APPLICATION OF HIGH STRENGTH NIOBIUM GRAIN-REFINED STEELS TO A RE-DESIGN OF THE SINGAPORE NATIONAL STADIUM ROOF

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Abstract

The new 55,000 seat National Stadium is the centrepiece of the Singapore Sports Hub project, due to be completed in early 2014. The roof structure for the stadium is a highly efficient dome spanning 310 m with an elevation of 85 m from the ground level. When complete, the fixed roof will be the largest clear span dome roof in the world and supports a symmetrical movable roof in two halves. The fixed roof structure is formed by a series of steel arch trusses varying in depth from 5 m at the centre to 2.5 m at the base and is supported by a post-tensioned concrete ring beam. The trusses are constructed from circular hollow section (CHS) elements and span in multiple directions to form a highly efficient braced dome structure.

Due to the movable roof there is a regularly varying load on the structure. Whilst the structure is not highly fatigue sensitive, fluctuating loads had to be carefully considered in the design to ensure that they could be accommodated within the design of the connections.

The construction design was based on S355 steel using cold-formed CHS members with hot finished S355 CHS elements used only for thicker sections. Whilst serviceability issues such as deflection and fatigue were design considerations, the structure was predominantly strength governed.

This paper considers the potential savings in material use that could have been made if higher strength, high elongation steel had been economically viable and could have been competitively tendered. It is shown that it could be possible to make a significant saving on element steelwork if hot finished niobium grain-refined S500 can be adopted for similar projects in the future.

Introduction

This paper investigates the implications of substituting S355 carbon steel with high strength S500 steel in the design of the long-span Singapore Sports Hub National Stadium (SSH NST) roof.

Using high strength steel for such structures will be shown to provide a reasonable reduction in steel tonnage, due not only to the increased capacity of elements but also the positive feedback into the design associated with a reduction in self-weight of the steel structure. However, it will be identified that there are limits on potential savings imposed by reduced stiffness (section classification, global buckling, and slender bracing elements), fatigue and practical limits on plate thicknesses.

Project Overview

The Singapore National Stadium will form the centrepiece to the new Singapore Sports Hub and lies at the heart of the 35 ha sports precinct, Figure 1.



Figure 1. Master plan of the Singapore Sports Hub.

National Stadium Roof Structure - Overview

The National Stadium (NST) has a 310 m diameter spherical steel dome roof. The roof rises to a height of approximately 85 m from pitch level, or 73 m from its supporting level 3 post-tensioned concrete Ring Beam. The roof comprises both a fixed roof and two movable roof components totalling approximately 8,100 tonnes of structural steel, excluding connections, Figure 2.

The fixed roof spans clear across the stadium with no support taken from the stadium seating 'bowl' concrete superstructure and supports the movable roof via a series of 'bogies' running on the parallel 'runway trusses' that span perpendicular to the pitch axis.



Figure 2. Section through NST.

The structural dome form of the roof imparts large tensile forces into a post-tensioned concrete ring beam at level 3, approximately 9 m from ground level, which acts to restrain the roof from spreading. There is an opening in the roof which is approximately 220 m long by 82 m wide over the football pitch, athletics track and part of the stand in the south-west of the stadium. Six runway trusses span perpendicular to the pitch axis, five of which span across the opening. These five runway trusses provide support for the runway beams and track that carry the two halves of the movable roof structure which open and close symmetrically. The runway trusses are at approximately 48 m centres. A fundamental principle in the design of the fixed and movable roofs has been to create a very stiff fixed roof and a flexible movable roof structure. This is in order to minimise the tendency of the movable roof to rack or skew and jam during operation. This is discussed further in a later section.

Fixed Roof Structure, Figures 3-5

All loads on the roof structure are transmitted to the concrete structure at level 3 by a network of triangular primary trusses creating a very stiff 3-dimensional shell or dome structure.



Figure 3. Fixed roof primary trusses and ring beam.

These primary trusses comprise:

- Six 'runway trusses' (RWT) spanning across the stadium perpendicular to the pitch axis;
- 'Transverse trusses' which are the two trusses parallel to the pitch and form the long edges of the roof opening;
- 'Diagonal trusses' linking corners of the rectangular forms described by the Transverse and runway trusses;
- 'Interceptor trusses' which define the junction between the fixed roof cladding supported on the secondary trusses above and the PTFE fabric clad Giant Louvres below.

The primary trusses form the principal load carrying steel members in the roof. They vary in both depth and width with a minimum depth of approximately 2.5 m at the base nodes and a maximum depth of approximately 5.0 m at the centre of the dome. All trusses are 3D triangular trusses fabricated from CH sections with chord sizes of 457 mm diameter and 508 mm diameter. Spanning between the runway trusses and diagonal trusses, the secondary trusses directly support the fixed roof cladding system. The secondary truss top chords provide support to the roof cladding at 6 m centres.

The vertical loads on the roof are primarily resisted by compression with the system of primary trusses and secondary trusses acting together to form a dome or shell structure. The thrust from the dome is then balanced by a combination of axial load in the post-tensioned reinforced concrete ring beam and portal action between the ring beam and supporting columns.



Figure 4. Fixed roof secondary structures.



Figure 5. Primary and secondary truss detail.

Fixed Roof Connections

A number of different connection types were initially investigated for the complex geometry of the tube-to-tube connections of the fixed roof, Figure 6. Three key factors were assessed when selecting the connection detail to use:

- Fatigue sensitivity: Use of stiffener plates, slotted plates and cruciforms within connections can greatly reduce the fatigue life of connections;
- Ease of fabrication: Fabricator's preference for profile cut members rather than fabricated plate nodes;
- Ease of design: Designs with clear load paths and the ability to design using published methods were preferred.



Figure 6. Complex multi-CHS node connections.



Figure 7. Thickened can approach to connections.

A connection formed from one thickened member through the connection, and profile cutting and welding all other members to it, was selected as the preferred fabrication option and the least fatigue sensitive detail. The thickened member through the connection is referred to as a "thickened can", Figure 7. Figure 8 illustrates the finite element model used to design the complex multi-CHS nodes.



Figure 8. Finite element model of a complex connection.

This paper undertakes a preliminary assessment of which connections could potentially be reduced by adjusting the current strength utilisations to reflect the extra bending and punching capacity associated with S500 steel.

Scope of Study

As noted above, the steel structure for the SSH NST roof is split into three distinct components; the movable roof, fixed roof and giant louvres. The fixed roof accounts for approximately 70% of the total steel tonnage, based on the S355 reference design, and is the focus of the redesign carried out in this study, Figure 9.



Figure 9. Fixed roof finite element model.

One of the most significant imposed loads on the fixed roof is that generated by the movable roof. The study considers both the open and closed positions of the movable roof and the forces generated by long-term movements of the ring-beam (creep and shrinkage).

The study has been broken down into two distinct stages; an **initial study** that reviews the impact on typical elements such as truss chords and braces and an **optimisation phase** that determines the steel tonnage of the fixed roof using high strength steel.

Initial Study

This initial phase encompasses a review of Singapore design codes for the design of truss structures (also known as lattice structures) with high strength steels.

Typical elements are identified in the structure and comparisons made between S355 and S500 high strength steel.

Optimisation

In the second stage, Arup undertook a rigorous redesign of the Singapore Sports Hub National Stadium roof and have provided an in-depth investigation of certain aspects of its structural behaviour such as fatigue, dynamic wind response and buckling behaviour.

Parameters

Reference Design

The study has been conducted on the fixed roof structure redesign using optimisation software developed by Arup specifically for this project. The S355 design was repeated to remove section size adjustments to elements in the final design which were the result of aesthetic or buildability considerations.

A target strength utilisation of 85% was adopted (5% design contingency and 10% allowance for locked-in stresses) as this matches what was undertaken for the roof's final design.

Materials

The following materials have been used as part of the study:

- Carbon steel: S355 to EN10210-1 and EN10219-1 (used in the reference design);
- High strength carbon steel: S500.

The density for all grades of steel has been assumed to be 7.85 tonnes/m³.

Strength. The mechanical properties adopted in this study are shown in Table I.

Grade	Nominal Yield Strength (N/mm ²)	Nominal Ultimate Tensile Strength (N/mm ²)	Young's Modulus (kN/mm ²)	Initial Imperfection Ratio
S355 (cold-formed)	355	510	205	L/260
S355 (hot-finished)	355	510	205	L/720
S500 (hot-finished)	500	620	205	L720

Table I. Mechanical Properties

Buckling imperfection factors have been back-calculated from the Robertson constant in accordance with Appendix C of BS5950.

The BCA design guide on use of the *Alternative Steel Materials to BS5950* has an upper limit on yield strength of 460 N/mm² for structures that assume plastic design. This limit does not apply since we have used elastic analysis to determine the distribution of forces in the structure.

The assumed variation in yield strength (provided by CBMM) with plate thickness is given in Table II.

Steel Grade	Thickness (mm) (less than or equal to)	Design Strength – p _y (N/mm ²)		
	16	355		
8355	40	345		
	63	345		
	80	325		
	100	315		
	150	295		
	16	500		
S500	40	485		
	63	470		
	80	455		
	100	440		
	150	425		

Table II. Variation in Yield Strength with Thickness

This study assumes that the appropriate Factory Production Control systems are in place for the steel, such that the variability in properties is equivalent to that in EN 10025 and assumed in related design codes.

<u>Ductility</u>. Ductility is of paramount importance to the roof's design since it is relied upon to redistribute stresses at nodes and mobilise the more efficient axial only load-path. Ductility tends to reduce with increased yield strength; fortunately this effect does not seem to be an issue for the S500 steel since tests show that the nominal elongation at failure is 25% which exceeds the minimum code requirement of 15%. It is also clear from Table I that the tensile strength to yield strength ratio exceeds the BCA requirement of 1.2 based on nominal values.

<u>Impact Toughness.</u> CBMM have advised us that they predict that the Charpy impact test results would be within the range of 200 to 300 J at -40 °C and in excess of 27 J at -50 °C. Should this be the case, then the subgrade would be equivalent to NLH. The minimum service temperature for Singapore was taken as 20 °C. The corresponding maximum thicknesses, t_{max} for each subgrade are given in Table III. As the connections are tubular nodal joints the maximum permitted thicknesses are half the reference maximum thickness, t_1 (k=0.5).

Grade	Charpy Impact Energy	t ₁ (mm)	k	t _{max} (mm)
S355J0H	27 J at 0 °C	72	0.5	36
S355J2H	27 J at -20 °C	104	0.5	52
S355K2H	40 J at -20 °C	124	0.5	62
S500NLH	27 J at -50 °C	111	0.5	55

Table III. Sub-grade Thickness Limits

<u>Durability</u>. It is assumed that the durability performance of S500 and S355 will be similar and that the same paint protection system would be adopted.

<u>Chemical Composition.</u> The typical chemical composition of S500 steel has been provided by CBMM and is summarised below.

		Tabl	le IV. S	500 Ch	emical	l Comp	osition	(wt.%)		
/In	Si	S	Р	Ni	Cr	Mo	V	Cu	Ν	N

Note: Ouantities	of S. P. N	i. Cr. Mo). Cu. N are residual levels allowed from scrap pick up.

The Carbon Equivalent (CEV) has been calculated from:

0.20 0.015 0.015

$$CEV(\%) = \%C + \frac{\%Mn}{6} + \frac{\%Cr + \%Mo + \%V}{5} + \frac{\%Cu + \%Ni}{15}$$
(1)

0.10 0.10 0.05 0.020 0.10 0.007

CEV

0.289

0.025

It is clear that the steel satisfies BCA requirements, specifically:

- Carbon content that does not exceed 0.24%;
- Carbon Equivalent that does not exceed 0.55%;
- Phosphorus less than 0.035%;
- Sulphur less than 0.035%.

С

0.10

0.85

<u>Weldability</u>. The study assumed that appropriately qualified production and welding procedures would be developed for the steels that take account of the particular chemistry and heat treatment of the high strength steels. It is assumed these welding procedures result in joints (weld metal and heat affected zones) with comparable mechanical properties to the parent materials.

A minimum wall thickness of 6 mm has been adopted in the design which matches that in the reference design. Selection of this limit was based upon the level of inspection that is possible using standard ultrasonic non-destructive testing equipment.

Initial Study

Material Strength Curves

In order to understand the potential benefits of using higher strength steel for a predominantly compression structure, it is informative to consider the strut buckling curves for different materials. Figure 10 shows how the compressive strength of a CH section, p_c varies with slenderness and material grade.

The weighted average slenderness of the reference design is 53. (Slenderness, λ , is the ratio of the effective length of a column to the least radius of gyration of its cross section.) At this level of slenderness, there is a significant strength benefit in changing from cold-formed S355 to hot finished S500 with the compression strength increasing by 57% from 266.7 N/mm² to 419.2 N/mm².

It is also apparent that removing residual stresses by hot finishing has a significant benefit. For members with slenderness ratios in excess of 66, it can be seen that hot finished S275 actually has a higher compressive strength than cold-formed S355.

Figure 10 and Figure 11 illustrate that the maximum benefit in terms of member weight is found in stocky members with slenderness ratios in the range 50-60. For the most slender braces in this structure ($\lambda \sim 170$), compressive strength is controlled, primarily by Euler buckling and the increase in strength is reduced to 14%.



Figure 10. Strut buckling curves for circular hollow sections (CHS).



Figure 11. Relative compressive strengths compared to cold-formed S355 CHS.

N-M Interaction Diagrams

Even though the structure is predominantly an axial one, the members will still be subjected to applied bending moments and we, therefore, need to consider more than just its compression strength. Interaction diagrams describe the capacity envelope for a steel section and represent its ability to accommodate combinations of applied bending and axial forces. Put simply, the larger the envelope, the greater the capacity.

The following paragraphs discuss N-M (axial load – bending moment) diagrams for typical elements in the structure.

<u>Typical Chord.</u> Figure 12 shows the N-M interaction diagram for a typical chord section within the clad section of the fixed roof. The envelope shown is for a utilisation of 1.0 in accordance with 4.8.3.3.1 and 4.8.2.2 of BS 5950 Part 1. The step in moment capacity at zero axial load is due to plastic capacity being permissible when an element is subjected to tension. As previously discussed, the target was for members to have a maximum utilisation of 0.85 and it can be seen that the member forces satisfy this with very few exceptions.

It can be seen that for the same section size ($457 \times 32 \text{ mm}$) there is a significant increase in capacity on both the compression and tension sides of the plot. Using S500 steel it can be seen that this section could be reduced to a $457 \times 20 \text{ mm}$ CHS (a reduction in weight of 38%) and still achieve the target utilisation.

<u>Typical Brace</u>. Figure 13 shows the interaction diagram for a typical brace within the runway trusses. In this case because the member has an increased slenderness, the increase in capacity is less on the compression side than it is on the tension side of the diagram. It can, however, be seen that the wall thickness of the section could be reduced from 8 to 6 mm whilst still enveloping the member forces and allow a 25% reduction in the weight of the element.

<u>Typical Slender Brace.</u> Figure 14 shows the N-M interaction diagram for a slender brace used in the top surface of the fixed roof. It can be seen that there is significant potential for increasing the capacity of the member in tension using S500 steel. However, the section is presently limited by minimum wall thickness rules. Therefore, this additional capacity could only be realised by changing the section library to include smaller diameter sections.

The capacity of the connections would be limited by the can capacity which decreases with decreasing brace diameter and the degree to which the tension capacity could be practically increased is somewhat limited. In this study, sections of this type have not shown any saving in material quantity.



Figure 12. N-M interaction diagram for stocky section¹, 457 CHS (λ =36).

¹ Tensile axial forces positive



Figure 13. N-M interaction diagram for member with intermediate slenderness, 168 CHS (λ =119).



Figure 14. N-M interaction diagram for a slender member, 139.7 CHS (λ =172.4).

Local Buckling Effects in Cross Sections

BS5950 controls local buckling effects of cross sections by placing limits on the D/t (diameter/wall thickness) ratio for circular hollow sections which are a function of the yield strength of the material. Any section which falls outside of these limits is classified as slender and effective section properties are defined which limit the percentage of the section capacity that can be utilised.

Figure 15 and Figure 16 show the compression and bending strengths respectively of a truss' chord section for varying thickness. It can be seen that S500 steel sections will be classified slender at lower D/t ratios than S355 sections. It is, therefore, clear that a S500 design would be most efficient by concentrating on the use of a more compact section library.

It can also be seen that the reason the minimum chord size in the S355 reference design was taken as 457 x 10 mm CHS was that it is the first section above the limit for slender sections in bending. In S500 steel, this section is actually marginally slender for both axial compression and bending and would not be allowed by the code to utilise its full strength.



Figure 15. Compressive strength limits for local buckling. Compression strength/yield strength ratio vs thickness (λ =36).



Figure 16. Bending strength limits for local buckling. $M_c/P_vZ = (plastic moment capacity)/(first yield moment).$

Shear Capacity of Sections

Typically elements are subjected to very low shear since global shear forces are generally carried via axial forces in bracing elements. As all shear forces are less than 0.6Pv there is no need to consider any reduction in moment capacity due to shear-moment interaction². This is true for both the S355 and S500 designs.

Impact on Connection Design

It is outside the scope of this study to complete a redesign of the fixed roof connections. However, to allow the member design to be completed, it is important to consider what changes to the connection design would be required to accommodate the change in material grade for the members.

Different fabricators have different preferences for the way that they wish to form connections but it is thought that canned connections discussed earlier have been effective and it is assumed that this type of connection would also be used for any redesign using S500 steel.

Connection weight adds to the total dead load of the structure which therefore has an impact on the size of the members required. For this type of construction, it is suggested that the strength of the connections should at least match the strength of the members used.

² Shear capacity, Pv = 0.36 pyA in accordance with BS5950

<u>Strength.</u> ³CIDECT Design Guide 1 (2nd Edition) restricts the specified nominal yield strength of the material for CHS connections to 460 N/mm². CIDECT's capacity formulae are based on S355 material and it therefore applies a 0.9 reduction factor to the capacity of connections formed with S460 steel.

CIDECT places restrictions on strength capacity in order to limit the amount of deformation of the chord which occurs during plastification of the CHS cross section. Furthermore, it presumes that higher strength material may have a lower ductility and therefore less capability to accept deformation.

As discussed earlier, the S500 material used in this study has good ductility. As such, it is thought that the above restriction is conservative and it may be reasonable to take advantage of the additional yield capacity of the section but combine it with a strength reduction factor of 0.88. This would, of course, require agreement with the local authorities.

The strength of the connections in the fixed roof structure was generally controlled by plastification of the chord face rather than punching shear imposed by the brace onto the chord.

CIDECT's formulae for the strength of the connection in resisting plastification is proportional to $p_y t^2$ and means 500 N/mm² steel would provide an 11% increase in the strength of the connection. Assuming the same chord and brace diameters are used, this increase in strength could then be directly translated into a reduction in material usage.

<u>Fatigue</u>. For steel strengths up to around 700 N/mm², the fatigue strength of a connection is not dependent on its material strength and is dependent only upon the fluctuating stress in the connection. Therefore, where connection designs are controlled by the fatigue stresses there would be no apparent benefit in increasing the material strength.

However, the majority of the connections are not governed by fatigue stresses and it is therefore important to consider that if a reduction in thickness is permitted for strength, whether fatigue stresses would then dominate and prevent the full increase in strength from being realised.

Critical stresses often occur in the chord rather than the brace section itself. For X, T and K connections, the stress concentration factors provided by DNV RP C203 are proportional to $1/t^{1.4}$ in the worst case. Using this ratio of stress as a basis it is possible to make informed preliminary estimates of which connections would become fatigue critical as the thickness required for strength decreases.

³ Comite International pour le Developpement et l'Etude de la Construction Tubulaire

Initial Study Conclusions

Based on an initial comparison of sectional capacity and preliminary investigation of connection capacities, the following conclusions can be drawn:

- Significant savings in material would be expected for strength-governed stocky members;
- Savings in slender compression members would be possible, but it is anticipated that these would be more limited;
- If S500 steel were used for the connection design, savings in material within the connections would be possible but these would be less direct than for member design;
- Fatigue would govern a larger proportion of elements of a S500 design than for the S355 reference design;
- Minimum section sizes for the reference design were determined by the ability to carry out post-welding inspection and to avoid using slender sections. For S500 steel, these requirements govern a higher proportion of the elements resulting in a design which is less optimised for strength than the S355 design.

Optimisation Study

Preliminary Optimisation

The initial studies indicated that S500 could provide a significant increase in capacity for many of the elements and hence a significant saving in tonnage. However, as the sectional area of the members decreases, so does their stiffness. This leads to amplification of loading effects such as p-delta (see below) and resonant response to fluctuating wind loads (see below) which increase in importance as the stiffness of the structure decreases. Conversely, the change in weight of steel decreases the total dead load on the structure meaning that further savings are possible. To ensure that this latter effect has been fully accounted for, optimisation runs were started from a position where all members were given a minimum possible section size and the load increased only as far as it needed to for each design iteration.

As a result of increases in load effects due to decreases in stiffness, it has not been possible to fully realise all of the initial savings once these effects have been taken into account.

The following sections discuss the changes in stiffness that occurred with the change in material strength and the effect that this has had on the design. A final tonnage of steelwork is given and discussed afterwards.

Serviceability Limit State Design

Generally, the roof steelwork is governed by strength requirements and once designed it has been checked to ensure that deflections are reasonable for cladding and supported structure.

An insight into how the stiffness of the structure has changed with the increase in strength of the material can be gained by comparing the deflection of the structure in a few key load cases.

Figures 17 to 19 show the deflected shapes for the total dead load deflections and that of a wind load case which governs many of the element designs. It can be seen from Table V that whilst deflections due to applied loadings generally increased, the re-designed structure remains very stiff with live load deflections which are less than L/3600, compared to a limit of L/360 or L/500 for normal roof structures (where L is the roof span of 310 m).

Load Case	S355	S500
Self-weight	96.6	108.4
Total DL + LL Open	244.3	272.8
Total DL + LL Closed	329.8	385.2
Wind Load*	66.1	75.9

Table V. Comparison of Peak Resolved Deflections in Structure (Values in mm)

* critical North-South wind load cas



Figure 17. Total dead load deflection (movable roof open) for S500 design.



Figure 18. Total dead load deflection (movable roof closed) for S500 design.



Figure 19. Deflection due to critical wind load case in S500 design.

Dynamic Augmentation

Wind loads for the design of the roof structures have been derived from wind tunnel testing using an influence surface approach. Time histories of the measured wind pressure across the model are analysed and multiplied by weighting functions which consider the structural behaviour of the structure in order to find the most onerous set of realistic loadings to apply to the structure. An example of a simple influence surface and the resulting pressure distribution used in design is shown in Figure 20.



Figure 20. Influence surface (left) and corresponding pressure distribution from wind tunnel results (right).

Although the roof structures are not aerodynamically unstable, they are sufficiently lightweight and have a low enough frequency that their resonant response to fluctuating wind needs to be considered at the ultimate limit state. Resonant responses to the first 15 natural modes of vibration of the structure were calculated and then combined using a square root sum of squares approach to find the total peak additional dynamic force in the members.

These results were then used to find an equivalent dynamic augmentation factor to use for design. It was found that applying a factor of 1.3 to results of the static loads from the wind tunnel was sufficient to envelope the dynamic effects in the members.

<u>S500 Design</u>. The use of higher strength steel has resulted in a reduction in stiffness of the members. Consequently, the natural frequency of the structure has decreased as has the modal mass. This means that the resonant response to the wind would be expected to increase.

It was not possible as part of this study to recalculate the modal responses for each of the modes of the roof. As a result, an increased dynamic augmentation factor of 1.4 has been estimated for use in the design. It is believed that this is sufficiently conservative to cover the increase in resonant response that would occur for the lighter structure.

P-delta

P-delta covers the second-order geometric effects which cause changes in base shear and overturning moment as a result of sway of the stadium roof. The roof is sufficiently stiff that it does not exhibit significant p-delta behaviour. However, under normal Ultimate Limit State (ULS) factored loads it has a global buckling mode where $\lambda_{cr}^4 < 10$ and is therefore classified as "sway sensitive" by BS5950.

 $^{^{4}\}lambda_{cr}$ – elastic critical load factor = ratio of elastic buckling load to factored loading

Since $\lambda_{cr} > 4$, p-delta forces can be accounted for via either the Amplified Sway Method or the Analytical Method as detailed in Section 2.4.2.7.1 of BS5950. We have chosen to adopt the Amplified Sway Method and verified that this is conservative by comparing a selection of answers to the Analytical Method.

The Sway method amplifies loadings that induce sway effects (such as wind and patterned live load) by k_{amp} which is calculated as:

$$k_{amp} = \frac{\lambda_{cr}}{\lambda_{cr} - 1} \tag{2}$$

The fundamental buckling mode is shown in Figure 21 below.



Figure 21. Fundamental buckling mode: closed roof.

The results from this method have been verified as conservative by comparing them to the results from a p-delta analysis that uses the forces from elastic analysis to approximate geometric stiffness and combines this with the global stiffness matrix so as to account for geometric non-linearity.

The comparison has been undertaken for load-cases that generate the greatest sway effects such as patterned live-load and asymmetric wind loading.

<u>S500 Design</u>. The reduction in axial stiffness of the members caused by reducing sections sizes has meant that there has been a reduction in the global buckling load factor compared to the S355 reference design, Table VI. This resulted in an increase of approximately 5% in the sway amplification applied to lateral loadings and reduced the effectiveness of the change in steel grade in saving material.

Grade	λ _{cr}	k _{amp}
S355	6.41	1.185
S500	5.54	1.248

Table VI. Critical Buckling Load Factor

Buckling

Three different types of buckling mechanism have been considered in the roof's design. These are:

- Local buckling which is satisfied by limiting the bending capacity of semi-compact sections;
- Single element buckling such as a brace buckling between node points. This is addressed by the member design checks;
- Global buckling such as a truss buckling laterally when the top chords are in compression.

<u>Global Buckling Restraint Forces.</u> There are two mechanisms for global buckling failure, which are:

- Elastic buckling failure (when $|\lambda_{cr}| < 1.0$);
- Plastic buckling failure (when residual stresses and amplifications of fabrication/ construction imperfections result in elements exceeding their member capacity).

Linear modal buckling analysis has been undertaken by Arup's GSA software buckling analysis for a number of load cases that generate either the greatest bending in a truss or combined maximum axial force. The minimum load factor from this analysis is 4.05 which demonstrates that the structure is not at risk from elastic buckling failure.

<u>Initial Imperfection</u>. BS5950 has four different strut-curves that account for different levels of residual stresses, along with an allowance for fabrication/construction tolerances. It is possible to equate these to a single value initial imperfection via:

$$x_0 = \frac{r_y}{v} \frac{a}{1000} L_d$$
(3)

For cold-formed and hot finished CHS structures this value can be taken as $L_d/262$ and $L_d/720$ respectively, where L_d is the buckling length.⁵

Based on first principles and EuroCode 3 it has been possible to show that the initial imperfection would be amplified by applied loading in such a way that the buckling restraint forces can be determined by multiplying the analysis mode shapes by:

$$q_i = x_0 \frac{1}{\lambda_{cr} - 1} \tag{4}$$

Global buckling forces, calculated as described above, were then added (+/-) to those generated by p-delta forces and each member capacity checked. Member capacities are checked with forces, including global buckling restraint forces, since it is feasible to have a global imperfection occur simultaneously with local element imperfections.

<u>S500 Design.</u> The lowest buckling mode in the fixed roof occurs in runway truss RWT1 (Figure 3) as the top chords buckle laterally under compression, Figure 22.

Following initial optimisation runs for strength, it was found that the elastic critical load factor for this mode had fallen to $\lambda_{cr} = 2.89$. Although this buckling mode is really a local mode rather than "global" buckling, this load factor is lower than the minimum load factor of $\lambda_{cr} = 4.0$ upon which the design was based. As a result, the reduction in mass of the members within RWT1 over the pitch has been limited by the need to maintain the truss stiffness.

If the design had originally been conceived using S500, measures would have been taken to increase the width and depth of this truss to ensure that its design was controlled by strength rather than stiffness.

 r_y = radius of gyration, y = distance from neutral axis to extreme fibre (D/2), a = Robertson constant



Figure 22. Lowest buckling mode: compression buckling of top chords of RWT1.

In other areas of the roof, the reduction in initial imperfection associated with the use of hot finished sections allowed a reduction in the buckling restraint forces that helped to reduce the overall tonnage.

Fatigue

Fluctuation in stress in the fixed roof structure is primarily caused by the changing gravitational loading caused by the movable roof being moved between the open and closed position.

One primary load cycle is caused by moving the roof from the open to closed position and back to open. Allowing for an initial commissioning period and annual maintenance movements, the 50 year design life is expected to generate 10,700 of these loading cycles. Whilst this is a relatively small number of load cycles, the associated stress ranges are sufficiently high that fatigue needed to be considered in the design.

For both welded and bolted steel structures, the fatigue life is normally governed by the fatigue behaviour of the joints, including both main and secondary joints. A factor of safety on design life of 2.0 has been used for non-critical inspectable connections and 10.0 for critical uninspectable connections.

In order to allow the connection design to achieve these safety factors in a reasonably practicable manner, members were sized so that their axial stress range due to movement of the movable roof did not exceed 100 N/mm².

An indication of the fluctuating stresses caused by the movable roof opening and closing is provided in Figure 23.

<u>S500 Design</u>. The increase in strength from the use of S500 steel has allowed member sizes to be reduced. As a consequence, the fluctuating stresses in the members have increased. This means that a larger proportion of the members are now controlled by a fatigue criteria rather than strength.

Approximately 7% of the members within the optimisation set are controlled by fatigue criteria when using S500 steel (Figure 24). This compares to 3% of the members being controlled by fatigue in the reference design. As indicated in Figure 25, in the S500 design 9% of the member weight is within members governed by fatigue criteria compared to 5% in the S355 design.

Connections

For the construction design, approximately 35% of the connections, which were designed by hand calculation, were governed by fatigue with the remainder governed by strength requirements.

Using the relationships between strength and can thickness outlined previously, it has been possible to make an estimate of the connection weight that would be needed if the cans used were S500 steel.

It was found that for cans fabricated in S500 steel the connection weight would contribute an additional 18-20% to the self-weight of the structure over and above that calculated based on the centreline geometry of S500 members.

This compares to the 17.5% allowance that was made in the construction design using S355 steel for the cans.

Due to requirements to procure the steelwork in advance of the final connection design being completed, the construction connection design contains areas which are not heavily utilised and with a different project program, a thinner can could have been justified. Therefore, a more complete comparison between the material strengths would suggest a minimum connection allowance of 15-16% for S355 members and connections and 18-20% for S500 members and connections.



Figure 23. Variation in axial stress in a runway truss due to movement of the movable roof (S500).



Figure 24. Proportion of members controlled by strength and fatigue criteria.



Figure 25. Proportion of steel mass controlled by strength and fatigue criteria.

Results of Optimisation and Discussion

The final design has found that for S500 steel with the same section library as the reference design, it has been possible to complete the fixed roof design with 4644 tonnes of steelwork for the fixed roof structure. As shown in Table VII, this amounts to a saving of nearly 15% on member size or 13.6% on total steel tonnage once the connections are accounted for.

Therefore, by using S500 steel for both the members and the connections, it would be possible to save 855 tonnes of steel within the fixed roof structure.

The following sections outline where savings in steelwork have been made and highlight areas where amendments to the design may have allowed further savings to be made.

Areas of Roof Where Savings Have Been Made, Figures 26 and 27

As indicated in Figure 26, the S500 design has allowed a reduction in mass of some elements of up to 62%, compared to their weight in the S355 design. As an individual value this is towards the very top end of what would be expected for axially governed sections, based on the sectional capacity comparisons conducted in this paper.

It can be seen that the largest changes in sectional mass were concentrated in the chords of the secondary trusses. The runway truss member sizes have tended to be controlled by fatigue requirements and have, therefore, seen more modest changes in size. This resulted in a marginal tendency for the structure to span in a more North-South direction creating a positive feedback loop which reinforced the initial savings in size in the secondary chords.



Figure 26. Plan of fixed roof showing changes in structural mass per element (%).



Figure 27. Plan of fixed roof showing changes in structural mass per element (kg).

Figure 28 provides an indication of the relative change in mass of the elements within runway truss 2 (RWT2). It is clear from this figure that the most significant savings occurred in the top chords of the structure. No change in section size in the bottom chords of the truss within the clad roof has taken place because these sections were already at minimum section sizes (457 x 10 mm CHS), even in the S355 design. Over the pitch, it has not been possible to reduce the size of the bottom chords because these are limited by the 100 N/mm² stress range for fatigue requirements



Figure 28. Section through RWT2 showing changes in structural mass per element (%).

The fact that the use of higher strength steel has only allowed modest savings in the selfweight of the bracing members suggests that, economically, the best overall solution may be to use a mixture of steel grades depending on the element. For example, cold-formed S355 could be used for slender bracing members (273 mm diameter and below) and hot finished S500 steel could be used for stockier chord elements.

Distribution of Utilisation

As noted earlier, both the S355 and S500 designs have been carried out using the same section library. This section library was based upon maintaining a minimum wall thickness of the circular hollow sections for welding and inspection purposes. Furthermore, a consistent outside diameter has been used for the chord members throughout the fixed roof over the pitch and underneath the clad roof (508 mm and 457 mm respectively).

The target strength utilisation for optimisation that has been used is 0.85 ("Parameters", above). Due to the fact that thickness and diameter both change in discrete steps, it was inevitable that many members would have lower utilisations than this value. Furthermore, some member sizes have been governed by minimum section rules and as a result are not fully utilised.

Figure 29 below provides an indication of the distribution of utilisation across the roof for both the S355 reference design and the S500 redesign.



Figure 29. Distribution of utilisation with number of elements.



Figure 30. Distribution of utilisation with mass of steelwork.

It can be seen from Figures 29 and 30 that the S355 design generally has a higher member utilisation than the S500 design, with 70% of the total tonnage contained in members with a utilisation greater than 0.6. For the S500 design, the comparative value has reduced to 50% of the total tonnage. The weighted utilisation for the S355 and S500 designs are 0.646 and 0.564 respectively.

The apparent reduction in structural efficiency is due to the fact that more of the sections in the S500 design have been reduced to their minimum size, meaning the saving in steel weight is less than might otherwise be possible. For example, the S500 design has 61% more members which use the minimum chord size of 457×10 mm CHS than the S355 reference design.

This suggests that further savings in steel weight could be made for a S500 design by varying the section library accordingly. One of the key areas where savings would be possible would be by reducing the outside diameter of the chord members within the clad roof and over the pitch. A minimum section size of $406 \times 8 \text{ mm}$ CHS could then be used in place of a $457 \times 10 \text{ mm}$ CHS.

Areas of Further Study/Savings

The preceding discussion has shown that the use of S500 steel has the potential to offer significant savings in the material quantities required in the roof structure design. Although the original intent of the study was to carry out a like for like comparison of the fixed roof design, there are a number of areas where further development of the design could allow further savings to be identified. These are discussed and evaluated in the following paragraphs.

<u>Movable Roof.</u> The patch loading applied by the varying position of the movable roof is one of the most significant load cases applied to the fixed roof structure. The self-weight of the steelwork within the movable roof accounts for approximately 55% of its total dead load.

If S500 steel were used within the movable roof to reduce its self-weight, not only would there be savings in the weight of the movable roof itself, but there would also be a reduction in loading on the fixed roof. Assuming a similar 15% reduction in the self-weight of the movable roof structure, this would amount to approximately 180 tonnes of further savings within the movable roof panels and an 8% reduction in the total dead load of the movable roof.

This would create a positive feedback loop which would allow further savings in the fixed roof structure.

<u>Section Library</u>. As discussed above, by changing the section library used for the fixed roof structure and reducing the minimum allowable section sizes, it would be expected that further savings in material usage would be identified.

<u>Geometry Developments.</u> As discussed above, the amount of steel required in the smallest runway trusses across the pitch is controlled by the need to maintain its stiffness. In a design for construction using S500 steel the geometry of this truss would be investigated to increase its stiffness with increased width and depth, with a view to allowing a reduction in the total material usage.

<u>Further Study Results.</u> Arup have carried out additional design calculations to evaluate the effectiveness of designing the movable roof in S500 steel and making changes to the section library in the fixed roof. In order to ensure that comparisons are fair, no changes to geometry of the trusses have been made as part of the study.

Whilst the movable roof has not been re-designed in detail using S500 steel, it could reasonably be assumed to achieve a similar change in element mass as the fixed roof has seen. As noted in Table VII, it is predicted that the change in self-weight alone would allow a further 59 tonnes of steel to be removed from the fixed roof design in addition to the 180 tonnes that could be saved in the movable roof itself.

Finally, a study was undertaken where all 457 mm diameter sections were substituted with 406.4 mm CHS elements and was combined with the effect of changing the movable roof to S500 steel. These combined effects allowed a significant change in the mass of the elements which previously used a $457 \times 10 \text{ mm}$ CHS resulting in a reduction in steel tonnage of more than 400 tonnes.

	Member		Connection			
Grade	Mass (tonnes)	% Change of Design	Mass (tonnes)	% Change of Design	Total (tonnes)	% Change from Base Design
S355 members and connections	5451	0.0	861 (16% allowance)	0.0	6312	0.0
Hot formed S355 members and connections	5285	3.0	861	0.0	6146	2.6
S500 members and connections	4644	14.8	813 (18% allowance)	5.6	5457	13.6
S500 and S500 movable roof	4586	15.9	813	5.6	5398	14.5
S500 with movable roof and reduced section library	4171	23.5	813	5.6	4983	21.1

Table VII. Summary of Fixed Roof Tonnages



Figure 31. Distribution of utilisation with mass of steelwork.

It can be seen from Figure 31 that the distribution of member utilisation with a reduced section library is more concentrated in the 0.7 to 0.8 range than the initial S500 design. It is not, however, as highly optimised for strength as the S355 reference design was. With the reduced section library, it was found that 12% of the mass of the elements were governed by fatigue compared to 3% in the S355 reference design.

In total, the reduced section library and re-designed movable roof is predicted to allow a saving of 1329 tonnes of steel in the fixed roof – over 21% of its mass – in addition to the savings in the movable roof structure.

Overall Efficiency of the Design

Owing to its domed construction, the Singapore Sports Hub National Stadium roof was already an efficient design when measured on the basis of material used. This study has shown that by redesigning the structure with higher strength steel it would be possible to make further savings in material usage.

Roof structure projects take many different forms and place varying demands on the requirements of the substructure design. Furthermore, complexity of geometry and connection design can have significant implications on the total cost of fabrication. Once the fact that structures are located across the world including regions with different climates and loading regimes is taken into account, it becomes difficult to make any comparison between them that is truly fair.

Having said this, one means of comparison that provides some insight into the relative efficiency of a design, is to compare the amount of material used with the span of the structure. Figure 32 shows a graph of other built examples of long span roofs and the mass of steelwork required in comparison to their span.



Steel roof - Span vs Weight

Figure 32. Comparison of roof span and structural steel weight per unit area for selected projects.

A general rule of thumb for a long span roof up to 200 m, is that it will require approximately 1 kg/m^2 for every metre that it spans⁶.

It can be seen from the plot that with a total material usage of 130 kg/m^2 for a 310 m span, the S355 design compares very favourably to the rule of thumb. The S500 design would be even lighter with a material usage of 117 kg/m². Taking account of a S500 movable roof and section library changes would allow the total to reduce to 106 kg/m² equivalent to only 0.34 kg/m² for every metre of its span.

Conclusions

When complete, the National Stadium roof structure will be an iconic structure of national and international significance utilising the latest innovations in wind engineering applied to parametric structural engineering design and optimisation to create a 310 m span roof with a structural weight per square metre of footprint that would be considered efficient for a 100 m span.

This study has shown that potential savings in material use could have been made if higher strength steel had been economically viable and the project could have been competitively tendered on this basis. This study has shown that if the design had been completed using hot finished S500 steel in place of cold-formed S355, a reduction of 13.6% of the total tonnage of steel within the fixed roof structure could have been realised. By extending the use of S500 to include the movable roof and revisiting the section library used, the saving could be increased to 21.1% of the total tonnage.

For a roof of this scale, the structure represents a significant proportion of the total cost and such savings demonstrate that if it can be shown to be economically viable, S500 steel would have the potential to deliver improvements in the efficiency of future steel projects.

List of Relevant Standards

BS 6399 Part 1:1996 Loadings for Buildings, Code of Practice for Dead and Imposed Load.

BS 6399 Part 2:1997 Loading for Buildings, Code of Practice for Wind Loads.

BS 6399 Part 3:1988 Loading for Buildings, Code of Practice for Imposed and Roof Load.

CP3 Chapter V-2:1992 Code of Basic Data for the Design of Building, Wind Loads.

BS 5950-1:2000 Structural Use of Steelworks in Building – Part 1: Code of Practice for Design – Rolled and Welded Sections.

BC1:2008 Design Guide on Use of Alternative Steel Materials to BS5950.

BS EN 1993-1-1:2005 Design of Steel Structures, General Rules and Rules for Buildings.

⁶ Area measured as plan area covered by the structure. Weights include connections.

BS EN 10164:2004 Steel Products with Improved Deformation Properties Perpendicular to the Surface of the Product, Technical Delivery Conditions.

BS EN 10210-1:2006 Hot Finished Structural Hollow Sections of Non-Alloy and Fine Grain Steels, Technical Delivery Conditions.

BS EN 10219-1:2006 Cold Formed Welded Structural Hollow Sections of Non-Alloy and Fine Grain Steels, Technical Delivery Requirements.

AWS D1.1:2005 Structural Welding Code Steel.