tension in the cables is in general possible with a good level of accuracy.

### Table 6.4 Stay tension obtained from measured natural frequencies

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<thead>
<tr>
<th>Vibration mode</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
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<tr>
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<td>4100 N</td>
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<td>12.8</td>
<td>18.9</td>
<td>25.2</td>
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<td>38.6</td>
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<td>5.7%</td>
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<td>16.6</td>
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<td>33.9</td>
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A second set of tests were carried out to study the variation of natural frequency with stay geometry and pretension. Accelerometers were positioned at 9 equally spaced locations along the column. The modal identification was performed using ARTeMIS Extractor software through a decomposition method in the frequency domain, FDD (Frequency Domain Decomposing). Frequency Domain Modal Analysis technique was used to extract singular values of the spectral densities matrices. Eigen frequencies were obtained by Peak picking and respective modes were estimated, which are represented in Figure 6.21.

**Figure 6.21 Vibration modes for column C07, 10 mm diameter stay**

**Numerical modal analysis**

Considering PSSC C07 with 4 arms and a 10 mm cable diameter, an FE model was developed in ABAQUS according to the geometry defined for the specimens and the supports at the extremities and below the lower arm at mid span. At the extremities the displacements were considered fully restrained and the rotations were free. Under the mid-span vertical arm, the displacements were considered as fully restrained and the rotations were partially restrained using rotational springs with stiffness 50 kNm, which best fitted the modes and frequencies obtained from measurements.

The mode shapes for column C07, 10 mm diameter cable are shown in Figure 6.22. Table 6.5 compares experimental and numerical results for natural frequencies.
Table 6.5  Eigenfrequencies of calibrated models compared to measurements for columns with one set of cross-arms

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<td>0.1</td>
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<tr>
<th></th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>Mode 4</th>
<th>Mode 5</th>
<th>Tension [N]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>3.8</td>
<td>5.9</td>
<td>5.9</td>
<td>7.4</td>
<td>9.8</td>
<td></td>
</tr>
<tr>
<td>Model</td>
<td>3.83</td>
<td>5.86</td>
<td>5.92</td>
<td>7.37</td>
<td>9.48</td>
<td></td>
</tr>
<tr>
<td>Error [%]</td>
<td>0.8</td>
<td>0.7</td>
<td>0.3</td>
<td>0.4</td>
<td>3.27</td>
<td></td>
</tr>
</tbody>
</table>

Discussion

The work demonstrated that Equation (8-1) can be used to relate the natural frequencies to the tension in the cables and can thus be used as a check during installation that the cables are being preloaded to the correct level. A representative set of geometries and cable pretension forces was chosen in order to obtain natural frequencies and mode shapes, both by ambient vibration tests and by FE analysis. Both experimental and numerical results showed that the natural frequencies tend to increase when increasing the pretension from $T = 2$ kN to $T = 15$ kN.

6.3 Task 4.2 Prestressed trusses

Research on prestressed steel arched trusses dates back to the 1990s, when an Australian research team investigated the structural response of arched structures erected through the tensioning of a cable in their bottom chord\(^{[56,57]}\). The prestressing technology applied within WP4.2 is based on a methodology developed and effectively utilised by one of the project partners, S-Squared\(^{[58,59]}\).
6.3.1 Task 4.2.1 Tests on Individual Truss Members

Laboratory testing

A series of tensile and compressive member tests on HSS tubular elements containing prestressing cables was carried out. A detailed account of the investigation is given in D4.2.1\[^{60}\]. A total of 22 specimens were tested under axial loading (12 in tension and 10 in compression). The steel tubes were hot-finished S460 and S690 SHS 50×50×5, with 7-wire strand Y1860S7 steel prestressing cables. The key variables of the cable-in-tube specimens were the steel strength (S460 and S690), initial prestress level (no cable, $P_{\text{nom}} = 5$ kN, $0.5 P_{\text{opt}}$ and $P_{\text{opt}}$) and the presence of grout. The prestress level $P_{\text{opt}}$ is the optimum prestress force that causes the cable and the tube to yield simultaneously when the system is under tension, hence maximising the extent of the elastic range \[^{61}\]. A list of the test specimens is provided in Table 6.6.

Table 6.6 Summary of the Prestressed Cable-in-Tube Test Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Nominal length (mm)</th>
<th>No. of cables</th>
<th>Grouted (G)/ Non-grouted (NG)</th>
<th>Target prestress (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T460NGN</td>
<td>2000</td>
<td>0</td>
<td>NG</td>
<td>N/A</td>
</tr>
<tr>
<td>T460NG0</td>
<td>2000</td>
<td>1</td>
<td>NG</td>
<td>$5 (P_{\text{nom}})$</td>
</tr>
<tr>
<td>T460NG1</td>
<td>2000</td>
<td>1</td>
<td>NG</td>
<td>$95 (0.5P_{\text{opt}})$</td>
</tr>
<tr>
<td>T460NG2</td>
<td>2000</td>
<td>1</td>
<td>NG</td>
<td>$191 (P_{\text{opt}})$</td>
</tr>
<tr>
<td>T460G1</td>
<td>2000</td>
<td>1</td>
<td>G</td>
<td>$95 (0.5P_{\text{opt}})$</td>
</tr>
<tr>
<td>T460G2</td>
<td>2000</td>
<td>1</td>
<td>G</td>
<td>$191 (P_{\text{opt}})$</td>
</tr>
<tr>
<td>C460NG0</td>
<td>1000</td>
<td>1</td>
<td>NG</td>
<td>$5 (P_{\text{nom}})$</td>
</tr>
<tr>
<td>C460NG2</td>
<td>1000</td>
<td>1</td>
<td>NG</td>
<td>$191 (P_{\text{opt}})$</td>
</tr>
<tr>
<td>C460G0</td>
<td>1000</td>
<td>1</td>
<td>G</td>
<td>$5 (P_{\text{nom}})$</td>
</tr>
<tr>
<td>C460G1</td>
<td>1000</td>
<td>1</td>
<td>G</td>
<td>$95 (0.5P_{\text{opt}})$</td>
</tr>
<tr>
<td>C460G2</td>
<td>1000</td>
<td>1</td>
<td>G</td>
<td>$191 (P_{\text{opt}})$</td>
</tr>
<tr>
<td>T690NGN</td>
<td>2000</td>
<td>0</td>
<td>NG</td>
<td>N/A</td>
</tr>
<tr>
<td>T690NG0</td>
<td>2000</td>
<td>1</td>
<td>NG</td>
<td>$5 (P_{\text{nom}})$</td>
</tr>
<tr>
<td>T690NG1</td>
<td>2000</td>
<td>1</td>
<td>NG</td>
<td>$86 (0.5P_{\text{opt}})$</td>
</tr>
<tr>
<td>T690NG2</td>
<td>2000</td>
<td>1</td>
<td>NG</td>
<td>$172 (P_{\text{opt}})$</td>
</tr>
<tr>
<td>T690G1</td>
<td>2000</td>
<td>1</td>
<td>G</td>
<td>$86 (0.5P_{\text{opt}})$</td>
</tr>
<tr>
<td>T690G2</td>
<td>2000</td>
<td>1</td>
<td>G</td>
<td>$172 (P_{\text{opt}})$</td>
</tr>
<tr>
<td>C690NG0</td>
<td>1000</td>
<td>1</td>
<td>NG</td>
<td>$5 (P_{\text{nom}})$</td>
</tr>
<tr>
<td>C690NG2</td>
<td>1000</td>
<td>1</td>
<td>NG</td>
<td>$172 (P_{\text{opt}})$</td>
</tr>
<tr>
<td>C690G0</td>
<td>1000</td>
<td>1</td>
<td>G</td>
<td>$5 (P_{\text{nom}})$</td>
</tr>
<tr>
<td>C690G1</td>
<td>1000</td>
<td>1</td>
<td>G</td>
<td>$86 (0.5P_{\text{opt}})$</td>
</tr>
<tr>
<td>C690G2</td>
<td>1000</td>
<td>1</td>
<td>G</td>
<td>$172 (P_{\text{opt}})$</td>
</tr>
</tbody>
</table>

T = tension, C = compression

The SHS 50×50×5 HSS tubes were cut to the required length and then welded onto 20 mm thick end plates with stiffeners added (Figure 6.23). An Enerpac PTJ-6S pneumatic stressing jack was used to prestress the cable-in-tube systems. The stress in the cables was held by an anchoring system, which consisted of a chuck and three wedges, as displayed in Figure 6.24.
Prior to the member tests, the following material tests were carried out:

- Tensile coupon tests on material extracted from the flat faces and corner regions of the HSS tubes were conducted to determine their basic engineering stress-strain response.
- Tensile tests on 2 m long specimens of the 7 wire strand Y1860S7 prestressing steel cables were carried out to determine its nonlinear stress-strain response.
- Standard cube tests were performed to measure the grout strength of the Portland based grout, with water to cement ratio of 0.35, used for the grouted specimens. Based on the grout tests, it was decided to carry out the cable-in-tube systems 10 days after prestressing and grouting, when the grout strength was found to be around 50 MPa.

A total of 12 cable-in-tube specimens were tested under axial tensile loading. Accurate measurements of the geometric dimensions of the test specimens were made prior to testing. The actual initial prestress force $P_i$ was also measured for each of the tested cable-in-tube specimens. All tests were performed in an Instron 2000 kN hydraulic testing machine, where displacement control at a uniform rate of 0.5 mm/min was used to drive the testing machine. The axial load and displacement were read from the loading machine directly, and the tube strains were obtained from four strain gauges fixed to the four faces of the tube at mid-length.

To investigate the compressive response of the HSS cable-in-tube systems, representing the bottom chord elements in a truss under uplift forces (e.g. due to wind), a total of 10 specimens were tested. All tests were performed in an Instron 2000 kN hydraulic testing machine. The geometric dimensions of the compressive test specimens, as well as the initial bow imperfection amplitude $\omega_i$ were measured. Pin-ended boundary conditions were applied through hardened steel knife edge supports, which allowed only in-plane rotation of the members about one axis. Strain gauges attached to the four faces of the tubes at mid-height measured the strain, and a string potentiometer measured the lateral deflection at mid-height during the tests. The applied load was read directly from the loading machine.
**Experimental results and discussions**

The measured overall axial load-displacement response of the cable-in-tube tensile specimens are shown in Figure 6.25 for the S460 and S690 specimens, where the influences of adding a prestressing cable, increasing prestress level and adding grout may be seen. Comparisons of the results of the T460NGN and T460NG0 specimens and the T690NGN and T6990NG0 specimens show that the addition of the prestressing cable increased the tensile capacity of the system by approximately 30%. This is essentially due to the fact that the prestressing cable, with a fracture load of 270 kN, will also contribute to resisting the applied tensile load. As illustrated by specimens T460NG1, T460NG2, T690NG1 and T690NG2, increasing the level of initial prestress, whereby the yielding of the tube is delayed and brought closer to the yielding point of the prestressing cable, increases the extent of the elastic region of the system’s response. This increase in the elastic response range reduces the axial deflection required to reach ultimate load. The presence of the grout, as observed in specimens T460G1, T460G2, T690G1 and T690G2 members, had a minimal effect on the strength of the cable-in-tube system, but was found to improve their ductility. Comparison of the results for S460 and S690 shows that the S460 specimens gained more improvement in their yield strength (61% for T460NG2, 39% for T460NG1 and 10% for T460NG0 members) than the S690 specimens (32% for T690NG2, 25% for T690NG1 and 6% for T690NG0 members), revealing that lower strength steels in fact benefit more from the addition of cables and prestressing than higher strengths.

The load-lateral deflection curves for the S460 and S690 compressive specimens are shown in Figure 6.26. Prestressing is detrimental for self-anchored cable-in-tube systems in compression. However, the observed reduction in resistance was not as large as the applied initial prestress. The presence of grout was shown, as expected, to increase the compressive resistance.

![Figure 6.25 Comparison of the axial load-displacement responses of the S460 and S690 tensile specimens](image-url)
6.3.2 Task 4.2.2 Tests on prestressed trusses

A total of four HSS trusses of 11 m length were tested. A detailed account of the investigation is given in D4.2.2 [62]. The arched truss with no prestressing cable, Truss 1, was used as a reference specimen. A nominal prestress level of 5 kN was applied in Truss 2, where the benefits obtained from the mere addition of the prestressing cable were demonstrated by comparisons with the measured response of Truss 1 with no prestressing cable. To assess the influence of the applied prestress level, two different prestress levels, $0.5P_{\text{opt}}$ and $P_{\text{opt}}$ were applied to Truss 3 and Truss 4, respectively. The structural elements comprising the arched trusses were S460 hot finished SHS. The top chord elements were SHS 70×70×6.3 and the bottom chord elements were SHS 50×50×5. The bracing elements were SHS 40×40×2.9, apart from those at the end supports which were SHS 50×50×5. The overall configuration of the assembled arched trusses is presented in Figure 6.27. All joints between the various truss elements were full penetration butt welds. For ease of transportation, each truss was fabricated in three pieces which were assembled by bolted steel plates (Labelled EP1 and EP2 in Figure 6.27).

![Figure 6.26 Load versus lateral displacement responses of the S460 and S690 compressive specimens](image-url)
Figure 6.27 Schematic drawing of the arched truss specimens

The tests were carried out in a purpose-built rig that was designed to examine the behaviour of the HSS prestressed arched trusses subjected to vertical downward loading. Figure 6.28 shows the general test set-up configuration. Vertical downward loading was applied to each truss through five point loads on the bottom chord at the locations of the hydraulic jacks as shown in Figure 6.28. A lateral support system was developed to maintain the overall lateral stability of the tested trusses during loading and to ensure in-plane behaviour. A series of lateral restraint cables were used, connected to the truss top chord at one end and supported by the test rig at the other end. A total of eighteen lateral restraint cables, nine on each side of the top chord, placed at discrete locations as shown in Figure 6.28, were deemed necessary. All truss specimens were extensively instrumented which allowed a full representation of the response of the HSS prestressed arched trusses to be obtained. The prestressing cables located within the bottom chord of the fully assembled trusses were first anchored in place at both ends, and then prestressed.

Experimental results and discussions

The load versus vertical displacement curves of all the truss specimens at each of the loading positions (loading points 1-3) are shown in Figure 6.29, where the influences of the addition of the prestress cable and the varying applied prestress loads on the measured response are illustrated. In-plane failure modes were observed for Truss 1, Truss 2 and Truss 4, as shown in Figure 6.30. Truss 3 showed evidence of out-of-plane deformations, as shown in Figure 6.31, and was therefore unable to reach the system’s ultimate load. The influence of prestress, which is to delay the yielding of the bottom chord and hence extend the elastic range of the system response, may be seen in Figure 6.29.
Figure 6.28 Truss test rig set-up

Figure 6.29 Load versus vertical displacement curves of the four tested trusses
Vibration tests

To investigate the effects of the addition of prestressing cables and increasing prestress levels on the vibration behaviour of the prestressed arched trusses, measurements of their dynamic properties were carried out. The focus of this study was to determine the natural frequencies and associated mode shapes of the prestressed arched trusses. A modified impact hammer modal testing method was employed. This involved a rubber hammer, which caused free vibration of the prestressed arched trusses, and accelerometers placed at discrete locations on the trusses, which measured the resulting system response. The recorded acceleration versus time responses were used to extract the natural frequencies of each of the tested trusses by means of the Fast Fourier Transform (FFT), which translates the measured acceleration from the time domain into the frequency domain. The measured natural frequencies corresponding to modes 1 and 2 of the three trusses are reported in Table 6.7, where the results of the frequency analysis using the FE analysis package ABAQUS are also provided for comparison purposes.
Table 6.7 Measured natural frequencies of the truss specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mode 1</th>
<th></th>
<th>Mode 2</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test (s(^{-1}))</td>
<td>FE (s(^{-1}))</td>
<td>Test (s(^{-1}))</td>
<td>FE (s(^{-1}))</td>
</tr>
<tr>
<td>Truss 1 (No cable)</td>
<td>17.1</td>
<td>17.10</td>
<td>39.1</td>
<td>47.8</td>
</tr>
<tr>
<td>Truss 2 ((P_{\text{num}}))</td>
<td>16.8</td>
<td>17.10</td>
<td>36.6</td>
<td>49.1</td>
</tr>
<tr>
<td>Truss 3 (0.5 (P_{\text{opt}}))</td>
<td>16.8</td>
<td>17.30</td>
<td>37.7</td>
<td>46.9</td>
</tr>
</tbody>
</table>

Measurements of the mode shapes corresponding to the extracted natural frequencies were also carried out. The results showed that the addition of the cable and increasing prestress levels had an insignificant effect on the modal properties of the prestressed high strength steel arched trusses.

6.3.3 Task 4.2.3 Monitoring of long-term performance of prestressed trusses

Owing to a downturn in the market for the project partner S-Squared, it was not possible to gain access to monitor structures in practice as no suitable structures were installed during the project. As an alternative to this, greater emphasis was placed on laboratory testing, with the number of individual truss member tests increased from 10 to 20, though with a commensurate decrease in the number of truss tests (from 6 to 4) to maintain approximately the same level of material usage. Prestressing was initially going to be performed by specialist fabricators, but it was decided to carry this out in-house; while this required considerably more resources, the experience gained helped to direct the research in terms of practical levels of prestress that are achievable, reliability of end anchorage, the importance of grouting, health and safety aspects and so on. Key lessons learnt, which are relevant to practical prestressing operations, are as follows:

- Losses in the applied cable prestress were encountered upon the removal of the stressing jack for the case of both the individual members and the arched trusses. This loss of prestress force was mainly attributed to the slipping of the prestressing cable in the wedges of the adopted cable anchoring system. It was found that the amount of prestress loss depends on the applied prestress and the length of the specimen. The higher the applied prestress level and/or the longer the specimen length, the lower the percentage loss in the prestress. Owing to their length, the prestress losses in the trusses were relatively low.
- The application of prestress higher than the target value was therefore necessary to achieve the final desired level. It was also found that hammering in the wedges inside the chuck provided a firmer grip on the cable and reduced the level of prestress loss.
- Vibration of the tubes with a wooden plank or similar was found to facilitate the grouting process and ensure that the tubes were entirely filled by the grout.

Finally, although the full-scale monitoring of the trusses was not possible, shorter term and smaller scale monitoring was carried out in the laboratory. The variation of the cable prestressing force with time was measured for one of the tested prestressed arched trusses. The force in the cable of the truss prestressed to \(P_{\text{opt}}\) (Truss 4) was monitored over a duration of three weeks. The cable was prestressed to an average value of 152.7 kN. During this period, the temperature in the laboratory ranged from 10 to 32°C. The cable prestressing force decreased by 0.97 kN on average, 0.6% of the initial prestress level. The rate of prestress loss decreased steadily with time; it was highest within the first five days and flattened out subsequently.

6.3.4 Task 4.2.4 Numerical analysis on prestressed trusses

Numerical modelling of the prestressed truss was carried out using the general purpose FE package ABAQUS \([31]\). Full details are given in D4.2.4 \([63]\). The experimental results of the four vertically loaded prestressed trusses were used to calibrate the FE models.

Development of the FE model

An initial study based on a simplified truss model without considering the construction joints of the bottom chord over-predicted the stiffness of the structure. The following issues were investigated in order to achieve better predictions: i) the FE element type, ii) the simulation of the connections of the bottom chord and iii) the effective stiffness of the cable. ABAQUS offers a wide range of element types, suitable for solving many different problems. The two following truss models, with different FE element types for the truss members and the cable, were investigated:

- A model with beam elements (B31) for the truss members and truss elements (T3D2) for the cable (this truss model is symbolised as B hereafter).
- A model with shell elements (S4R) for the truss members and solid elements (C3D8R) for the cable (this truss model is symbolised as S hereafter).
Model B is simpler and requires less computational time than model S. Model S, though, is able to accurately capture the localised deformations at the joints, thus accounting for the real stiffness of the truss joints and partially rectifying the overly stiff response of the models employing beam elements.

The material properties used in the FE models were derived from the material coupon tests. For the truss members, the stress-strain response of the S460 50×50×5 tensile flat coupon tests was adopted for the truss members. The cable was found to have a 0.2% proof strength of 1700 MPa on average. As a significant variation in elastic properties had been reported from the tests of cables of varying lengths, it was considered necessary to investigate the effect of the cable’s elastic material properties on the structural behaviour of the truss within the FE investigation. A modified 2-stage Ramberg-Osgood model was used for the description of the cable’s material response. Cables with Young’s Modulus of 90 GPa (Ec90), 130 GPa (Ec130) and 215 GPa (Ec215) were studied. ABAQUS requires the material properties input to be in the form of points on a true stress-logarithmic plastic strain curve. For both the cable and the truss members, the multi-linear approximated stress-strain curve was therefore transformed to true stress-logarithmic plastic strain.

Together with the development of the numerical truss model, the structural response of the construction joint of the bottom chord was also simulated, thus allowing the evaluation of its effect on the overall stiffness. Solid elements (C3D8R for the plate elements and the stiffeners and C3D10 for the bolts) were used for the development of the connection model, as shown in Figure 6.32. Given that the initial stiffness of the connection was of interest, only the elastic material properties of the steel components were incorporated. The joint was loaded under tensile static loading and the initial stiffness was found by plotting the incremental load (N) against the gap opening in the middle of the plates (mm). The axial stiffness was found to have an average value equal to 590,000 N/mm. In order to quantify the contribution of the joints’ stiffness to the stiffness of the bottom chord, an initial simplified study considering the bottom chord as a straight member with two springs of stiffness \( k = 590,000 \text{ N/mm} \) was conducted and found that the stiffness was over-predicted if the connections of the bottom chord were neglected. To this end, models where the connections were simulated with springs with an axial stiffness 590,000 N/mm (spr590) and models where the connections were neglected (sprNO), were investigated, thus allowing the evaluation of the effect of the joints’ stiffness on the overall stiffness.

![Figure 6.32 FE model of the connection of the bottom chord (left) and the deformed shape of the connection - gap opening (right)](image)

Real structural members are never perfectly straight and this deviation from the ideal geometry needs to be accounted for as it can significantly affect the ultimate response when failure is due to buckling. For prestressed trusses, both out-of-plane and in-plane initial geometric imperfections have to be considered. Based on similar studies\(^{[64]}\), the imperfections were incorporated into the numerical models through an initial linear eigenvalue buckling analysis, from which the relevant mode shapes corresponding to the observed failure modes were extracted and introduced as initial geometric imperfections in the subsequent nonlinear static analysis. The mode shapes of Figure 6.33 were used for the introduction of the initial geometric imperfections. In the designation used hereafter, the first number following the word “imp” indicates the magnitude of the out-of-plane geometric imperfections, whilst the second number is the magnitude of the in-plane geometric imperfections, with \( L \) being the relevant buckling length. In addition to the initial geometric imperfections, residual stresses can affect the structural response and should be considered for an accurate numerical investigation of the structural behaviour. Given that the hot-finished HSS specimens studied in HILONG were found to have small values of residual stresses, it was decided that residual stresses do not need to be included in the FE models.
For the evaluation of the structural response of HSS prestressed trusses, a nonlinear static analysis, was performed using the modified RIKS procedure (a variation of the classic arc length method) \cite{65}.

**Validation of the FE model**

Each of the four numerical truss models (T1 (no cable), T2 (truss with a cable at $P_{\text{nom}}$ prestress level), T3 (truss with a cable at 0.50 $P_{\text{opt}}$ prestress level) and T4 (truss with cable at $P_{\text{opt}}$ prestress level)) were validated against the respective experimental results. The results were plotted in load-displacement curves, thus allowing the direct comparison between experimental and numerical data. The effects of the FE element type and the flexibility of the bottom chord’s connections on the overall performance were evaluated for all trusses. For T2, T3 and T4, the effect of cables with different Young’s Modulus on the overall structural response was also investigated. The load-displacement curves at the midpoint (L3) of all trusses are depicted in Figure 6.34.

As can be observed for Truss 1, the numerical model with beam elements for truss members (model B) over-predicts the stiffness of the system, whereas the numerical model with shell elements (model S) leads to a more flexible response, closer to the experimental observations. With the incorporation of the bottom chord joints as springs of axial stiffness 590,000 N/mm in model S, the initial stiffness of the system is reduced further, matching precisely the experimental curve. The same conclusion with regard to the over-prediction of the overall stiffness of the structure by model B can be drawn also for Truss 2. Moreover, for Truss 2, the FE results achieve better agreement with the experimental ones - both in terms of ultimate load and displacement at failure load - for the cable with the lowest Young’s Modulus. The applied prestress in Truss 3 and Truss 4, leads to a stiffer overall response of the structure compared to the response of Truss 2, whereas the effect of the cable’s Young’s Modulus on the overall structural behaviour is more significant in Truss 2 than in Truss 3 and 4. It has therefore been concluded that a model with shell elements for the truss members, with an axial spring in place of the connections of the bottom chord and a cable with Young’s Modulus of 90 GPa leads to very good overall agreement with the test results. This model was therefore used to investigate the appropriate amplitudes of the initial out-of-plane and in-plane geometric imperfections that give the closest agreement with the experimental results.
Upon establishing the numerical model that is capable of replicating accurately the truss performance, ten different combinations of out-of-plane and in-plane initial geometric imperfections...
were investigated. The ultimate load \( F_u \) and the initial stiffness \( k_{\text{initial}} \) were very well replicated in most of the cases, whilst, as anticipated, the displacement at the maximum load appeared to be more sensitive to the initial geometric imperfections. It should also be noted that for in-plane initial geometric imperfections higher than out-of-plane imperfections, less conservative results were obtained. It was concluded that the model with out-of-plane imperfection \( L/750 \) and in-plane \( L/1500 \), gave very good agreement with the test results \( F_u(\text{FE})/F_u(\text{exp}) \): mean 0.98 with COV 4.2\%, \( k_{\text{initial}}(\text{FE})/k_{\text{initial}}(\text{exp}) \): mean 1.02 with COV 4.3\%). These imperfection amplitudes were therefore introduced as initial geometric imperfections in the subsequent parametric studies.

The failure modes were also successfully replicated in all cases, as shown in Figure 6.35. The following differences in behaviour were apparent:

- Truss 2 has increased initial stiffness, increased ultimate load and significantly increased displacement compared with Truss 1,
- Truss 3 has similar initial stiffness, slightly higher load and lower displacement compared with Truss 2,
- Truss 4 achieves the maximum load at the lowest displacement compared with the other trusses.

![Figure 6.35 Truss 2: In plane buckling (top), Truss 3&4: Out-of-plane buckling (bottom)](image)

**Parametric studies**

Having successfully validated the FE truss models, an extensive parametric study was conducted in order to investigate the effect of key parameters on the structural response. The parameters included the applied prestress level, the span-to-depth ratio, the steel strength of the truss members, the truss shape and the section sizes for the top and bottom chord \( (A_{\text{top}}/A_{\text{bottom}}) \). The increase in resistance and the reduction of the midpoint displacement at failure load that arise from prestressing were also studied for a wide range of structural configurations likely to occur in practice.

A simplified model comprising beam and truss elements was used for the execution of the subsequent parametric studies, due to the very high computational cost associated with modeling the entire truss using shell elements. To quantify the effect of the various parameters, the increase in resistance was expressed in terms of the ultimate load of the un-prestressed truss, whereas the reduction of the midpoint displacement at failure load was normalised with the relevant displacement of a truss with a cable prestressed to \( P_{\text{nom}} \).

It was shown that increasing the level of prestress leads to a higher ultimate load at smaller strains. Prestress led to enhanced performance for all the steel strengths studied. The influence of the truss configuration (shape, curvature) was also examined, leading again to the conclusion that prestress can give an improved ultimate performance and contribute to overcoming potential deflection limit
problems. The effect of the $A_{\text{top}}/A_{\text{bottom}}$ ratio was investigated, which showed that care is needed in the design: if the top chord fails by premature buckling, the benefits of prestress cannot be realised as the truss fails at small strains. When the anticipated failure mode is yielding of the tensile bottom chord, the truss exhibits a more ductile structural response with increased resistance for increasing levels of prestress.

Hence, it can be concluded that the application of prestress can increase the elastic load-carrying resistance of a structure, without substantially increasing its structural weight. It can therefore be utilised to overcome potential deflection limit issues in HSS structures, in which it is likely that SLS implications are more pronounced than in conventional strength steel structures. This means that the design criteria can be met using smaller span-to-depth ratios, hence reducing the length and cross-section dimensions of the truss diagonals and resulting in lighter and more elegant structures, thus pushing further the limits of modern structural design practice.

**Vibration considerations**

The dynamic response of prestressed trusses was numerically investigated using the FE software ABAQUS. Based on both the experimental and the numerical studies, it was concluded that the introduction of cables in the bottom chord of the trusses does not cause any significant change in the dynamic response (natural frequencies and mode shapes) of the trusses regardless of the applied prestress level. Hence the dynamic response of the prestressed arched trusses does not differ from the response of conventional steel trusses and therefore, no further research on vibration control needs to be carried out. Nevertheless, if deemed necessary, various technologies to mitigate vibration in steel structures are currently available\cite{66,67} and could be adopted.

### 6.3.5 Design guidance for prestressed trusses

**Design procedure**

Based on the tests and numerical modelling described in the preceding sections, a design process for long span trusses of tubular members was developed. Full details are given in D4.3b\cite{68}. The design process applies to trusses in one of the three following cases: i) without any cable inserted in the bottom chord, ii) with a cable in the bottom chord at nominal prestress level $P_{\text{nom}}$ and iii) with a cable in the bottom chord at optimum prestress level $P_{\text{opt}}$.

As a first step, the designer selects the geometry (i.e. span, depth, configuration of diagonals, number of girders and curvature of the bottom and top chord) of the truss. Like in conventional truss design, the final choice is a compromise between aesthetics, economy, weight, fabrication cost and so on, and is usually the result of a number of iterations.

The second step involves the determination of the relevant actions for both the Ultimate Limit State (ULS) and Serviceability Limit State (SLS). Various loading combinations should be established and compared, from which the most critical loading cases, in both the gravity and uplift directions, are able to be identified. It should be noted that for some cases, where all the load combinations result in net downward forces, the uplift ULS and SLS can be ignored and only the gravity ULS and SLS need to be considered in the truss design. For conceptual design, it may be assumed that only point loads at the joints of the top chord act on the structure. If a concentrated load acts transversely on a top chord member, that member needs to be designed as a beam column rather than a truss member in pure compression. Herein, it will be assumed that loads are acting only at joint locations and hence only truss action will be considered.

The next step is to determine the cross-section sizes of the truss members according to the identified ULS. The procedure starts with the design of the bottom chord, which depends on whether a cable is to be inserted in the bottom chord. With the assumption that the truss behaves macroscopically like a simply supported beam, subjected to uniform gravity or uplift loading $q_{\text{Ed}}$, the design bending moment at midspan at ULS can be found by the formula $q_{\text{Ed}}L^2/8$, where $L$ is the span of the truss. Hence, the design tensile (or compressive) load in the bottom chord, $N_{\text{Ed,bottom},t}$ (or $N_{\text{Ed,bottom},c}$), equals the maximum gravity (or uplift) bending moment, $M_{\text{Ed,gravity}}$ (or $M_{\text{Ed,uplift}}$), divided by the lever arm (i.e the depth of the truss at mid-span).

When there is no cable inserted in the bottom chord, the cross-section of the bottom chord ($A_{\text{bottom}}$) in the considered steel strength ($f_{\text{y,t}}$) can be evaluated from Equations (8-2) and (8-3), being checked against the tensile and compressive design loads, respectively.
When there is a cable prestressed to $P_{\text{nom}}$ in the bottom chord, the cross-sectional area of the cable ($A_{c}$) and the tube ($A_{t,\text{bottom}}$) can be evaluated, by neglecting the contribution of $P_{\text{nom}}$ prestress force and assuming that the bottom chord will fail when the tube yields, as shown in Equation (8-4). In compression, Equation (8-5) should be satisfied.

$$Ed,bo \leq pl,Rd, = (bo \times yt)/ 0 \quad (8-2)$$

$$Ed,bo \leq Rd,bo$$

$$Ed,bo \leq Rd,bo \quad (8-3)$$

where $N_{pl,Rd,\text{bottom}}$ is the design plastic resistance, with $\gamma_{M0}$ being the partial factor for resistance of cross-sections with a recommended value of 1.0 and $N_{b,Rd,\text{bottom}}$ the design buckling resistance of the bottom chord member, calculated in accordance with EN 1993-1-1.

When the inserted cable is preloaded to $P_{\text{opt}}$ in the bottom chord, the cross-sectional area of the tube and the cable can be determined according to Equation (8-6), whereas the $P_{\text{opt}}$ force can be evaluated from Equation (8-7). The design compressive load, $N_{Ed,\text{bottom},c}$ should not exceed the buckling resistance of the bottom chord, as stated in Equation (8-8), where $N_{b,Rd,\text{bottom},p}$ is the design buckling resistance of the bottom chord taking due account of the prestress force.

$$Ed,bo \leq pl,Rd,bo = bo (yt + cy)/ 0 \quad (8-4)$$

$$Ed,bo \leq Rd,bo \quad (8-5)$$

where $f_{yt}$ is the yield stress of the tube, and $N_{b,Rd,\text{bottom}}$ is defined as in Equation (8-3), being the tube buckling strength based on EN 1993-1-1, without taking into consideration the presence of the cable due to the negligible magnitude of the prestress level.

When the inserted cable is preloaded to the optimal prestress level (i.e. applying the prestress force $P_{\text{opt}}$), the cross-sectional area of the tube and the cable can be determined according to Equation (8-6), whereas the $P_{\text{opt}}$ force can be evaluated from Equation (8-7). The design compressive load, $N_{Ed,\text{bottom},c}$ should not exceed the buckling resistance of the bottom chord, as stated in Equation (8-8), where $N_{b,Rd,\text{bottom},p}$ is the design buckling resistance of the bottom chord taking due account of the prestress force.

$$Ed,bo \leq pl,Rd,bo = (bo \times yt + cy)/ 0 \quad (8-6)$$

$$Ed,bo \leq Rd,bo \quad (8-7)$$

$$Ed,bo \leq Rd,bo \quad (8-8)$$

$N_{b,Rd,\text{bottom},p}$ may be determined using the modified Perry-Robertson approach proposed by Gosaye et al. (69) and Wang et al. (70), working in conjunction with the codified column buckling curves in EN 1993-1-1. The method uses the same framework as Eurocode 3 for column design, but accounts for the influence of the prestressing cables and grout.

The top chord is designed next. The top chord should be able to resist the maximum tensile and compressive design loads ($N_{Ed,\text{top},t}$ and $N_{Ed,\text{top},c}$), which correspond to the uplift and gravity ULS, respectively. The tensile and compressive design criteria are shown in Equations (8-9) and (8-10), respectively. Note that under the gravity ULS, in order to have a ductile failure mode, buckling of the compressive top chord (both in-plane and out-of-plane) should occur after the yielding of the tensile bottom chord.

$$Ed, \leq Rd, = ( \times yt)/ 0 \quad (8-9)$$

$$Ed, \leq Rd, \quad (8-10)$$

where $A_{t,\text{top}}$ is the cross-sectional area of the top chord member, and $N_{b,Rd,\text{top}}$ is the design buckling resistance of the top chord, calculated according to EN 1993-1-1.

The design force in the diagonals ($N_{Ed,\text{diagonals},t}$ and $N_{Ed,\text{diagonals},c}$) can be derived from a simple static analysis. Using Equations (8-11) and (8-12), the cross-sectional area of the diagonals $A_{\text{diagonals}}$ can be evaluated.

$$Ed, ia, nais, \leq Rd, ia, nais = ( ia \times yt)/ 0 \quad (8-11)$$

$$Ed, ia, nais, \leq Rd, ia, nais \quad (8-12)$$

where $N_{b,Rd,\text{diagonals}}$ is the buckling resistance of the diagonal members under compression, as defined in EN 1993-1-1.
Finally, a deflection check under service loads should also be performed. A summary of checks is shown in Figure 6.36.

Based on the procedure described earlier, worked examples on Warren trusses were developed. Three different spans (20 m, 50 m and 80 m) and three prestress cases (no cable, \( P_{nom} \), \( P_{opt} \)) were examined. Steel strengths S235, S275, S355 and S460 were considered for the truss members. The trusses were designed for permanent and variable actions, and were based on a design example for truss structures published by SCI\[^{[71]}\], D4.3b\[^{[68]}\] gives the worked examples.

In order to check the validity of the design procedure, FE models were created in ABAQUS for the worked examples. The models were similar to the simplified models which was used for the parametric studies except that elastic-perfectly plastic material behaviour was assumed. Nonlinear static analysis with initial geometric imperfections was performed for each of the studied trusses. The results were used to check that the ultimate load of the structure was higher than the predicted design load. Figure 6.37 shows that, for the 80 m span truss, the design load is higher or equal to the ultimate load in all cases. As anticipated, the truss without any cable inserted in the bottom chord and with the largest bottom chord, appears to be the stiffest (largest inclination of the linear part of the curve), whereas the truss with \( P_{opt} \) prestress level is also able to achieve the design load, even though it is lighter (smaller cross-section for the bottom chord). It is worth noting that the failure load of the same truss without any cable inserted in the bottom chord, which is also indicated in Figure 6.37 as “no cable (2)”, is approximately the half of the case when there is a cable prestressed to \( P_{opt} \). The same truss at the \( P_{nom} \) prestress level, indicated as \( P_{nom} (2) \), has also been added in Figure 6.37 for comparison purposes.

**Figure 6.36 Summary of checks for a long span truss (with or without prestress)**
Figure 6.37 Load - displacement curves of 80 m span truss

Figure 6.37 shows the benefits of the use of prestress. By comparing the load displacement response of the no cable (2) truss with that of the $P_{\text{nom}}$(2) truss, it is clear that the insertion of a nominally prestressed cable in the bottom chord of a truss increases significantly its ultimate resistance, albeit attained at very large displacements. By introducing $P_{\text{opt}}$ prestress to the cable, the benefit in the reduction of the displacements at failure load can be seen by comparing the $P_{\text{nom}}$(2) with the $P_{\text{opt}}$ truss. It is worth noting that for the trusses studied, the introduction of $P_{\text{opt}}$ prestress reduces the midspan deflection at ultimate load by about 50% on average, compared with when the cable is only nominally prestressed. In addition, 10-15% material savings arise from the introduction of an optimally prestressed cable compared to a conventionally designed truss without any cable in the bottom chord.

As anticipated, the selection of truss members in higher strength steel leads to a reduced total truss weight, as indicated for the 80 m span truss in Figure 6.38. When the strength changes from S235 to S460 there is a significant reduction in weight of about 35%. This means that important material savings can be made from the use of HSS. However, the reduction in cross-section size means that stiffness related issues such as excessive deflections are more pronounced and hence the benefits of the increased material strength may not be fully exploited if SLS rather than ULS govern the design. This issue can be addressed using the prestressing technology. In particular, the combination of HSS with the prestressing system, which leads to significantly reduced displacements, may lead to a smart, efficient and economic structural system.

![Figure 6.37 Load - displacement curves of 80 m span truss](image_url)
7 WP5 EXPLOITATION OF RESEARCH RESULTS

7.1 Objectives of WP5

The objectives of this WP were to present the results of the project in a form that can be used by a wider audience. The key activities were:

- Preparation of design examples which illustrated the benefits of using HSS in certain applications.
- Contributions to future revisions to Eurocode 3.
- A seminar, with presentations by researchers and practitioners.
- Preparation of journal papers.

7.2 Task 5.1 Design examples

A collection of eight design examples was prepared for D5.1\cite{72}. It is expected that they will be valuable educational tools showing the step-by-step design procedure for HSS members (beams and columns) and joints. The examples comprise:

- Example 1: Square hollow tension member (S460) welded to a connection plate (S355).
- Example 2: As Example 1, except the tension member is S690 steel.
- Example 3: Internal I shape column in a multi-storey building from S460 where the beam spans and loading about each axis are identical; there are no out-of-balance loads and so the column is subject to axial load only
- Example 4: Top chord member in tubular truss, subject to a high compression force and external loads
- Example 5: Rectangular hollow section (S460) in compression demonstrating the Continuous Strength Method (CSM)
- Example 6: Rectangular hollow section (S460) in bending demonstrating the CSM
- Example 7: Square hollow section (S460) under combined loading demonstrating the CSM
- Example 8: Crocodile Nose connection

Whilst the first four examples do not involve the technologies specifically studied in HILONG, they nevertheless demonstrate the weight saving of using HSS. It should be noted that the limited range of sections available in HSS compared to conventional strength steels can limit the weight savings achievable. Also, for tension members, the design of the connection will often be governing, which can again limit the achievable weight savings by using a higher strength member.

Design examples were also developed to demonstrate the new design methods developed for HSS prestressed cable stayed columns and prestressed arches; these are presented in D4.3a\cite{54} and D4.3b\cite{68} respectively.

7.3 Task 5.2 Contributions to European Standardisation

The work completed in HILONG has been discussed at meetings of the Working Group (WG) for EN 1993-1-12 over the last two years. The main contributions are summarised in D5.2\cite{73}.

The HILONG research has made a useful contribution to the understanding of HSS members, connections and structural systems. In order to carry out Eurocode reliability analyses, material strength data were collected from steel producers in Europe. Strength data were also extracted from numerous mill certificates and research papers. This enabled a mean ratio of measured to minimum specified strength, accompanied by standard deviations, to be derived for each strength grade of steel.

Under WP2, EN 1993-1-1 and EN 1993-1-12 design rules were confirmed. Concerns were raised over the level of reliability of the effective width formulation for Class 4 cross-sections. These results were presented to the WG for EN 1993-1-12 and the relevant deliverables will be forwarded to the WG once the final report is approved.

The design rules for tubular joints in EN 1993-1-8 are based on experimental and numerical investigations using steel S235 and S355. The work carried out in WP3 has made a significant contribution to the gap in knowledge of the behaviour of tubular joints in higher strength members. Under WP3, the application of these design rules for tubular joints made from HSS was confirmed.
WP4 studied structural systems and developed design methods for PSSCs and prestressed trusses, aligned where appropriate to the provisions in Eurocode 3.

7.4 Task 5.3 Public Seminar

7.4.1 Preparation for the seminar

A seminar was held on 30 June 2015 to disseminate the findings of the HILONG Project. A full account of the event is given in D5.3[74]. The venue was Imperial College London. This was an ideal location because it is located in central London with good public transport links. Also, holding the seminar at Imperial College meant that a number of post-graduate students from Imperial College attended, in addition to construction practitioners. The workshop was publicised extensively by:

- Emailing contacts on SCI’s and other HILONG partners’ contact databases.
- Personal invitations.
- Advertisements in The Structural Engineer and New Steel Construction journals (see Figure 7.1).
- LinkedIn.

In order to attract delegates, Continuing Professional Development Certificates were available for those requiring proof of attendance.

In total, 124 people registered, of which 20 were speakers or project partners. The delegates came from a wide range of organisations:

- Designers (this was the majority of delegates, e.g. Arup, WSP, Aecom, Flint & Neill, Waterman).
- Steelwork manufacturers (e.g. Westok, Doshi).
- Steel producers (Tata Steel, ArcelorMittal).
- Contractors (e.g. Laing O’Rourke).
- Local authorities (e.g. Southend-on-Sea).
- Steelwork fabricators/contractors (e.g. Severfield, Billington Structures).
- Universities (Derby, Brunel, RWTH in addition to the partner universities).
- Ministry of Defence.
- Norwegian Steel Association.
- British Constructional Steelwork Association.
7.4.2 Content of the seminar

The contents of the seminar were designed so that the information presented was varied, giving a ‘complete’ picture of the use of HSS in structures, and not just the topics studied in HILONG. Presentations included guidance on specification, design and fabrication as well as a comprehensive presentation of the results of the research carried out in HILONG. Case study information about HSS structures was also included and Tata Steel, Vallourec and ArcelorMittal presented details of their HSS product range. Each talk was limited to 20 minutes to keep the audience attentive. The full programme is given in Figure 7.2.
Tuesday 30 June 2015  
Room 201, Skempton Building,  
Imperial College London  
Registration from 09.30

<table>
<thead>
<tr>
<th>Time</th>
<th>Session</th>
<th>Speaker/Institution</th>
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<tbody>
<tr>
<td>10.00</td>
<td>Welcome</td>
<td>Nancy Baddoo (SCI)</td>
</tr>
<tr>
<td>10.10</td>
<td>Recent experiences with long span structures</td>
<td>Fergus McCormick (Buro Happold)</td>
</tr>
<tr>
<td>10.30</td>
<td>Friends Arena and other recent projects using HSSs (HSS)</td>
<td>Lars Cederfeldt (Sweco Structures AB, Sweden)</td>
</tr>
<tr>
<td>10.50</td>
<td>Properties and specification of HSS</td>
<td>Nancy Baddoo (SCI)</td>
</tr>
<tr>
<td>11.10</td>
<td>Vallourec hot formed hollow sections in HSS</td>
<td>Christian Remde (Vallourec, Germany)</td>
</tr>
<tr>
<td>11.20</td>
<td>Coffee Break</td>
<td></td>
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<tr>
<td>11.45</td>
<td>Designing HSS structures: Eurocode rules and practical guidance</td>
<td>Prof. Leroy Gardner (Imperial College London)</td>
</tr>
<tr>
<td>12.05</td>
<td>Fabrication of HSS</td>
<td>Dan Hudson (Severfield UK Ltd)</td>
</tr>
<tr>
<td>12.25</td>
<td>Design methods for HSS cross-sections</td>
<td>Dr. Sheida Afshan (Imperial College London)</td>
</tr>
<tr>
<td>12.40</td>
<td>ArcelorMittal HSS open sections – product range and recent projects</td>
<td>Marc May (ArcelorMittal Europe, Luxembourg)</td>
</tr>
<tr>
<td>12.55</td>
<td>Experiences of HSS structures in Scandinavia</td>
<td>Prof. Milan Veljkovic (Luleå Univ. of Technology, Sweden)</td>
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<tr>
<td>13.10</td>
<td>Lunch</td>
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<td>14.10</td>
<td>Utilisation of HSS in tubular trusses</td>
<td>Dr. Kristo Mela (Tampere Univ. of Technology)</td>
</tr>
<tr>
<td>14.25</td>
<td>Innovative structural systems for HSS: Prestressed stayed columns</td>
<td>Prof. Luis Simões da Silva (Univ. of Coimbra, Portugal)</td>
</tr>
<tr>
<td>14.45</td>
<td>Innovative structural systems for HSS: Prestressed trusses</td>
<td>Dr. Marios Theofanous (University of Birmingham)</td>
</tr>
<tr>
<td>15.05</td>
<td>Tata Steel HSS tubes</td>
<td>Dave Chapman (Tata Steel Tubes)</td>
</tr>
<tr>
<td>15.15</td>
<td>Innovative cross-sections and connections in HSS</td>
<td>Prof. Milan Veljkovic (Luleå Univ. of Technology, Sweden)</td>
</tr>
<tr>
<td>15.35</td>
<td>Discussion, access to further resources and closing remarks</td>
<td>Nancy Baddoo (SCI)</td>
</tr>
</tbody>
</table>

Figure 7.2  Workshop programme

### 7.4.3 Feedback from delegates and follow-up activities

The seminar was well received by students and practitioners alike. There was a lively question time after many of the presentations. Examples of the feedback received by email were:

I attended the HILONG seminar at Imperial College last week (which I thought was excellent).

Yesterday myself and two others from Reids attended the SCI high strength steels in long span structures talk. The talk was excellent with a lot of very interesting speakers. So thanks to yourself and everyone else involved in organising it!


### 7.5 Publications arising from work carried out in HILONG

The following papers have been prepared in relation to HILONG:
Journal Papers


Gkantou, M., Wang, J., Theofanous, M., Gardner, L. and Baniotopoulos, C. Structural behaviour of high strength steel hollow sections under compression and uniaxial bending. Proceedings of the Institution of Civil Engineers (abstract accepted)

Gkantou, M., Theofanous, M. Antoniou, A. and Baniotopoulos, C. Numerical investigation of High Strength Steel (HSS) square and rectangular hollow section stub columns. Proceedings of the Institution of Civil Engineers (abstract accepted)

Conference Papers


Antoniou, N., Theofanous, M., Baniotopoulos, C., Gardner, L., Numerical investigation of high strength steel tubular members, CESARE’14 International Conference Civil Engineering for Sustainability and Resilience, 2014


MSc Dissertations


MEng research project 2013/14, Elias Baramilis, Numerical modelling of high strength steel stub columns

MSc Civil Engineering 2013/14, Nicola Wall, Design of a High Strength Steel (HSS) Industrial Building

MSc Civil Engineering and Management 2013/14, Sasidharan Manu Plavinkoottathil, Structural Design of an Arena by using High Strength Steel
8 EXPLOITATION AND IMPACT OF RESEARCH

Despite steels of higher strength than S355 being available for many years, the market share is still surprisingly low in the main construction segments. One of the main obstacles to the use of HSS is that many structural elements are governed by stiffness considerations rather than strength, hence buckling or deflection criteria prevent the benefits of using HSS from being fully exploited. Additionally, the design rules for HSS are unnecessarily conservative and based on few test data, thus preventing efficient design in many circumstances. Further obstacles to the wider use of HSS include the higher price/tonne, inexperience in fabrication, limited sources of supply, fewer section sizes available, and a general reluctance to change within the notoriously conservative construction industry.

Most designers know little about higher strength steels: there is a widespread lack of knowledge about the properties of modern higher strength steels and about recent projects in which they have been used. Replicating a design in S355 steel will not always lead to an optimum design in S460, and rarely in higher strength steels. For the higher strength steels, the use of ‘traditional’ structural sections like hollow rectangular and square sections, and open I sections will not allow the higher strength to be exploited since local buckling will govern design. It is not always easy or obvious how to design structures using higher strength steels in such a way as to achieve maximum cost savings. It not only involves designing to optimise the amount of strength which can be exploited, but also requires minimising fabrication costs, ensuring stability during erection, and being aware of differences in sections/plates availability, (slightly) different welding procedures, etc.

The HILONG project has sought to overcome some of the obstacles relating to design limitations due to buckling and deflections by studying innovative structural arrangements and cross-sections which suppress buckling and reduce deflection. Once construction practitioners are able to design more efficient HSS structures, demand will grow and the procurement obstacles are likely to reduce in response.

The work completed in WP2 adds to the body of data on HSS cross-sections and will contribute to the next revision of Eurocode 3 with respect to the rules on ductility requirements, section classification, effective width formulae and axial load and bending moment interaction. The database of material strength properties collected for the reliability analysis will also be useful in the future for reliability assessments for HSS design rules, and has already been shared with HILONG’s sister RFCS project RUOSTE.

The work completed in WP3 sets the foundation for promoting the use of innovative cross-sections in HSS rather than just continuing to use the standard rectangular and circular hollow and I-sections designers are familiar with. This is significant because polygonal cross-sections are not only less susceptible to local buckling but also enable joints to be made more easily. They also give freedom to optimise the cross-section (external dimension, thickness) and ability to fabricate the cross-section wherever a press-braking facility is available, with plates delivered in sizes required for optimal fabrication. The tests on the HSS trusses are valuable since there are very few data on the performance of any types of HSS joints. The work on the CN connection will be very useful for trusses made from conventional as well as high strength steel members.

WP4 studied the application of prestressing technologies to HSS columns and trusses, although the outcomes of the work are also valuable to any strength steel and will help to promote the use of prestressing in general. Prestressing can significantly increase the load-carrying resistance of a truss or column without increasing its structural weight. Prestressing a truss can help to overcome potential deflection limit issues which enables design criteria to be met using smaller span-to-depth ratios, hence reducing the length and cross-section dimensions of the truss diagonals and resulting in lighter and more elegant structures, thus pushing further the limits of modern structural design practice.

The results of the HILONG project have shown there is much potential for designing more efficient higher strength steel structures and the potential scope of application of the results of this project are far reaching. Further dissemination activities are required to enable designers to make use of the recommendations and techniques arising from the HILONG research.
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REFERENCES

8. The BRITe project, Post-Tensioned Steel Trusses for Long Span Roofs, Innovation Case Study No. 9, CRC Construction Innovation, Australia, 2006 (http://eprints.qut.edu.au/27130/1/27130.pdf)
11. HILONG D1.2.1: Serra, M, WP1: Design of two case studies in conventional steel, 2013
26. EN ISO 3834: Quality requirements for fusion welding of metallic materials, CEN
EN ISO 6892-1: Metallic materials - Tensile testing - Part 1: Method of test at room temperature, CEN, 2009

HILONG D2.2: Gkantou, M., Antoniou, N. and Baniotopoulos, C., WP2: Report on numerical modelling of HSS members (material, stub columns and beams), 2014


EN 1990 Eurocode: Basis of Structural design. CEN, 2002


HILONG D3.2.2 and D3.2.3: WP3: Manoleas, P. and Koltsakis, E., WP3: Joints in HSS tubular and innovative trusses: Tests and analysis, 2016


J. Wardenier, JA Packer, XL Zhao, and GJ Van der Vegte, Hollow sections in structural applications. CIDECT Zoetermeer, Netherlands, 2000.


EN ISO 6892-1: Metallic materials. Tensile testing. Method of test at ambient temperature, CEN, 2009

EN ISO 377: Steel and steel products. Location and preparation of samples and test pieces for mechanical testing, CEN, 2013

EN 10218-1: Steel wire and wire products. General. Test methods, CEN, 2012

EN 10002-1: Tensile testing of metallic materials. Method of test at ambient temperature, CEN, 2001


Smith, R.J., Ellis, J.S., McCaffrey, G.T., Buckling of a single-crossarm stayed column, Journal of the Structural Division, 101, 249-268, 1975

Hafez, H.H., Ellis, J.S., Temple, M.C., Pretensioning of single-cross arm stayed columns journal of the structural division, 105, 359-375, 1979


Brettle, M.E. and Brown, D.G. Steel Building Design: Worked examples for students, In accordance with Eurocodes and the UK National Annexes, Steel Construction Institute, UK, 2009

HILONG D5.1: Baddoo, N., WP5: HSS worked examples, March 2016

HILONG D5.2: Baddoo, N., WP5: Contributions to European standardisation, July 2015

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HILONG investigated innovative cross-sections and structural arrangements which enable the benefit of high strength to be maximised by suppressing buckling and reducing deflection. The study focussed on long span applications, such as trusses for stadia, exhibition halls etc. The strengths studied were between S460 and S690.

Tests and numerical studies on stub columns, beams and beam-columns confirmed current guidelines in Eurocode 3 and also led to the development of a less conservative deformation-based design method.

The performance of different types of joints in two HSS trusses was investigated. One truss comprised conventional tubular sections and the other a U-shaped bottom chord and semi-closed polygonal top chords; these innovative cross-sections facilitate easier fabrication of the joints compared to conventional tubular joints. Tests were also carried out on a crocodile nose connection, which is a novel type of connection suitable for joining tubular diagonal members to a gusset plate.

The performance of prestressed stayed columns and prestressed trusses was studied experimentally and numerically to assess the extent to which they enable a greater proportion of the higher strength to be utilised by suppressing buckling and limiting deflection. Vibration tests explored the dynamic response of these prestressed systems.

Two case studies (a long span roof truss and a prestressed cable stayed column) demonstrated the potential for savings in weight, cost and environmental impacts through the use of HSS and the technologies studied in HILONG. The studies clearly demonstrated the potential advantages of HSS structural systems, but careful design is required to maximise these advantages.