STEEL News & Notes
SINGAPORE STRUCTURAL STEEL SOCIETY

Contents

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The President's Corner

The Singapore Structural Steel Society will be celebrating our 30th anniversary this year. We have travelled a long way to where we are now. Membership have grown over the years and we are proud to note the number of corporate members has exceeded 100, and for individual members beyond 300. Together, we have successfully contributed to the completion of numerous structural steel projects that our members are involved in. Indeed an achievement that we should be proud of.

The Structural Steel Fabricators Accreditation Scheme (SSFAS) has gained recognition by industry players as the firms under our scheme have upgraded the skills of its workforce as well as investing in machines and facilities to boost their productivity so as to take on bigger jobs. Over the years, the society has accredited more than 100 steel fabricators under this scheme.

The structured training courses for Structural Steel Supervisors and Structural Steel Engineers provide a good training ground for the practitioners with the necessary skills to supervise structural steelwork and in assisting firms to meet the skilled personnel requirements in the SSFAS. These courses also attracted other industry personnel to improve their knowledge in structural steel construction, thereby enhanced their capabilities and skills to supervise these works. To date, there are more than 1500 practitioners having gone through these two courses.

The introduction of the Structural Eurocodes for Singapore to replace the existing British Standard design codes will benefit structural steel design for building and construction. With BCA setting the migration to the Eurocodes by April 2015, the society and BCA Academy jointly organised EC3 & EC4 courses for the benefit of the building and construction personnel. These lectures were conducted by Professors and well qualified structural steel practitioners, who are members of the SSSS. We would like to thank them for their efforts.

Our yearly scholarship fund raising golf tournament to fund needy and outstanding students to pursue their undergraduate and polytechnic studies have benefitted many students. This is done through the generous contributions from members of the society.

To recognise the creative and innovative projects of our members and the building construction industry, the society regularly organises the Structural Steel Excellence Awards competition to showcase a selection of their best projects. These competitions are well participated and due recognitions are accorded to deserving entries and awarded with uniquely designed trophies. The Structural Steel Excellence Awards were previously held in 2002, 2004, 2007, 2009, 2010 and 2012. This year the award ceremony will be held on 28 August 2014.

We also provide a platform for members to network with fellow members and for them to experience knowledge related to structural steel construction. These are done through regularly organised site visits, evening talks, members' and corporate members' nights, seminars and conferences. The success of these events required the active participation of members. We thank our members for their overwhelming support.

Moving forward, the society will continue to pursue areas of interest with key players in the building and construction industry and will engage in activities that would bring benefit to our society. With your continuous participation, we will continue to look forward to a better future for all of us.

Anthony Tan
President, Singapore Structural Steel Society

SSSS New Corporate Members as of 10 July 2014

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Chin Yuan Metal Pte Ltd	18 Tuas Street, Singapore 638455 Phone: 6265 0550 Fax: 6262 0550	12-Apr-2012	Corporate	CM-207
Steeltech's Engineering Contractors Pte Ltd	50 Serangoon North Ave 4 #06-16, First Centre, Singapore 555856 Phone: 6854 3774 Fax: 6570 1006	10-May-2012	Corporate	CM-208
CCECC Singapore Pte Ltd	21 Bukit Batok Crescent #23-80, WCEGA Tower, Singapore 658065 Phone: 6324 5152 Fax: 6324 6242	29-May-2012	Corporate	CM-209
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Platec Engineering Pte Ltd	No. 8A Tuas Avenue 13 Singapore 638981 Phone: 68613966 Fax: 68613969	07-Mar-2014	Corporate	CM-233
Everbright Metal Works	No. 11 Sungei Kadut Street 5 Singapore 728957 Phone: 6368 1145 Fax : 6368 1146	06-May-2014	Corporate	CM-234
AVEVA Pte Ltd	16 Collyer Quay #18-00, S'pore 049318 Phone: 6818 9960 Fax: 6818 9041	06-May-2014	Corporate	CM-235
XINHE Engineering Pte Ltd	280 Woodlands Industrial Park E5 #08-43, Harvest @ Woodlands Singapore 757322. Phone: 6339 0060 Fax: 6339 0070	05-Jun-2014	Corporate	CM-236
HIROSE (Singapore) Pte Ltd	200 Cantonment Road #06-06/07, Southpoint, Singapore 089763 Phone: 6225 8655 Fax: 6222 3008	10-Jul-2014	Corporate	CM-237
Shandong Hangxiao Steel Structure Co. Ltd	No. 8 Yunxi Road P.R. China 266300. Phone: 86 0532 87273788	10-Jul-2014	Corporate	CM-238

London King's Cross Station - The Western Concourse Roof

Authors: Mike King and Alex Reddihough

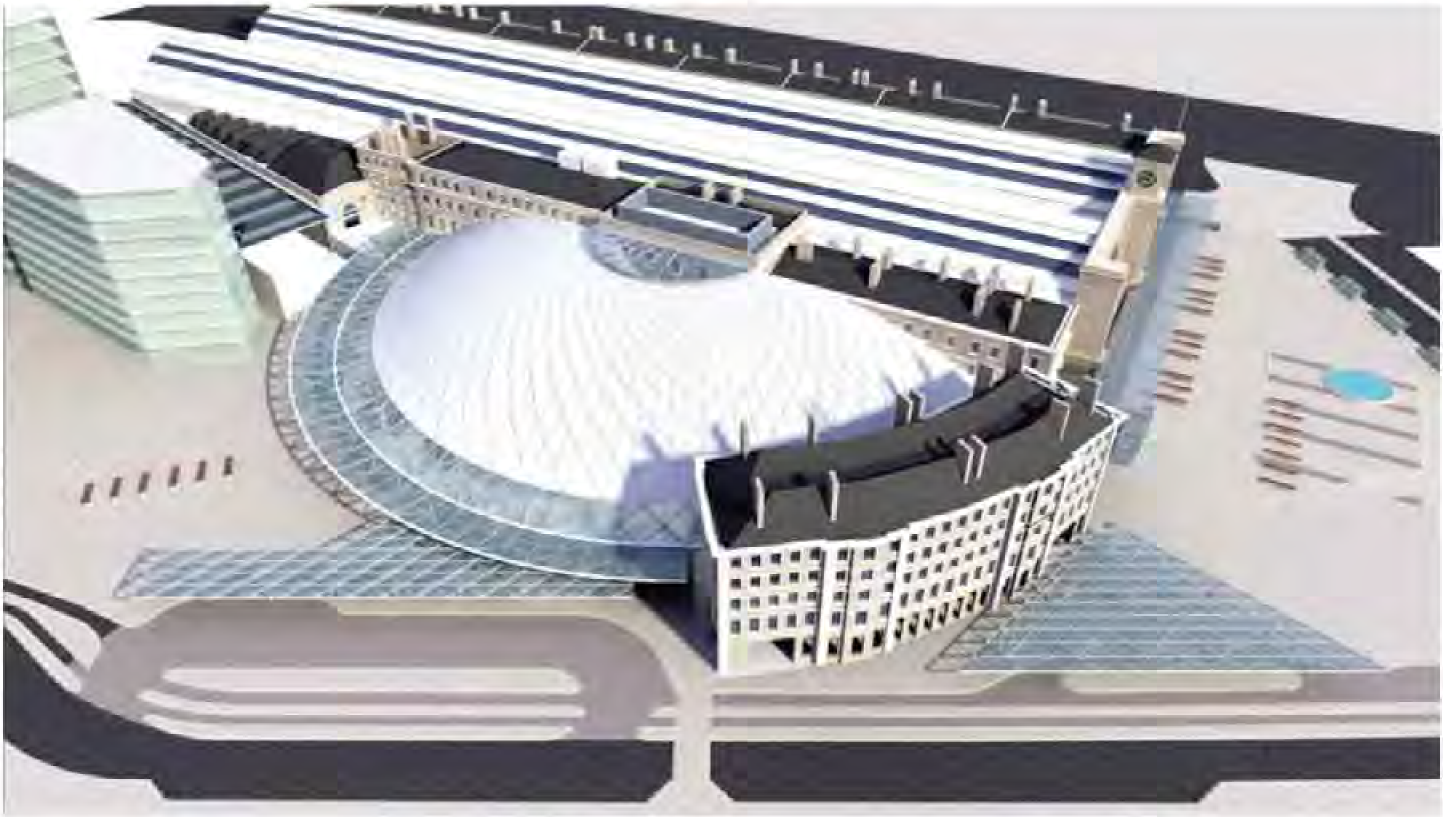

Figure 1

Drivers of the design

The roof design evolved through seamless collaboration between JMP and Arup, driven by the need to work within and respond to the following constraints and challenges:

- create a long-span structure that would bridge fully over the London Underground northern ticket hall "box", already under construction at the time the Western Concourse roof was being designed
- develop an efficient and elegant structural scheme that did not apply any loads to the Grade I listed Western Range façade, and would also fit within the curved form of the Grade II listed Great Northern Hotel (Figs 1 - 2)
- create an architecturally welcoming space that was also visually and operationally unifying, forming a hub to serve both the suburban and mainline intercity platforms, which had always been disconnected. The semicircular plan thus created aids pedestrian flow between these two parts of the station as well as being a generous space for people waiting for their trains or arriving passengers.

Evolution of the design

The team worked together for many months, though the light, dynamic diagrid shell form came together relatively quickly. As well as the obvious architectural benefits of the semicircular plan geometry in terms of pedestrian and passenger flow, there were also great engineering advantages.

Key among these was that, as well as creating a thin shell structure, the doubly-curved S-shaped section (Fig 3)and semicircular plan form act to carry most of the roof load away from the WR façade and support it at the perimeter. Ideally, for structural efficiency, such a shell roof would form a complete circle, but the functional and geometrical constraints imposed by the presence of the existing buildings required it to be cut into a semicircle at the WR façade.

This meant that a structurally stiff edge to the cut shell was required where it abuts the WR façade. This was achieved through deep vertical truss elements, also glazed to enclose the building envelope and enable views from the Western Concourse to the WR façade (Fig 4).

Figure 2

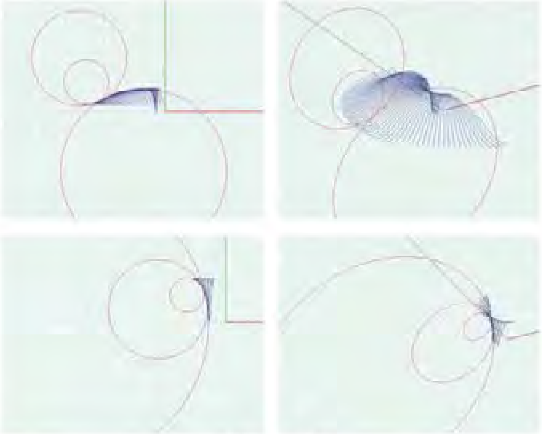

Figure 3

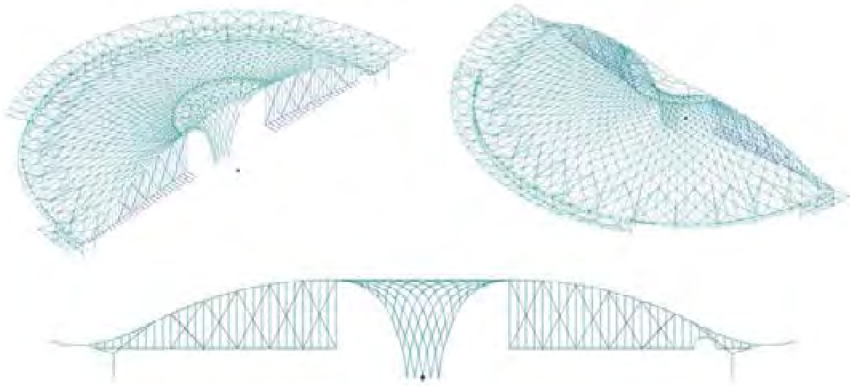

Figure 4

Figure 5

1. Architect's design concept for Western Concourse roof.

2. The original taxi rank and parcels yard alongside the Western Range buildings became the site of the Western Concourse. The Great Northern Hotel can be seen to the right.

3. How the diagrid shell geometry is created by three tangential circles rotated about a vertical axis (a, b); how the funnel geometry is created by three tangential circles rotated about a vertical axis (c, d).

4. Analysis model: (a) isometric of concourse roof, showing cut edge and funnel; (b) isometric showing opposite aspect of concourse roof; (c) cut edge of the semicircular shell.

The funnel

The central support to these trusses, and to the semicircular skylight above, is arguably the most dramatic structural and architectural element of the roof structure. The "funnel" was developed in response to the challenge to create an efficient structural support at the centre of the roof as well as a strong architectural focal point.

It is easy to see it becoming a symbol of King's Cross Station, and is such a natural meeting point that anyone meeting someone at the station will almost certainly use the words "*...I'll meet you under the funnel at King's Cross*" (Figs 5-6).

Its structure is a natural extension of the diagrid shell form, curving from the horizontal diagrid at the edge of the roof skylight to near-vertical at the support at ground level. As the funnel structure is doubly-curved, it has strong resistance to out-of-plane buckling, enabling the use of relatively slender tubular steel sections.

The client required the steel tonnage to be benchmarked against other long-span roof structures so as to be satisfied that, as well as this being an iconic structure, the team was also delivering an efficient structural system.

Geometry and structural elements

The entire roof diagrid geometry and funnel form were developed and finalised through "sculpting" in Arup's 3-D structural analysis software, *GSA*. Conceptually, the roof structure is divisible into radial rib elements (primarily bending forces) and a diagrid (largely in-plane shell forces) (Fig 7). The former are fabricated as boxes to produce a more efficient section for bending and to visually distinguish them from the diagrid tubes, which are conversely optimised for axial loads.

The fabricated box rib radial sections are typically 150mm wide, varying from 250mm-450mm in depth in line with the changing bending moments. The diagrid tubes are standard circular hollow sections, varying between 139mm and 219mm in diameter.

Figure 6

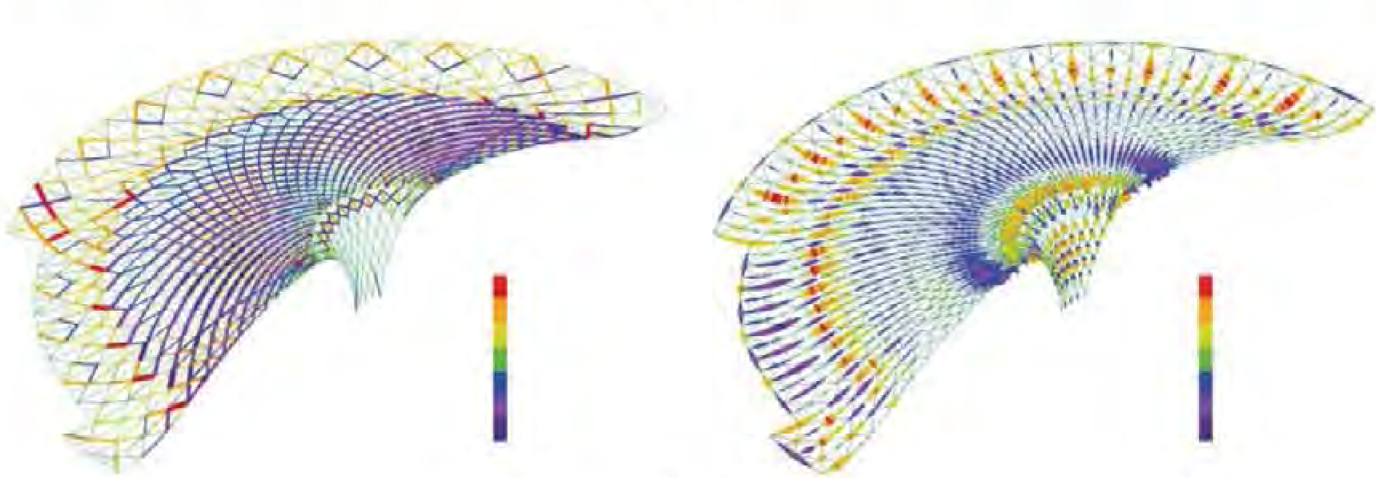

Figure 7

Figure 8

5. The funnel form.
6. The overall diameter of the roof is 138m: the north-south span of the roof is the longest at any railway station in Europe.
7. The diagrid shell axial forces (a) and bending forces (b).
8. Funnel element being fabricated.

The "tree trunk" columns at the bottom of the funnel need to resist large net lateral thrusts from the branch struts supporting the roof, thus enabling shell action in the roof diagrid. These forces are up to 600 tonnes at the top of the columns in the radial direction, and produce considerable bending moments in the column itself and large overturning loads at the baseplate. The restraint forces in the minor axis (circumferential direction) are more modest – "only" 90 tonnes in the horizontal direction.

The carefully-shaped tapered ovoid section makes these columns look deceptively slender for the forces carried. At the base, a typical tree column is 1.4m on the longer axis and 0.6m wide, skilfully fabricated from large CHS sections connected by curved plate. The baseplates measure 1.8m x1.0m.

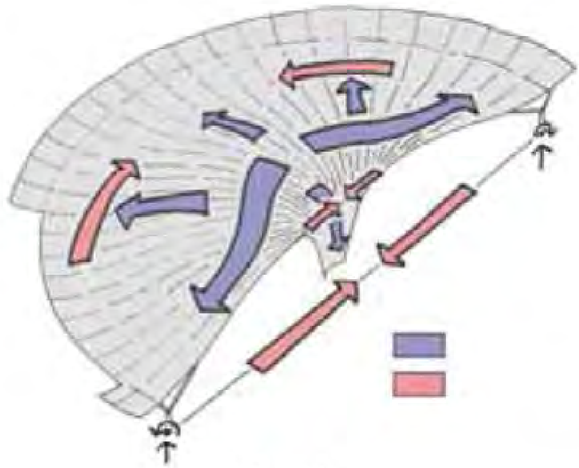

Figure 9

Figure 10

Figure 11

The branches are pin-ended at the connection to the diagrid shell to allow the roof to articulate, and avoid bending forces being transferred from the diagrid radial members into the branches themselves.

All but two of the tree columns are identical. Two "super-tree" columns with only two forward-facing branches, carrying significantly larger forces than the typical case, stand 114.7m apart on opposite sides of the funnel, and provide the edge restraint to the shell adjacent to the existing WR building. The super-tree columns are larger – 1.9m x 0.65m maximum dimension, with a 3.35m long baseplate – and each is 54.6m from the closest point on the funnel.

Some of the trees are carried directly on dedicated concrete bored pile foundations, while others are supported on the basement concrete box structure of the London Underground Northern Ticket Hall. These foundations were all built as part of the London Underground works, also designed by Arup.

Figure 12

The long span, wider across the WR building façade than any other railway station in Europe, presented several structural challenges. A key part of the analysis involved checking for global and local buckling of the elements under the very high loads. This was also carried out using GSA, in combination with a custom-built automated spreadsheet which analysed every element of the roof under around 100 separate load combinations.

Connections

The connections between the tree column branches and trunk have to transfer significant forces from several directions down to the foundations, and are among the largest and most visible parts of the concourse roof structure.

It was decided that a solid cast "node" (Figs 10-11), sculpted to smoothly transition the geometry between branches and trunk, was the best solution, though it is not something often found on such a scale in a modern building. A 3-D finite element of each node was analysed to optimise the plate thickness and geometry within the constraints of the casting process.

The detailed design of the roof required close collaboration with the architect, as all the structure is fully exposed. No bolts are visible from the underside, as all connections are hidden within the structural members themselves.

The constantly changing geometry of the roof required careful grouping of connection types to give some uniformity to the connection design while still achieving an efficient and lightweight roof. The overall result is a very clean structure, with no interruptions to the curving geometry of the diagrid (Fig 12).

9. Global force diagram.
10. Perimeter "tree" columns, 12m apart and 52.1m from the centre of the funnel.
11. Solid cast node, showing lifting eyes.
12. All connections are hidden within the structure.

Figure 13

Figure 14

Figure 15

Construction

The construction of the roof on such an extremely constrained site while maintaining station operations required careful planning and sequencing. A huge scaffold was erected, onto which the prefabricated "ladders" of the roof structure were dropped in and connected in situ. Once the shell was complete, the scaffold was gradually removed to let the roof settle under its own weight; the recorded deflections all within the limits predicted by the analysis model.

Reference

(1) EVANS, P *et al.* Super subterranean hub: updating King's Cross St Pancras. *ICE Proceedings: Civil Engineering, 164*(CE2), pp73-80, May 2011.

13. Prefabricated roof structure being placed on scaffolding.
14. The complete roof structure in place.
15. Placing roof cladding elements.
16. The completed Western Concourse just prior to opening.

Figure 16

Impact of Structural Eurocodes on Steel Structures

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The introduction of Structural Eurocodes to replace the existing British Standard design codes will benefit the construction industry in the long run. However, in the short term, to help our engineers migrate over to the new Eurocodes, it is useful to conduct a comparative study on both Eurocodes and British Standards to identify some of the key changes and impact on the performance of the steel structures. In this connection, this paper investigates three key design aspects comprising flexural buckling resistance of compression members with Grade S460 steel, shear buckling resistance of plated structures, web bearing and buckling resistance of the plated structures under transverse force. It is found that the flexural buckling resistance, web bearing and buckling resistance determined from Eurocodes 3 are much larger than those from BS5950. However, for shear buckling resistance of plated structures, Eurocode 3 produced a lower value compared to BS5950.

Keywords: Eurocodes, BS5950, flexural buckling resistance, shear buckling resistance, web bearing and buckling resistance.

1. Introduction

Eurocodes are a set of harmonized technical design rules established by the European Committee for the standardization of design of structures and structural components in the European Union. The main objective of drawing up these set of harmonized technical specifications for building and civil engineering works is to eliminate technical barriers and to promote trades across member countries. Each Eurocode contains provisions which are open for national choice. Such provisions contain country specific data on those parameters, known as Nationally Determined Parameters, and such data could include weather aspects, seismic zones, geo-informatics, etc. in a relating National Annex. The National Standard implementing Eurocodes must comprise the full, unaltered text of that Eurocode, including all annexes. All European countries are required to develop their own National Annexes. Although a non-European country, Singapore has chosen to adopt the Eurocodes as her national design codes and after an initial 2 year co-existence period, the Eurocodes will be fully implemented in Singapore by 1 April 2015.

Although many of the Eurocodes design rules are based on the same design concepts as the British Standards, the Eurocodes have incorporated more up-to-date research on many respects of structures behavior. The Eurocodes clauses are structured slightly differently in which they specified principles that must be satisfied first and followed by application rules that offer a way of satisfying the principles. The Eurocodes are also less prescriptive than the British Standards, with more design aspects left open to the engineers to decide. Consequently, for this reason, it is useful to discuss the design approaches and compare the results obtained by both codes to assess the impact of making some of the key changes. Slender or thin-walled elements under compressive stresses are susceptible to buckling; hence it is imperative to evaluate accurately the buckling strength in order to optimize the design. For structural element, there are basically two different buckling modes, the overall buckling and local buckling. Overall buckling includes lateral buckling, torsional buckling and other buckling modes. Local plate buckling may occur on the slender elements first before the overall beam or column buckling, or yielding.

In this paper, the key changes for three design aspects using both Eurocodes and BS5950 are investigated in details, and they are all related to structural member buckling resistances. Flexural buckling resistance of compression members with grade S460 steel, shear buckling resistance of plated structures and web bearing and buckling resistance of the plated structures under transverse force are compared using BS5950-1 and EC3.

2. Flexural buckling resistance of universal columns with S460 steel

The behaviour of high strength steel (Figure 1) with respect to effect of buckling and residual stresses is studied by Grotmann D, Sedlacek, G (1994) and Beg D, Hladnik L (1995). It is found that HSS has a better performance than ordinary steel, which can be interpreted as the smaller influence of imperfections, such as residual stresses. The residual stresses of HSS were lower if made dimensionless with respect to the strength of the steel, as shown in the work by Clarin M. (2004). This is reflected in the calculation of flexural buckling resistance by using higher buckling curves for S460 according to EN 1993-1-1, shown as Table 1. Therefore, the buckling resistance of columns with high strength steels are expected to be higher than those of columns with normal steels.

According to BS5950-1, members in compression are analysed by considering the nonlinear diagram of steel behaviour, the accidental eccentricities as well as the shape of cross-sections. Design value of compression buckling resistance is determined by:

$$N_{b,Rd} = Ap_c \qquad (1)$$

Where, A is the gross cross-sectional area; p_c is the compressive strength determined from Table 23 and Table 24 in BS5950-1, which is related to the buckling curves, the design strength f_y and the slenderness λ.

The principle in EC3 for determining the buckling resistance of compression members is similar with that of BS5950-1.

According to EN 1993-1-1, for compression members the design buckling resistance should be taken as:

$$N_{b,Rd} = A\frac{\chi f_y}{\gamma_{M1}} \quad (\gamma_{M1} = 1.0) \qquad (2)$$

Where, A is the gross cross-sectional area; χ is the reduction factor related to the non-dimensional slenderness of the member:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \qquad (3)$$

where

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$$

$$\lambda_1 = \pi\sqrt{E/f_y} = 93.9\varepsilon \qquad \varepsilon = \sqrt{235/f_y}$$

α is the imperfection factor.

The buckling curves defined by EC3 are equivalent to those given in Table 24 in BS5950-1. However, except curves a, b, c and d, a more favourable buckling curve a_0 for S460 steel is introduced in EC3.

To compare the compression buckling resistance calculated from BS5950-1 and EC3, a column with pinned boundary conditions is analysed. A series of Rolled H-sections (356×406UC) and Rolled I-sections (457×191UB) with grade S460 and S355 steel are adopted as the internal column. Assuming the nominal buckling length is 7.5m. The buckling resistance are illustrated in Figure 2.

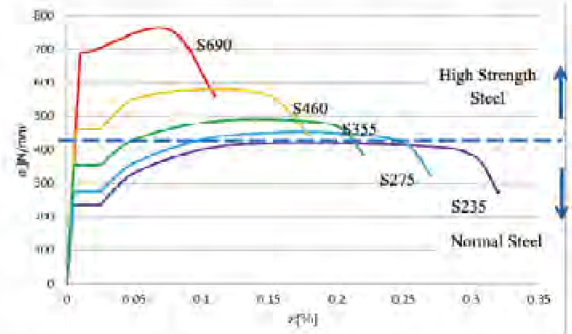


Figure 1: Typical structural steel material properties

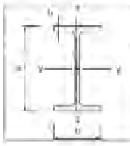
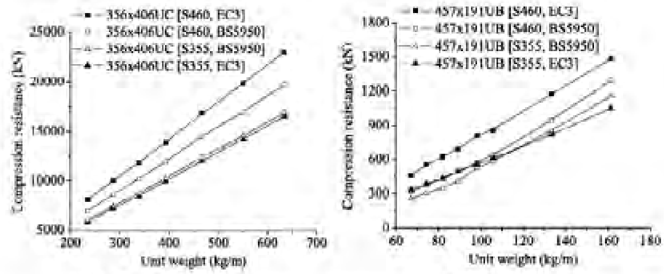
Cross-section	Buckling about axis		Buckling curves					
			EN 1993-1-1			BS 5950		
			Limits	S235 S275 S355 S420	S460	Limits	S275 S355	S460
	Rolled I-section	y-y z-z	$t_f \le 40$mm	a b	a_0 a_0	$t_f \le 40$mm	a b	a b
		y-y z-z	40mm < $t_f \le$ 100mm	b c	a a	t_f >40mm	b c	b c
	Rolled H-section	y-y z-z	$t_f \le 100$mm	b c	a a	$t_f \le 40$mm	b c	b c
		y-y z-z	t_f >100mm	d d	c c	t_f >40mm	c d	c d

Table 1: Selection of buckling curve for cross-sections

It can be seen from Figure 2 that the buckling resistance obtained from EC3 has a significantly improvement compared with that determined from BS5950-1. The buckling resistance for S460 steel calculated from EC3 is about 15% higher than that obtained from BS5950-1. However, for S355 steel, almost identical values are obtained from both codes. This is because that a favourable buckling curve a_0 is adopted for S460 steel in EC3, as mentioned in above.



(a) Rolled H-sections (b) Rolled I-sections

Figure 2: Compression buckling resistance

3. Shear buckling resistance of plates

The usual slender design of the web panels in plate structures, such as plate girder (Figure 3), makes the web susceptible to instability phenomena: shear buckling (Figure 4). Buckling in slender plates is a local and sudden phenomenon followed by large out-of-plane displacements and loss of stiffness. Slender plates are capable of carrying considerable post-buckling loads due to stresses in the inclined tension fields as shown in the work by Alinia MM, Habashi HR (2009).

Figure 3: Plate girder

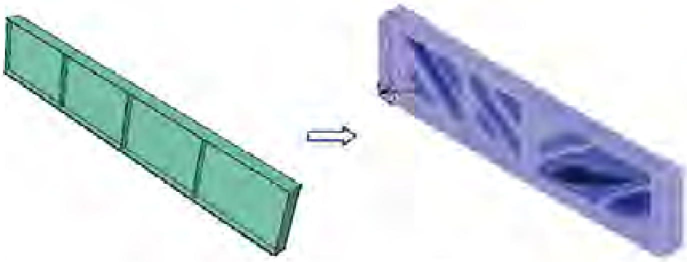


Figure 4: Shear buckling

In BS5950-1, if the web depth-to-thickness ration $h_w/t_w > 70\varepsilon$ for a rolled section, or 62ε for a welded section, it should be assumed to be susceptible to shear buckling. The factor ε is taken as $\varepsilon = \sqrt{275/f_y}$, which is different from the expression in EC3. The shear buckling resistance $V_{b,Rd}$ of a web with or without intermediate transverse stiffeners may be taken as the simple buckling resistance V_w given by:

$$V_{b,Rd} = V_w = h_w t_w q_w \qquad (4)$$

Where, q_w is the shear buckling strength which should be obtained from Table 21 or Annex H.1 of BS5950-1 depending on the values of h_w/t_w and a/h_w where a is the stiffener spacing. In EC3, two methods are provided to determine shear resistance of a cross-section. One is the plastic shear resistance of cross-sections where the web is not prone to shear buckling. The other one is the shear buckling resistance of cross-sections where the web is prone to shear buckling. Plate with $h_w/t_w > 72\varepsilon/\eta$ for unstiffened web, or $h_w/t_w > 31\varepsilon\sqrt{k_\tau}/\eta$ for a stiffened web, it becomes susceptible to shear buckling. Where, k_τ is the shear buckling coefficient defined in Annex A3 of EN 1993-1-5; η is taken as 1.0 given in UK National Annex.

There are many tension field theories which aim to describe the ultimate resistance of plates under shear, as shown in the work by Höglund (1981). The resistance of steel beams to shear buckling is covered by the stiffened plate rules in EN 1993-1-5 and is based on the rotated stress field theory proposed by Höglund. In this method, the shear resistance $V_{b,Rd}$ comprises contributions from the web $V_{bw,Rd}$ and the flanges $V_{bf,Rd}$. The expression given in EN 1993-1-5 takes the following form, clearly identifying the two separate contributions:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \eta \frac{f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}} \quad (5)$$

With:

$$V_{bw,Rd} = \chi_w \frac{f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}} \quad (6)$$

$$V_{bf,Rd} = \frac{b_f t_f^2}{c} \frac{f_{yf}}{\gamma_{M1}} \left[1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right] \quad (7)$$

The value $V_{bw,Rd}$ depends on web slenderness and end post condition. Rigid end posts are typically used at the ends of girders to improve their shear resistance. The reduction factor χ_w considers components of pure shear and anchorage of membrane forces by transverse stiffeners due to tension field action. The partial safety factor γ_{M1} is taken as 1.0. The value $V_{bf,Rd}$ depends on the plastic resistance of the flanges bending out of its plane. However, the contribution to shear resistance from the flange is much less than the web and can always be conservatively ignored to avoid the additional calculation effort. Hence, the shear resistance of cross-sections can be transferred as:

$$V_{b,Rd} \approx V_{bw,Rd} = \chi_w \frac{f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}} \leq \eta \frac{f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}} \quad (8)$$

From equation (8), it's noted that the shear buckling resistance can never be larger than the plastic shear resistance of the cross-section $\eta \frac{f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}}$.

EC3 classify the beam end support conditions as rigidity and non-rigidity according to the end post could or not provide adequate anchorage for the longitudinal component of the tensile stresses developed in the web during the post-buckling range. The reduction factor χ_w of rigid and non-rigid end post is determined from different expressions, as shown in Table 5.1 and Figure 5.1 of EN 1993-1-5 depending on web slenderness $\bar{\lambda}_w$. The web slenderness $\bar{\lambda}_w$, for both unstiffened and stiffened webs is given by:

$$\bar{\lambda}_w = \frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}} \quad (9)$$

For beams without intermediate stiffeners and longitudinal stiffeners at supports only, this expression could simplify to:

$$\bar{\lambda}_w = \frac{h_w}{86.4 t_w \varepsilon} \quad (10)$$

To compare the shear buckling resistance calculated by the methods given in EC 3 and BS5950-1, a simply supported plate girder with grade S275 steel is analysed. To find the connection with BS5950, the following expression is used:

$$V_{b,Rd} = \chi_w \frac{f_{yw} h_w t_w}{\sqrt{3}\gamma_{M1}} = h_w t_w f_{sb} \quad (11)$$

With: $f_{sb} = \dfrac{f_{yw}\chi_w}{\sqrt{3}\gamma_{M1}} = 158.8\chi_w$

The stiffener spacing is assumed to be equal to web depth, hence $a/h_w = 1$. Then, the shear buckling coefficient k_τ is $5.34 + 4.00(h_w/a) = 9.34$. Hence, the equation (9) can be rewritten as $\bar{\lambda}_w = h_w/105.7t_w$. Then, the Table 5.1 of EN1993-1-5 can be rewritten as shown in Table 2 using the factor h_w/t_w and the shear buckling strength f_{sb} (q_w) of the plate girder obtained from EC3 and BS5950-1 are given in Figure 5.

Table 2: Shear resistance function for S275 steel by h_w/t_w

	χ_w	
	Rigid end post	Non-rigid end post
$\frac{h_w}{t_w} < 87.7$	1	1
$87.7 \leq \frac{h_w}{t_w} < 114.1$	$87.7\frac{t_w}{h_w}$	$87.7\frac{t_w}{h_w}$
$\frac{h_w}{t_w} \geq 114.1$	$\frac{1.37}{0.7 + 105.66 h_w/t_w}$	$87.7\frac{t_w}{h_w}$

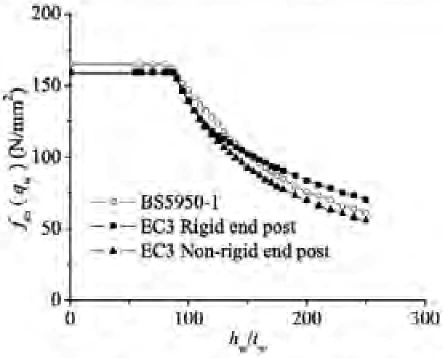


Figure 5: Shear buckling strength curves for S275 steel

It can be seen from Figure 5 that the values of shear buckling strength obtained from EC 3 in plateau are generally smaller than that determined from BS5950-1. For non-rigid end post girder, the values of shear buckling resistance are consistently lower than that got from BS5950-1. However, for rigid end post web, with the increasing of the ratio h_w/t_w, shear buckling resistance may exceed the value obtained from BS5950-1. It means that the design shear resistance obtained from EC3 is conservative compared with BS5950-1 only for welded stocky beams.

4. Web buckling resistance under transverse loading

Buckling of the web happens when the web is too slender to carry the transverse loading being transferred from the flange, shown as Figure 6. In BS5950-1, the web buckling occurs if the flanges are restrained, and the buckling resistance of the unstiffened web should be taken as:

$$F_{Rd,BS} = \frac{25\varepsilon t_w}{\sqrt{(S_s + nk)\, h_w}} F_{w,Rd} \quad (12)$$

With

$$F_{w,Rd}=\left(S_s+nk\right)t_w f_{yw} \qquad (13)$$

Where, Ss is the stiff bearing length determined from Figure 13 of BS5950-1; k is $t_f + r$ for rolled section, or t_f for weld section; nk is the additional length assuming a dispersion of the bearing at 1: n through the flange thickness.

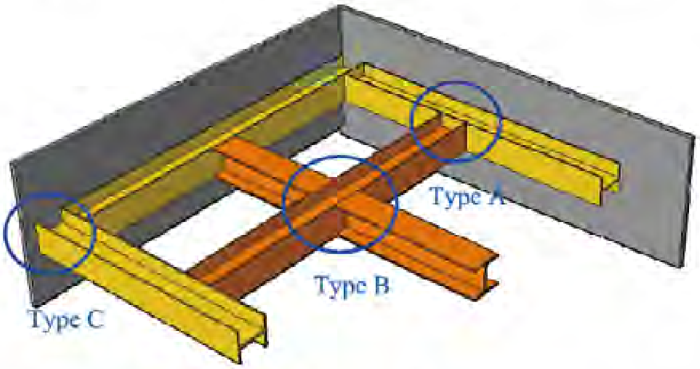


Figure 6: Typical strut-waler connections

In EC3, three types of load application are provided: (a) load is applied through one flange; (b) through both flanges and transferred through the web directly; (c) through one flange adjacent to an unstiffened end (Figure 6). With a reduction factor χ_F, the design resistance to local buckling under transverse loading should be determined as:

$$F_{Rd,EC}=\chi_F\frac{l_y t_w f_{yw}}{\gamma_{M1}} \qquad (14)$$

Where

$$\chi_F=\frac{0.5}{\bar{\lambda}_F}\le 1.0 \qquad (15)$$

$$\bar{\lambda}_F=\sqrt{\frac{l_y t_w f_{yw}}{F_{cr}}} \qquad (16)$$

$$F_{cr}=0.9k_F E\frac{t_w^3}{h_w} \qquad (17)$$

k_F is a factor related to load application;
l_y is the effective loaded length calculated as follows (Figure 7):
For load application type (a) and (b), it should be taken as:

$$l_{y1}=s_s+2t_f\left(1+\sqrt{m_1+m_2}\right) \qquad (18)$$

For load application type (c), it should be taken as the smallest value obtained from equation (18), (19), and (20), which is given in the Corrigendum to EN 1993-1-5 (CEN, 2009).

$$l_{y2}=l_e+t_f\sqrt{\frac{m_1}{2}+\left(\frac{l_e}{t_f}\right)^2+m_2} \qquad (19)$$

$$l_{y3}=l_e+t_f\sqrt{m_1+m_2} \qquad (20)$$

Where, l_e is a factor related to k_F; the factor m1 is taken as $\frac{f_{yf}b_f}{f_{yw}t_w}$, and m_2 is taken as 0 for $\bar{\lambda}_F\le 0.5$, or $0.02\left(\frac{h_w}{t_f}\right)^2$ for $\bar{\lambda}_F>0.5$. It should be noted that $\bar{\lambda}_F$ and m_2 are inter-dependent. The plate slenderness $\bar{\lambda}_F$ depends on l_y, which in turn is affected by m_2. To compare the web buckling resistance predicted by EC3 and BS5950-1, an expression can be proposed as following:

$$R=\frac{F_{Rd,EC}}{F_{Rd,BS}} \qquad (21)$$

Based on equation (12) and (13), the web buckling resistance given in BS5950-1 can be written as:

$$F_{Rd,BS}=\frac{25\varepsilon t_w}{\sqrt{\left(S_s+nk\right)h_w}}\left(S_s+nk\right)t_w f_{yw} \qquad (22)$$

Similarly, with $\chi_F\le 1.0$, from equation (14)-(17), the web buckling resistance obtained from EC3 can be rewritten as:

$$F_{Rd,EC}=\frac{0.5\sqrt{0.9k_F E t_w}}{\sqrt{l_y h_w f_{yw}}}l_y t_w f_{yw} \qquad (23)$$

If $\chi_F>1.0$, equation (23) is not applicable. Then,

$$R=\frac{F_{Rd,EC}}{F_{Rd,BS}}=\sqrt{0.275k_F}\sqrt{\frac{l_{y,EC}}{l_{y,BS}}} \qquad (24)$$

Where, E is taken as 210GPa; $l_{y,BS}$ is taken as $S_s + nk$. For unstiffened web with load application through one flange web, the buckling coefficient k_F is taken as 6. Then the ration R is only related to the effective loaded length l_y as follows:

$$R=\frac{F_{Rd,EC}}{F_{Rd,BS}}=1.284\sqrt{l_{y,EC}/l_{y,BS}} \qquad (25)$$

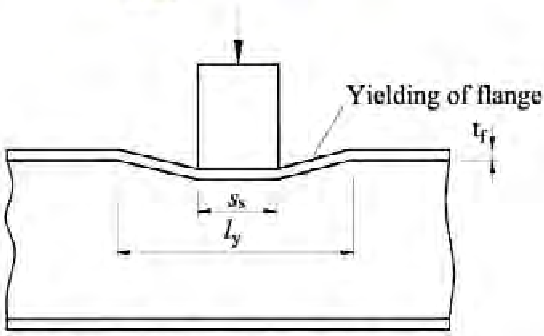


Figure 7: The effective loaded length

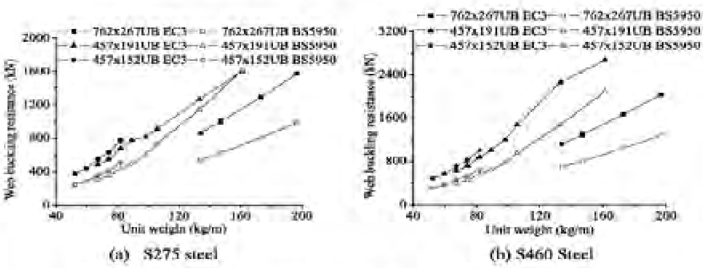


Figure 8: Web buckling resistance of rolled section

Based on analysis of various steel sections, it is found that the effective loaded length obtained from EC3 is larger than that calculated from BS5950-1. Therefore, as shown in equation (24), the value of R is greater than 1.0, which means the web buckling resistance of EC3 is higher than that of BS5950-1. Taking the universal beams 762×267UB, 457×191UB and 457×152UB for instance, the calculation results of web buckling resistance under transverse loading are shown in Figure 8. In Figure 8, the load application type (a) is adopted according to EC3, and n is taken as 5 according to BS5950-1 for calculating $l_{y,BS}$.

It can be seen that the web buckling resistance of EC3 is always larger than that of BS5950-1 for both S275 and S460 steel sections. About 40% improvement of web buckling resistance can be achieved by using the equation of EC3.

5. Conclusions

The impact of Eurocodes on steel structures is investigated in this paper. The analysis of flexural buckling resistance of uniform compression members with grade S460 steel, the shear buckling resistance of plated structures, and the web bearing and web buckling resistance of the plated structures under transverse loading are conducted based on EC3 and BS5950-1. Based on the results of analysis, the following conclusions can be drawn.

(1) Considering the effect of residual stress, a favourable buckling curve a_0 is adopted to calculate the flexural buckling resistance for compression members with S460 steel in EC3. Therefore, the buckling resistance obtained from EC3 has a significant improvement compared with that determined from BS5950-1.

(2) Support condition (rigidity or non-rigidity) has a significant influence on determining shear buckling resistance. Under the same web depth-to-width ratio h_w/t_w, the shear buckling resistance for non-rigid end post web is consistently lower than the value obtained from BS5950-1. However, the shear buckling resistance of the rigid end post web may exceed the value obtained from BS5950-1 if the ratio h_w/t_w exceeds a certain value.

(3) The resistance to transverse loading is determined by two main factors, the effective loaded length l_y and the reduction factor χ. The deduction and worked example show that the effective loaded length and reduction factor for rolled section predicted by EC3 are always larger than the length predicted by BS5950-1. Therefore, the resistance to transverse loading obtained by BS5950 is more conservative compared with EC3.

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Structural Steel Excellence Awards 2014

Healthcare or Education Structures

NUH Medical Centre:
Structural Steelwork of a 19 Storey (Including 2 basement) Medical Centre
Location: 1E Kent Ridge Road, Singapore
Client(s): National University Health System
Architect(s): Consultants Incorporated Architects + Planners
Contractor(s): Penta-Ocean Construction Co Ltd
Structural Engineer / Steel Fabricator:
TY Lin International Pte Ltd / Yongnam Engineering & Construction (Pte) Ltd

Stamford American International School: 8 Storey Steel Structure Building
Location: 1 Woodleigh Lane, Singapore
Client(s): Stamford American International School Pte Ltd
Architect(s): Architects 61 Pte Ltd
Contractor(s): Lend Lease Singapore Pte Ltd
Structural Engineer / Steel Fabricator:
CKM Consultants Pte Ltd / TTJ Design & Engineering Pte Ltd

Temasek Polytechnic West Wing 1 & 2:
2 Blocks of 5 Storey Teaching Facilities
Location: 21 Tampines Avenue 1, Singapore **Client(s)**: Temasek Polytechnic
Architect(s): Forum Architects Pte Ltd
Contractor(s): V3 Construction Pte Ltd
Structural Engineer / Steel Fabricator:
RSP Architects, Planners & Engineers Pte Ltd / Hwee Metal Works Pte Ltd

SIT@ TP: 5 Storey Block of teaching facilities and academic offices
Location: 21 Tampines Avenue 1, Singapore
Client(s): Singapore Institute of Technology **Architect(s)**: Forum Architects Pte Ltd
Contractor(s): V3 + LHL Builders Pte Ltd
Structural Engineer / Steel Fabricator:
RSP Architects Planners & Engineers / Hwee Metal Works Pte Ltd

Community or Residential Structures

Dawson – Contract 2: Skybridge
Location: Dawson Road, Singapore
Client(s): Housing Development Board (HDB), Singapore
Architect(s): SCDA Architects Pte Ltd **Contractor(s)**: Sun Huan Construction Pte Ltd
Structural Engineer / Steel Fabricator:
Ronnie & Koh Consultants Pte Ltd / TTJ Design & Engineering Pte Ltd

Lincoln Suite – Sky Gym: Proposed Condominium Development
Location: 1 & 3 Khiang Guan Avenue, Singapore
Client(s): Phileap Pte Ltd **Architect(s)**: ONG & ONG Pte Ltd
Contractor(s): Koh Brothers Building & Civil Engineering Contractor (Pte) Ltd
Structural Engineer / Steel Fabricator: ARUP Singapore Pte Ltd / Sterling Engineering Pte Ltd

RV Residence – Steel Trellis: Proposed Condominium Development
Location: 467 River Valley Road, Singapore
Client(s): Asiawide Resources Pte Ltd **Architect(s)**: ONG & ONG Pte Ltd
Contractor(s): Lee Kim Tah - Woh Hup JV
Structural Engineer / Steel Fabricator: KTP Consultants Pte Ltd / Sterling Engineering Pte Ltd
Other(s): MERO Asia Pacific Pte Ltd

Commercial or Retail Structures

Capital Gate – Abu Dhabi : Mixed Commerical Use (Office & Hotel)
Location: Abu Dhabi, United Arab Emirates
Client(s): Abu Dhabi National Exhibition Centre (ADNEC)
Architect(s): Robert Matthew Johnson Marshall (RMJM)
Contractor(s): Al Habtoor Engineering Enterprises C.O.L.L.C.
Structural Engineer / Steel Fabricator:
Robert Matthew Johnson Marshall / Eversendai Engineering Pte Ltd

Concourse Skyline: Architectural Façade & Water Feature
Location: Concourse Skyline at Beach Road, Singapore
Client(s): Hong Fok Land Ltd **Contractor(s)**: Maincon Building Pte Ltd
Architect(s): Architect 61 Pte Ltd / Sitetectonix Pte Ltd (Landscape Architects)
Structural Engineer / Steel Fabricator:
CKL Consultants Pte Ltd / Landscape Engineering Pte Ltd

The Tube (Orchard Gateway): The Tube is a 53 Metres link bridge across Orchard Road
Location: 277 & 218 Orchard Road, Singapore **Architect(s)**: Tange Associates / AWP Pte Ltd
Client(s): UE Orchard Pte Ltd **Contractor(s)**: Hyundai Engineering & Construction Co. Ltd.
Structural Engineer / Steel Fabricator:
Meinhardt (S) Pte Ltd / TTJ Design and Engineering Pte Ltd

SEACARE: Hotel Building with steel transfer trusses
Location: 52 Chin Swee Road, Singapore
Client(s): Seacare Co-operative Ltd **Architect(s)**: Archispace + Designs Pte Ltd
Contractor(s): Union Contractor (s) Pte Ltd
Structural Engineer / Steel Fabricator:
ECAS-EJ Consultants Pte Ltd / WY Steel Construction Pte Ltd

Structural Steel Excellence Awards 2014

Arts / Entertainment /Sports / Leisure Structures

Singapore National Stadium Roof: Integrated Sports Hub
Location: 2 Stadium Walk, Singapore **Client(s):** SportsHub Pte Ltd
Architect(s): Arup Associates + DP Architects Pte Ltd
Contractor(s): Dragages Singapore Pte Ltd
Structural Engineer / Steel Fabricator:
ARUP Singapore Pte Ltd / Yongnam Engineering & Construction Pte Ltd
Other(s): AECOM (Landscape Architect)

Roof Over Parade Ring: 95.88 Metres Large Span Steel Structure
Location: Kranji Road (Singapore Turf Club), Singapore
Client(s): Singapore Turf Club **Architect(s):** ONG & ONG Pte Ltd
Contractor(s): WY Steel Construction Pte Ltd
Structural Engineer / Steel Fabricator:
KK Lim & Associates Pte Ltd / WY Steel Construction Pte Ltd

Sun Pavilion at Gardens by the Bay: Canopy for Cactus Garden
Location: 18 Marina Gardens Dr, Singapore
Client(s): Gardens by the Bay **Architect(s):** Grant Associates (Landscape Architects)
Contractor(s): Thompson Pte Ltd
Structural Engineer / Steel Fabricator: CHC Consultants with Passage Projects / Artfair

Infrastructure or Transportation Structures

Mumbai International Airport Terminal 2: International Terminal Roof Steelwork
Location: Mumbai, India
Client(s): Mumbai International Airport Limited
Architect(s): Skidmore Owings & Merrill
Contractor(s): Larsen & Toubro
Structural Engineer / Steel Fabricator:
Geodesic Yongnam Structurals Pte Ltd / Yongnam Engineering Sdn Bhd

Tuas West Extension MRT Depot: Roof Steel Trusses
Location: Tuas Link 4, Singapore
Client(s): Land Transport Authority Singapore **Architect(s):** DP Architects Pte Ltd
Contractor(s): Jurong Primewide Pte Ltd
Structural Engineer / Steel Fabricator:
Parsons Brinckerhoff Pte Ltd / TTJ Design & Engineering Pte Ltd

Pedestrian Overhead Bridge at SportsHub: Bridges
Location: 2 Stadium Walk, Singapore
Client(s): Singapore Sports Hub **Architect(s):** DP Architects Pte Ltd
Contractor(s): Dragages Singapore
Structural Engineer / Steel Fabricator:
Arup Singapore Pte Ltd / Hetat Pte Ltd

Others

Laxness Nd-PBR Plant: Building a new Neodymium - based performance Butadiene Rubber (Nd-PBR) plant in Singapore
Location: Tembusu Road, Jurong Island, Singapore **Client(s):** Lanxess Butyl Pte Ltd
Architect(s) / Contractor(s): Foster Wheeler Asia Pacific Pte Ltd
Structural Engineer / Steel Fabricator:
Foster Wheeler Asia Pacific Pte Ltd / TTJ Design & Engineering Pte Ltd

Singapore LNG Terminal:
Singapore LNG, Receiving, Reloading, Storage and Regasification Terminal
Location: Merianti View, Jurong Island, Singapore
Client(s): Singapore LNG Corporation Pte Ltd **Contractor(s):** Samsung C&T Corporation
Structural Engineer / Steel Fabricator:
Steen Consultants Pte Ltd / TTJ Design & Engineering Pte Ltd