TIED-ARCH BRIDGES WITH JUMBO SHAPES AS ARCH MEMBER – STATE OF THE ART AND DEVELOPMENTS

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SUMMARY

The tied-arch bridge is a structural solution developed since over a century, but recently has experienced a considerable renaissance thanks to its aesthetic value as a landmark object with a relative cost efficiency compared to other landmark solutions of the same span category.

A recent trend in Europe has been the use of structural heavy shapes as arch member for the span range between 50 and 120 meters. Several projects are presented in this paper, with the common feature to be inspired by a pragmatic design with efficient and economic steelwork detailing. The inclined hangers form of a dense network, which comparing to the traditional vertical hanger arrangement allows amongst other for lower bending moments in the tie and the arch. The use of high-strength steel for the arch permits weight savings translating into project economies. The hangers are attached to the arch simply via gusset plates welded directly into the profile's chamber between flanges and web.

The arch members are spliced by means of full penetration groove welds, knowing that the challenge of welding such thick steel requires preparation and the availability of skilled workmanship. The choice of the steel grade must be appropriate in terms of toughness, keeping in consideration that the welded connection may result in stricter toughness requirements at the steel purchase. Recent welding tests have been performed to further qualify the butt-weld joints of Jumbo sections, aiming also at quantifying the residual stress patterns after welding procedure. Some studies concerning the geometrical shape of the weld bevel preparation and of the cope hole to optimize the fatigue resistance will be presented.

Keywords: Arch bridges, High-strength steel, Welding, Jumbo structural shape.

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Figure 1: Left: Road bridge WD-431 over Highway A1 (Pyrzowice – Piekary) – span 62 m [32];
Right: Rail bridge E-30 over Vistola River in Krakow – main span 116m [39]

1. INTRODUCTION

The shape of the arch is of essential importance for the historical development of bridge engineering. The introduction of wrought-iron in civil construction during the industrial revolution gave designers the possibility to conceive new structural forms beside the traditional ones realized with stone or masonry arches. In fact beside the deck constructed above the arch, it was now possible to realize the arch standing above the deck, with this second suspended to the first by means of tension elements. This permitted to create a new answer for the situations where the construction height below the deck was very limited.

The use of iron, a material with substantial resistance under tensile force, disclosed also a new possibility for the arch standing above the deck – the tied-arch bridge concept (also referred as bowstring bridge). The horizontal component at the arch impost, traditionally overtaken by neighbouring arches or directly equilibrated by support reactions in the fondations, could be attached in the horizontal deck and compensated with the opposite horizontal component caused at the other arch impost. This creates a so-called self-equilibrated structure for vertical loads (meaning that under vertical loads only vertical reactions are induced in the supports), a major feature for designer.

The first major project implementing consciously this concept goes back to 1849 with the Windsor Railway Bridge (Great Western Railway line), design by I.K. Brunel (Figure 2). After major refurbishment in 1991, the bridge is still in service today [45].

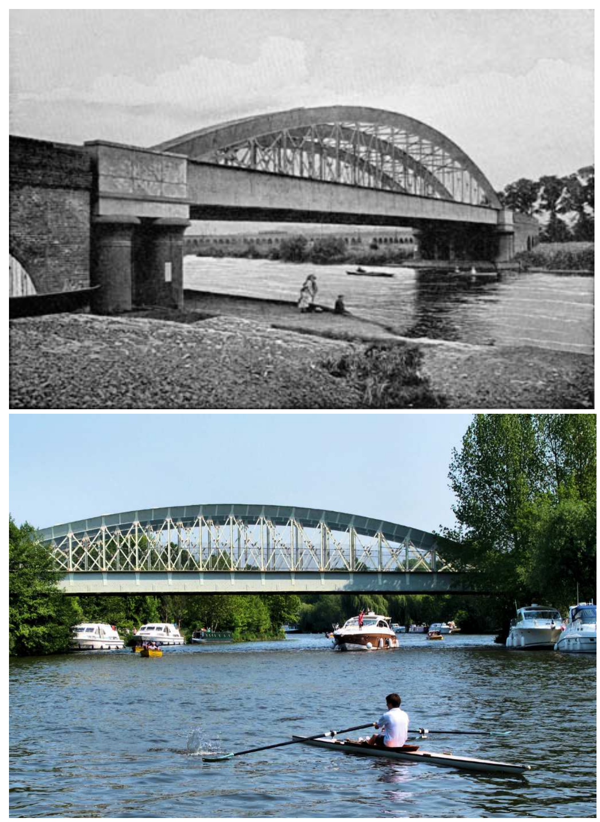


Figure 2: Windsor Railway Bridge at the time of construction (1849) and today (2019) in UK, span 62m [45].

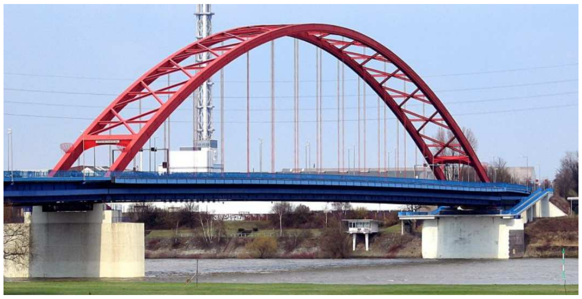


Figure 3:“Brücke der Solidarität” in Duisburg, Germany (1950), span 256m, with a Langer hanger arrangement [46].

The parameters are still interesting today: span is 62m, width is 10.7m, the rise of the arch is 1 / 7 of the span, 393 tons of wrought-iron (592 kg/m^2). The Windsor Rail bridge has a series of vertical hangers transferring the loads from the deck to the arch, but Brunel also had foreseen diagonal elements to improve the behavior and increase the stiffness of the system. Nevertheless in the following decades the typology, that widespread for the realization of tied-arch bridge around the world, was the one patented by Langer in 1859, with uniquely vertical hangers at a fair constant spacing.

Nielsen studied the use of inclined hangers (Figure 4) as alternative to vertical hangers and their capacity to reduce the bending moment in the arch and tie. Based on his patent of 1926, various bridges were realized starting in the ’30 with spans in the range 60…120m. The use of the Nielsen hanger arrangement remains nevertheless limited compared to the predominant Langer’s vertical arrangement.

Figure 4: The new bridge over Panaro in Bomporto and Ravarino (Modena), (2018), span 80m, with a Nielsen hanger arrangement [42].

Network arch bridges (Figure 5) were developed mainly by Tveit in the ’50s and ’60s [1] as a further development of the Nielsen system, whereas each hanger shall cross at least two others in its trajectory. The shape of the net has been deeply studied and several suggestions are in the literature; for instance by Tveit [9], [23] Teich [22] and Brunn und Schanack [21][10].

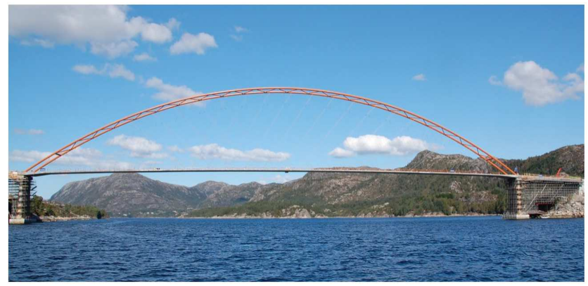


Figure 5: Brandanger network arch, Norway (2012), span 220m – defined as the world’s most slender arch bridge [44].

In this system the bending moment in the arch are minimal compared to traditional vertical systems. Thanks to the fact that the arch is working mainly under axial compression, Tveit identified the high-strength wide flange H beams as an economic solution. The first documented project goes back to Tveit and Teich for the Åkvik Sound bridge (2001) with a span of 135m (Figure 6).

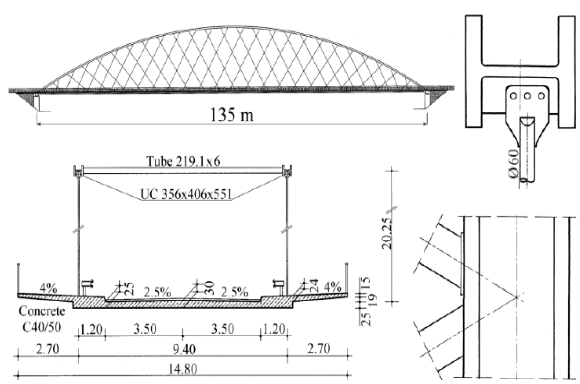


Figure 6: First documented project of Network Arch Bridge using rolled sections - Peer Tveit – Stephan Teich: Project of Åkvik Sound bridge (2001) [44].

It is worth mentioning that in France, road autorithies SETRA promoted the radial hanger arrangement as as an alternative to the vertical one (Figure 7), with the major advantage to simplify the hanger-arch connection amongst as they cross always at the same angle of 90° [17].

Figure 7: Pont de franchissement de l'Orb à Bedarieux – J. Berthellemy (2009), span 90m [34].

Berthellemy further developed this configuration for the low span range with the invention of a specific cruciform hanger connection with variable dimensions. In a nutshell, the main advantages are that it can be executed and assembled easily by the steel fabricators, it can take some compression and is fatigue-resilient and easily inspectable. This hanger was used for bridges up to 90m span and hangers up to 17m length without any problem of wind-induced resonance effects [34].

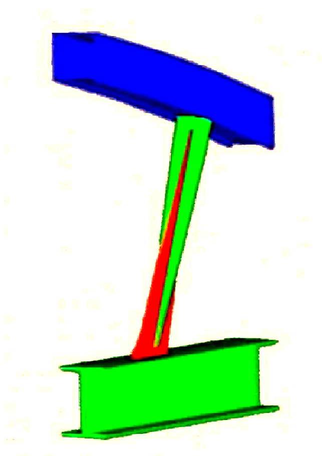


Figure 8: Concept of flat cruciform hangers with variable section by Berthellemy, SETRA [34].

2. SPECIFICITY OF JUMBO SHAPES AS ARCH MEMBER

2.1 Availability of rolled structural shapes

Rolled structural shapes (L, I, H, U) were developed at the end of the 19th century, answering the need to simplify shapes built up from plates assembled together by rivets. The advantages in terms of weight savings, fabrication simplification and cost reduction were integral to the acceptance of rolled shapes in every field. Today, the geometric range of available H structural shapes is extensive (beam height 80…1150mm, flange width 50…450mm, flange thickness 4…140mm) with a well-established presence of production sites around the world, making structural shapes a well-known standard products known by Engineers and Contractors [7].

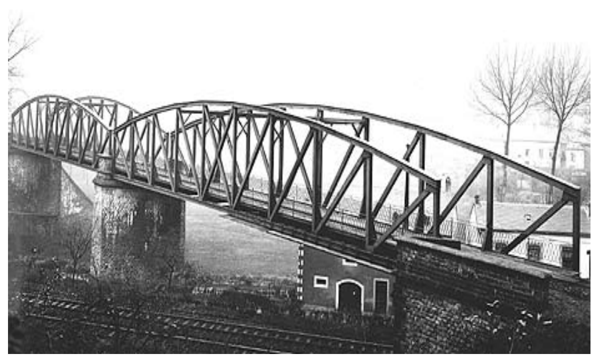


Figure 9: Charly's bridge in Dommeldange – Luxembourg (1902) – first bridge realized with parallel flange H beams.

In addition to the extension of geometric properties, the continuous development of optimized rolling procedures proposed more advanced steel grades over the decades. Since the '90s, thermo-mechanical rolling has become a standard for the most advanced plants. In order to enhance the benefits of thermomechanical rolling, the quenching and self-tempering process (QST) was developed specifically for structural shapes with thick flanges [12]. Implementing this innovative procedure, made it possible to economically obtain high steel strengths (up to 485MPa) for heavy sections without the costly addition of alloying elements [27]. Today the latest status is Grade 80 (550 MPa) [41], [28],[32].

Table 1: Steel grades available for Jumbo Structural shapes with thermo-mechanical steel [28].

	Yield point, ksi [MPa], min.	Tensile strength, ksi [MPa], min.	Elongation min 8 in. [200 mm], %
A913 Gr. 50	50 [345]	65 [450]	18
A913 Gr. 65	65 [450]	80 [550]	15
A913 Gr. 70	70 [485]	90 [620]	14
Grade 80	80 [550]	95 [655]	13

2.2 Curving of rolled structural shapes

The first step to obtain an arch member is to curve it to the desired bending radius. Whereas rollers are the most common tool to curve structural shapes, for Jumbos this procedure is not possible since the forces required would be excessive. The curving is therefore applied on a gag press (Figure 10), which can be explained as a series of three-point bending process slighlty above the yielding point, done with a close spacing so to achieve a continuous curvature along beam axis. Since the gag-press corresponds to imposing a cold-deformation to the structural steel, it was necessary to investigate its impact for a special applications such as arch members.

Figure 10: Curving of heavy structural sections.

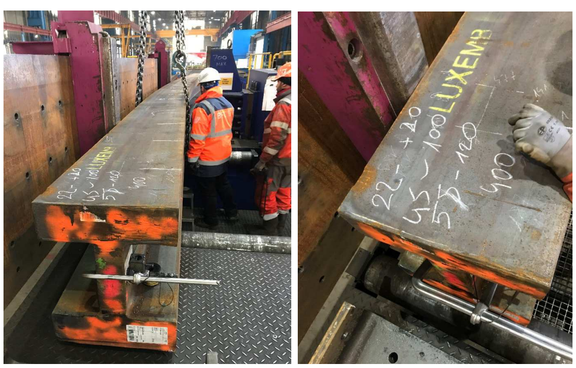


Figure 11: Curving beam test with the gag-press.

The curving procedure currently in use for composite bridge girders has been re-evaluated for this specific use with real scale tests (Figure 11) and numerical simulations (Figure 12, Figure 13). The criteria established were to:

- Ensure that maximum plastic strain stays below 2.0% to avoid any impact in design concerning the toughness and ductility [];
- Limit the plastic strain peaks at load application points to maximum 1.3 times the average of plastic strain in the most external fiber. This was a self-imposed criteria for quality control.

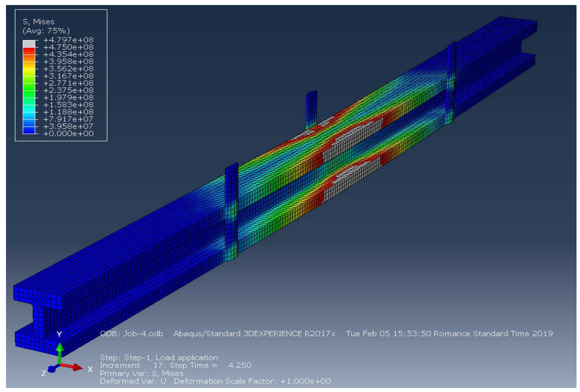


Figure 12: Numerical simulation of the curving test.

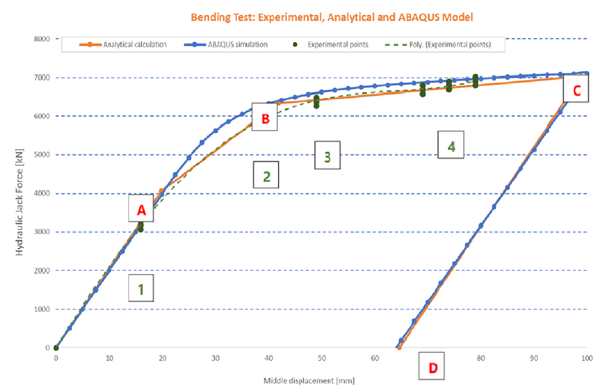


Figure 13: Comparison of results and simulations.

These requirements impose an homogeneous yielding of the extreme fiber all along the beam axis as well as limit the maximum local curvature. In order to achieve this, the procedure at the gag-press was adapted as it follows:

- The distance of the supports at the gag press shall be 10 times the dimensions of the beam, so to allow for a longer yielded zone in the middle and lower influence of shear force. In practice, this imposes higher overlengths which have to be scrapped after cutting to final length;

- The increment f the gag press shall be smaller than 1.25 x flange width / beam height, respectively for curving happening along the weak / strong axis. This allows for a significant overlapping of the plastic hinges and an homogeneous path along the extreme fiber;
- The final value of curvature shall be achieved in at least 3 steps, so to allow for a smooth quasi-static procedure.
- The final curving radius shall be greater than the minimum showed in Figure 14, to ensure to limit the maximum plastic strain.

The procedure described has been adopted for the fabrication shop for curving radius between 200m and 50m for all steelwork corresponding to Execution Class 3 / 4 according EN1090-2 [].

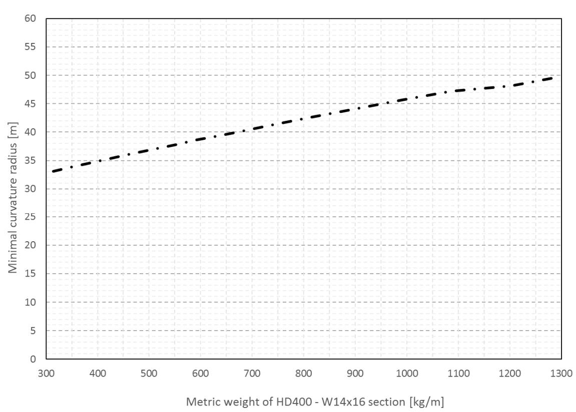


Figure 14: Minimal bending radius for HD400 – W14x16

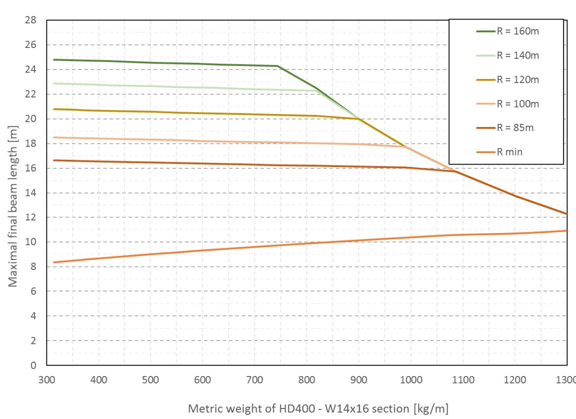


Figure 15: Maximal lengths for HD400 – W14x16

2.3 Weld-bevel preparation of rolled shapes

After curving of the beam, it is possible to obtain the weld bevel preparation as well as the cut to final length by means of a traditional oxycutting robot adequatly programmed (Figure 16). The complexity of the operation is linked with the fact that the beams are curved and therefore the weld bevel preparation occurs along an inclined plane.

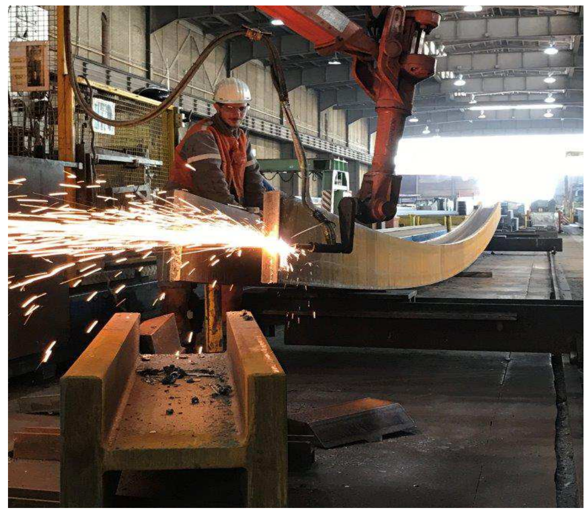


Figure 16: Fabrication of heavy structural sections into arch bridge members: chamfering.

Till the arch is working mainly in compression which could be transmitted by contact with perimetral welds designed to overtake the bending moments, in the current practice the arch member splices are designed as full penetration butt weld joints according [14].

Figure 17: Detail of the weld bevel preparation.

3.4 Cope hole- considerations for stress design

For Structural shapes with flange thickness over 50mm (1 inch), it is highly recommended to provide cope holes (Figure 17) in order to have a proper welding procedure as well as avoid excessive triaxial constraint in the flange due to transversal retreat of the web weldment. The important size of these cope holes is relevant for the design of compact section, as a significant part of the web is subtracted. The first step was then to establish the section capacity for different internal forces based on a standard AICS cope hole [8][31].

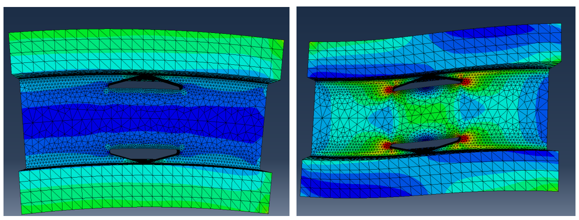


Figure 18: Example of ABAQUS simulations showing Von-Mises stresses done for different elementary load cases: Left: Bending strong axis, Right: Shear strong axis.

When it comes to the full section capacity, normal force and bending moment are linked. It is of high importance to take into consideration the net section reduction under pure normal force (which is close to -6%, Table 2) as tension / compression members could have rather constant action throughout their length. When it comes to the bending moment, as current practice it is important to place the splice away from the maximum bending moment.

Table 2: Simplified proposal of cross section reduction in the net-section of the cope hole for different actions

HD400 / W14x16 sections **Cope hole: max 50mm high, max 300m long**		
Normal force	A_{net} / A_{tot}	**0.938**
Bending strong axis	$W_{pl.y\ net} / W_{pl.y}$	0.947
Shear strong axis	$V_{Rd.y.net} / V_{Rd.z}$	**0.275 ***
Bending weak axis	$W_{pl.z\ net} / W_{pl.z}$	0.973
Shear weak axis	$V_{Rd.z.net} / V_{Rd.z}$	1.0

** This value is advised to avoid excessive yielding in the corner of the cope hole - it depends on the exact cope hole detail and loading combination so in case of important shear action it is strongly recommended to look at this aspect into details.*

When it comes to shear in the plane of the web, according to the performed simulations in the reduced section the web carries about 75.5% of the shear, whereas the rest is transferred via the thick flanges. Still, shear forces in this plane cause local effects around the opening (called also Vierendeel bending moments). These can easily lead to yielding of the cope hole ends –not a capacity problem but may become a fatigue or deformation problem.

2.4 Cope hole – considerations for fatigue design

Having a cope hole reduces significantly the detail category linked with the joint detail (Figure 20), since the geometrical discontinuity leads to a stress hot spot. Whereas guidance concerning the geometry is given in the AISC specifications [14], no requirement concerning the cope hole geometry is given in the Eurocode approach [15].

The reduction of the cross-section capacity at a plastic stage mainly depends on the outer dimensions of the cope hole. The stress peaks conversely are highly influenced by its shape: for this reason, it has therefore been decided to investigate the most adequate geometric shape compliant with fatigue design.

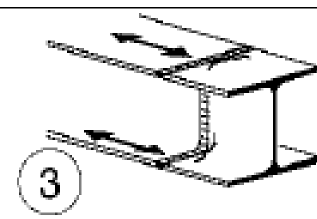
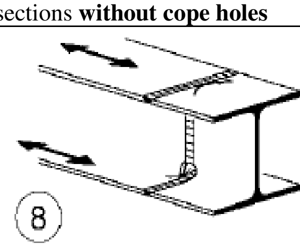
Finishing type	***A***	***B***
 Full pen butt welds of rolled sections **without cope holes**	**112**	**90**
 Full pen butt welds of rolled sections **with cope holes**	**90**	**80**
Size effect for t > 25mm	$k_s=(25/t)^{0,2}$	

Figure 19: Excerpt of Eurocode approach for detail classification of butt-weld joints with rolled sections [15].

Finishing type A	*Finishing type B*
- All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT.	- The height of the weld convexity to be not greater than 10% of the weld width, with smooth transition to the plate surface. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT.

Figure 20: Eurocode approach for detail classification of butt-weld joints with rolled sections – Finishing Type [15].

For the base material, the cope hole as long as it is rounded can be considered as a material discontinuity. Therefore it is advisable to calculate the stresses in the net cross-section and verify it as the detail category "Structural element with holes subject to bending and axial forces – 90" [15][11]. For the weld itself, the evaluation can be done according to the Eurocode approach [19], [15], and in particular the Annex B of [15] which indicates the reference value. It is quite clear to understand that the hot-spot stress factor assumed for the weld is 1.25, as it corresponds to the ration 112 / 90 and 100 / 80 amongst the categories of Figure 21 and the corresponding categories of Figure 19 with the cope hole (second row).

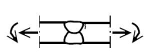
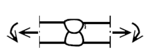
Detail category	Constructional detail	Description	Requirements
112	1 	1) Full penetration butt joint.	1) - All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides, checked by NDT. - For misalignment see NOTE 1.
100	2 	2) Full penetration butt joint.	2) - Weld not ground flush - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides. - For misalignment see NOTE 1.

Figure 21: Detail categories for use with geometric (hot spot) stress method – Extract of [15]

Four different shapes have been analyzed (Figure 22) inspired by the literature [11][31][20] [6]. The height of the cope hole is kept constant at 50mm since the practice has shown that is appropriate for a proper welding and control of the joint without becoming excessive in term of cross-section reduction. An initial length of the cope hole of 100…150mm has been assumed to allow for a decoupling of web and flange welding to avoid triaxial stress concentration due to retreat of the web weld.

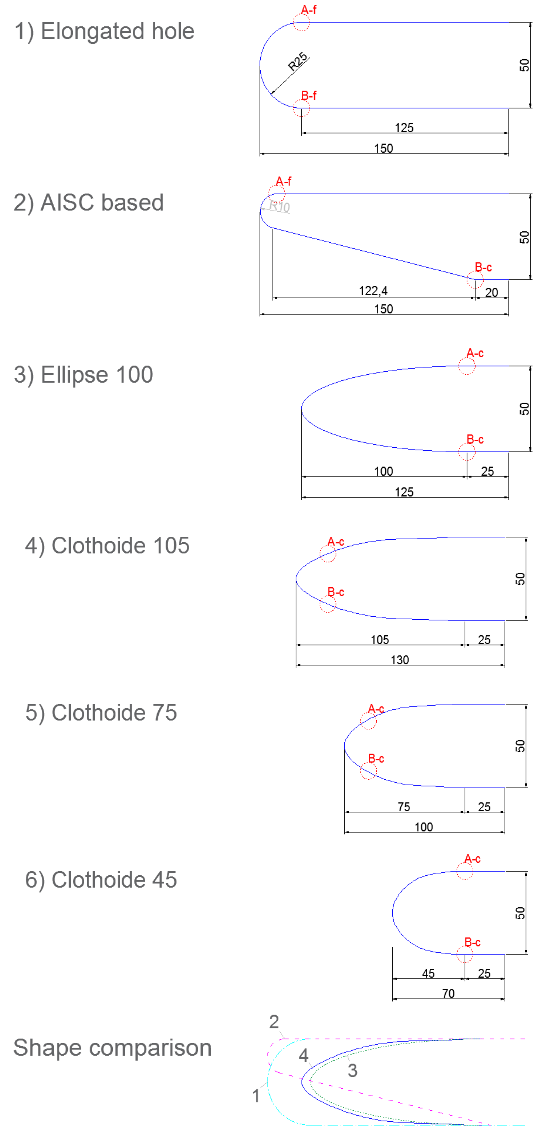


Figure 22: Shapes analyzed for the cope hole ("A" is on the web side, "B" on the flange side. "f" can be compared with a round hole, "c" is a specific curve)

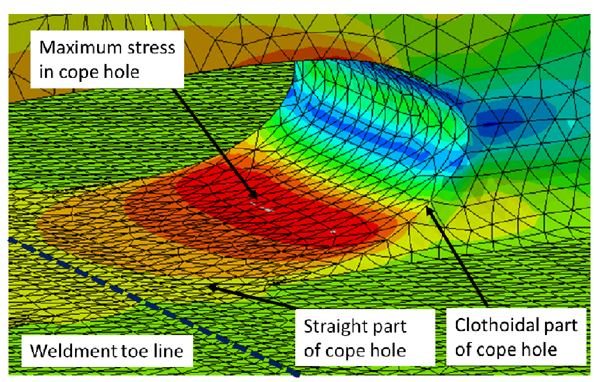


Figure 23: Detail of the ABAQUS model used to calculate the hot spot stress

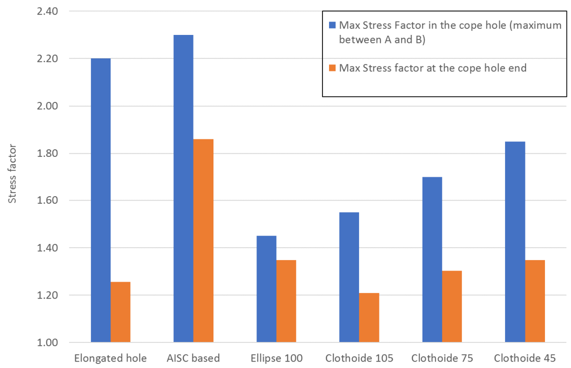


Figure 24: Maximum hot-spot stress factors in the cope hole (blue) and at the cope hole end (orange) for the different shapes analyzed

Figure 24 shows the results in terms of peak stress factor for the analyzed cope hole shapes: the blue ones indicates the maximum hot-spot stress registered and the orange one the value obtained at the end of the cope hole (where the weld toe is supposed to start).

- For the elongated hole, as expected the peak is close to the rounded part (for the given geometry is closed to 2.2, as per the technical literature), but it decreases quickly along the straight part.
- in the "AISC based" form the lower inclination towards the flange seems not optimized for high-cycle fatigue as it creates a stress peak at the cope hole end-flange junction. In the case considered the flat part before the weld toe is just 20mm, so the hot spot stress factor is not far below The flat part shall be longer before the weld toe.
- The ellipse shape is the shape which allows for the lowest stresses in the cope hole, with a stress factor. This value is consistent with the estimation that can be obtained based on the graph in [6], which already confirms as the elliptical shape is effective in reducing the stress peak. Nevertheless in this case the peak would be when the ellipse meets to the straight part, and therefore the stress at the end of the cope hole.
- The clothoide shape is less effective than the elliptical shape but permits to have the stress hot peak in the rounded part and therefore far from the straight part at the cope hole end. In particular the long clothoidal shape can allow an improvement of the fatigue detail compared to the elongated hole, nevertheless it does not seem enough to justify the classification into a better category detail and it shall be solved how the shape can be implemented in practice.

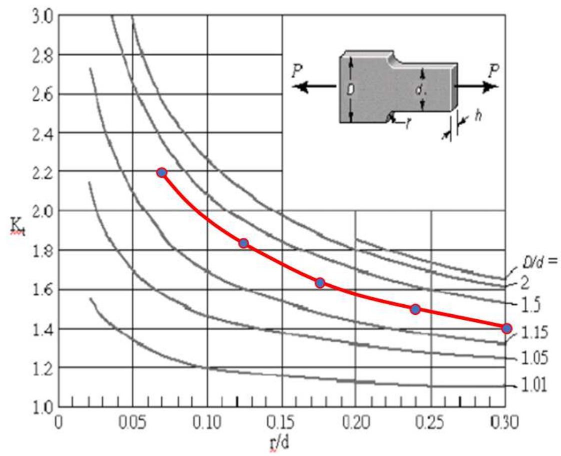


Figure 25: Example of notch factor from the literature – modified extract from [3]

On the basis of this analysis, the analysis was concentrated on the elongated cope hole configuration due to their easiness for designers and fabricators. As mentioned before, the cope hole height is considered as fix to 50mm, as it is the "minimum" height ensuring a correct welding execution. Nonetheless, designers could take also smaller values provided they can ensure a correct execution procedure.

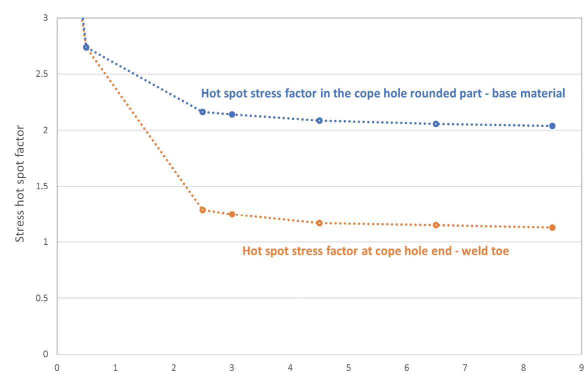


Figure 26: Hot-spot stress factors in the cope hole (blue) and at the cope hole end (orange) for the elongated hole in function of the Length / height ratio.

The next step was therefore to investigate the optimal length of the cope hole. Figure 21 shows the evolution of the hot-spot stress in function of the Length / height ratio (reminder: the calculation done before was for a length / height ratio of 3). Since the objective is to have a hot spot stress factor for the weld toe below 1.25, it is advisable to have a ratio higher than 3. For the current practice, we suggest a ratio of 4 to ensure that the ratios are below 1.25 Figure 26.

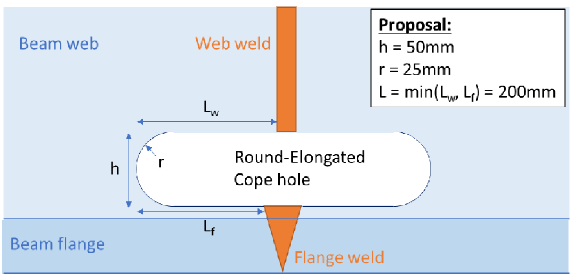


Figure 27: Proposal of cope hole adapted to HD400 – W14x16 structural shapes according Eurocode fatigue design categories

From this study it was possible to identify that the elongated hole with a geometric configuration according Figure 27 appears as an appropriate solution the current Eurocode approach if the stresses are calculated with the net cross-section.

2.5 Butt-Welding of rolled shapes

Heavy structural shapes have been welded successfully in North America since over twenty years also for tension member with splicing done both in the workshop and on the construction site [2][4][5][20][29].

Recent important projects in Europe have shown that it is possible and economically convenient also in this context, provided that adequate preparation in the design, detail and execution phase is ensured throughout the project[27]. In order to demonstrate the adequacy of the welding procedure and justify the presence of the cope hole, recently an investigation was launched to estimate the residual internal stresses after welding [40].

Figure 28: Welded Mock-up out of HD400x744 [36]

From the mock-up test (Figure 28) it was possible to confirm by measuring the internal residual stresses after welding (

Figure 29) that the internal stresses are in most of the cross-section amongst 100 and 200 MPa, tension or compression. As expected, the last part welded are in tension whereas the first ones are under compression. Some parts of the sections reach internal stresses up to 300 MPa, but most of it is clearly below 0.5 * fy = 250MPa which is the standard value accepted as internal stresses due to rolling process (Figure 30). The test was very successful as it demonstrates that an adequate welding procedure with the cope hole leads to quite reasonable values of the internal stresses also for very thick structural shapes.

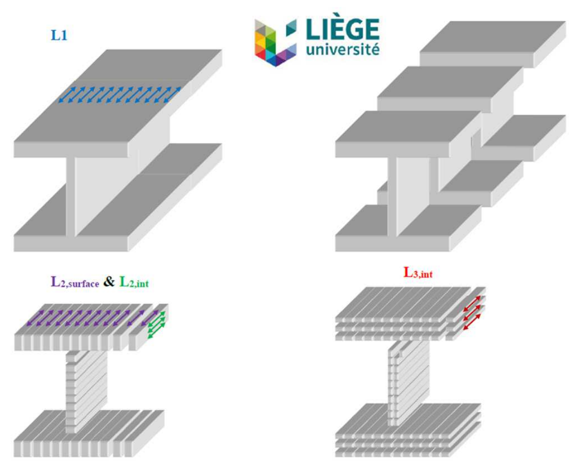


Figure 29: Procedure to evaluate internal stresses in the welded section [40]

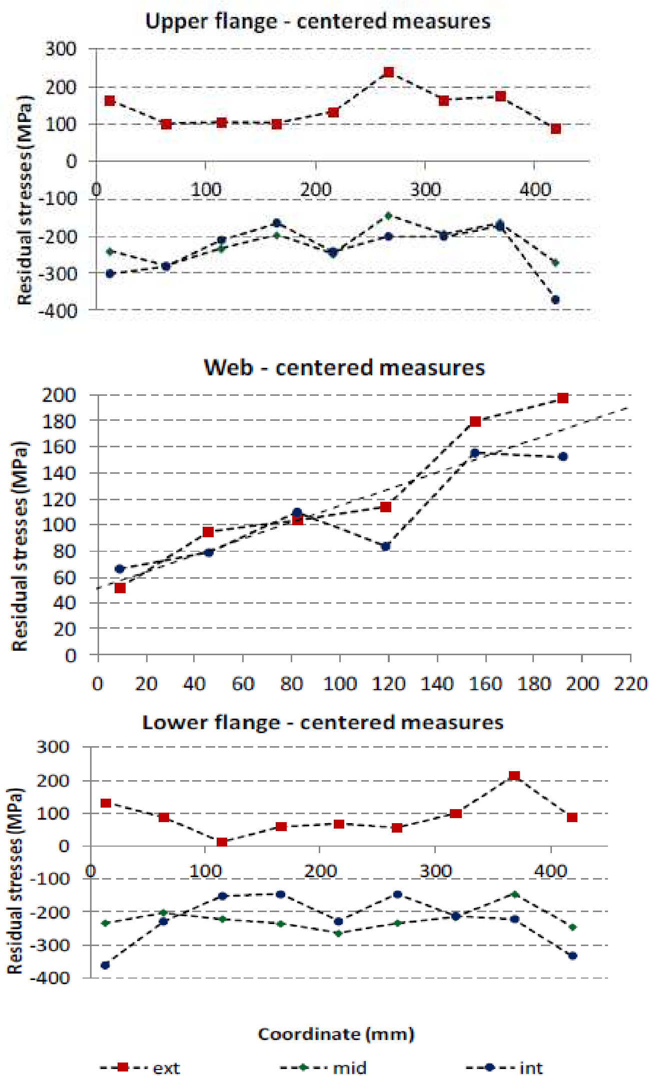


Figure 30: Internal stress patterns in the section [40]

In a first step, the experimental results were compared with a simplified stress-block model. In this model the section is divided in 7 parts (3 parts for each flange, 1 for the web). A simplified welding step is foreseen where each part causes a weld retreat force on the rest of the section. The residual stress caused by the weld retreat of one part is estimated to be 50% of the yield point of the material in the cold situation. The restraint given by the rest of the section is estimated on the basis of the ratio part being welded / part already welded and is therefore increasing during the welding procedure. Exception is done for the web, where the restraint is halved thanks to the cope hole which separate the web and the flanges.

Table 3: Simplified stress block model (Values in MPa)

	Step 1 - Welding Int Flange	Step 2 - Welding Mid Flange	Step 3 - Welding Ext Flange	Step 4 - Welding Web	Stress-block result	Measured average
Ext Flange			250	-31	219	212
Mid Flange		125	-187.50	-31	-93	-104
Int Flange	62.5	-125	-62.50	-31	-155	-163
Web				193	193	179
Int Flange	62.5	-125	-62.50	-31	-155	-164
Mid Flange		125	-187.50	-31	-93	-122
Ext Flange			250	-31	219	170
Restraint	25%	50%	100%	84%		
Yield strength	500	500	500	460		
Reduction	50%	50%	50%	50%		

This simplified model explained in Table 3 gives already a good accuracy (deviation from the average values between 3 and 18%) and can explain the main stress patterns which are formed in the cross-section during welding.

Currently the R&D Center is working to develop an FEM model with the scope to simulate the internal stresses obtained during the welding procedure in a more accurate way [40].

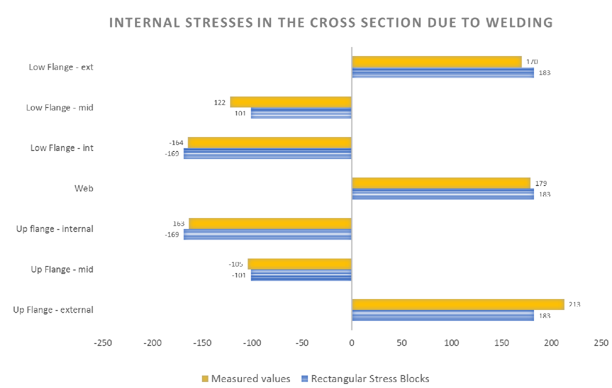


Figure 31: Results obtained with the stress block model compared with the measured values

3. REALIZED PROJECTS

3.1 Project in New Zealand

The first known bridge realized with structural shape for the arch bridge in New Zealand is Mangamahu River Bridge in 2008 (Figure 32, Figure 33). Chief designer was M. Chan from Holmes Consulting Group [44].

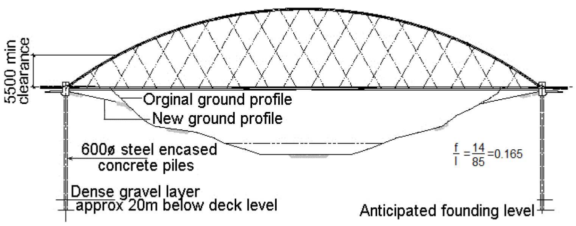


Figure 32: Mangamahu River Bridge in New Zealand – Span 85m [44].

Inspired by Tveit's ideas of structural optimisation, the design minimized the bending moment of the system based on following three aspects:

- Vertical arches anchored in the tie;
- horizontal bracing as closely spaced tubular truss to reduce out of plane buckling
- Quite dense network hanger arrangement

These three structural features can by resumed as the essence of the tied-arch bridge built with rolled sections in the following years.

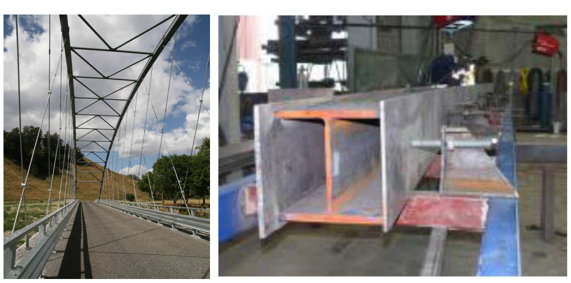


Figure 33: Detail and arch member at gag-press [44].

3.2 First projects in Poland

Between 2007 and 2009, four network arch bridges with HD profiles for the arches were designed in Poland. These designs were favored over other bridge types and justified answering the demand for landmarks at motorway junctions or obviate visibility [39].

Figure 34: Bridge WD-57 in Rawicz (PL) over expressway S5 (Poznań –Wrocław) – 75m span [38]

The first realized has been the bridge WD-431, completed in 2008. It is located in a mining area and connects the A1 motorway to the route Pyrzowice - Piekary. The bridge consists of two independent parallel decks crossing the highway. The span of each superstructure is 62.0 m, with a height of the arc of 9.3 m. This results in a ratio H / L = 1 / 6.67 (= 0.15), the distance between the arches is S = 8.94 m. The sheets were constructed from hot rolled UC 356 × 406 × 634 profiles (similar to HD 400 × 634) in steel grade S355J2. The carriageway was designed as a concrete slab construction with longitudinal pre-stressing in-situ.

Based on this first positive experience, the following three bridges were realized:

- Overpass WD-10 of a highway over the A2 motorway of the Mińsk-Mazowiecki ring, completed in August 2008. The viaduct has a span of 65.0 m, the ratio H / L of the arches is 10.795 m height 1/6. The distance of the arches is 8.95 m. The arches were constructed from hot-rolled HD 400 × 677, steel grade S355J2. The deck is a longitudinally prestressed concrete slab.

- Overpass WD-57 (Figure 34) of the DK36 on the S5 expressway near Rawicz, completed in September 2009. The viaduct has a span of 75.0 m, the ratio H / L of the arches with 12.89 m height is 1 / 5.82. The distance of the arches is 14.20 m. The arches were built from hot-rolled HD 400 × 990 steel grade S460M. In this structure, the deck was designed as a composite slab with rolled sections of the HE series as tie and cross beams.
- Overpass WD-137 viaduct (Figure 35) with the ramps of branch "Ropczyce-Sędziszów" via the A4 motorway in the section Tarnów Rzeszów, completed in October 2009. The viaduct consists of two independent decks, each with a span of 72.0 m. The ratio of height to span of the arches is H / L = 1 / 5.2, the distance between the arches is 8.62 m. The arches were constructed from HD 400 × 744, steel grade S460M. The deck was de-signed as a longitudinally prestressed concrete slab with variable thickness, since it is increased in the axis of the arch.

Figure 35: Bridge WD-137 over Highway A4 (Tarnów Rzeszów) – span 72 m.

The horizontal wind-bracing to avoid out-of-plane buckling of the rolled sections consist out of tubes. For the hangers, usually screwed bars are used (e.g., Macalloy system in grade 460 or 520, M48 to M85 [18]). Although solutions with welded square or flat bars are also used in Poland, they have not yet been used in combination with HD profiles. The arrangement of the hangers and struts is comparable to that of slender box sections as a bow. The main difference to the welded box girder (apart from the slenderness of the bows) is the way in which the arch form is manufactured, namely by cold bending the profiles.

In the workshop, the gusset plates are welded into the profile chambers, similar to a standard often used stiffness. The connecting element has the advantage that it remains completely visible and thus easy to inspect (Figure 36, Figure 37, Figure 38). After completion of the steel construction work in the factory, the corrosion protection is applied, and the segments are dispatched to the construction site where they are assembled together to form the arch.

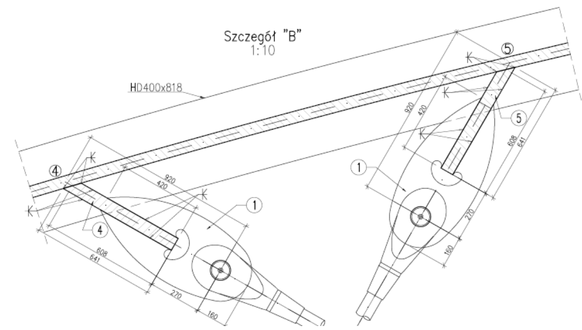


Figure 36: Details of Hanger attachment to the arch

Figure 37: Details of Hanger attachment to the arch during fabrication

Figure 38: View of the hanger attachment close to an arch splice on the construction site

3.3 Erection on the construction site

Various technologies have been implemented for the erection phase. A very interesting possibility is the assembly of the arch on the ground and afterwards the lifting of the complete arch in one piece (Figure 39, Figure 40) – in this case of course the danger of non-elastic behavior or instability during the construction phase must be checked carefully. Alternatively, the arc segments are welded together in the end position (Figure 41) as usual as for the common box girders. Till the arch is working mainly in compression, for main bridge member it is advisable to establish a full capacity splice and therefore the connection is realized by means of a full penetration weld.

Figure 39: Erection of pre-assembled arch for WD-57

Figure 40: Erection of pre-assembled arch Bridge Zatorze in Elblag – Span 50m

3.4 Largest projects built

Amongst the bridges built up to date with rolled sections as arch member, the MS-15 viaduct (Figure 42), which carries two separate lanes of the S51 (Olsztyn-Olsztynek) expressway between Olsztyn-East and Olsztyn-South across the river Łyna, has the longest span.

Figure 41: Erection oft the arch segments by welding in its final position - MS-15 across the Łyna, - span 120 m

The steel construction consists of two separate net arch structures, each with a span of 120 m. For the arches presents an axis-distance of 13.40 m and a height of 21 m. They are realized with profiles HD 400 × 744 and HD 400 × 1086 in steel grade S460HISTAR® [32], a thermomechanical fine grain steel with a guaranteed yield strength 460 MPa also for larger thicknesses than for EN10025-4 and enhanced weldability.

With a flange thickness for this project up to 125 mm, the increased yield strength compared to usually EN 10025-4 (which would guarantee 385 MPa) allows significant savings in reducing the required steel cross section. The building was recently completed, and the experience gained suggests that larger spans can be possible.

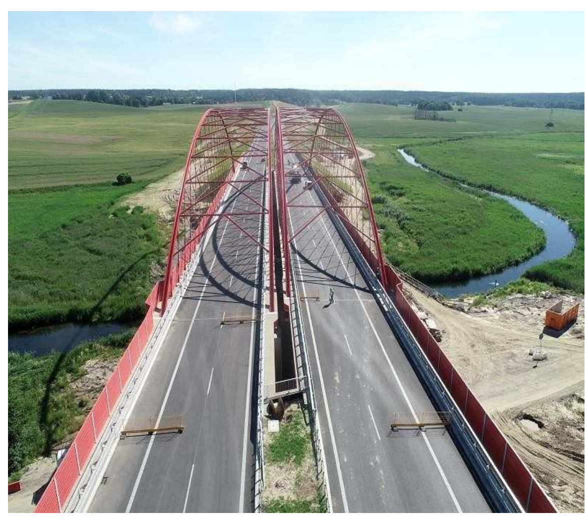


Figure 42: MS-15 across the Łyna, - span 120 m [39]

Figure 43: Rail bridge E-30 over Vistola River in Krakow – main span 116m

In 2018, the first railway network arch bridge with rolled sections was realized in the frame of the modernization of the railway line E30 in Krakow (Figure 43). It consists of three adjacent superstructures (two single-track and one double-track) with three subsequent independent fields each, which are all independent from each other. Thus, the bridge consists of nine individual independent decks, in the range of a single-track superstructure with 49.5 m span to the double-railed superstructure with 116 m span. The deck consists of prestressed concrete, whereas for the arches rolled shapes up to the thickest profiles HD400x1299 series in steel grade HISTAR®460 were used [32].

In the arch-deck connection, the HD profiles are transformed in an innovative solution into halved HL920 profiles with the same material thickness. The innovation consists that the halved section has composite dowels in the web. This allows continuous load transfer and connection to the concrete deck (Figure 44). Design of such composite dowels can be made in accordance to the technical literature [37].

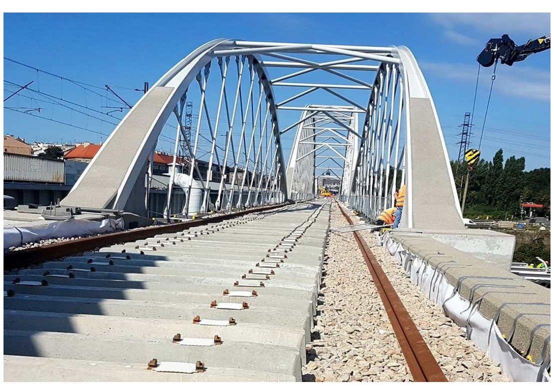


Figure 44: Rail bridge E-30 over Vistola River in Krakow – main span 116m [38]

Table 4: Overview of projects built in Poland between 2008 and 2019 as network arch bridge with rolled sections

Name	Traffic type	Span	Deck width	Rise / span	HD400 section	Design Yield strength
Lowicz	Road	46	19.18	0.22	744	325
Zatorza Elblag	Road	50	18.28	0.17	744	400
WD-431	Road	62	8.94	0.17	634	325
WD-10	Road	65	8.95	0.17	677	325
Suprasl	Road	70	16.32	0.19	744	450
WD-137	Road	72	13	0.19	744	400
WD-57	Road	75	14.2	0.17	990	385
MS-15	Road	120	13.4	0.18	1086	450
E-30	Railway	49.5	10	0.16	677	460
E-30	Railway	116	10	0.17	1299	450

3.5 Credits and acknowledgments

The WD-57 bridge was manufactured by ARCADIS Sp. Z o.o, design was created by Paweł Pilecki and Piotr Myko and tested by Tomasz Kaczmarek. The bridge over the Supraśl was designed by Krzysztof Topolewicz. The viaduct WD-431 was designed by COMPLEX PROJEKT Biuro Projektowo-Konsultingowe Sp. Z oo. - design engineers were Tomasz Kaczmarek and Monika Szymoniak. The project was reviewed by Andrzej Radziecki and Zdzisław Kondrat. The WD-10 building was designed by Andrzej Kulawik Biuro Inżynierskie Zabrze as part of the doc-umentation produced by Dopravoprojekt Bratislava (project engineer: Andrzej Kulawik, re-viewed by Tomasz Kaczmarek). The WD-137 viaduct was also purchased from ARCADIS Sp. Z o.o. under the leadership of COMPLEX PROJEKT Biuro Projektowo-Consultingowe Sp. z oo z o.o. Katowice (editor of the project: Mariola Sygit and Tomasz Kaczmarek, examined by Andrzej Radziecki). The MS-15 bridge with a span of 120 m was designed in 2017 by Krzysztof Topolewicz at TOP PROJECT. Piotr Gosławski was the planner on behalf of Banimex for the executed alternative for the Poronin bridge. The innovative and challenging design of the railway bridges in Krakow is by Radosław Sęk (RS Project).

The authors especially thank the companies and individuals mentioned above for sharing their experience and knowledge, without which this report would not have been possible.

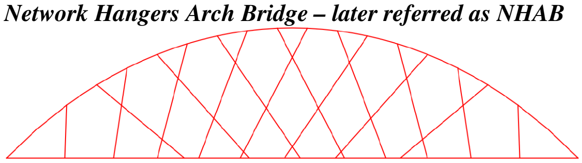
4. CONSIDERATIONS ABOUT HANGER ARRANGEMENT

4.1 Scope

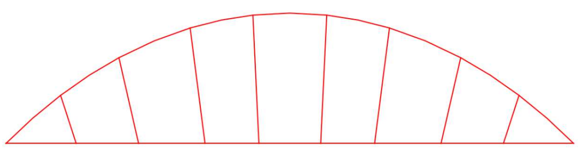
As it could be seen in the previous chapter, all realizations were done with the network tied-arch typology. It is quite intuitive to understand that for this case, since the arch is working mainly under central compression, stocky section in high-strength steel are a valuable option.

Nevertheless, in many cases other hanger arrangement is preferred or traditionally used for several reasons. The question therefore arises whether the use of structural shapes is possible and economical also for these. To answer it, a parametric study was launched to study various possible configurations (Figure 45) and their impact on the design.

Network Hangers Arch Bridge – later referred as NHAB



Radial Hangers Arch Bridge – later referred as RHAB



Vertical Hangers Arch Bridge – later referred as VHAB

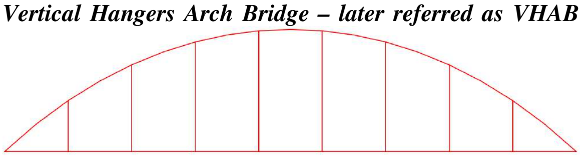


Figure 45: Basic shapes considered in the parametric study

An internal tool was created in order to make linear analysis of a plane arch with free geometric variables. The arch was loaded in all points of the deck in order to generate the envelope of the internal forces and calculate the maximum allowable stress. The transversal repartition of the bridge is linear between the two arches.

This tool was used to make a parametric study for the network, vertical and radial hanger arrangement of a bridge having the same span. The loads used for this analysis was self-weight, a standard permanent weight and Eurocode road traffic loads. Beside the determination of the steelwork consumption, a price estimation for each of the solutions was done based on the internal available information gathered in the realized projects.

4.2 Steelwork consumption / cost vs Form

The NHAB configuration permits to reduce the steelwork weight significantly compared to the other solutions (Figure 46). This result shall be nevertheless mildered when it comes to the global cost (Figure 47). In fact, the NHAB reduces the weight of the arch and the tie thanks to a significantly lower bending moment, but it increases the number and weight of the hangers – which have a much higher unitary cost due to material and installation cost.

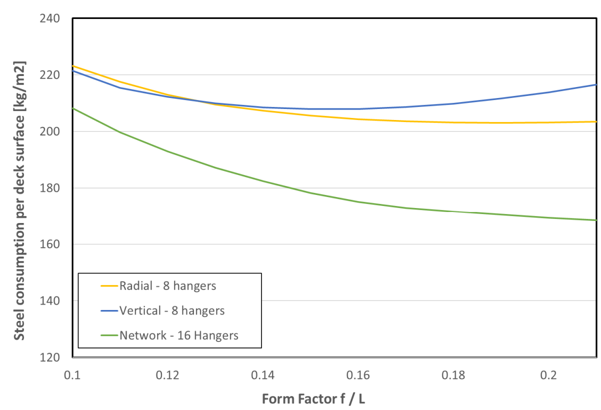


Figure 46: Steelwork consumption vs form factor

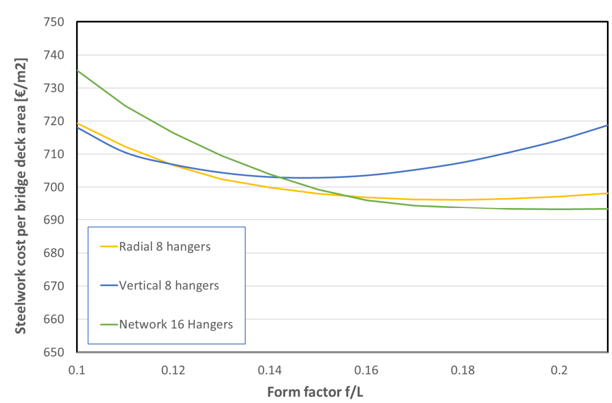


Figure 47:: Steelwork cost vs Form factor

In the frame of this study, the difference in cost amongst the different solutions is not very relevant. Nevertheless, we could outline the following trends:

- NHAB shall be preferred with form factors f/L between 0.14 and 0.25. Higher form factors shall lead to more economic solutions. NHAB with form factors below 0.14 does not appear to be optimized;
- VHAB can be the optimal solution in particular for shallow arched for form factors between 0.1 and 0.13. The form factor shall be limited to 0.18 as their economic efficiency decreases with increasing form factor;
- RHAB is a valuable alternative for all form factors.

4.3 Hanger spacing

The spacing of the hanger is a major parameter since it governs the value of bending actions in the arch and the tie (Figure 48).

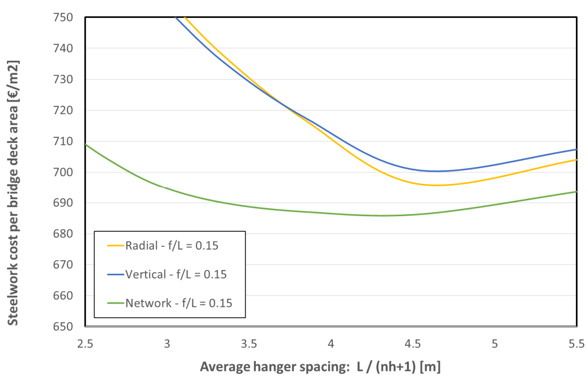


Figure 48: Steelwork cost vs Average hanger spacing

- For VHAB and RHAB the Average hanger spacing shall be larger than 4m, optimal spacing shall be between 4 and 6m.
- NHAB: optimum spacing is between 3.5 and 5m. The guidance given in the literature (e.g. Teich [22]) according to the bridge span in general appear as appropriate.

4.4 Hanger force variation

An important parameter for designers is the variation of the forces in the hanger under traffic loads. It must be premised that in the design of the configurations, only tension was admitted in the hangers under the envelope of permanent loads and traffic loads. The variability of the hanger force gives an indication about the need and the value of the prestressing force to input (Figure 49).

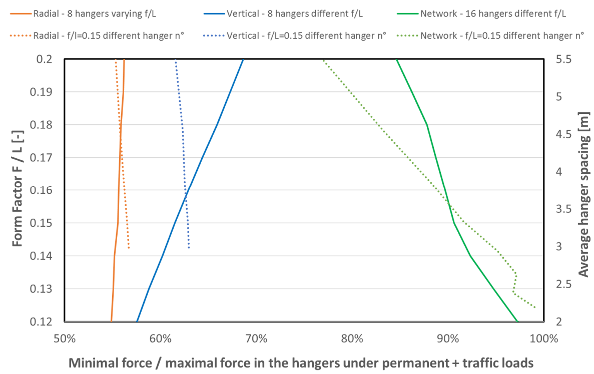


Figure 49: Hanger force variation vs Form Factor and average hanger spacing

Following findings could be gathered:

- NHAB shall not be used for a form factor below 0.15 and / or average hanger spacing below 3m as the variability in the hanger force under traffic loads exceed 90%. Above this threshold it is complicated to design an adequate pre-stressing force.
- Conversely, VHAB shall not have form factors greater than 0.18 because the hanger force variation increases with the form
- RHAB appear as the most stable solution as the shall not exceed 60% as long as the average hanger spacing stays above 3m. For this solution it shall be possible to avoid the pre-stressing of the hangers.

4.5 Optimal cross-section choice for the arch

The parametric study highlights that the eccentricity in the arch (defined as the ratio M / N for the load case maximizing the stress in the arch) is mainly linked with the average hanger spacing – which is a very intuitive finding. On the other side, the capacity of the cross-section to resist bending compared to axial force can be expressed as the ratio of the section modulus on the cross-section area.

The arch is considered as optimized when it is mainly subject to axial force and the bending moment plays a secondary role. This qualitative statement has been translated into the requirement that the bending stresses represent less than 33% of the maximum stress obtained under permanent and traffic loads (reworded: the stresses due to axial force is twice bigger than the one due to bending). Fixing this parameter permits plotting the required minimal W / A ration in function of the average spacing, Figure 50. This graph shows a general tendency which can be approximated with a line.

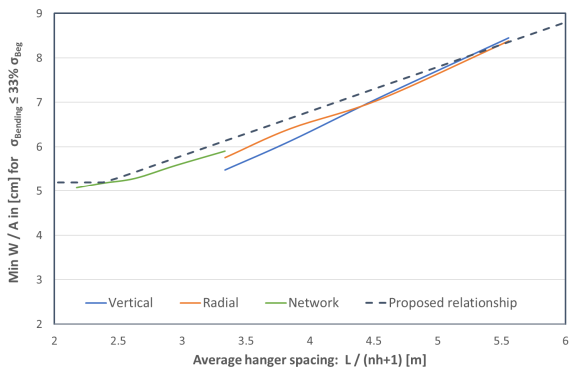


Figure 50: Required minimal W / A to have less than 1/3 of stresses due to bending in the arch in function of the average hanger spacing.

It can be further considered that the W / A is mainly linked with the shape of the cross-section (if the thickness remains in the usual field). Under this assumption, the advised characteristic height of the arch cross-section can be calculated for different cross-section shapes in function of the average hanger spacing, Figure 51.

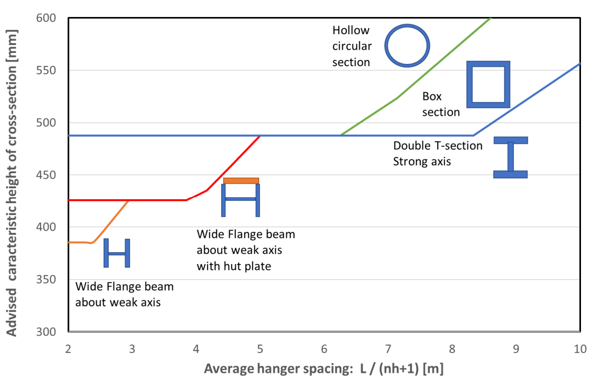


Figure 51: Steelwork cost vs Average hanger spacing

For average hanger spacing larger than 5.5…6m box welded sections appear as the appropriate solution. Circular hollow section in general are less optimized than the box section from the structural point of view, the choice in this case is mainly linked with the aesthetics.

But it can be clearly seen as with lower hanger spacing the wide flange beams are the optimal solution, as the bending moment becomes minor and the section modulus of the section is enough.

4.6 Conclusions of the parametric study

- Network hanger arrangement is in general the optimal solution from the structural point of view, but the design must be done with attention to avoid resulting in higher costs than the other solutions. In particular the arch shall not be too shallow (rise / span greater than 0.15) and the number of hangers not excessive;
- Radial hanger arrangement appears as a better option than the vertical hanger arrangement in all aspects of the structural behavior; from the structural point of view they are the solution to consider beside network arches;
- Pre-stressing of hangers in the network arrangement appears unavoidable. Conversely, for the case of the radial hanger arrangement, it shall be possible to avoid it as the variation in the hanger force is limited.
- Independently of the hanger arrangement chosen, the main variable for the arch design is the average hanger spacing. Wide Flange Beams set along the weak axis are the most economical solution for average hanger spacing up to 3m (this is typically the case of network arch bridges). In case of a reinforcing plate put on top of the wide flange beam only on the upper side, they represent an optimized solution up to 5m hanger spacing (so they can be used also for vertical and radial hanger arrangement).

The authors of this study advise the designers to use these findings as a basis for the conception of arch bridges with heavy shapes, but like to remind two main assumptions taken:

- The study was done for road bridges where the deformability does not play a role for the design.
- The arches are supposed to be braced out of the plane and in the plane a simplified safe rule was used.

5. CONCLUSIONS

This article presents the experiences of the network arch bridges with rolled sections. The solution has been welcome over the past decade with spans reaching up to 120m. The success is due to the economy of the solution linked with simple, effective and fatigue resistant steelwork details.

In this study it has been showed that rolled sections can also be used in other types of tied arch-bridges, namely with vertical hanger arrangements or radial vertical arrangements.

6. ACKNOWLEDGMENTS

The authors especially thank the companies and individuals mentioned above for sharing their experience and knowledge, without which this report would not have been possible.

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