TUBULAR STRUCTURES XI

EDITORS:
J.A. PACKER AND S. WILLIBALD

TUBULAR STRUCTURES XI

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Tubular Structures XI

Editors

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Preface

These Proceedings contain the papers presented at the *11th International Symposium and IIW International Conference on Tubular Structures*, held in Québec City, Canada, from August 31 to September 2, 2006. The 10 previous symposia, held between 1984 and 2003, are described in the "Publications of the previous symposia on tubular structures". This series of symposia or conferences began when the International Institute of Welding (IIW) decided to hold its 1984 International Conference on the topic of Tubular Structures, as part of the IIW Annual Assembly that year in Boston. So successful was the meeting that the tubular structures community decided to hold further conferences independently of IIW, but immediately prior or after the IIW Annual Assembly and in the same country. This practice continued from 1986 to 1996, with the exception of ISTS6 in 1994. The eighth, ninth and tenth symposia, from 1998 to 2003, had no relationship to the IIW Annual Assembly timing or venue. This 11th Symposium, or ISTS11, has again returned under the umbrella of the IIW Annual Assembly, from whence it originated, as the topic for the 2006 IIW International Conference. Throughout its 22-year history the frequency, location and technical content of all the symposia has been determined by the IIW Subcommission XV-E on Welded Tubular Structures.

This 11th Symposium marks the first time since the inaugural conference that the meeting has been held in North America. This has been made possible by the generous financial support of a number of industry associations: IIW, Comité International pour le Développement et l'Étude de la Construction Tubulaire (CIDECT), Steel Tube Institute of North America, American Institute of Steel Construction (AISC) and the International Tube Association. In addition several corporate sponsors provided key funding: Atlas Tube Inc., IPSCO Inc., Welded Tube of Canada, Novamerican Steel Inc., Russel Metals and AD Fire Protection Systems. In-kind support was also provided by the Canadian Institute of Steel Construction (CISC) and the University of Toronto. From modest beginnings this Symposium has now grown to become the principal showcase for manufactured tubing and the prime international forum for discussion of research, developments and applications in this field.

This volume contains 83 peer-reviewed papers, which have all been reviewed by two international experts in the field. Of these, 75 have resulted from an initial "Call for Papers", which produced a record number of submitted Abstracts for this symposium series. Two further papers were invited Keynote Addresses: the Houdremont Lecture (selected by IIW) given by Professor Jeffrey Packer of Canada, and the Kurobane Lecture (selected by the ISTS International Programme Committee) given by Professor Peter Marshall of the U.S.A. The "Houdremont Lecture" (or the "Portevin Lecture" in odd years) is the IIW International Conference Keynote Address which has been incorporated into the Opening Session of the IIW International Conference since 1964. The "Kurobane Lecture" is the International Symposium on Tubular Structures Keynote Address which was inaugurated at ISTS8 in 1998. Besides financial support, CIDECT has traditionally maintained a close interest in the ISTS and the CIDECT Annual Meeting has also been held in Québec City in 2006 in conjunction with ISTS11, as was also done in 2003 with ISTS10. Starting with ISTS9 (in 2001) and repeated at ISTS10 (in 2003), CIDECT has sponsored and administered a "CIDECT President's Student Awards" competition. In this, university students world-wide have been invited to submit entries pertaining to their tubular structures research and the three students judged to be the finalists then completed full papers and were invited to make presentations at the ISTS. For ISTS11, CIDECT has expanded this to two competitions: a Research Award Competition and a Design Award

Competition. A judging panel established by CIDECT has resulted in the three finalists from each of these competitions submitting papers and hence six Student Awards papers are also included in these Proceedings. The two winners of these competitions are announced at ISTS11, after the publication of this book. For a limited time after ISTS11, further details on the Symposium are available at: www.ists11.org

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The Editors hope that the contemporary applications, case studies, concepts, insights, overviews, research summaries, analyses and product developments described in this book provide some inspiration to architects, developers, contractors, engineers and fabricators to build ever more innovative and competitive tubular structures. This archival volume of the current "state of the art" will also serve as excellent reference material to academics, researchers, trade associations and manufacturers of hollow sections in the future.

Jeffrey A. Packer & Silke Willibald
Editors
University of Toronto, Canada
May 2006

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Publications of the previous symposia on tubular structures

M. A. Jaurrieta, A. Alonso & J.A. Chica (Eds.) 2003. *Tubular Structures X*, 10th International Symposium on Tubular Structures, Madrid, Spain, 2003. Rotterdam: A.A. Balkema Publishers.

R. Puthli & S. Herion (Eds.) 2001. *Tubular Structures IX*, 9th International Symposium on Tubular Structures, Düsseldorf, Germany, 2001. Rotterdam: A.A. Balkema Publishers.

Y.S. Choo & G.J. van der Vegte (Eds.) 1998. *Tubular Structures VIII*, 8th International Symposium on Tubular Structures, Singapore, 1998. Rotterdam: A.A. Balkema Publishers.

J. Farkas & K. Jármai (Eds.) 1996. *Tubular Structures VII*, 7th International Symposium on Tubular Structures, Miskolc, Hungary, 1996. Rotterdam: A.A. Balkema Publishers.

P. Grundy, A. Holgate & B. Wong (Eds.) 1994. *Tubular Structures VI*, 6th International Symposium on Tubular Structures, Melbourne, Australia, 1994. Rotterdam: A.A. Balkema Publishers.

M.G. Coutie & G. Davies (Eds.) 1993. *Tubular Structures V*, 5th International Symposium on Tubular Structures, Nottingham, United Kingdom, 1993. London/Glasgow/New York/Tokyo/Melbourne/Madras: E & FN Spon.

J. Wardenier & E. Panjeh Shahi (Eds.) 1991. *Tubular Structures*, 4th International Symposium on Tubular Structures, Delft, The Netherlands, 1991. Delft: Delft University Press.

E. Niemi & P. Mäkeläinen (Eds.) 1990. *Tubular Structures*, 3rd International Symposium on Tubular Structures, Lappeenranta, Finland, 1989. Essex: Elsevier Science Publishers Ltd.

Y. Kurobane & Y. Makino (Eds.) 1987. *Safety Criteria in Design of Tubular Structures*, 2nd International Symposium on Tubular Structures, Tokyo, Japan, 1986. Tokyo: Architectural Institute of Japan, IIW.

International Institute of Welding 1984. *Welding of Tubular Structures/Soudage des Structures Tubulaires*, 1st International Symposium on Tubular Structures, Boston, USA, 1984. Oxford/New York/Toronto/Sydney/Paris/Frankfurt: Pergamon Press.

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Organisation

This volume contains the Proceedings of the **11th International Symposium on Tubular Structures – ISTS11** held in Québec City, Canada, from August 31 to September 2, 2006. ISTS11 has been organised by the International Institute of Welding (IIW) Subcommission XV-E, Comité International pour le Développement et l'Étude de la Construction Tubulaire (CIDECT) and the University of Toronto, Canada.

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Tubular brace member connections in braced steel frames

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ABSTRACT: Diagonal bracings are extremely popular elements for lateral load resistance in steel-framed buildings. In turn, the most common shape used for bracing members is the hollow structural section. While the design of such members is straight-forward, the design of gusset-plate connections at the member ends is controversial and the fabrication of these connections can be expensive. This paper reviews the current "state-of-the-art" for the design of such connections, under both static and seismic loading conditions, and for fabricated and cast connections.

1 INTRODUCTION

The total global output of welded tubes, which represent the manufacturing process used for most of the world's structural tubing, has been approximately constant – despite some fluctuations – over the last 10 years: 40.1 million metric tons in 1995 and 41.1 million metric tons in 2004 (IISI 2005). In this same period, however, the world production of crude steel has increased by 41%, from 752 million metric tons in 1995 to 1,058 million metric tons in 2004. Thus, in 2004 welded tubes represent about 4% of the total steel market, but a very important component of the structural steel sector. While some countries have decreased welded tube output in the last decade (e.g. U.S.A.), there has been a huge increase in production in China (by 245% over the period 1995–2004). National production statistics, for the 10 leading countries, are shown in Figure 1 (IISI 2005). These figures do not include other (less-common) types of hollow sections (e.g. seamless tubes and fabricated sections). While not all of these tonnages will be used for structural purposes, the data is indicative of local consumption and export levels.

In steel structures the most common applications for welded tubes are as columns, in trusses and as lateral bracing members, where the structural engineer can take advantage of excellent properties in compression and the architect can utilize aesthetic qualities in exposed steelwork. Simply-connected steel frames are typically laterally-braced with diagonal members as shown in Figure 2. The ends of the Hollow Structural Section (HSS) bracings are then usually connected to the steel frame via gusset plates, as shown in Figure 3. The design of the bracings, as compression or tension members, is performed in accordance with applicable national or regional structural steel specifications. For low-rise structures with lateral loads governed by static

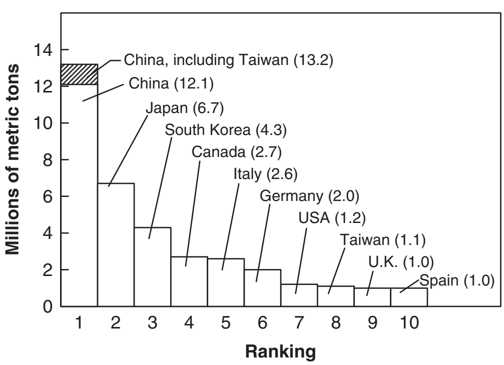


Figure 1. The 10 leading producers of welded tubes, by country, for 2004 (IISI 2005, Table 29).

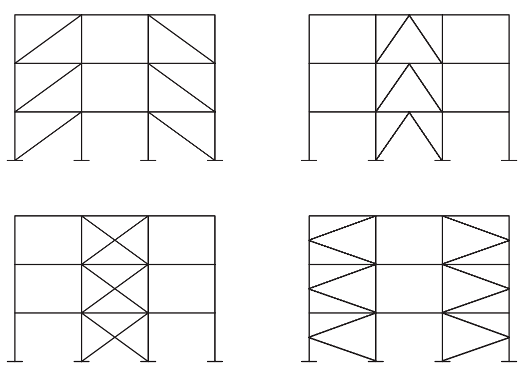


Figure 2. Typical configurations of concentrically-braced steel frames using hollow sections as bracings.

Figure 3. Statically-loaded steel frame, braced with diagonal hollow sections.

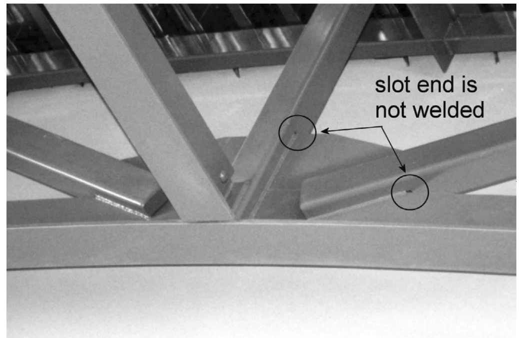


Figure 4. Square HSS diagonals with slotted ends, in a roof truss.

(wind) loading, bracing member selection will often be controlled by maximum permitted member slenderness limits. (For example, in Canada $(KL/r)_{max} = 200$ in compression and, generally, 300 in tension (CSA 2001)). In structures with lateral load design governed by seismic actions, bracing member selection will be further restricted by limits on the slenderness of the member cross-section. For example, for moderately ductile concentrically braced frames in Canada, where moderate amounts of energy are dissipated through yielding of bracing members with $(KL/r) \leq 100$, the flat width-to-thickness ratio of square and rectangular HSS must be $\leq 330/\sqrt{F_y}$ and the diameter-to-thickness ratio of circular HSS must be $\leq 10{,}000/F_y$. These cross-section slenderness limits, in which the yield stress F_y is expressed in MPa or N/mm^2, are considerably lower than the normal Class 1 limits (CSA 2001). In current U.S. provisions for "special" and "ordinary" concentrically braced frames, these cross-section slenderness limits are even more restrictive: $286/\sqrt{F_y}$ for square/rectangular HSS and $8{,}800/F_y$ for circular HSS (AISC 2005a).

2 GUSSET PLATE CONNECTIONS TO THE ENDS OF HOLLOW SECTIONS – STATIC LOADING

Single plates are often inserted into the slotted ends of a round or square HSS, concentric to the axis of the HSS member. This is done both in roof trusses (typically to avoid round-to-round HSS tube profiling associated with directly-welded members, as shown in Figure 4) and in diagonal bracing members in braced frames (Figure 3). This inserted plate is frequently then connected to a single gusset plate, usually by bolting. In such situations a bending moment is induced in the joint by the eccentricity between the plates which must be considered. Under compression loads the plates need to be proportioned as beam-columns, and assuming that both ends of the connection can sway laterally relative to each other. This is frequently overlooked, leading to periodic structural failures, but the American HSS Connections Manual (AISC 1997, Chapter 6) is not guilty of this omission and gives a reasonable and simple design method. Alternatively, the single gusset plate attached to the building frame can be stiffened, typically by adding another transverse plate along at least one edge of the gusset, thereby giving the gusset attached to the building frame a T-shape in cross-section.

With regard to the performance of the HSS in such connections, load is only transmitted initially to a portion of the HSS cross-section, thereby creating a shear lag effect which may result in a lower HSS capacity in both compression and tension. For tension loading on the HSS member, the effective area (A_e) is determined by the net area (A_n) multiplied by a shear lag factor, U. For the latter, the most recent specification version is given by AISC (2005b). These U factors have been revised by AISC from the previous specification (AISC 2000), where U had an upper limit of 0.9. Based on the work of Cheng & Kulak (2000) the U factor can now be taken as 1.0 for connections to circular HSS with a sufficiently-long inserted plate and weld length (L_w). Table 1 shows the current AISC U factors for circular HSS compared to those from other Canadian codes/guides, and Figure 5 illustrates the geometric parameters used. For the shear lag effect, Eurocode 3 (CEN 2005) only addresses bolted connections for angles connected by one leg and other un-symmetrically connected tension members.

North American specifications have gone through many revisions (Geschwindner 2004) concerning the design methods for the limit state of tensile fracture affected by shear lag. Table 1 illustrates the two main prevailing methods: based on the connection

Table 1. Shear lag design provisions for circular and elliptical hollow sections.

Specification or design guide	Effective net area	Shear lag coefficients	Range of validity
AISC (2005b): Specification for Structural Steel Buildings		$U = 1 - \frac{\bar{x}}{L_w}$ for $1.3D > L_w \geq D$ $U = 1.0$ for $L_w \geq 1.3D$ (for circular HSS)	$L_w \geq D$
CSA (1994): Limit States Design of Steel Structures		$U = 1.0$ for $L_w/w \geq 2.0$ $U = 0.87$ for $2.0 > L_w/w \geq 1.5$ $U = 0.75$ for $1.5 > L_w/w \geq 1.0$	$L_w \geq w$
CSA (2001): Limit States Design of Steel Structures	$A_e = A_n \cdot U$	$U = 1.0$ for $L_w/w \geq 2.0$ $U = 0.5 + 0.25\,L_w/w$ for $2.0 > L_w/w \geq 1.0$ $U = 0.75\,L_w/w$ for $L_w/w < 1.0$	no restrictions
Packer and Henderson (1997): Hollow Structural Section Connections and Trusses – A Design Guide		$U = 1.0$ for $L_w/w \geq 2.0$ $U = 0.87$ for $2.0 > L_w/w \geq 1.5$ $U = 0.75$ for $1.5 > L_w/w \geq 1.0$ $U = 0.62$ for $1.0 > L_w/w \geq 0.6$	shear lag not critical for $L_w < 0.6\,w$

$T_r = \phi\, A_e\, F_u$ (AISC (2005b) Specification, $\phi = 0.75$) or $T_r = 0.85\, \phi\, A_e\, F_u$ (CSA (2001) Specification, $\phi = 0.9$), where T_r = factored tensile resistance, F_u = ultimate tensile stress and ϕ = resistance factor.

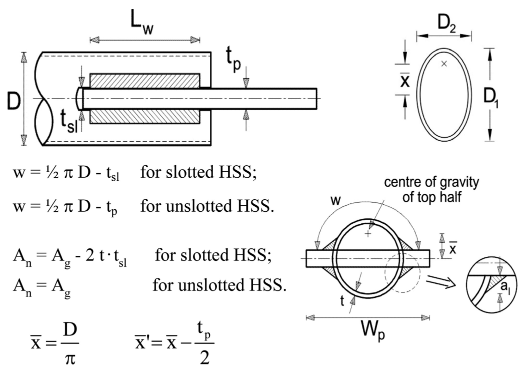


Figure 5. Important geometric parameters influencing connection design.

eccentricity (AISC) or based on the distance between the welds (CSA). In this table it can be seen that the Packer and Henderson (1997) approach is just a modification of the CSA (1994) method. Note that the resistance factor of $\phi = 0.75$ for AISC (2005b) is approximately the same as $(0.9)(0.85) = 0.765$ for CSA (2001). The other tensile limit state for these connections is "block shear" (or tear-out) and the current North American and European design provisions are given in Table 2. As can be seen, all use a design model based on the summation of the resistance of the part in tension (where all use the net area in tension multiplied by the ultimate tensile stress) and the resistance of the part in shear. The latter can be calculated based on the net/gross area in shear multiplied by the shear yield stress/shear ultimate stress, depending on the specification. At present the American and Canadian specifications use a common design model but quite different resistance factors. (The Canadian resistance factor is currently under review).

A study of both concentric gusset plate-to-slotted tube and slotted gusset plate-to-tube connections, under both static tensile and compression member loadings, using both round and elliptical HSS, has been underway at the University of Toronto since 2002. The connection fabrication details investigated, which include both end return welds and connections leaving the slot end un-welded, are shown in Figure 6. The latter fabrication detail is extremely popular in North-America because it provides maximum erection tolerance if the plate-to-HSS joint is fillet welded on site. Sometimes the slot end is roughly cut, in practice, as shown in Figure 7, but this has negligible influence on the connection capacity in quasi-static loading as cracking (leading to ultimate failure) initiates at the end of the weld, not at the end of the slot (see Figure 7). For dynamic loading situations, however, it is recommended that the slot end be drilled or cut accurately. Complete details of the experimental testing program can be found elsewhere (Willibald et al. 2006) but examples of the two classic failure modes are shown in Figure 8.

The experimental program by Willibald et al. (2006) concluded that the block shear design model (Table 2), although based on limited correlations, was suitable, particularly if predictions were calculated using a theoretical fracture path excluding the welds. Further proposals have been recently made to improve the

Table 2. Block shear (tear-out) design provisions.

Specification or design guide	Block shear strength
AISC (2005b): Specification for Structural Steel Buildings	$T_r + V_r = \phi\, U_{bs}\, A_{nt}\, F_u + 0.6\, \phi\, A_{gv}\, F_y \leq \phi\, U_{bs}\, A_{nt}\, F_u + 0.6\, \phi\, A_{nv}\, F_u$ with $\phi = 0.75$ and $U_{bs} = 1$
CSA (2001): Limit States Design of Steel Structures	$T_r + V_r = \phi\, A_{nt}\, F_u + 0.6\, \phi\, A_{gv}\, F_y \leq \phi\, A_{nt}\, F_u + 0.6\, \phi\, A_{nv}\, F_u$ with $\phi = 0.9$
Eurocode (CEN 2005): Design of Steel Structures – General Rules – Part 1–8: Design of Joints [a)]	$T_r + V_r = \frac{1}{\gamma_{M2}} A_{nt}\, F_u + \frac{1}{\gamma_{M0}} \frac{1}{\sqrt{3}} A_{nv}\, F_y$ $\gamma_{M0} = 1.0$ and $\gamma_{M2} = 1.25$

[a)] Design rule for bolted connections differs slightly.
T_r = factored tensile resistance, V_r = factored shear resistance, A_{nt} = net area in tension, A_{nv} = net area in shear, A_{gv} = gross area in shear and F_y = yield tensile stress.

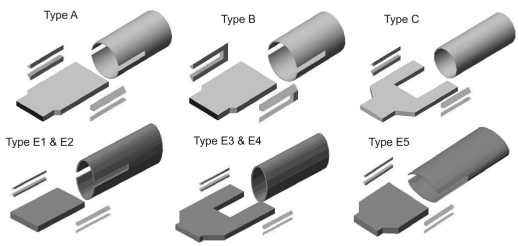


Figure 6. Fabricated connection details investigated.

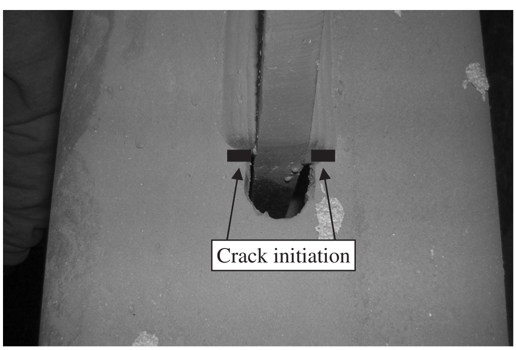
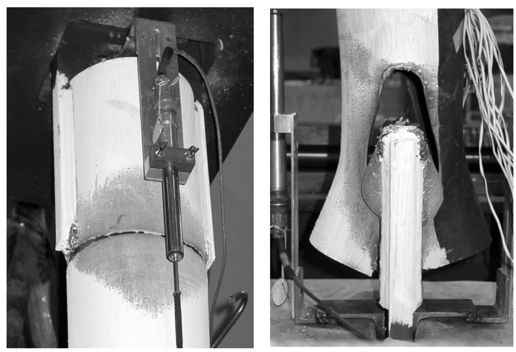


Figure 7. Example of an inserted plate with the HSS slot end roughly cut and left open.

general block shear model in Table 2 (Franchuk et al. 2004, Driver et al. 2006) by adjusting the shear resistance term. It should be noted, however, that their recommendations are based only on bolted connection data. These experiments by Willibald et al. (2006) also confirmed that both the AISC (2005b) and CSA (2001) shear lag factors (Table 1) were excessively conservative, as has been noted by other researchers.



(a) Typical Circumferential Failure (CF) of the HSS, induced by Shear Lag

(b) Typical Tear-Out (TO) Failure along the Weld

Figure 8. Failure modes for gusset plate-to-HSS connections in tension.

The better shear lag factor method was that by AISC, but Willibald et al. (2006) suggested that the existing formulation could be much improved by reducing the connection eccentricity $\bar{x}$ term – used to calculate U – to $\bar{x}'$, as shown in Figure 5. This essentially accounts for the thickness of the gusset plate, which is often substantial relative to the tube size. Interestingly, a very similar conclusion has just been reached by Dowswell & Barber (2005) for slotted rectangular HSS connections, whereby they propose an "exact" $\bar{x}$ term calculated by using a distance from the edge of the gusset plate to the outside wall of the HSS. Dowswell & Barber (2005) verify their proposal by showing improved accuracy relative to published test data by others.

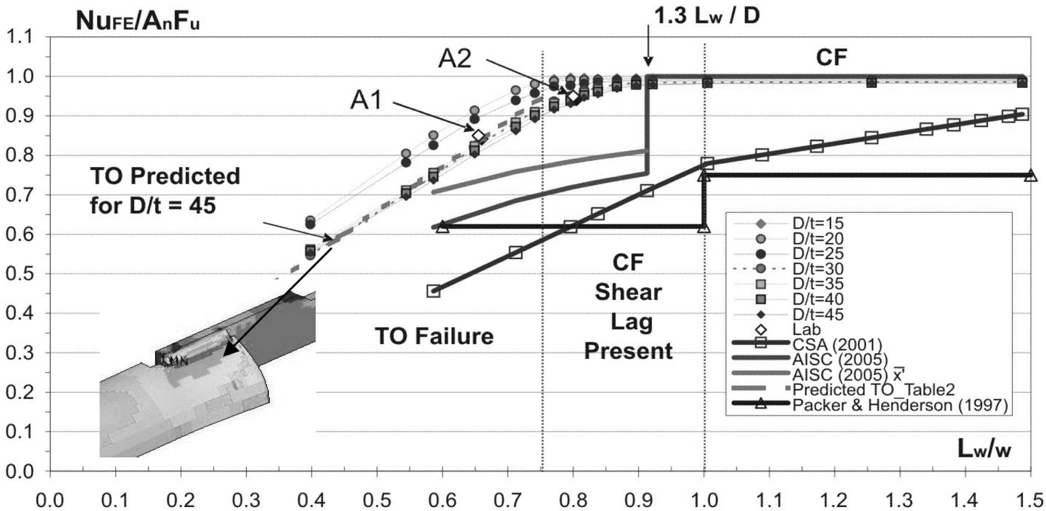


Figure 9. Results of parametric FE analysis and experiments (A1, A2) for connection type A (see Fig. 6) [Tension loading; circular HSS with the slot end not filled: a very popular bracing member detail in practice.] N_{uFE} = connection ultimate strength by FE analysis.

Following experimental research on the connection types shown in Figure 6, an extensive detailed numerical study followed on the same connections using non-linear Finite Element (FE) Analysis (Martinez-Saucedo et al. 2005). A full parameter study expanded the total experimental and numerical database to over 700 connections (Martinez-Saucedo et al. 2006). The FE models revealed a gradual transition between the failure modes of block shear/tear-out (TO) and circumferential tension fracture (CF), with the latter sometimes influenced by the shear lag phenomenon (see Figure 9). A continual monotonic increase in the connection capacity was achieved as the weld length increased. The transition point between these failure modes depended on factors such as: the connection type, the weld length, the tube diameter-to-thickness ratio and the connection eccentricity, $\bar{x}$ (the latter having a strong influence for elliptical HSS). This gradual transition between the failure modes is in contrast to the behaviour given by design models in current specifications, since these specifications do not consider a gradual change between these limit states. Thus, a more unified and less conservative design model for slotted gusset plate HSS connections can be expected in the near future. Figure 9 also confirms that a value of $U = 1.0$ (hence 100% of A_nF_u) for circular HSS with $L_w/D \geq 1.3$ (AISC 2005b) is indeed correct, and for all practical tube diameter-to-thickness (D/t) ratios. However, the conservative connection capacity predictions by over-estimating the severity of the shear lag effect at $L_w/D \leq 1.3$ are very apparent.

3 GUSSET PLATE CONNECTIONS TO THE ENDS OF HOLLOW SECTIONS – SEISMIC LOADING

If the results in Figure 9 are re-plotted in terms of $N_{uFE}/A_g\ F_y$, where A_g is the tube gross area, then it can be shown that long plate insertion lengths can achieve tension capacities very close to $A_g\ F_y$, even for this connection type with an open slot end. However, in tension-loaded energy-dissipating braces the connection will be required to resist an even greater load of $A_g\ R_y\ F_y$, where R_y is a material over-strength factor to account for the probable yield stress in the HSS bracing. This value of R_y is specified as 1.1 in Canada (CSA 2001), and 1.4 (for A500 Grades B and C (ASTM 2003)) or 1.6 (for A53 (ASTM 2002)) in the U.S. (AISC 2005a). The Canadian value is too low, based on personal laboratory testing experience, and a realistic value for the mean expected yield strength-to-specified minimum yield strength ratio is around 1.3, for CSA-grade HSS (CSA 2004). Tremblay (2002) reported a mean over-strength yield value of 1.29 for rectangular HSS surveyed, and Goggins et al. (2005) have reported a mean over-strength yield value of 1.49 for rectangular HSS (Europe), but the latter was the result of specifying low grade 235 MPa steel. The high U.S. values were determined by a survey of mill test reports by Liu (2003) and are not surprising because, in a market like North America with several different steel grades and production standards, manufacturers will produce to the highest standard and work to

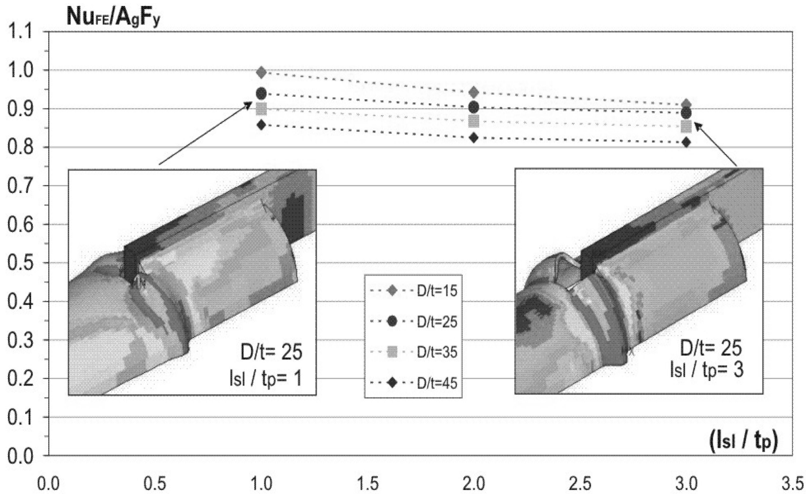


Figure 10. Results of parametric FE analysis for connection type A (see Fig. 6). [Compression loading; circular HSS with the slot end not filled: a very popular bracing member detail in practice.] N_{uFE} = connection ultimate strength by FE analysis, l_{sl} = length of open slot and t_p = plate thickness.

a "one product fits all" approach. (For example, in her survey Liu's ASTM A500 data all pertained to Grade B tubing, whereas manufacturers will knowingly produce to meet the higher Grade C strengths). AISC, however, has now introduced another material over-strength factor, R_t, to account for the expected tensile ultimate strength relative to the specified minimum tensile strength (AISC 2005a) with these values being 1.3 for ASTM A500 Grades B and C and 1.2 for ASTM A53. This R_t factor is applied to fracture limit states in designated yielding members – such as bracings in concentrically braced frames where circumferential fracture (CF) is a design criterion. Thus, applying capacity design principles to preclude non-ductile modes of failure within a designated yielding member (bracing) and setting the resistance factors $\phi = 1.0$, one obtains the following, to avoid circumferential fracture of the HSS at the gusset plate (refer to the equations below Table 1):

AISC (2005a):
$\phi\, R_t\, F_u\, A_e \geq \phi\, R_y\, F_y\, A_g$, hence for ASTM A500 HSS and setting $F_y \leq 0.85F_u$, $A_e \geq 0.92A_g$
CSA (2001):
$(0.85\, F_u\, A_e)\, R_y \geq\, R_y\, F_y\, A_g$, hence for CSA HSS and setting $F_y \leq 0.85F_u$, $A_e \geq 1.00A_g$

In the case of AISC (2005a) the handling of ϕ factors in the above inequality is still under debate. By retaining ϕ factors, and using a lower ϕ factor for the fracture criterion (the left hand side of the inequality), the required $A_e > A_g$. A good discussion of ϕ, R_t and R_y factor selection is given by Haddad and Tremblay (2006).

From the above, one can see that the required minimum effective net area – after consideration of shear lag and application of the U factor – is near, or above, the gross area of the HSS bracing.

In compression, type A connections (see Figures 6 and 10) can be shown to achieve capacities that also approach $A_g\, F_y$, provided the length of the open slot is kept short (in the order of the plate thickness) and the tube is relatively stocky (see Fig. 10). However, despite the achievement of high compression load capacity this is accompanied by considerable plastic deformation in the tube at the connection, which is likely to also render the connection unsuitable for use in energy-dissipating brace members.

Fabricated end connections to tubular braces, in concentrically braced frames, hence have great difficulty meeting connection design requirements under typical seismic loading situations. Reinforcement of the connection is then the usual route. It is difficult to fabricate cover plates for round HSS members so square HSS with flat sides have become the preferred section, resulting in costly reinforced connections as shown in Figure 11. Moreover, recent research on the performance of HSS bracings under seismic loading *still* concentrates on square/rectangular hollow sections (Goggins et al. 2005; Elghazouli et al. 2005; Tremblay 2002). A drawback of using cold-formed, North American square/rectangular HSS is that they have low ductility in the corners and are prone to fracture in the corners after local buckling during low-cycle fatigue.

A clear improvement is to use cold-formed circular hollow sections, which do not have corners, and to

Figure 11. Fabricated square HSS gusset connection for seismic application. (Photo courtesy of Pierre Gignac, Canam Group, Canada).

attempt to avoid reinforcement. Yang & Mahin (2005) recently performed six tests on slotted square HSS and slotted circular HSS under seismic loading and highlighted the improved performance of the circular member, which was "much more resistant to local buckling". Additionally, the use of ASTM A53 Grade B (ASTM 2002) pipe, which is readily available in the U.S. but not Canada, provides a suitably low nominal F_y/F_u ratio of 0.58, which makes the connection much more resistant to fracture at the critical net section and a real design option without reinforcement. ASTM A53 Grade B can be compared to the popular ASTM A500 square HSS Grade C which has a nominal F_y/F_u ratio of 0.81. North American-produced square/rectangular HSS are also known to have poor impact resistance properties since, unlike their European cold-formed counterparts, they are normally produced with no impact rating (Kosteski et al. 2005). Regardless of the section shape and steel grade chosen for energy-dissipative bracings, it is clearly preferable to specify a maximum permissible material strength on engineering drawings, as per Eurocode 8 (CEN 2004).

The use of fabricated, slotted circular HSS gusset plate connections, without reinforcement, is hence being further explored at the University of Toronto. Fabrication with the slot end un-welded (i.e. without an end return weld) is a very popular practice in North America, so special details are being investigated which still permit this concept yet provide a net area (A_n) equal to the gross area (A_g) at the critical cross-section, such as shown in Figure 12. As can be seen in Figure 12, a small gap is still provided at the tube end for fit-up, but the weld terminates at the end of the gusset plate which corresponds to a tube cross-section where the gross area applies.

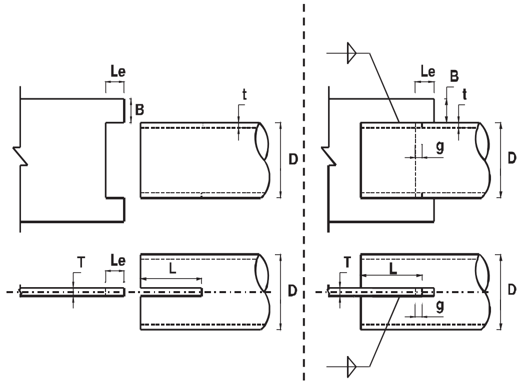


Figure 12. Fabricated connection detail using an over-slotted circular HSS but with $A_n = A_g$ at the weld termination.

4 CAST STEEL CONNECTIONS – SEISMIC APPLICATIONS

Cast steel joints have enjoyed a renaissance in Europe in conjunction with tubular steel construction, mainly as truss-type nodes in dynamically-loaded pedestrian, highway and railway bridges where fabricated nodes would have been fatigue-critical. Another popular application has been in tree-like tubular roof structures where the smooth lines of a cast node have great architectural appeal. Cast steel connectors to tubular braces under severe seismic load conditions have not been used to date, but cast steel connections represent a solution to the design dilemma of fabricated bracing member connections and these can be specially shaped to provide material where it is particularly needed. Types currently under investigation at the University of Toronto, which are designed to remain elastic under the full seismic loading regime, are shown in Figure 13. These connectors are being made using a sand-casting process, which involves some post-casting machining and hole drilling finishing operations. By mass-producing cast end connectors, to suit popular circular HSS bracing member sizes, an economic and aesthetic solution can be reached that still allows the use of regular HSS members and avoids the use of alternatives like buckling-restrained braces, which require pre-qualification by testing and a high level of quality assurance (AISC 2005a). Cast end connectors thus represent another exciting development in the evolution of tubular steel construction. Current work in Canada on cast connectors to tubular members is summarized elsewhere by De Oliveira et al. (2006).

Further research on cast steel nodes, mainly oriented to wide flange beam-to-column moment

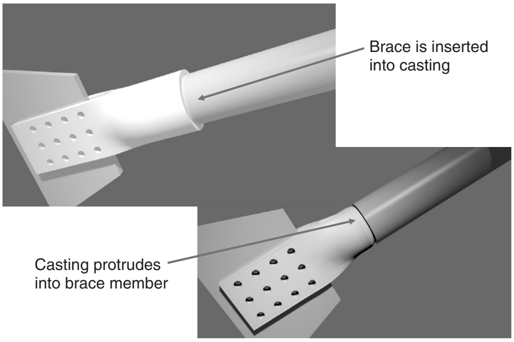


Figure 13. Cast steel connections to tubular braces for seismic load applications.

connections and primarily for seismic applications, is also underway at present at the University of Arizona (Sumer et al. 2004, Fleischman et al. 2006). Such cast modular components avoid the use of diaphragms and special detailing requirements, which are currently advocated for moment connections to HSS columns under seismic loading (Kurobane et al. 2004). It should be noted that this latter publication (CIDECT Design Guide No. 9) does not cover connections to tubular braces under seismic loading.

Another innovative connection solution for wide flange beam-to-HSS columns has been launched by California-based ConXtech Inc., termed the SMRSF. With this, a pre-engineered collar connection is fitted around 102 mm or 203 mm square HSS columns and bolted together on site, resulting in very fast construction times. Although it uses machined components that are shop-welded in place, rather than cast components, this connection is also pre-qualified for use as a fully-restrained, Special Moment Resistant Frame connection under the latest FEMA and AISC seismic provisions. Novel connection solutions such as these herald a potential paradigm shift in HSS construction technology.

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Tubular structures for liquid design architecture

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ABSTRACT: This paper deals with two applications of the technology of tubular structures, as it has been developed over the recent decades for applications in architecture thanks to the fundamental work of civil engineering scientists. Recent architecture is greatly influenced by 3D modelling computer programs which leads to 3D liquid design architecture. In structural engineering terms the design of structures will be governed by liquid design whims of architects, that do not conform to any of the efficient schemes developed in recent years, like space frames, shell structures, tensile or compression structures. Rather the form is not structurally logical. The form is simply given and has to be realized. This realization will require a mix of many primary structural systems. The two examples will illustrate this combined approach: the steel structure for the central body of the Great Hall of the Rabin Center in Tel Aviv (Israel) and the Glass House of Malmö (Sweden).

1 INTRODUCTION

The influence of 3D design in structures is greatly powered by new 2D computer design programs which can determine architectural building forms in almost every imaginable shape. As the main driving force of architects is sculptural, the realization of these dreams has become the task of structural designers, without very much feedback. Of course there are always easier ways to reach a stable structure, but usually the architect wants to see his liquid form realized. It is exactly at this point where the journey of the structural engineer starts. The path is more difficult than it has ever been. The result: an improbable structure, surprising and unbelievable. Nevertheless, after a demanding discovery route for the structural engineer only the result matters.

2 RABIN CENTER, TEL AVIV, ISRAEL

2.1 *Introduction*

In November 2002 we received tender drawings of a design by architect Moshe Safdie from Boston, USA as a part of the Yitzhak Rabin Center in Tel Aviv. The design of the building was an elaboration and extension of a former auxiliary electricity plant near a university campus, in order to become a memorial building for the late prime minister Yitzhak Rabin who was murdered in 1995, on November 4th. He was seen as a peace maker and was awarded the Nobel price for Peace (1994). His activities led to the so-called "Oslo peace talks". The tender we received provided for two building parts: the "Great Hall" and the "Library". These two big rooms both have large glass façades facing south towards the valley below. Both hall designs have remarkable and plastically designed roofs that resemble dove wings as a tribute to Rabin. Moshe Safdie is well known since he designed the 'Habitat' of Montreal as a part of the World Exhibition of 1967 when he was a 27 year old architect (Kohn et al. 1996).

2.2 *Tender drawings*

We had worked for Safdie before on the glass cone of the Samson Center in Jerusalem, overlooking a valley adjacent to the old city near the Jaffa Gate. In our eyes he is an almost prophetic designer who designs beautiful interior spaces. But he is also a perfectionist who supervises all of his delegates in his projects all over the world. He is gifted with a desire for ruthless accuracy in the engineering stage. The Samson glass dome, overlooking Jerusalem with its golden colour in the afternoon, is used for marriage feasts and other celebrations and is a great success. Safdie was very satisfied with our alternative design proposals and with the realized accuracy.

The complicated liquid design roofs of the Rabin Center contained in the tender were analyzed by ARUP

Figure 1. Exterior of the Samson Center, Jerusalem.

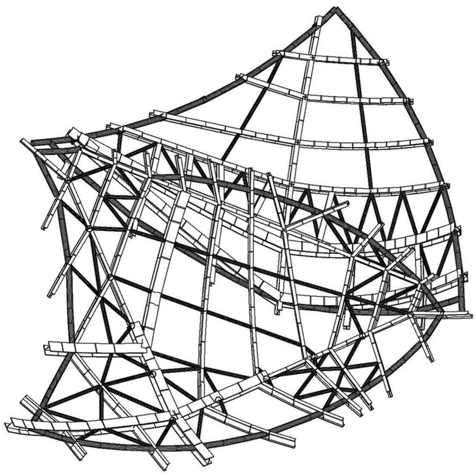

Figure 2. Tender drawing of “the Library” made by ARUP.

New York to be made of a system of arbitrary open steel profiles with a layer of concrete on top. The specification left the roof cladding up to the contractors. On top of this the architect requested a seamless solution in the roof.

For two months we did not give the tender drawings and the thick specification much notice. We thought it was better to leave this project to one of our newly appeared competitors. The “seamless” requirement would make any prefabricated system very difficult and the success would depend entirely on local labor and supervision, which we do not like as a producer of industrial and prefabricated systems. However, the client and his building manager kept on reminding us of the tender date. They even postponed it for one month. It seemed difficult for them to find a trustworthy answer, according to their message. Finally it annoyed us that we would not be able to find a proper technical solution for a seamless fluid roof. This was too much of an intellectual challenge for us as technical designers and inventors.

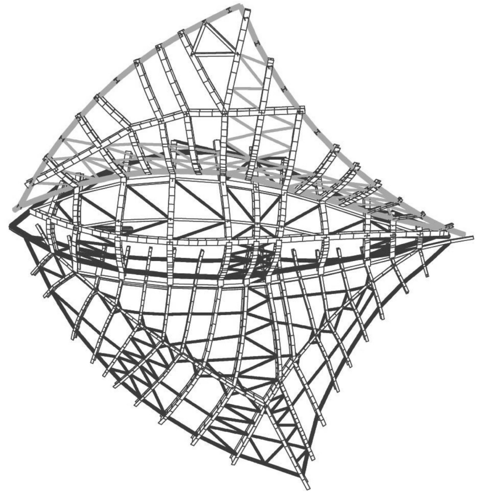

Figure 3. Tender drawing of “the Great Hall” made by ARUP.

2.3 *Alternative idea*

In a few brainstorms we came to the following basic idea: make the roofs as giant surfboards of foam with stressed GRP skins on both sides. The size of the roofs, subdivided into 5 different roof wings was a maximum of 30 × 30 m. Each wing had a maximum length of 30 m and a width of 15 to 20 m. It was a self-secluded form in itself – the very realization meant a possible step forward in larger sized architectural objects. It was the technology and the resulting technical product that was fascinating.

In a month we organized three successive brainstorms on the product idea, the structural concept and the logistics & pricing. These meetings were open discussions in which all concerned parties had to make up their mind whether they were able to live up to their part of the job and to estimate the financial complications. We noticed that all members of the team were a bit afraid of the adventure, which became apparent when the first cost estimations were added up. We decided to work out and price our stressed sandwich skin alternative as well as the original tender specification of the steel structure with a non-described, free covering as a variation. The deadweight of the steel structure was estimated by ARUP, so a price for the original with cladding variation was easy to make. The cladding we proposed for the original tender design was derived from the mega-sandwich idea, but now in a thinner scale version of 50 to 80 mm thickness, as it only needed to span the space between the steel structure elements (max. 3 m).

The budget calculations came out on a level of 2.5 million Euro for the original design with a thin 80 mm thick GRP sandwich cladding instead of concrete. The

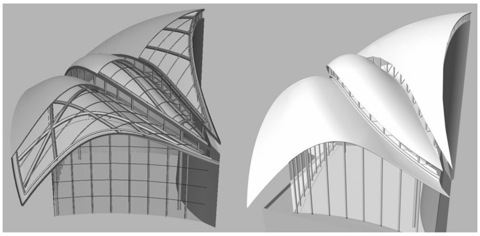

Figure 4. Two construction types: a steel structure of circular hollow sections (left) and the structural sandwich structure (right).

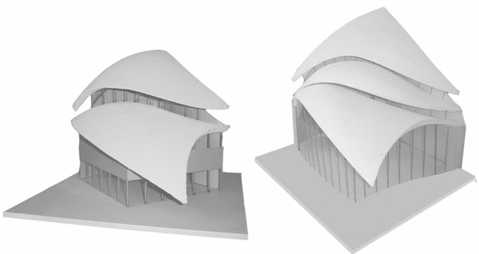

Figure 5. Models made for the tender of ‘The Library’ and “The Great Hall”.

alternative design with the full load bearing stressed skin sandwich would add up to more than 4 million Euro, largely due to the high estimates of the production of the polyester parts. We argued that the maximum extra costs could not exceed one million Euro, resulting in a total price of 3.5 million Euro. We were sure that any architect would fall in love with the alternative idea of the self-supporting stressed skin sandwich. This was what we faxed to Israel, just in time before the tender closing date, accompanied by a letter explaining the two quoted systems.

2.4 *Amazing solution*

Only two days after the tender closed we received a telephone call from the local representative architect Avi Halberstadt, speaking on behalf of Moshe Safdie. He gave us the compliment that the architect saw the alternative proposal as *“an amazing solution”*. Apparently they had not thought about this concept themselves and this was the most satisfactory compliment we ever received. For us, whether we were awarded the contract or not, intellectually we had already won. Halberstadt invited Eekhout to come over to Tel Aviv for a meeting, so he could present his ideas to the building commission. At the presentation he showed the polystyrene models that Haiko Dragstra had machined in a demountable model scale of 1 to 40. The model showed that the corner details in the design had not yet been accurately designed and that the overall stability was not satisfactory. The design needed a lot of engineering work. The building commission went into a separate meeting.

2.5 *Extremely innovative but too expensive*

After one hour of fierce discussions, the outcome was that the tender original with the GRP covering was practically on the average tender price level. On the other hand they noted that the alternative proposal was indeed very attractive from the viewpoint of its extremely innovative design and construction, but was priced one million Euro over budget. We responded that this was known from the date we sent in the quotation. Knowing the intellectual value of the alternative proposal, it would have been stupid to sell it at a lower price than the tender proposals. Usually technical alternatives are more efficient solutions for the contractors and tend to be lower in price than the original. A more expensive alternative is rare and hence extraordinary. Since most of the tender results in Israel showed that Octatube was the most expensive party, it was and still is, a sport to get the order. Starting with the highest price and the best technology, may end with a contract at a compromised price. We also loose projects as our competitors can copy our technology after one completed project and execute this without the necessary research and without the higher Dutch labour costs of Octatube. But in the case of the wings, the alternative idea was to become a technical world novelty and Moshe Safdie understood this. He stated that the idea was unbelievable and never done before to his knowledge. If someone could make it work in his opinion, it was to be Octatube. He remembered our collaboration for the Samson Center two years before and he had made some phone calls to other Israeli architects to check our reputation. We had the complete support of the architect. He was our product hero, protecting us with an umbrella. The response of the chairman of the building committee was to come up with different logistics for the GRP sandwich proposal in a manner that the price level could be lowered to 2.5 million Euro. He suggested that it might be possible to transfer the foam machining and the GRP production to Israel in order to reduce costs for shipment and labour at the same time. This was the message we took home 29 April 2003. The committee wanted to speed up their decision, so we had to come up with an alternative idea within one or two weeks.

2.6 *Rethinking*

Back in Delft we discussed the consequences with our engineers and the external team members. The plan was born in the airplane from Israel to the Netherlands. If we could build the GRP sandwich roofs, it would be a hit on the world market. We were prepared to transfer more labour to Israel in order to reduce costs and talk to new Israeli partners if our current partners would let us down in order to realize this proposal.

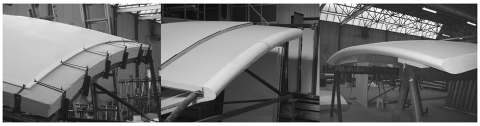

Figure 6. Three prototypes (left: stressed membrane; centre: to be locally produced; right: prefab sandwich).

First we could try to decompose the big wings into transportable components, which we could assemble on-site on a jig and finish the broken GRP layers and give the shells a final top-layer or top-coat. We could have machined the polystyrene blocks and we could also set up an Israeli GRP plant in Tel Aviv on the building site. The most likely position to assemble a wing would be in a vertical position. More or less like stacking bricks, we would assemble the wall from placing the polystyrene blocks on top of each other. This way both polystyrene skins could be treated simultaneously and we could control shrinking of the foam. The subcontractor was not experienced in estimating larger productions than mock-ups. The bottom price of our subcontractor for composite productions in Israel did not give much hope either. At the same time the usual squeezing of tender prices came about, which forced us to land on another price level altogether. We decided upon a steel space frame with a locally made sandwich panel system on top, forgetting the world novelty of the stressed skin sandwich, just to stay in the race. Based on this price and on our abilities Moshe Safdie was convinced that Octatube could do the best job. We had to deliver further design development and prototypes of the construction of the Great Hall, assuming that the details of the Library would follow those of the Great Hall. Just for intellectual reasons we added the prototype of the stressed skin sandwich.

2.7 *Production and installation*

The largest challenge for Octatube proved to be the *central body*: the central part of the Great Hall. Due to the large forces from the upper and lower roof wings amongst others, that this part of the roof has to cope with, a steel tubular structure was the only solution to make this span possible. This resulted in a complex structure of tubular steel, later to be fitted with thin GRP panels. According to the 3D computer data, various tubes were to be rolled in both 3D (two directions) and 2D (one direction). Because accurate 3D rolling is a rather complex procedure, the 2D rolled tubes possessed a greater accuracy. The 3D tubes, mainly situated along the length of the central body, at best approached the desired shape, therefore had to be connected to the accurately shaped 2D tubes. As if the 3D shape of the central body alone was not complicated enough! Nonetheless, after a lot of thinking and welding an impressive "artwork" came into being.

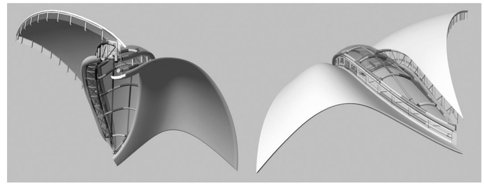

Figure 7. Renderings of the roofs for the Great Hall with special view of the *central body*.

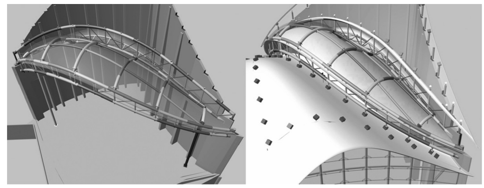

Figure 8. Two more views of the *central body*, the small cubes (image right) mark the position of the steel *inserts* to connect the GRP shells to the steel of the columns and the *central body* (right).

Figure 9. Test assembly of the *central body* (spanning 30 m) in Lelystad at Holland Composites.

At Holland Composites the entire central body was assembled in order to fit the panels. After every panel was fitted, the structure was disassembled and transported to Tel Aviv. In Israel it was to be assembled in two parts, hoisted on its position and only later would these two parts be connected.

After the production of the roof parts for the lower wing of the Library, discrepancies between the theoretical drawings and the practical distortions and tolerances from shrinking of the polyester resin in the vacuum bags were measured. Tolerances because of warping of the negative moulds resulted in unforeseen deformations of the produced GRP components. These components together had to form the ruthless smooth surface of the complete wing in the end. All aspects were approached in an engineering manner: measuring, analyzing problems and deducing solutions. Improvisations bring the Law of Murphy nearer. Analytical engineering in the best traditions of the TU Delft made the initial amazing, improbable design solution finale a reality. The resulting design is a combination of structural design with architectural flavour,

Figure 10. GRP roof segment assembled on a temporary steel structure at Holland Composites in March 2005.

incorporating the technologies from aeronautics, ship building, industrial design and geodetic surveying and poses an example of multiple innovation of technology, thanks to the involvement of co-makers Octatube International, Holland Composites Industries and Solico Engineering.

2.8 *Assembly*

Due to the experimental character of the production process and unfamiliarity with the consequences of vacuum deformation, we decided to perform a test-assembly or pre-assemblage on the premises of Holland Composites in Lelystad. The fitting took place on a positive steel frame, the shell would therefore be curved upward. When a technician fell, he would not fall in the shell, falling instead off the shell. Subsequently we would gently turn the shell over with a mobile crane, by means of three temporary hoisting fixtures in the shell. From the pre-assembly we could also draw conclusions regarding the theoretical versus the practical measurements of the individual segments. All the segments were produced on individual foam moulds and they all had their own shrinkage and shrink-direction. Yet together, these segments had to form the unforgiving smooth surface desired by the client and architect. It was exciting to see if the total fitting of the individual deformed segments would still form a smooth surface when the entire shell was assembled. In order to acquire this smooth surface we indeed needed a solid frame with clamps in order to force the segments into the desired position. In general the segments proved to be somewhat smaller than intended. When filling up the seams during assembly a bigger seam meant more fibre (due to the required ratio between fibre and resin) thus causing a larger weight of the shell.

The connections between the individual segments can be divided into connections in the length and connections in the width of the segments. On the side of the segments a groove has been made of 220 mm width and 15 mm depth. In this groove a prefabricated reinforcement of 200 mm width and 10 mm depth (of high density glass fibre meshes that have been vacuum injected with resin) was placed. This reinforcement was glued and clamped by bolts. After the segments of the two wings in Lelystad were fitted on the steel frame, the frame was dismantled and shipped in special containers to Tel Aviv. The build-up in Tel Aviv had to take place on the south side of a tall wall of the building. The segments were assembled inversely, measured, touched-up and finished with the structural reinforcement meshes and filler. Next, the shells were turned over and identically finished on the other side. After hoisting onto the Library, the shell was positioned on a steel sub-structure, which in turn rests on a concrete wall with a much larger tolerance difference. Until the end, theoretical drawings remained the decisive factor. In all phases of engineering, production, assembly up until the hoisting and positioning, theoretical drawings were always present. Building parts were simultaneously produced in locations all over the world. In this project the steel was manufactured in Delft, the glass in Luxembourg and Belgium, the polyester segments in Lelystad and the concrete in Tel Aviv.

Figure 11. Hoisting of one of the roof shell wings.

Figure 12. During installation of the roofs the *central body* was temporarily supported by scaffolding.

Figure 13. The Yitzhak Rabin Center in November 2005.

2.9 *Combination of factors*

The design is the result of a combination of architectural design, building technical design, structural design, material design, with major influences from aeronautical and yacht design, from mechanical engineering (machining moulds) and industrial design, composite production design, assisted by the newest techniques of geodetic surveying to accurately measure the 3D forms in any stage of production, assembly and installation, all smeared by standard computer software like Maya and Rhinoceros and static analysis programs, including Computational Fluid Dynamics. In all of the successive steps of the process many of the used technologies have applications of theoretical developments done in more fundamental design or science areas. The bold design proposals from this design process challenge the more fundamental partners in the design process to come up with new answers. This process was an illustration of the "Delft Silicone Valley-effect" that the TU Delft as a whole has, with its range of faculties, on the world novelty of the end result. The co-makers Octatube, Holland Composites and Solico joined forces and developed this very experimental project with high economic risks, but with great endeavour and the eternal optimism of designers envisaging a new future in the thrill of this experimental design and build process.

3 GREEN HOUSE, MALMÖ, SWEDEN

Not only is the shape of the Green House in Malmö special, the technical aspects of the project are also very interesting. The control of the complex geometry as well as the structural design were very carefully approached to ensure that both were up to the high standard a project like this was asking for. The goal set by landscape architect Monika Gora was to develop a structure with as much transparency as possible. She asked Ian Liddell of Buro Happold of Bath (UK) to support her design in a structural scheme suitable for tender. A major consideration was the realization of the glass house with flat glass quadrangular panels. This model was sent for tender on the international market. Some Swedish companies as well as Octatube tendered.

Figure 14. Transparent model of the Green House describing the final geometry.

Although the price of Octatube would not have been the lowest, it was supported by great experience in the field of tubular structures and glass structures. Also juggling with 3D geometries is fairly common business in the engineering department of Octatube. The project was awarded to Octatube. At the start of the redesign phase after contracting, Monika Gora provided a small physical surface model of the Green House and a 3D CAD drawing that Octatube could work with. As the 3D drawing was not nearly accurate enough to develop a working basis for the further engineering process, a new computer model had to be developed. This was done using sophisticated design software (like Maya and Autodesk Mechanical Desktop). Rendered visualizations helped to finalize the shape.

Early in the design process the decision was made to use only flat glass panels for the outside surface. Although Octatube had ample experience with twisted and bent glass panels, the use of warm-bent glass panels was ruled out, as the cost for cladding the complete building with free-formed panels would be astronomical. Cold-twisted panels (panels pressed into shape on site) might have been an option but were also ruled out, as their twisted surface shape is difficult to control. Cold-twisted panels normally form a hyperplane (known from tent constructions), which would contradict the chosen shape of the Green House completely. The challenge therefore was to develop a geometry for the glass panels that would follow the original shape as close as possible, while keeping all the glass panels flat.

The challenge could have been solved easily if the surface was divided into triangles. This easy solution was abandoned, as triangles would mean a lot more glass divisions with extra silicone seams and glass nodes, which would lessen the aspired transparency of the surface. Instead, a natural growth algorithm was used to develop a surface consisting of flat panels with four corners, starting at the highest point of the Green House and working onward and downward to both

Figure 15. Test assembly of the steel structure in Delft.

ends of the building. A couple of different divisions of the surface were worked out, adjusting the parameters for the generation of the 4 corner panels each time to arrive at a solution which had logical divisions and endings at all places. By using a natural growth algorithm it was made sure that all panels were perfectly flat, while every corner point of every panel still rested perfectly on the originally chosen surface!

The glass panels are each made from two laminated, heat-strengthened glass panes with each having 8 mm thickness. To get an as transparent façade as possible, all glass panels were made from a special, extra white (low-iron) glass. Also the glass for the doors and the louver systems at the bottom are made from extra white glass. The glass material of these panels has a very low level of iron, reducing the normal, slight green tint of the glass to almost zero. The high glass thickness was necessary, as the site location of the Green House is exposed to high wind loadings, as it is situated directly on the shore of the Öresund between Sweden and Denmark.

For the glass node, a special clamped connection in the seam between the glass panels was chosen. This connection consists of stainless steel plates, both inside and outside of the glass, with a plastic spacer in between. Four clamped glass nodes are fixed to one spider with four arms. Each of the spiders is laser cut from a steel plate and bent into the necessary shape. Due to the complex geometry of the surface none of the spiders is the same. A central bolt to the steel structure inside the Green House then fixes the spider. The clamped connection gives the possibility to use the glass surface to stiffen the whole structure, as each glass panel can transmit forces in its plane through the plastic spacer to its neighbouring panel. This would have been impossible with a drilled connection due to the high stresses around the holes in the glass. The glass surface is therefore used as a continuous shell.

The structural design of the tubular steel structure of the Green House started with the already developed divisions of the glass surface as a boundary condition. The structure had to follow the glass seams, as the spiders had to be fixed to it. Different possibilities were discussed:

Figure 16. Close-up of the glass panels and the supporting steel structure during installation.

1. delta trusses as columns,
2. delta trusses for the backbone,
3. plane trusses for the columns,
4. a single layered space frame for the whole surface.

In the end a very simple and clear structure of a double tubular spine and single tubes as legs was chosen. Along the ridge of the Green House (the backbone) a double beam from CHS profiles ø168 mm was designed. As columns behind each glass seam, CHS profiles ø159 are used. Overall this choice leads to a very clean appearance of the Green House, with a very high transparency of the façade. To minimize the steel structure, the glass surface was used as a shell to reduce overall deflections. The connections between the different steel parts are made by sleeve connections, to achieve a continuous appearance of the CHS profiles. All connections of the structure are designed to be moment-resisting, to reduce the profile dimensions as much as possible.

For production and assembly the highest accuracy had to be administered. Due to the choice to keep all parts as small as possible almost all adjustment space was left out. The glass panels were fitted into the glass nodes with a tolerance of ± 1 mm. This tolerance between the glass and the plastic spacer of the glass nodes was then filled up by silicone strips, as otherwise the glass surface could not work as a shell and reduce deflections of the whole structure. The glass seams between the glass panels were made weatherproof by using structural silicon sealant.

To make sure that all parts fitted together and all connection points for the spiders were fixed at the right

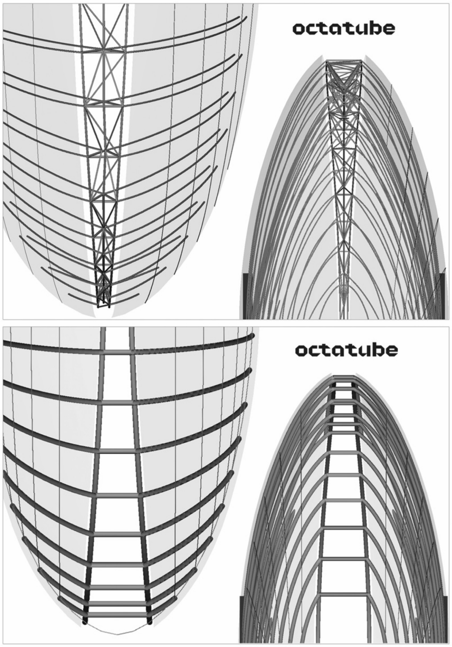

Figure 17. Tubular scheme 2 (top) and 4 (bottom).

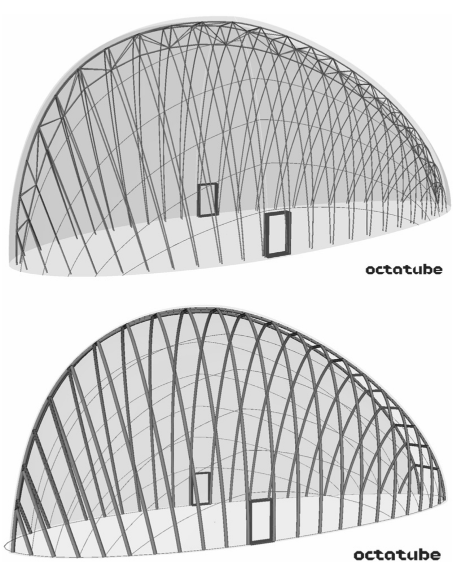

Figure 18. Tubular scheme 2 (top) and 4 (bottom).

place, pointing in the right direction the whole steel structure was set up in Delft before being galvanized and coated. After all dimensions were checked, the parts were galvanized and then powder coated to achieve the best possible surface finish.

Transport to Sweden and assembly on site went very smoothly in the late summer of 2005. Last finishing touches as well as the technical installations were done in the winter. The outside landscape gardening was finished in November, the inside in January 2006. The official opening of the Green House was in February 2006.

Figure 19. The Green House just after completion.

4 CONCLUSION

Realizing two structures in liquid design form has taught the engineers that the combined virtues of different structural schemes have to be employed, and that insight into tubular structures has become ever so complicated. By combining the structural action of shells and space frames, and even for structures with bent elements, the resulting structures could be made. The engineering required a great many bent-tube pieces in very different bending radii. The accuracy required for producing structures assembled from bent elements of different radius is well known. Recognizing mismatches and dealing with them in order to arrive at a structure with a high tolerance, dictated by the use of frameless glazing that cannot allow larger tolerances than 1 to 2 mm at very seam, requires very careful production and frequent 3D surveying during production and assembly stages. The result, however, shows a great optical logic.

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Tubular steel roof for Spencer Street railway station in Melbourne, Australia

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ABSTRACT: With its striking visual appearance, the redeveloped Spencer Street (Southern Cross) Station in Melbourne, Australia is now a distinctive landmark and a world-class transport facility, and opened in early 2006. Through a sequence of staged deliveries, the station remained fully operational throughout the construction phase. The new station brings together the best elements of a modern, international-style station, creating a user-friendly facility that will be comfortable, convenient and safe for passengers. This paper describes the design and construction of the unique and innovative domed tubular roof structure, comprised of spine trusses along the platforms and long span arches across multiple railway lines.

1 BACKGROUND

Spencer Street Station is the key transport hub for both the Melbourne metropolitan and the Victorian regional and interstate rail network. Upgrading of the old station (including its renaming to Southern Cross Station) has included revamping of commuter traffic flows from an underground basement access to a modern open-air transport interchange facility where the platforms are accessed via a newly constructed elevated concourse. The track network and platform layouts are predominantly unchanged by the redevelopment; however, new infrastructure has been built for ticketing, administration and retailing facilities. A feature of the new station is the total enclosure of the station, comprising an entire city block, with a spectacular wave-form roof enclosed on all sides by a glass façade.

The project is a Public Private Partnership between the Victorian Government and the Civic Nexus consortium. The Civic Nexus consortium comprises a maintenance and retail operator with the design and construction being undertaken by Leighton Contractors. The design consultants for the project are as follows:

– Project Architect – Grimshaw-Jackson Joint Venture
– Structural Engineering – Winward Structures
– Mechanical and Electrical Engineering – Lincolne Scott
– Track and Signalling – GHD
– Durability and Rail Engineering – Maunsell
– Wind Tunnel Testing – MEL Consulting
– Environmental Engineering – Advanced Environmental Concepts
– Shop Detailing (Roof) – Precision Design Australia.

2 PRELIMINARY DESIGN DEVELOPMENT

The main feature of the station is its large wave-form roof. Occupying a complete city block and covering 36,000 square metres, it is not only an architectural icon of Melbourne, but performs an important functional requirement for the station. The concept for the roof developed by Grimshaw Jackson was to not only provide an architectural feature but also to provide a form that would allow gases from both diesel locomotives and in some instances steam locomotives to be exhausted without the use of costly mechanical fans or extraction systems. Grimshaw's past experience with railway stations in both Europe and the UK indicated that covered stations with large barrel vaulted structures tended to trap diesel fumes – hence the development of the domed or individual mogul-shaped roof which allows the fumes to rise to isolated points which can then be extracted from the station.

Computer modelling in the form of computational fluid dynamics (CFD) confirmed that this could be achieved. The roof moguls at Spencer Street reach a height of up to 24 metres above the platform levels at their highest point, dropping to a minimum height of 6 metres in some places. The roof is also required to be elevated at both the northern and southern ends to clear the existing Collins Street and Bourke Street bridges.

In order to assist the roof ventilation under prevailing cross winds, the moguls are offset across grid lines. This also reduces the risk of fumes being sucked back into the station environs under a cross-wind through an adjacent mogul. At the apex of each mogul or dome is a louvre cap which enables fumes and/or gases to be extracted, but which must also operate to prevent wind-driven rain from being blown into the station.

The roof structure essentially comprises a series of structural steel tubes forming a two-way net system. The original roof design was cut back to make way for a future commercial development on the western side of the station above the existing rail lines. This structure is an elevated slab which will support a future 10 storey building above. Introduction of this structure to the project after the initial design was commenced created a problem for the main station roof, as the largest internal bay of the roof now became an end bay. This resulted in higher stresses in the main roof rafters, requiring increased wall thicknesses in a substantial amount of members. A number of options were investigated, with the final solution adopted being to "lean" the roof on to the substantially stiffer elevated deck. This avoided having to use heavy wall pipe but meant that temporary works in the form of a series of arch ties could not be removed until the elevated deck had been completed.

3 ADOPTED DESIGN

The roof is set out on a geometric grid, with a series of five grid lines running down the centre of every alternate platform in a north–south direction. The platforms are set at a 12 degree skew to the adjacent Spencer Street. Grid lines labelled A to E run in the east–west direction. A total of 29 column supports for the roof are located at the intersection of all grid lines. Grid lines numbered 1 to 5 run in the north–south direction and are the central axis for large triangulated trusses referred to as spine trusses. The 12 degree skew has resulted in a complex intersection of trusses at the corner of Spencer Street and Collins Street. A series of curved roof rafters, referred to as primary arches, run in the east–west direction and are spaced equally at 4 metre centres. These arches are not true arches as such, as their geometry has a reverse curve as part of the arch shape. Again, these intersect with the spine trusses at a 12 degree skew in plan. A series of short secondary members between adjacent arches help form the complex two-way system. Columns at 40 metre centres along the spine trusses support the truss in its Y-shaped form. Columns are cantilevered out of the ground from a rigid pile cap and piled foundations.

The structure described above forms the main substructure, or the bottom of the roof's three layers. The second layer above the structure is a ceiling consisting of a series of triangulated ceiling panels. Over 7000 ceiling panels of similar design but each with their own unique geometry provide both an architectural ceiling and an insulating barrier for the station roof as well as providing a work platform to enable construction of the third and final layer of roof sheeting above. The ceiling itself also contains an important diagonal structural brace which is connected to the main supporting steel substructure. The architect intended that this diagonal bracing member should be hidden from view, although it is an essential element of the roof structure. An important feature of the ceiling panel design is that it allows air to flow and naturally ventilate the station environs. This was achieved by maintaining nominal gaps between panels to allow air to track through the gaps into the ceiling space, and rise to the top of each mogul before being vented out through the louvre cap.

The third and final layer is a weatherproof skin in the form of a metal deck to the station roof. The chosen product had to have the ability to be manufactured and installed to suit the continually changing geometry of the wave roof. It also needed to have a 25 year warranty. The choice of roof sheeting was a key issue in the final design solution. A standing seam aluminium tray deck system, stainless steel shingles and zinc roof sheeting were the main alternatives considered. A number of early concepts using GRC panels, tension fabrics and sandwich panels were also considered, but dismissed as being either too difficult to weatherproof, not able to meet the warranty requirements or unable to meet the defined geometry of a flexible roof structure capable of accommodating thermal movements, settlement of the structure under self weight and fabrication/erection tolerances.

Other key features of the design include a siphonic drainage system. The steep roof sheeting drains to gutters located each side of the spine truss. Outlet discharge points are located at each column, where a sump gives way to a siphonic outlet discharging into the column arm down to the base of the column, piped along the platform into an abandoned subway tunnel, and then discharging to a common surge pit on the western side of the station.

A clear skylight system is located above each grid line, running north–south along the centre of alternate platforms. The skylight sits above each line of spine trusses. A number of solutions for the skylight were investigated. A solution suggested by the architect was the use of ETFE cushions. This is a tensioned fabric technology, with each cushion consisting of a twin layer fabric structure of ETFE (basically clear Teflon) in the shape of a cushion. A continual air supply is pumped into the cushion to maintain both shape and strength to the finished product. Grimshaw had successfully used this technology in a number of projects in the United Kingdom – most notably at the Eden Project in Cornwall, which is an environmentally controlled biosphere constructed entirely of EFTE cushions over a steel framework. Alternatives such as traditional glazing were also investigated for the skylights. However, the logistics of installing large sheets of glass in-situ on the roof together with the continuously changing geometry of the trusses would have made glazing a difficult option. The roof itself is a flexible structure, both in its final configuration and during the various stages of construction. This could have potentially created additional problems for glazing and sealants at construction completion. The flexible nature of ETFE ensures that movements of the roof will not impede the performance of the skylights.

Some statistics for the roof are:

- Approximately 3200 tonnes of structural steel
- Roof area of 36,000 square metres of which 7000 square metres are skylights
- 29 supporting columns
- 207 primary arches
- 1370 secondary members and 1600 diagonal braces
- 7050 ceiling panels.

4 STRUCTURAL DESIGN

The roof structure consists of structural steel tube, with the predominant size being 356 mm OD circular hollow section (CHS). This size includes the main boom members of the spine trusses and all primary arch members (curved roof rafters). Wall thicknesses vary from 6.4 mm to 12.7 mm; however, some heavier wall tube up to 23.8 mm was used in some of the flatter arches. Secondary and diagonal bracing members in the roof proper consist of 168 mm diameter CHS of varying wall thicknesses between 4.8 mm and 25.0 mm.

The main support for the roof is a series of triangulated spine trusses running in a north–south direction along the centre of the platforms. The inverted triangle section is 8 metres in width at the top, with a depth varying between 4 metres at the columns and 2 metres at mid-span. These trusses also form an undulating profile arching between column supports along the platform. The spine trusses consist of 356 mm diameter members for the main chords and cross member of the triangle, with 273 mm diameter vertical members and 168 mm diameter plan bracing.

The roof structure is unique in that the primary arches spanning between spine trusses span one way in the construction condition. Once all the lacing members (i.e. secondary and diagonal members) are installed, the roof mogul begins to form a two-way system in the final condition. The roof mogul acts as a shell transferring the bulk of the roof loads back to the four columns in the corners of each dome, thus substantially reducing the amount of vertical load or dead load to the spine trusses on the grid lines. This was a chief architectural requirement, as the architect wished to see only a two-way net system/structure from beneath. The trusses therefore became a Vierendeel truss, with no diagonal bracing in elevation but diagonal bracing in plan to achieve the lateral restraint required by the out-of-balance forces provided by a flat arch opposing a steep arch on either side of the truss.

The choice of typical key connection types was critical early in the design process. These included the following connections:

Primary arch to spine truss connection. A bolted halving joint was chosen as it provided a seat for the primary arch to land, enabling quick bolting and securing of the arch.

Primary arches up to 40 metres in total length were generally broken into three sections for transport. This was not just due to its length, but also because the varying radii of most arches produced an over-width transport problem. Depending on the location of these joints, some varied from a full moment connection to pure compression, depending on the load in the arch and the point of contraflexure (virtually all arches contained a reverse curve in geometry). To maintain a consistent approach it was decided to butt weld all these joints on site. This also removed the difficulty where a connection of a secondary/diagonal member at a node point clashed with a primary arch connection, had these been bolted together. This decision was made during the early design development phase, before the geometry of the roof was fixed and the nodal points defined.

Spine truss to spine truss main chord or boom member connections between adjacent truss lengths. A spliced angle plate/cruciform connection was chosen to allow some construction tolerance as trusses were erected over each column first with an infill section placed between them. The installation of the infill spine truss was one of the more complex lifts on site and needed to be bolted and secured in the limited time slot available during the night occupation. The connection was therefore detailed to allow a rapid bolting and securing of the truss.

Connections of vertical, diagonal and main chord members in each truss were fully welded. The use of similar tube sizes avoided issues with punching shear failure modes in the majority of cases. However in some connections, in particular in and around the column where a diagonal brace was installed, heavy wall pipe and/or stiffener plates were installed as part of the design and fabrication process.

Primary arch connection to diagonal roof bracing. An innovative connection detail was developed for this connection. As described previously, the diagonal brace sits above the main structure hidden within the ceiling panels. The diagonal bracing is an important element in transferring load from the roof shell back to the columns. Some of the diagonal roof members in and around the columns carry substantial loads. This connection therefore induces an eccentric load on the primary arch. Due to this substantial load and the varying orientation of cleat plate connections for the diagonal braces, it was decided to locate individual cleats around a circular stub upstand welded on top of the arch. This stub also provided for a further upstand to pick up a roof purlin at a higher level again.

5 FABRICATION

Structural steel tube was purchased from OneSteel, with some of the heavier wall pipe being sourced from overseas. On delivery of the tube, it was transported to a fabricator in South Australia to be cold-rolled into its correct shape and radius. Once this was completed the members were then transported to various fabricators in Victoria, Tasmania and New South Wales for the next stage of fabrication.

The columns were made in two parts: the bottom section was a circular tapered tube, and the top section was a fabricated box structure forming the Y-shaped arms. The fabrication process was made difficult by the 12 degree skew of the column arm – because the cross section of the arm was not rectangular but effectively a rhombus. The fitting of the internal services including the siphonic downpipes and an electrical conduit also made the fabrication process difficult. Once on site, the bottom section was installed, the top part of the "Y" inserted via a spigot connection into the bottom pedestal section, services connections were made and then the bottom pedestal section was filled with concrete, pumped from the bottom.

The primary arches were set out in a workshop to their completed final shape. The shop drawings were used to then set out all connection points and all cleat connections for secondary and diagonal members were fabricated in the shop, including the end connections for bolting to the spine trusses. Arches were then cut into three sections for transport, painted and then sent to site with set-out marks for welding on site.

The fabrication of the spine trusses was a more complex process. The triangulated trusses were fabricated in an inverted position. The shop detailer was able to provide additional set-out information to enable the fabricator to set up the truss in purpose-made jigs. Tube elements were cut with ends profiled to match the complex connection points around the main boom members. Some node points on the main boom member had five different tubes connecting at the same point. The structural designer also had to nominate a priority for members coming into that joint, together with the specified welding type and extent. Set-out of the halving joint stub for connection to the primary arches was critical, as the connection relied on the full mating of matching plates in the field prior to bolting. Taking into account the 12 degree skew and the continually changing pitch of the arch and the undulating spine truss, this was no mean feat. Once the set-up was done, the trusses were fully welded, lifted and rotated into their upright position and the steelwork for the gutter beams was fixed. Finally, the truss was painted, installed into a transport frame and loaded on to a barge for its trip to the Port of Melbourne, for its final delivery to site via road transport at night.

Connection of the spine truss to the Y-shaped column arm was the one of the most complex connections. The concept, both architecturally and structurally, was to provide a three-point pin connection, one at the "crutch" of the Y and the top two at the column "ears". However, lining up or matching three pins in this configuration within 2 mm oversized holes was almost impossible owing to fabrication tolerances, deformation of items during both transport and erection, and thermal movement. It was therefore decided to provide an architectural pin connection only. This involved large steel billets being shaped and fitted to the spine trusses and then welded to the column arm on site. This required a detailed survey of both the spine truss at the end of fabrication and the as-installed column arm to confirm whether any clashes or excessive gaps were likely to occur prior to erection of the element.

Secondary and diagonal members were simpler, being straight members with bolted cleat connections at both ends. However, the logistics of tracking these members through the fabrication, painting, expediting and final erection processes was a massive task as all elements were of varying length and had their own unique position on the roof. Similarly, the ceiling panels were relatively straightforward to fabricate but each had its own unique geometry resulting in its unique position on the roof.

6 CONSTRUCTION

One of the key motives behind building a complete roof covering the entire Spencer Street Station site was to come up with a concept that was both architecturally acceptable and capable of being constructed over an operating railway station. Up to 60,000 commuters use the station per day, with 700 metropolitan trains serving six platforms and 240 interstate and country trains serving eight regional platforms.

This was a key issue that had to be taken into account throughout the entire design development process, from early concepts through to the final detailed design. An important part of the process was to engage the shop detailer early in the design process, to not only assist and provide input during the design development, but also to save significant time in the fabrication process. It reduced the amount of unnecessary and repetitive architectural drawing by making the shop detailer responsible for the roof geometry and all dimensional control. This also assisted greatly in the off-site fabrication process and the on-site survey installation by being able to interrogate any part of the theoretical model of the roof. The shop detailer therefore became an integral part of the design team with the architect and the structural consultant.

The choice of structural connection types and location of construction splices was another key factor in the design stage. Connections had to be located at convenient locations from both a design and installation point of view. Connection types had to allow for easy installation given the time constraints of constructing over live railway lines in limited blocks of time, as well as achieving the architectural and structural capacity desired by the roof designers. Bolted connections being the preferred choice.

The station constraints can be split into two main areas. Approximately one third of the roof was built over the electrified metropolitan lines, with the balance over the non-electrified country and regional platforms. The particular challenge over the electrified lines was that opportunities for installation were only available between the last train at night and the first train the next morning. Allowance within this period also had to be made for the overhead power to be isolated. This therefore further restricted the available window of opportunity to approximately two and a half hours each night to install all elements of the roof over the electrified lines.

To minimise the number of lifts over the metropolitan lines, it was therefore decided to construct large roof modules on the western side of the site, in the median strip of Wurundjeri Way, and to lift these large units into position with a 600 tonne crawler crane. The main spine trusses that supported the roof modules and ran along the length of the platforms were installed with this same crane. These large roof modules consisted of 4 primary arches with associated secondary and diagonal structural members and ceiling panels assembled as a complete unit (minus the roof sheeting) in the median strip ready for lifting.

The spine trusses were fabricated off-site in Geelong, Tasmania and New South Wales and towed to Port Melbourne on large barges. They were then transported at night to the site from the South Wharf, approximately 5 kilometres away, and lifted directly into their final positions. Each truss section was fabricated in 20 metre lengths. With 40 metres between column supports, a truss section was installed firstly over each column and then a 20 metre long infill truss was dropped into position and bolted. A similar process was adopted for installation of the remaining spine trusses over the entire project, but the full-time occupation of other areas of the station, such as over the regional platforms, enabled smaller cranes to be used.

Owing to the full-time occupation of regional platforms being attained in stages, it was more opportune to install the roof segments between trusses piece by piece. This meant the primary arch or main roof rafter (spanning from truss to truss) was installed first, followed by the series of secondary members and diagonal members, followed by the ceiling panels. The ceiling panel installation ensured that a working deck was in place to enable the roof sheeting to be installed during normal working hours, without the danger of dropping materials on to the public or workers below.

A series of safety nets were also installed immediately beneath the top of the trusses where the ETFE skylights were to be installed. Again, these nets enabled installation of the skylights to proceed during the day.

7 INNOVATION

The constraints of the site dictated that the main spine trusses had to run centrally down the platform centres on every other platform. This set the dimension/span between grid lines in the east–west direction, whereas along the length of the platform a nominal grid spacing of columns at 40 metres (and therefore main roof arches at 4 metre centres) was adopted in the north/south direction. The complex intersection at the corner of Spencer Street and Collins Street where the two spine trusses entwine presented its own construction challenges. This intersection was fabricated in one piece in the workshop off site, strategically cut to enable its transport to site, then reassembled on the ground beneath the supporting columns. The sections were then re-welded into one piece and lifted into its final position as a complete unit. The resulting effect is a remarkable feature of the main entrance to the station.

A full-scale prototype of an 8 metre by 8 metre section of the roof was constructed early in the design development stage. This was intended to not only provide a real-life part of the roof to be reviewed for its architectural aspects, but also to confirm concepts developed for the connection of steel members to steel members, and the connection of ceiling panels and roof sheet purlins to the main structure. Standard connections had to be adaptable for the full range of geometries encountered by intersecting members over the entire roof. The prototype also confirmed a connection detail for the triangulated ceiling panels that cover the entire roof. There are over 7000 individual triangulated ceiling panels, each with its own unique geometry, however connection details were required that would be applicable over the entire structure. From this prototype, more testing was done to replicate the performance and load capacity of the ceiling panels.

Although the ceiling panel was a relatively straightforward design, the connection details were unique. The connections had to allow for not only structural adequacy, but also for tolerances associated with steelwork fabrication and erection out of position. The connection also had to allow for movement as the structure was progressively loaded during construction and again at completion under in-service thermal and wind loading. A unique pin/barrel connection was developed which allowed rotation to suit the geometry of the roof, and lateral movement with an adjustable pin for construction tolerances.

A significant number of temporary works were required to stabilise the roof during the construction and to avoid excessive member sizes in the permanent structure to accommodate temporary construction loads. Temporary propping had to be minimised to avoid track pits and commuter traffic on platforms. Temporary works included:

- An arch tie "bow-string" which connected at the base of all primary arches (roof rafters). These were designed to take out the out-of balance lateral forces on the spine trusses prior to the adjacent bay being installed.
- A temporary spine truss prop located midway between columns to provide support for the spine truss until the roof modules were fully completed in adjacent areas of the roof. As described previously, the spine trusses do not have sufficient vertical capacity to carry the weight of the roof. At the completion of construction, the shell action of the completed roof structure transfers loads diagonally back to the columns.
- An adjustable jig stand was installed in the median strip of Wurundjeri Way to assemble the large roof modules for installation over the metropolitan lines. This jig had to replicate the changing geometry and the connection details of the spine trusses to which the roof modules would be landed. Again, this jig had to be uniquely developed for this roof. The jig was set at the spacing of the spine trusses where the roof modules were to be placed. The jig allowed for vertical adjustment to match the individual locations where the roof module would land in its final position. The roof modules were erected in the jig, but effectively floated in the jig, relying on the arch tie connected at the feet of each arch to maintain the horizontal spread of the arch. This ensured that once the roof module was removed from the jig on lifting, it did not spring apart or contract on lifting, thus maintaining its correct shape (spread on connections) which ensured a correct fit in its final position.

The two systems considered for the roof cladding were zinc and aluminium. Both met the long-term durability requirements for the station, and both were capable of being profiled to the unusually shaped roof. However, the support structure required by each system was totally different. Zinc sheeting, being a soft material, requires a fully supported surface in the form of a curved plywood surface. The ceiling panels for zinc sheeting would have therefore required a curved top surface. This could have been achieved by either curving the top surface of each individual ceiling panel (without visible steps at the joint of each panel) or by building the curved plywood surface in-situ after the ceiling panels were installed. The aluminium solution was a standing seam tray deck system, with traditional purlin supports at nominal spacings. Both these systems were trialled on the 8 metre by 8 metre prototype before the aluminium solution was selected.

The aluminium roof sheeting system adopted for the roof is a proprietary system called Kalzip. This system, supplied by Corus Bausysteme from Europe, was chosen for both its long-term durability and its flexibility in both design and installation to accommodate the continually changing shape of the Spencer Street wave roof in three directions. The support system of purlins was a series of circular steel purlins that snaked their way continuously along the entire length of the roof. More than 24 kilometres of uniquely shaped purlins in nominal 8 to 9 metre lengths were designed, drawn, fabricated and installed on the roof. The circular purlin was ideal for this roofing system as the clip for fixing the roof sheeting to the steelwork had to be preset for location and a correct angle in two directions. This entailed a complex survey set-out to orientate and place the clips prior to landing the roof sheeting.

The roof sheets were individually made. The width of each sheet between standing seams varies across the roof, as does the vertical curve, somewhat like the effect of peeling an orange. The profiling equipment for producing roof sheets was set up in an off-site facility. Flat coil was imported, then cut to length and shape, the standing seam was produced, before a taper was introduced and then finally a vertical curve. Each sheet was required to go through this four stage process. Sheets were then delivered to site and installed on the clips in their own unique position. A similar process was required for cladding the louvre caps, also in Kalzip. The system creates a unique finish for the outer skin of the station roof.

8 PHOTOGRAPHS

This section shows some of the main features of the truss roof and its connections.

Figure 1.   Erecting the spine truss.

Figure 2.   Insertion of an arch section.

Figure 3. Station in operation during construction.

Figure 4. Multiplanar tubular truss connection.

Figure 5. Tree columns supporting the roof.

Figure 6. Completed fully clad roof.

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João Havelange Olympic Stadium – Tubular arches for suspended roof

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ABSTRACT: This paper presents the basic concepts for the development of a Tubular Roof Structure for the João Havelange Olympic Stadium, which will be home of the 2007 Pan American Games in the city of Rio de Janeiro, Brazil, where the main design criteria were are safety and economy.

1 INTRODUCTION

1.1 *Pan American Games – 2007*

The city of Rio de Janeiro is known all over the world for the natural beauty of its beaches and mountains, together with hot climate and friendly people.

In 2007 Rio will host The Pan American Games. It is high the interest on this event since it is the first sport event of intercontinental interest that will take place in Rio de Janeiro second to the World Cup of soccer of 1950 which too place in another stadium, The Maracanã.

The roof project of the Olympic Stadium was developed with this objective of: bring the architectural and structural modernity to the world of sports.

As the budget to implement the necessary infrastructure for this event was small, this led to a detailed study of the project that had to be adapted to the available funds.

1.2 *Sequence*

This paper will be presented with emphasis on the following topics:

- Structural conception
- Construction methods
- Architectural and structural originality
- Respect to the environment
- Structural behavior – deflections and slenderness
- Better use of the materials and aesthetics.

2 STRUCTURAL CONCEPTION

2.1 *General data*

- Sport stadium with 45 000 capacity, meant to be the home of the Pan American Games – 2007 in the city of Rio de Janeiro, Brazil.

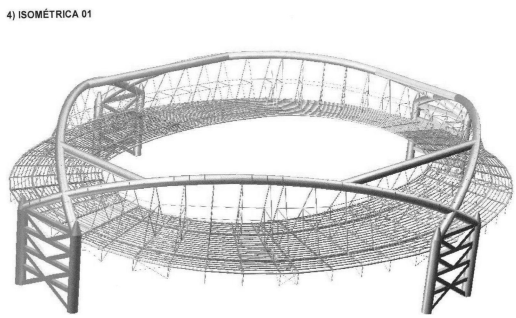

Figure 1. First version of the Arches.

- Structural Analysis: Computer Program: STRAP 12 Version
- Wind Tunnel: RWDI – Guelph city, Ontario, Canadá

2.2 *Historical*

The ideal geometric definition was one of the largest challenges in the development of the arches project. A first version was soon be discarded (Figure 1), because require to use tubes with very large diameters and therefore the structure's weight would be out of the usual parameters of weight per square meter.

A second version was developed with the use of arches composed of trusses of tubes with 500 mm diameter, however this version could not be adopted because of its serious geometric problems like the intersection of the arches (Figure 2).

2.3 *Final version*

- Finally the Structural/Architectural team decided, for lack of local qualified suppliers to adopt the following structural design
- The roof of the stadium would be composed of 42 trusses 50 meters long, 20 meters distant to each

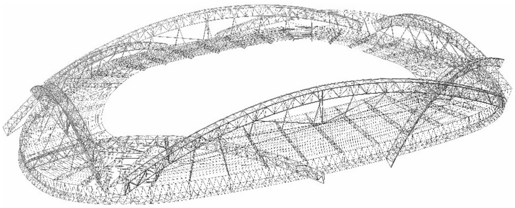

Figure 2. Second version of the Arches.

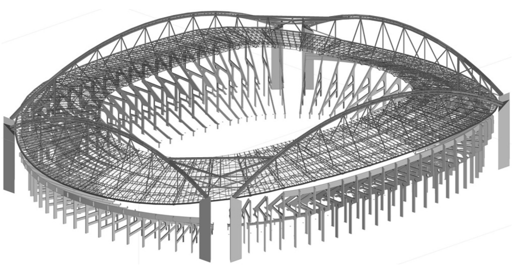

Figure 3. General view.

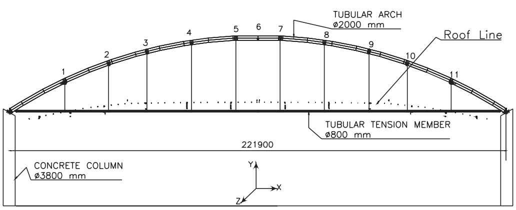

Figure 4. Arch No 1.

other and distributed in a radial pattern forming in plan a roof in the space of a ring with approximately 40 000 m^2.

This ring is supported by trusses, 4 great tubular arches.

ARCH No 1 – Formed by a 2000 mm diameter tube of variable thickness, with 221 meters span and 34 m high.

It Will be supported by 2 tubular concrete columns with 42 meters high, and 4,40 meters diameter.

ARCH No 2 – Formed by a 2000 mm diameter tube and variable thickness, with 163 meters span and 29 m high.

It Will be supported by 2 tubular concrete columns with 42 meters high, and 4,40 meters diameter.

2.4 *Structural behavior*

The Structure can be defined by four steel arched trusses with tubular sloped suspension hangers. The Tubular arches are supported by 8 massive concrete columns at the four corners of the Stadium. Beneath the arches are the egg shaped roof, suspended by steel hangers, 2 for each truss.

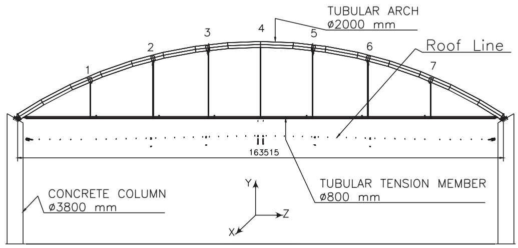

Figure 5. Arch No 2.

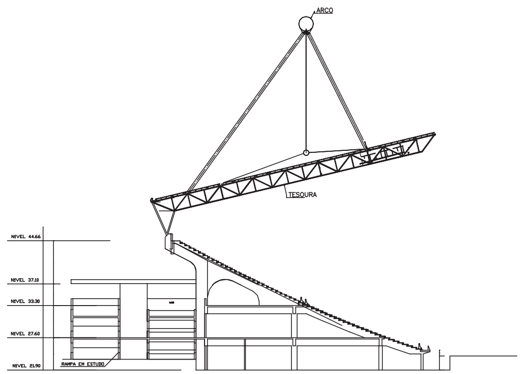

Figure 6. Typical section.

2.5 *Loads*

The Roof's project for Olympic Stadium took into account the proper distribution to the dead loads, live loads (0,25 KN/m^2), equipment, etc, thermal effect of the variation (+/−15° Celsius), 16 cases of wind effects, according to the results of the wind tests done by RWDI Engineering and Scientists (www.rwdi.com) in Canada.

They obtained $50 \times 4 = 200$ combinations which allowed static and dynamic analysis of the structure for the 1st and 2nd order.

3 CONSTRUCTIVE METHODS

3.1 *Fabrication shop*

The tubes will be formed and welded in a fabrication shop, where they will be dimensionally inspected and weld quality controlled.

After approved by the quality control inspector, they will be shipped to the construction site in 11 meter long segments.

3.2 *Field works*

At the site, the tube segments will be connected by welding, forming polygonal shapes 33 meters long.

The roof trusses will be raised to temporary supports until the final closure of the arch.

At the end of the erection of each arch and respective suspension hangers, connecting the roof, they will be loosened of the temporary supports. Control of deformations will be done in all stages of erection.

As soon as the structure has been stabilized, the roof tiles will be placed.

4 ARCHITECTURAL AND STRUCTURAL ORIGINALITY

Theatre of dreams, stage of emotions, these are the principles used in the development of the architectural project for the João Havelange Olympic Stadium, this monument to the sport will be prepared not only the sporting events of the Pan American Games, but also for official Games of soccer according to the regulations of FIFA (International Federation of Soccer).

The tubular arches will be reference for those who enter the city. In Brazil similar structures do not exist. The impact of the vision of the group of arches has for objective to give structural solidity to the notion and "I throw".

5 STRUCTURAL BEHAVIOR – DEFLECTIONS, SLENDER

5.1 *Structural model*

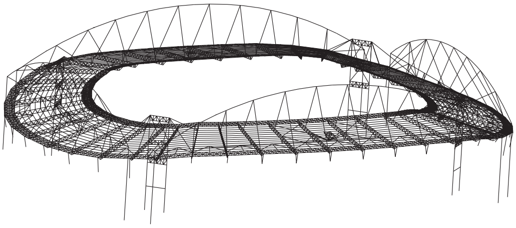

Figure 7. Structural model.

5729 nodes –
11 343 beams –
58 supports –
39 properties –

5.2 *Local buckling*

The instability modes for an axially compressed cylinder are mainly overall column buckling and local wall buckling.

In our case, the ratio of cylinder diameter to wall thickness, D/t is high, then the tubes can buckle in either as inelastic or an elastic shell buckling mode.

Elastic local buckling:
For the column buckling

$$\sigma xc = \frac{\pi^2 E}{(L/r)^2}$$

where:
L = is the length between ring stiffeners (20 m)

Inelastic Shell Buckling (Galambos 2004):
The inelastic buckling Stress of cylindrical shells is usually obtained in one of two ways.

Either the elastic formula is used with an effective modulus in place of the elastic modulus, or empirical relations are developed for specific classes of material.

The former approach is applicable only when the material stress-strain curves varies smoothly.

The slenderness parameters are usually either D/t or a nondimensional local buckling parameter, α, where:

$$\alpha = \frac{E/\sigma y}{D/t}$$

E = Modulus of elasticity
σy = Yield Strength (300 MPa)
D = External diameter of tube (mm)
t = Wall thickness (mm)

The structure of the arches was designed according API Code (API) which is recognized code worldwide.

The API Code (API) resulted as the best fit to a series of tests of fabricated cylinders with several strength levels, having the yield strength as part of the slenderness parameter in addition to α, where:

For $\alpha < \dfrac{E/\sigma y}{60}$

$$\sigma xc / \sigma y = 1{,}64 - 0{,}23/(\sigma y\ \alpha/E)^{0{,}25}$$

For $\alpha > \dfrac{E/\sigma y}{60}$

$$\sigma xc / \sigma y = 1$$

5.3 *Local buckling – bending (Galambos 2004)*

The buckling behavior of cylinders in bending differs from that of axially compressed cylinders in that bent cylinders have a stress gradient which is not present in axially cylinders and the cross section tends to ovalize.

The buckling behavior of cylinders in bending can be reasonably represented by a linear expression in terms of α for critical axial stresses.

For $\alpha > 14$ Mu/Mp = 1,0
For $\alpha < 14$ Mu/Mp = 0,775+0,016 α

Table 1. Critical stress in axially loaded steel shell.

D (mm)	t (mm)	α	$\sigma xc/\sigma y$
2000	19	6,65	0,903
2000	22	7,70	0,930
2000	25	8,75	0,952
800	19	16,63	1,000

Table 2. Critical moment capacity.

D (mm)	t (mm)	α	Mu/Mp
2000	19	6,65	0,880
2000	22	7,70	0,900
2000	25	8,75	0,920
800	19	16,63	1,000

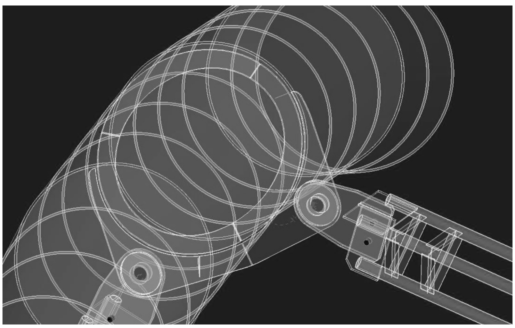

Figure 8. Design of the suspension ring.

5.4 *Imperfections*

For our design, imperfections are considered by specifying permissible out-of-roundness, using the API tolerances (Galambos 2004).

5.5 *Theoretical analysis*

For checking the design, models were developed using the method of the finite elements, for determination of the working stresses.

5.6 *Global Behavior and deflections*

The study of the deformations was accomplished in a meticulous way to allow a control of the serviceability limit state, tables were created containing the theoretical elastic and inelastic, 1st and 2nd order deformations in the vertical plans and horizontal to the arches.

The extensive data in the tables will be considered in the detailing drawings and in the production of the structural parts.

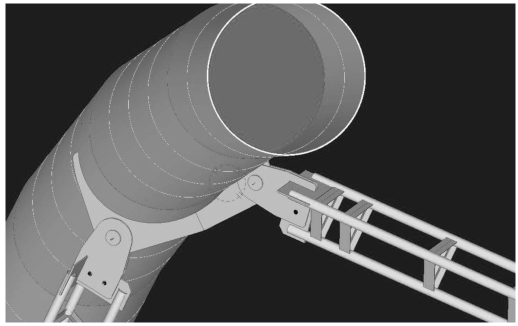

Figure 9. Detail of a suspension ring – Image.

Table 3. Theoretical vertical deflections – Arch 1.

	VERTICAL (mm)				
	41	5	6	7	8
DEAD LOAD	157	274	361	274	157
LIVE LOAD	70	95	123	95	70
TEMP +	−250	−277	294	−277	−250
TEMP −	250	277	294	277	250
WIND 1a	22	23	21	18	14
WIND 1b	26	44	50	41	39
WIND 2a	−12	−22	−26	−29	−27
WIND 2b	−35	−52	−65	−67	−57
WIND 3a	−33	−41	−48	−46	−41
WIND 3b	44	62	66	65	54
WIND 3c	62	96	142	114	111
WIND 4a	71	35	1	−30	−60
WIND 4b	76	70	40	−98	−115
WIND 4c	33	−30	−64	−85	−100
WIND 5	44	23	1	−25	−68
WIND 6	13	2	−12	−43	−46

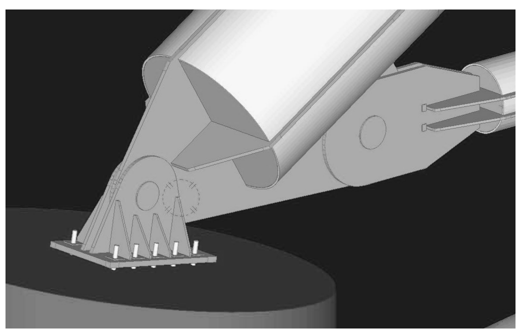

Figure 10. Detail of structural supports.

6 BETTER USE OF THE MATERIALS AND AESTHETICS

The use of circular tubes in the composition of arches is important, because through excellent structural

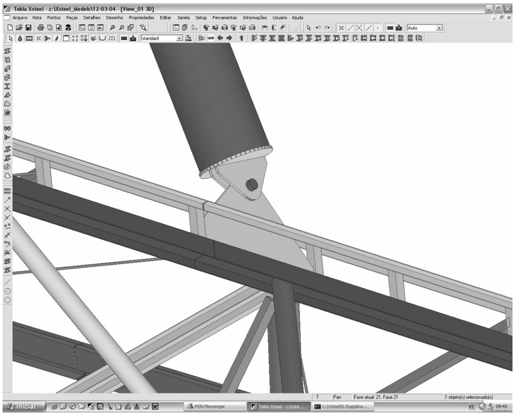

Figure 11. Detail connection for tension member x-suspension truss.

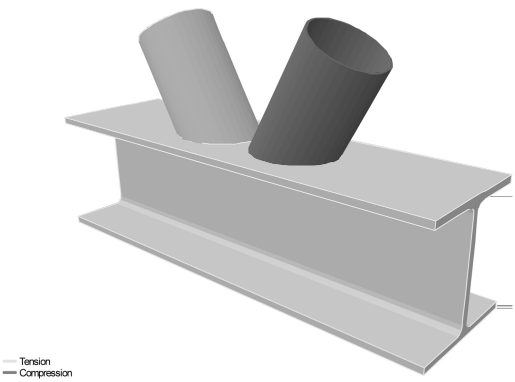

Figure 12. Detail connection for diagonal-chord x trusses.

characteristics they provide a modern and beautiful architectural aspect. In spite of Brazil not having tradition in major tubular structures, this solution was adopted for reasons of economy and beauty.

The connection details are individually identified allowing a correct structural assembly, easy maintenance and careful architectural finishing.

The choice of tubular steel structures considered easiness of handling the parts, speed of erection, less labor and ease interface with other materials.

7 ACTUAL STAGE OF CONSTRUCTION

Actually the construction is with the structure of concrete ready and manufacturing the pieces in steel, the assembly is foreseen to begin in May with end foreseen for December of 2006.

Figure 13. Image during the construction.

Table 4. Comparison between steel roof structures.

Stadium	Weight (ton)	Area (m^2)	W/A (kg/m^2)
Porto	4 050	36 000	112,5
Boavista	670	8 880	75,45
Aveiro	2 160	20 870	103,49
Coimbra	1 000	12 930	77,33
Leiria	2 600	20 700	125,6
Sporting	3 120	24 850	125,55
Faro	1 350	9 400	143,63
Benfica	4 620	29 380	157,25
EOJH-Brazil	3 700	39 000	94,87

8 CONCLUSION

After the sequence adopted is shown for the project of the João Havelange Olympic Stadium roof steel structure, we concluded that the analysis followed globally accepted concepts. However partly due to the favorable loading condition considered in Brazil (non-existence of snow, hurricanes or earthquakes) and mainly for the appropriate use of tubular structures, we achieved our objective of designing a light and economical structure.

Examples of similar structures built recently in Portugal are shown below.

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Roof of Santa Caterina market in Barcelona

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ABSTRACT: Santa Caterina market roof cover is a structure made of steel and wood. It is composed of a set of different wooden arches with straight and curved axis, supported by tubular steel beams of various cross-sections. These beams also hang from a set of three circular arches by means of a set of vertical hangers. The wooden arches are either tri-pinned or two-pinned. The steel arches are tied by cables which control the vertical deformations.

1 INTRODUCTION

The Institute of Barcelona Markets, controlled by the City council, is in a process of remodeling old markets around the city. Santa Caterina market is located in the centre of an old neighborhood that carries the same name.

The winning architect of the project, Enric Miralles, along with his wife Benedetta Tagliabue, conceived the roof as a curved and light surface that floated on a set of cables. Its outer surface is formed by ceramic mosaics that find their roots in the "trencadis" of Catalan architect Antoni Gaudí.

The initial idea materialized while the structure started to be conceived and the calculations were initiated (end of 1997). It was finally fixed as a set of wooden arches supported by steel beams, with these supported by three metallic arches through a set of hangers. The structure was completed in 2004, and the market opened to the public in May 2005. All the steel structure is made only of tubular sections and steel plates.

2 GENERAL DESCRIPTION OF THE STRUCTURE

The support at level zero consists of 13 columns:

Steel columns: Four curved tubular section columns curved are located in the main facade while two columns, the lower part made of concrete and the upper part of steel tubes, are located at the rear end of the building.

Concrete columns and beams: Two pre-stressed concrete beams are located on the sides of the building. One of 69 m long supported by 4 concrete columns, and another one of 30 m supported by 3 concrete columns of section 0.90 × 0.90 m.

V shaped beams: Taking advantage of the V shape of the zone at the lower edges of the roof, 6 steel beams were made to run the facade to the opposite end. They are distributed in the following way: The two outer beams are parallel to the market and the concrete beams, and the four other steel beams change in direction and bend to form a pair of rhombuses. Two beams begin from each of the central columns at the front. One of the two columns at the back of the structure has three beams and the other two.

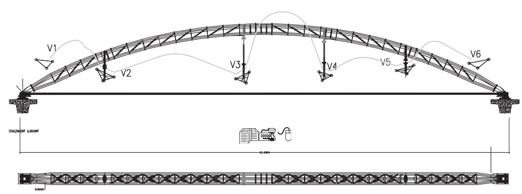

Figure 1. General cross-section of roof through the tubular steel arches.

Props: These are the set of bars that hold the roof uniting the far steel beams with the lateral concrete beams.

Wooden arches: The space between the steel beams is completed with wooden arches. The highest arch is tri-pinned and the lower arches are two-pinned.

Steel arches: Three arches have been designed in order to hold the four central beams by means of 12 hangers. The support points of each arch are tied at their base. The tension of these cables holds the arches firmly in place. These arches are unique in that they go in and out of the roof (see Figures 15 and 16).

3 ANALYSIS

The structure was analyzed by means of a three dimensional finite element model, using bar and shell elements. Twenty-six different load combinations were

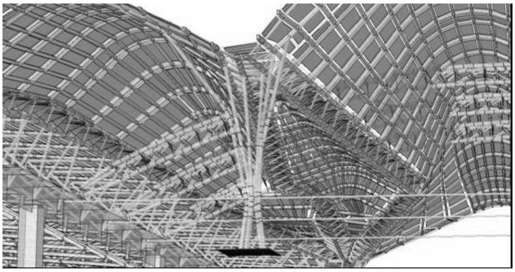

Figure 2. Numerical model.

taken into account: the wind load being the most disadvantageous in the analysis of the structure. The different thermal coefficients of wood and steel produced considerable stresses at the interface. The analysis of lateral buckling of the steel arches was difficult to achieve because horizontal loads, originating from roof movements, destabilize the arches if they do not have enough vertical load. Eurocodes have been used for loading hypotheses and analysis.

The stress distribution in the roof cover greatly depends on the way and the order that each of its components is placed during construction. If the steel beams are installed first, and are then released to start receiving loads, the wooden arches and the braces of the arches will only support loads near to them. If the cover is installed entirely and then released to start receiving loads, it will work as a whole set and it will behave as a big composite beam of steel and wood, where the braces and the cover (wood) work mainly in compression, while the steel beams will be working in tension.

4 THE CENTRAL STEEL ARCHES

4.1 *The arches*

The three arches are 42.78 m long, with a height of 6.02 m from the base of the arch (1/7 ratio), and a radius along its axis of 41.44 m (approximately 60° open). The section of the arch is composed of three tubes 219.1 mm in diameter and 25 mm thick, arranged in a shape like an equilateral triangle, with its base located at the top (see Figure 4).

The tubes form Howe-type trusses, in which the inclined members (tubes of 80 mm in diameter and 12 mm thick) are under compression and the vertical members, which are plates of 15 mm thickness, are under tension.

To prevent lateral buckling, calculations were made using classical formulae and non-linear analysis.

4.2 *Supports and cables*

The supports of the arches are “pin” type which allow rotation. One of them is a support guided in

Figure 3. The tubular arches and beams.

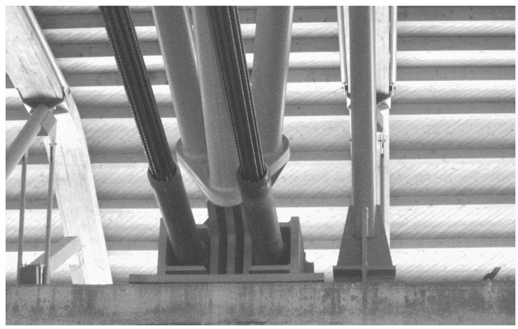

Figure 4. Arches supports.

the plane of the arch and the other one is restrained from translation. This way, the in-plane deformation of the arches does not affect the concrete beams horizontally.

A pair of tendons each one formed by 19 cables, of 15.24 mm diameter, resist the horizontal thrusts. The cable tension of 3000 kN (about, in service) controls vertical deflections of the arches, therefore, the beam’s deflections are controlled through hangers.

4.3 *The bracing*

The bracing between the arches has been solved by bridging members. These members (tubes) go from the centre of one triangle to the other, and when they reach the arch, the bar divides into two props, one directed to one upper tube and the other to the lower one (see Figure 6).

4.4 *The Hangers*

From the steel arches, 12 hangers descend (four per arch) and connect the arch to the steel beams, and work as a set of vertical elastic supports (see Figure 5). The

Figure 5. Connection of the hangers to the steel beams.

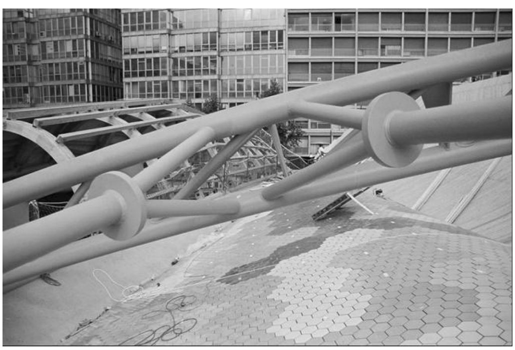

Figure 6. Local view of the bracing.

position of the hangers differs from one arch to another. Also, each one has a different length.

5 THE WOODEN ARCHES

5.1 *General description*

The curved shape of the cover is achieved by wooden arches. This wood is structural, and its cross-section

Figure 7. General view of the bracing.

Figure 8. General perspective under construction.

Figure 9. Steel beams.

is 200×400 mm as a flat beam. The arches are all different in length and shape. They are comprised, in general, of two straight sides and one curved part above flat aprons and one curved higher part. The arches are

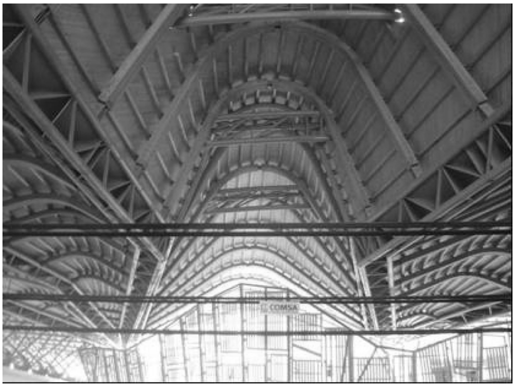

Figure 10. Steel beams.

Figure 11. Tubular steel beams. South facade.

articulated at the base for the purpose of not transmitting moments between the wood parts and the steel beams.

5.2 *Three-pinned arches*

The more inclined arches are three-pinned for the main purpose of releasing forces due to internal movement of the wood. Also, for construction, it was necessary

Figure 12. North facade.

Figure 13. Concrete beams and columns.

to have two independent pieces because of its length. The span varies between 6 m and 18 m and the height is between 3.80 m and 4.70 m from the base.

5.3 *Two-pinned arches*

The reduced arches are two-pinned to enhance their stiffness, because they form part of the central rhombuses that work like a great horizontal beam.

The maximum span is 10.94 m with a height of 1.69 m.

Figure 14. Concrete cantilever, tubular steel props and beam, and wooden arches.

Figure 15. General view of the Santa Caterina roof during construction.

6 THE STEEL BEAMS

The span is about 50 m but as the beams are curved their real length is 56 to 90 m. The cross-section of the beams is triangular. It is formed by three tubes of 219.1 mm diameter and of 25 mm thickness. The three tubes are joined by smaller pipes of 80 mm diameter × 12 mm thick, and triangular steel plates 20 mm wide. Every plate is different in shape. Buckling of the plates has been studied using non-linear analysis.

7 TUBULAR STEEL COLUMNS

These columns are shaped as bunches of independent steel tubes. For every load combination each tube works in a different way. Therefore the box needs to be

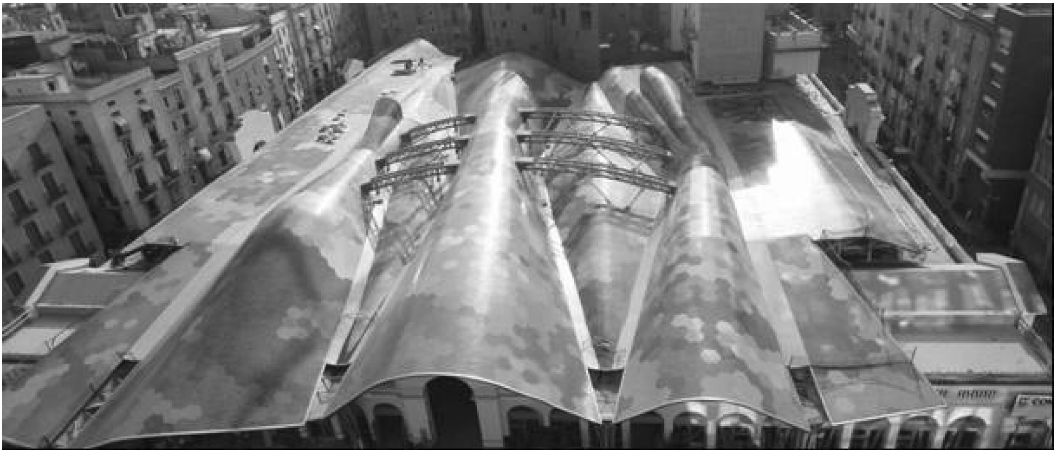

Figure 16. General view of the Santa Caterina roof.

studied carefully. The buckling has been studied using non-linear analysis. The geometry is complex and the connections have been studied in-depth.

8 CONCRETE BEAMS AND COLUMNS

Seven concrete square columns support two concrete beams (four columns for one beam and three columns for the other) that are perpendicular to the main entrance. The concrete beams support the big steel arches and the props then support the outer steel beams.

The concrete beams are T-shaped. Horizontal loads are basically supported by the flanges of the beams and the vertical loads by the whole beams.

The beams are post-tensioned and are 2.50 m wide × 1.20 m high. The flanges are 0.30 m wide and the webs are 0.90 m deep. The concrete beams are supported at the flanges, on roller supports. At each column there are two vertical supports to the flanges and two horizontal supports to the web of the beam. The torsional movement of the beam is prevented by the two vertical supports to the flanges with a span of 1.65 m. The lateral movement is prevented by the vertical placed supports. The longitudinal movement is free in each column except for one in each beam that is prevented from translation.

9 MATERIALS USED

The type of steel used is S355. The concrete used is C45/55 according to the Eurocode. The wood sections are type MC-30, (30 MPa).

10 CONCLUSIONS

- Tubular steel sections can be combined with wood to form suitable, with low weight, achieving complicated shapes and aesthetic forms.
- Wood and steel can work together but special care must be taken in the thermal analysis because of the different thermal coefficients of the materials used.
- The lateral buckling analysis of the arches is critical and should take into account horizontal loads perpendicular to the arches due to roof cover movements.

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Advantages of using tubular profiles for telecommunication structures

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ABSTRACT: The telecommunication industry is one of the fastest growing industries; consequently the telecommunication industry focuses on the towers for supporting the antennas. The focus is particularly on the total costs as well as production and erection time of the towers. However, aesthetics become more and more important since building permits become more difficult to achieve. Steel lattice towers are primarily produced of tubular or angular profiles. Compared to lattice towers of angular profiles the towers of tubular profiles have the advantages of lower wind resistance, increased buckling capacity and more aesthetic appearance. On the other hand, towers of angular profiles are easier to produce and demand less skilled people in the shop. Finally, angular profiles are less expensive per kilogram than tubular profiles. In practice the towers of tubular profiles are up to twice as expensive per kilogram as towers produced of angular profiles. However, towers of tubular profiles can compete with towers of angular profiles, but this calls for intelligent design and innovative solutions by the tower designer. This paper gives an overview of advantages and disadvantages of using tubular profiles compared to angular profiles as well as some practical examples on innovative connection details to reduce production costs. Furthermore some aspects regarding maintenance and problems regarding ice are mentioned. Finally, the difference between use of hot-rolled, cold-formed and welded profiles is discussed.

1 INTRODUCTION

The need for towers to support telecommunication antennas is increasing rapidly at the moment. The need for towers in Europe is big at the moment with the UMTS mobile communication system. Thus the need for towers is even bigger in the developing countries. This can be seen from the GSM world coverage (See figure 1).

The telecommunication industry focuses on the total cost and delivery time for the towers. This includes the manufacturing of the tower itself, but also foundation and erection, that all should be taken into account.

Figure 1. GSM world coverage (www.gsmworld.com).

Traditionally lattice towers have been produced of angular profiles, circular tubes or solid round bars.

In the very beginning, more than 100 years ago, the first steel lattice towers for telecommunications were produced of flat-sided profiles like the angular profiles since it was easy to produce and easy to assemble. However, some 50 years ago the first lattice towers were produced of tubular profiles and solid round bars in order to reduce the wind load and save material.

Nowadays towers are in most cases produced of tubular profiles in the northern part of Europe. In the UK and America the majority of the towers is however produced of angular profiles. The choice of structure is controlled by the options according to the national codes, manufacturing process but also traditions and innovations within the design.

In the following the angular profiles are considered as 90° angle profiles.

2 TOWERS FOR TELECOMMUNICATIONS

Towers for telecommunications are designed to withstand the wind load on antennas, cables, ladders etc. and on the structure itself. In some regions the towers are furthermore designed to withstand ice load and the combination of wind and ice load.

Since the towers carries antennas and often parabolas for microwave links the stiffness criteria is set up for the towers in order to be able to use the network under severe weather conditions. The stiffness criteria are often the design driver of the towers, especially when they carry parabolas and the height of the tower is more than 40 m.

The arrangement of the antennas is an important parameter in the design of the towers. The antennas are often arranged in one of the following two configurations:

- Road configuration covering two directions using two antenna directions.
- Normal configuration covering all directions using three antenna directions.

A triangular cross section enables the attachment of the antennas directly to the legs.

3 DESIGN BASIS

The basis of the design rules the design of the structures since the most cost efficient solutions are chosen.

Comparing the towers of angular profiles versus tubular profiles the following design parameters are important:

- Wind load
- Ice load
- Buckling capacity

3.1 *Wind load*

Apart from the wind load on the antennas, cables and other ancillaries, the lattice structure itself contributes significant to the wind resistance of the tower. The wind resistance of the flat-sided profiles such as angular profiles is larger than for the circular profiles. Consequently is the demand for the strength and the stiffness of the sections of the tower and the foundation dependent on the type of members.

The wind resistance of the lattice sections is dependent on various parameters: e.g. type of cross section, solidity ratio and type of members. The wind resistance is larger for square cross sections than for triangular cross sections. The drag coefficients for lattice bracing is decreasing for increasing solidity ratio in situations were the solidity ratio is moderate. Finally is the wind resistance for flat-sided profiles often up to 50 % larger that the circular profiles. For circular profiles the wind resistance is furthermore dependant on the wind speed – if the flow is supercritical of subcritical.

Figure 2 illustrates the drag coefficient dependent on the solidity ratio, type of cross section and profile. The values are based upon data from wind tunnel tests and are given in EC 3:Part 3-1 1997. For circular profiles the drag coefficient is dependent on the Reynolds number (proportional to the wind speed and the diameter) since the wind generates some turbulence around the cylinder which decreases the wind drag for larger circular profiles. Some codes like the old American TIA/EIA-222-F 1996 do not take this reduction of the wind load for circular profiles into account.

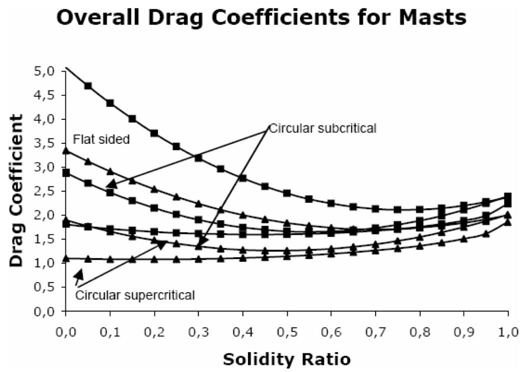

Figure 2. Drag coefficients for lattice triangular and square cross section.

3.2 *Ice load*

In some regions heavy ice load occurs on the structure and the dimensioning load can be the weight of the ice or the combination of ice load and wind load.

The weight per meter of the ice on a profile is dependent on the free surface area and since all the surfaces on angular profiles are exposed to ice load; the amount of ice on an angular profile is more than for the tubes. See figure 3.

The special considerations concerning ice load is further described in Nielsen, M.G. & Nielsen, S.O. 1998.

3.3 *Buckling capacity*

The design of the members in the bracing of lattice towers is normally controlled by their buckling capacity. Important parameters for the buckling capacity are radius of inertia, buckling length, eccentricity and the buckling curve.

When comparing a circular tube to a single angular profile with identical width and area of cross sections, the radius of inertia of a circular tube will typical be 10% larger than the radius of inertia about the strong axis of the angular profile and 70% larger compared to the radius about the weak axis. This result in a significant lower buckling capacity of the single angular profiles for the same distance between the bracing.

Furthermore are the diagonals for the sections with single angular profiles often eccentric loaded, which results in even lower buckling capacity.

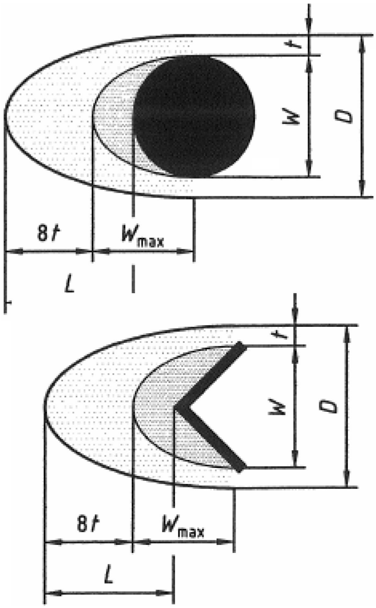

Figure 3. Ice accretion model for rime on circular and angular profiles (ISO 12492 2001).

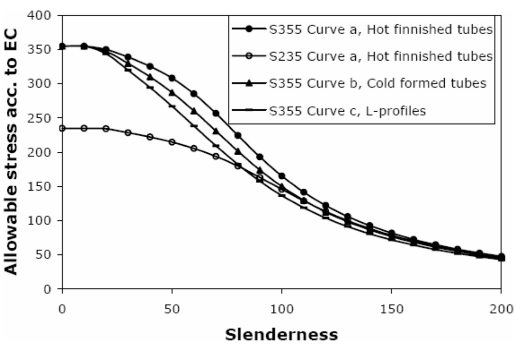

Figure 4. Buckling curves according to EC 3:Part 1-1 1992. The red curve is for tubes (hot finished) the light blue is for cold finished tubes, while the blue is for angular profiles and round bars. The green curve is hot finished tubes in steel quality S235.

The buckling curve according to EC 3:Part 1-1 1992 gives less critical stresses for angular profiles than the buckling curve used for hot rolled or even cold formed circular tubes. These results gives approx. 20% higher buckling capacity for the hot rolled circular profiles compared to the angular profiles for a typical slenderness of 60–120. The old American standard, TIA/EIA-222-F 1996, does not separate the buckling curves. Consequently it is more difficult for the towers with circular tubes to compete with the towers

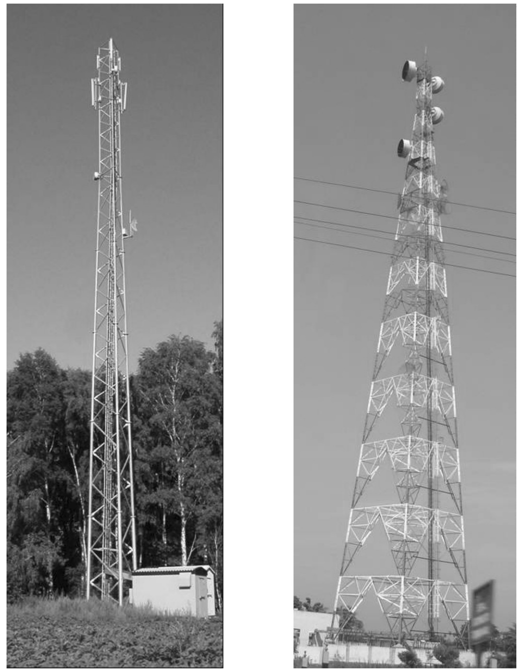

Figure 5. Standard towers of tubular (left) and angular profiles (right).

of angular profiles when the American standard is followed.

In order to meet the requirements laid down in the codes the design of towers of angular profiles demands more bracings and more members than for towers of circular profiles. This makes the towers of angular profiles more complicated to erect.

4 COMPARISON OF DIFFERENT STRUCTURAL DESIGNS

The production costs per kilo steel for the towers of angular profiles are relatively low since the angular profiles are cheep and the production of the joints is straightforward. Normally no welding is needed. The members can therefore be manufactured by automatic machinery in the industrialized countries or with less heavy machinery in developing countries.

However, the lattice towers of angular profiles are quite heavy and the number of bracing members is relative high. The lattice towers of circular tubes compete in this particular respect.

Figure 6 shows a number of different lattice towers of tubular profiles. Towers of circular tubes both as leg and diagonals members are often build using pattern no 1. However this pattern has some limitations if the tower is tall and has a strict rotation criterion. Since

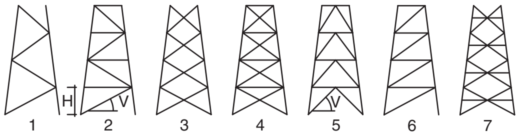

Figure 6. Typical lattice patterns (All diagonals can take compression as well as tension except pattern 4 that has special slender tension-only diagonal members).

Table 1. Comparison of different designs of lattice towers.

Type	I	II	III	IV	V
Cross section	Square	Square	Trian.	Trian.	Trian.
Leg members	Angle	Tube	Tube	Tube	Tube
Bracing configuration	Cross	Cross	Cross	V-bra.	V-bra.
Diagonal members*	Angle	S.tube	S.tube	S.tube	C.tube
Weight of steel (t)	3.6	2.9	2.7	2.4	2.0
Overturning moment, foundation (kNm)	810	670	590	530	420

*C.tube: Circular tube, S.tube: Square tube.

the most cost efficient solution in this case requires towers with a large face width. In this case the pattern no 3 has an advantage since it reduces the buckling length of the diagonals.

Table 1 from Støttrup-Andersen, U. 2000 shows the weight of steel and the overturning moment on the foundation for a typical 40 m lattice tower for mobile communications using five different layouts of the structure, where each design is optimized based on the different conditions.

The table shows that towers produced of circular tubes (Type V) are superior to angle profile constructions (Type I) with respect to weight of steel and overturning moment with potential savings of approximately 45%.

Furthermore, the tower of angular profiles (Type I) is more visible than the triangular tower of circular tubes (Type V) since the solidity of the tower is greater and the tower will look more massive with more and wider bracings. This is a disadvantage for the tower of angular profiles since aesthetics become more and more important in order to get the building permits.

5 DETAILS

In order to reduce the delivery costs it is not only important to reduce the costs of the raw material but also the costs of manufacturing. For the towers produced of angular profiles the costs of the manufacturing are rather low since normally the joints consists

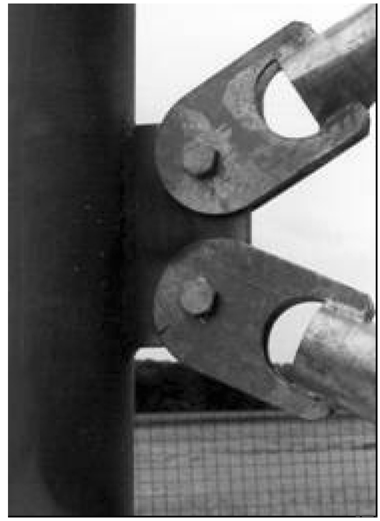

Figure 7. Traditional joint between diagonal and legs, both circular tubes (ONE, Austria).

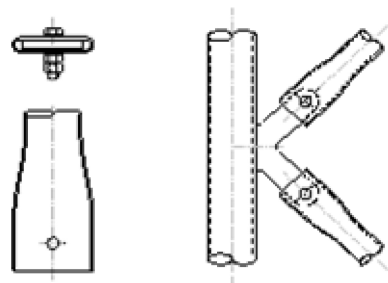

Figure 8. Special design of the joint without using welding on the diagonals.

of bolts and plates and no welding is included. The joints for lattice sections of circular towers are traditionally more complicated and time consuming. As an example hereof the bolted joint between the leg and diagonals is mentioned. This has traditionally been rather complicated with gusset plates welding etc. as shown in Figure 7 from a series of standard towers to Connect-Austria. However the towers were very competitive and more than 800 of these towers were delivered within a short period.

5.1 *Diagonals of circular tubes without welding*

Ramboll has introduced joints were the amount of welding is reduced: the ends of the circular tubes are squeezed together on a plate in order to have two plane faces in the ends of the diagonal.

The principle of the joint is shown in figure 8.

Figure 9. New joint between diagonal and leg, both circular tubes. BASE, Belgium.

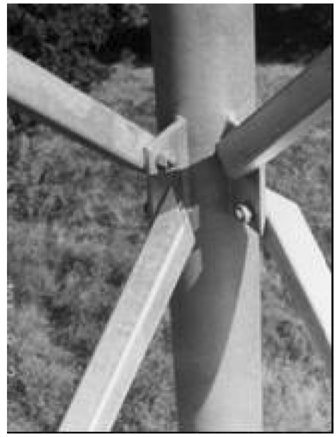

Figure 11. Eccentric joint between diagonal and leg, PANNON, Hungary.

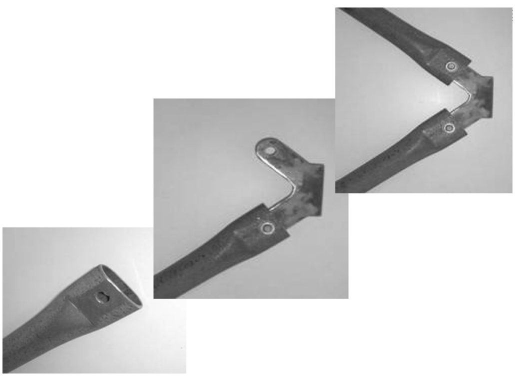

Figure 10. Principles of the assembly of the diagonals.

Figure 12. Joint in the intersection between cross bracing, PANNON, Hungary.

The diagonal is accordingly bolted to a special designed gusset plate welded to the legs as shown in figure 9.

The principle of the joint is more illustrative shown in figure 10. The gusset plate is prepared in a way that is possible to exchange the diagonals.

5.2 *Diagonals of square tubes without welding*

Another type of joint is used between the cross bracing of square tubes and circular leg members as shown in figure 11 and figure 12. Here no welding is used for the square tubes; only drilling and a saw cut under an angle of approximate 45° is needed. Eventhough this method brings eccentricity into the diagonals it has some clear advantages compared to angular profiles.

6 ERECTION OF TOWERS

The towers of tubes consist typically of less number of elements than the corresponding towers of angular profiles. The tower of angular profiles often with a square cross section and a large number of secondary bracing, whereas the towers of tubular members often has a triangular cross section and do not need secondary bracing. This implies that the number of elements in the tower of angular profiles often has the double or three times as many elements.

This makes the logistics more complicated and the erection more time consuming for the tower of angular profiles compared with the tower of tubular profiles.

However the towers of angular profiles have the advantage during transportation that the needed space is significantly lower for the tower of angular profiles compared with the tower of tubular profiles – however this does not always imply lower costs.

Figure 13. Erection of a tower of tubular elements in Pakistan using simple methods.

7 SPECIAL PROBLEMS FOR TUBULAR PROFILES

In some designs the designers have chosen not to protect the inner surface of the tubes but just closed the profiles e.g. with welded flanges etc. In case of weld porosities air will come into the tube from the exterior and water will condense in the tube. Apart from the corrosion problem the water can break the tube wall when it freezes and expands. This has been seen quite often and consequently all tubular structures should be equipped with holes for draining. It is especially important to drain the leg members of circular tubes in the bottom at the foundation, since here is a risk to collect water from the leg members in all the sections above. The hole for draining should as a minimum be Ø15 mm in order not to be blocked by leaves, dirt etc.

The inner surfaces of tubular profiles have to be protected against corrosion, but the inner surface is not as exposed to corrosion as the outside even if it is open in both ends. Hot dip galvanizing will be the best solution as protection against corrosion, since it is unrealistic to protect the inner surface by painting.

When the above mentioned special precautions are taken into account, experience has shown that lattice towers of circular tubes are maintenance free and can fully compete with towers of angular profiles.

8 CONCLUSIONS

The design of the structures is governed by the basis of the design rules since the most cost efficient solutions are chosen.

Lattice towers of tubular profiles have many advantages compared to towers of angular profiles, since the wind resistance is lower, the weight of steel is less and the overturning moment is less. These advantages can fully counterbalance the less expensive angular profiles; especially if the designer focus on the minimizing the labor cost for the manufacturing of the joints.

However some codes like the old American does not take due account to the decreased wind load and increased buckling capacity of the circular profiles compared to angular profiles. This has challenged the designer to make even better designs in order to be able to compete with the lattice towers of angular profiles. The new American code has diminished these advantages.

Our experiences show that lattice towers of tubular profiles are very competitive with towers of angular profiles.

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Survey of support structures for offshore wind turbines

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ABSTRACT: This paper aims to provide an overview of the state-of-the-art in offshore wind turbines, including an introduction to general design principles and the foundations. For use in shallow water, gravity foundations or buckets (caissons) are currently used, whereas monopiles are being applied for medium depth water. It is foreseen that future larger wind turbines (say 10 MW nominal power output, 160 m rotor diameter) in deeper water will be based on a tripod foundation. For very deep waters, such as around Japan, floating wind turbines are also being considered.

1 INTRODUCTION

1.1 *Wind energy*

Wind energy is quickly growing to the point where it is becoming a measurable contribution in the "war on global warming". Shell expects that by the year 2050 one third of all required energy will come from sustainable energy (Shell 2001).

After the first boost in the eighties in California, the focus of attention shifted to Europe, where the need for a more durable form of energy was felt early on. Here, research projects and positive tax climates resulted in the predominant position of Europe in the wind energy sector today and a thriving industry with a market share of about 90%. The "old" EU (15 countries, excluding the new members in East Europe) aim to have over 20% of the electricity production from renewable sources as early as the year 2010 (European Parliament 2001). A large part of that production will have to come from (on- and offshore) wind energy.

It is only recently that countries and companies outside of Europe are beginning to realise the potential. But nowadays an amazingly broad number of countries, including even countries like Iran, are participating. This can be seen in Figure 1, which shows wind turbines placed in a valley which acts as a giant wind funnel. Also in the USA (especially General Electric), India and China have started programs, companies and research projects to kick-start their wind industry, often with an amazing degree of aplomb.

Figure 1. Wind turbines in Iran.

1.2 *Offshore wind energy*

Offshore wind has been used by mankind as a prime energy source for a very long time – from the first sailing boats onwards. In the case of wind turbines, the drive towards the sea has two major causes. Firstly, the need to minimize negative effects, such as the impact on the horizon and, in the case of close proximity, shade and sometimes noise. This is especially the case in densely populated areas, where the need for electricity is largest. Secondly, the availability and quality (lack of disturbance) of offshore wind is better than on most onshore locations.

Since wind is not constantly available and wind turbines are stopped for maintenance and due to failures, the actual power output of a wind turbine is only a part of the rated nominal output (the electricity production under optimal conditions). In fact the actual power generation offshore is about 1/3 of the nominal power output. However, this compares quite favourably to onshore wind electricity production, where the actual output is about 1/4 of the nominal power output.

As a result, an important part of the future wind energy which Europe is aiming at can be expected to be generated offshore. This poses a new major opportunity for the offshore industry worldwide.

The current applications are for wind turbines with a diameter of about 90 to over 120 m and a rated nominal power output of 2.5 to 5 MW and over, per wind turbine. The current turbines are typically positioned in shallow (up to 5 m) to medium (5–20 m) water depth, at distances of 5 to 10 km from the shore.

It is worth noting that, unlike oil production platforms, wind turbines can be placed anywhere that sufficient wind exists and there is no necessity for placement in deep waters.

Although it would seem that the possibilities for placing wind parks are plentiful, in practice the available area is considerably diminished by shipping routes, fishing grounds, military zones and ecologically vulnerable areas (such as the Waddenzee in the north of The Netherlands), to the point where some wind parks will be placed in less favourable areas which are further from the shore (cabling), in deeper water (foundation and support structure) or at less favourable wind locations (lower electricity production).

1.3 *Cost of offshore wind energy*

The realisation of offshore wind parks is inherently more expensive than for onshore conditions for a number of reasons:

- Long cables are often necessary to connect the turbines to the onshore electricity grid. The cost of electricity cables can be 20 to 30% of the total costs of offshore wind energy and thus cabling is an important factor in the positioning of wind turbines.
- Installation and maintenance have to be carried out at sea which is quite expensive.
- In rougher offshore areas, such as the North Sea, larger maintenance campaigns are only possible for 6–7 months per year. Larger maintenance problems developing outside this period can result in several months of lost electricity production.
- The foundation and support, discussed in detail in Part 6 takes up also about 15% of the overall cost.

Altogether, the contribution to the electricity costs of the foundation, cabling, installation and operation & maintenance are about as high as that of the tower and turbine. These additional costs are the major reason for the higher cost of offshore wind energy, outweighing the effect of the better availability of the wind.

2 THE DESIGN PROCESS OF AN OFFSHORE WIND TURBINE

General structural design documents contain a number of basic elements. Slightly adapted towards wind turbines, these could be summed up as:

- A description of the project, containing aspects such as locations and other aspects affecting the design:
  - The overall capacity of the wind park
  - The general location
  - Number and position of wind turbines
  - Identification of nearby fishing grounds or ports, etc.
- An identification of design codes and standards to be used. In the case of wind turbines, a lot of codes deal with the load side of the design
  - Wind
  - Waves
  - Current
  - Control modes, such as for instance grid failure.

A few examples of relevant standards for the design of wind turbines are:

- IEC 2006
- GL 2004
- DNV 2004

The design process can be schematised as follows:

- Concept study, in which the main design parameters like diameter, installed power, control strategy, foundation type, etc. are used to define the basic design loads
- Pre-design phase in which the design is checked more carefully, now including the effects of wave loads, etc.
- Detailed design phase: all parts are designed in detail. Checks are run for both static (ultimate) and fatigue loads, the design is adapted and rechecked until a satisfactory design has been reached

3 WIND LOADING

Obviously, wind loading tends to be a fairly major issue wherever wind turbines are placed. For the tower and the support structure, the main wind loading on a wind turbine can be rather easily characterized by:

$$F = \tfrac{1}{2} \cdot \rho \cdot v^2 \cdot \pi \cdot r^2 \cdot C_t \tag{1}$$

Where :
$\rho =$ density of air: 1.225 kg/m^3
$v =$ wind speed in m/s
$r =$ the radius of the rotor in m
$C_t =$ drag coefficient

The upper limit value of C_t can be taken as 1.0 in certain circumstances, depending on the mode of the