ST-01:
STUDY OF MEGA-DECK FOR INTEGRATED DEVELOPMENT

Final Report

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The final draft of the report for this Final Year Project which will be presented to the Nanyang Technological University in partial fulfillment of the requirements for the Degree of Bachelor of Engineering

2012/13
This final year project is a study on structural truss decks which can be used to hold industrial loadings over existing developments. The study aims to find some of the more cost-effective structures – those with the longest span and least deflection under the maximum loading, using the least amount of structural steel. Steel sections underwent software analysis, while selection of joint connections was based more on industry research. However, instead of identifying the ultimate structurally-effective cost-efficient truss deck, the results show the range of prices for the structures analysed. Thus, this study gives an understanding of the minimal structural and financial requirements of realizing such an innovative idea.
ACKNOWLEDGEMENTS

This student would like to thank the following:

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LIST OF SYMBOLS

L    Span of structural truss deck

DL   Dead load

LL   Live load

WL   Wind load
1 INTRODUCTION

1.1 Objective of Study

The following is the project description provided during the Final Year Project (FYP) selection process:

“The use of structural truss to deck over a bus-terminal and train station has created usable land. However, the challenge is to develop a more economical and efficient structural truss, for the continuation of heavy industrial development. Besides depot and bus-terminal that could be deck over, the study should look into other low rise and large area developments, like port container yards.

The objective of this project includes:
1) Identifying various structural decks option suitable to deck over port container yard;
2) Comparing the various options and recommending one economical and effective option;
3) Designing and studying the effectiveness of the structural deck which includes running computer stimulations.”

1.2 Scope of Work

As this is a joint project between NTU and JTC Corporation, there are two supervisors – A/P Joseph Cheung Sai Hung and Mr. Ng Kian Wee. Discussion with Mr. Ng allowed the objectives of the study to be more focused, agreeing that the study is to be a simplistic and rough guideline in estimating the cost of building future structural mega-decks, specifically that of space trusses formed by structural steel. A/P Cheung had ensured that the checks done for this study were sufficient and also recommended ways that it can be further delved into.

The scope of work with respect to the objectives in Section 1.1 consists of:
1) Identifying a few space truss systems that can hold a higher range of loadings over a higher range of spans, based on literature reviews;
2) Identifying one truss design by subjecting the systems to a round of preliminary capacity and deflection checks via the software, SAP2000;
3) Modifying the steel section of chosen design to suit a higher range of loadings over a higher range of spans while conducting capacity, buckling and deflection checks on the above modified grids, via SAP2000;

4) Testing the robustness of the modified grids by increasing its maximum live loading;

5) Deciding on a joint method and understanding the technology and pricing behind it;

6) Estimating the “per square meter” cost of building each of the selected system, based on the steel sections and joints used, and construction machineries and methods required;

7) Identifying the most economical deck from among the analysed space truss systems by balancing economic feasibility with structural effectiveness.
2 LITERATURE REVIEW

The literature focuses on two main topics – understanding of the truss itself and the construction of the truss. For the first focus, a search on structural decks and space trusses mostly yielded studies on roofing decks which use truss systems. Their sole purpose is to provide shelter over large spans, and not to support imposed loads over a large area. Only one example of a load-bearing space truss was found – a tennis court built on the roof of United Arab Emirates’ Deira City Centre. Thus, in order to realise this project, it was important to understand the basic principles behind space trusses while keeping in mind the example of Deira City.

For the next focus, on the construction of structural decks and truss systems, part of the literature collected was obtained from industrial research of a few construction companies in Singapore, via either their websites or personal communication. As a result, there is a lack of citations for this portion, but the information quoted would have been ensured to be as accurate as possible.

2.1 Definition of Space Trusses

According to a lecture series by two professors from the Indian Institute of Technology Madras (2006), a space truss is a three-dimensional frame consisting of pin-connected members which have no moments or torsional resistance but carry only axial compression or tension. A space grid comprises of two or more sets of parallel beams on a single plane intersecting each other at various angles. External loadings are loaded normal to that plane. Thus, a space truss would need a minimal of two layers of space grids to form the chords, along with braces to connect these layers together. This is known as a double-layered grid.

One of this project’s outcomes is to provide a range of “per square meter” cost of building the deck. Logically, a double-layered grid would give the least weight per square meter, resulting in the least cost per square meter, as compared to three or more layers of grids. This logic is shown in examples of built structural decks, where most of them exist in the form of single-layered structures, such as the Al-Wahda sports hall at Abu Dhabi and the Terminal 5 roof at London’s Heathrow Airport. Thus, while future studies can look upon the effectiveness and cost-efficiency of multiple-layered grids, this FYP will attempt to find an effective and cost-efficient space truss with double-layered grids.
2.2 Advantages and Disadvantages of Space Trusses

The following were paraphrased from Tata Steel’s online resources (2012), selected from a list of advantages and disadvantages of space grids. Since space trusses are a combination of at least two space grids, as mentioned in the previous section, the following discussed points have relevance to this study.

The advantage of using space trusses is that all elements of the truss contribute to the load-carrying capacity, and loads are distributed more evenly to the supports, reducing the cost of the supporting structures. This is especially helpful when heavy moving loads, such as overhead cranes, are applied to the space trusses. As we will be applying large industrial live loads, this property is to our benefit. Furthermore, space trusses have many members, resulting in a high static redundancy. Thus, the failure of one or a limited number of elements will not lead to collapse of the entire structure. Failure modes include the buckling of a compression member. The survival of the structure in the event of a failure depends whether there exists an adequate alternative load path. Critical elements include those adjacent to column supports.

However, space trusses are not without disadvantages. They are expensive, compared with alternative structural systems, especially when used for short spans of 20 to 30m. Also, the number and complexity of joints can lead to longer erection times on site. This is very dependent on the truss system and steel sections chosen. Lastly, space trusses have a standardized modular nature, imposing geometric restrictions, making it difficult to use irregular plans shapes and impose control on the locations of the supports.

2.3 Topology of Space Trusses

Ramasamy et al. (2002) had detailed the decisions made in the construction of a space frame to support a concrete tennis court above the Deira City centre. One of the pivotal decisions was in selecting one out the six common topologies seen in Fig. 2.3.1. The stiff and structurally efficient topology of square on square, set diagonally (Type 2) had been chosen.

In Chapter 24 of Handbook of Structural Engineering (2005), Tien had provided a greater variety of topologies for double-layer grids along with how to choose the most suitable type. The ten different grid topologies have been split into three groups – latticed trusses (Fig. 2.3.2a to 2.3.2d), square pyramids (Fig. 2.3.2e to 2.3.2h) and triangular pyramids (Fig. 2.3.2i to 2.3.2j).
Fig. 2.3.1. The six grid arrangements normally used for space frames

(Ramasamy et al., 2002)

Again, the importance of choosing an appropriate grid topology was stressed upon as it “will have direct influence on the overall cost and speed of construction” (Tien, 2005). The factors to consider are the shape of building plan, size of span, supporting conditions, magnitude of loading, roof construction, and architectural requirements. The grid should also be built of relatively long tension members and short compression members. With respect to this FYP, here are the factors that were considered:

1) Shape of building plan – square structural deck will be built in order to simplify the methodologies for this project;

2) Supporting condition and architectural requirements – the architectural requirement of building the deck above current developments dictates the support conditions. As the land space below the deck has already been utilised, support columns will have to be built along perimeters.
Fig. 2.3.2. Framing system of double-layer grids (Tien, 2005)
The grid types suitable were types (a), (b), and (e) through (h). However, types (a) and (b) are considered too simplistic and weak due to the two planes lying directly on top of each other. Type (e) and (g) are the same as Type 1 and 5 of Fig. 2.3.1 respectively, both of which were not considered as superior as Type 2 of Fig. 2.3.1, but type (e) is the only square pyramid grid that does not have tension members (lower chords) that are longer than its compression members (top chords) which plays a part in grid effectiveness as said by Tien. Based on both sources, there were four grid topologies to choose from – Type 2 from Fig. 2.3.1, types (f), (g) and (h) from Fig. 2.3.2. The chosen topology will be explained in chapters 3.1 and 4.1 later on.

2.4 Configuration of Space Trusses

Tien (2005) also finds it “necessary to determine the depth and module size”, where “the depth is the distance between the top and bottom-layers and the module is the distance between two joints in the layer of the grid” as seen in Fig. 2.4.1 below. They are mutually dependent as the angle between the braces and the chords should be between 30˚ to 60˚, so that the forces in and lengths of the braces will not be too excessive, and so that there will not be too many braces overcrowding the grid.

Previous studies had been conducted on the depth and module relationships for concrete slab and steel purlin roofing systems. Fig. 2.4.2 summarises such relationships for grid of types (a), (b), and (e) through (h). Once the span of the structure, L, has been decided upon, the module number and span-depth ratio can be calculated. From there, the length of the modules and depth of the space truss can be determined.

Lan (1986) had derived the following formula in determining the range of span-depth ration for roofing systems composed of steel purlins and/or metal decks

\[
L/d = (510 – L)/34 ± 2
\]

(Eq. 1)

where \( L \) is the short span and \( d \) is the depth of the double-layer grids.
2.5 Types of Steel

Hollow sections are known to have certain advantages compared to other sections especially when the loads are applied at the joints as is the case for space trusses. The advantages are:

1) Less surface area than open sections, reducing the cost of maintenance and painting;
2) Higher torsional resistance;
3) More efficient in axial compressions;
4) Higher frequency of vibration than other open sections when under dynamic loading;
5) More aesthetically-pleasing.

Circular sections are used more often than rectangular ones as they have the added advantage of moments of inertia that are the same in all directions. Thus, circular sections of steel grades adhering to BCA Academy’s Design Guide (2008) will be used.

Based on the available design standards, it is also necessary to specify if the steel is hot-rolled or cold-formed. Cold-formed steel has the advantage of being formed at room temperature, making the material harder and stronger, decreasing the thickness of the material. Its lighter weight is more economical for mass-production, transportation and installation, making it the preferred option for this study.

Another point to consider is the use of unalloyed versus fine-grained steel. According to the ISF Welding Institute based in RWTH Aachen University (2004), welding problems caused by unalloyed steels are fewer and less complicated than that of fine-grained steels. Figure 2.5.1 illustrates these problems clearly. Welding ability of a steel type is taken into consideration because it is one of the more possible joint connection methods, to be discussed in the next section. Less complications lead to a lower likelihood of problems, thus reducing the need for maintenance and repairs. This means that unalloyed steel is more economical in the long run.
2.6 Types of Joint Connection Method

In the chapter Introduction to Space Frames, Ramasamy et al. (2002) split the connectors into modular and nodular. While their research was based on space frames, it is worth noting that space trusses are a subset of space frames, making it possible to utilise either methods. Modular units are quicker to assemble in the field and eliminate the need for expensive node connectors, but they are more for open sections due to the use of high-strength friction bolts and bolting for open sections is straightforward. In hollow sections, node connectors are more commonly used, among which are the MERO and Octatube systems, which have been used in Singapore. For example, the decks at Wheelock Place and Jurong East MRT Station use MERO’s KKW System, while the roofs at Raffles City Shopping Center and Forum Galleria use Octatube’s system. Both systems can be seen in Fig. 2.6.1.

Welding the joints of conventional steel sections is another possibility. Welding allows for greater structural rigidity and strength. However, its assembly, transport and specialised manpower costs are very high, making it unpopular.
Stacked end-flattened connections also exist for structures made of hollow steel sections. These are made by bolting the intersections of individual sections, unlike the modular system which bolts along adjacent or connecting steel sections, as can be compared in Fig. 2.6.2. According to Freitas et al. (2011), “one of the most common connections used for steel space truss is the connection obtained by staking end-flattened tubes and joining them with a single bolt” (p. 494). Its advantages lie in the easy transportation, fast assemblage, lowered costs and easily-available workforce. However, the strength of the structure is highly dependent on the bolt strength and due to the end-flattening, the steel tubes’ stiffness will be reduced, decreasing overall structural strength.
### 2.7 Design Checks

SAP2000 (CSI, 2009) provides design checks that are in accordance with EC3-2005. The first check needed for this project is the load-dependent design-analysis checks. The design algorithm of these checks are specific to structures constructed wholly of pin members and consists of the following, all of whose flow charts can be found in Appendix A:

1. Member design – consists of section classification;
2. Design axial resistance – consists of axial area, tension and compression check;
3. Design axial buckling resistance – consists of axial buckling check;

The values for the factors of the first six modes of buckling are then found by running the optimized structure through a separate buckling analysis.

Deflection has to be observed manually and the deflection limit has to be user-determined. As deflections due to permanent loads can be precambered during construction, deflection has to depend on vertical live loadings. New Steel Construction (2003) had published a technical article, part of it concerning vertical deflections based on BS 5950-1:2000. For rigid floorings, the total (dead and imposed loads) vertical deflection must be less than L/500. However, both RDA Projects and Ecophon suggest a live load deflection limit of L/500, to avoid bouncy floorings and to give ceilings a smooth appearance, respectively. In a bid for conservative design, the live deflection limit was then user-inputted at L/500.

### 2.8 Construction of Space Trusses

The considerations for the detailed design of space trusses previously discussed are material properties, element structural behaviour and dimensional accuracy (addressed by SAP2000 through their optimisation technique). Another thing to consider is on-site construction.

The construction method chosen depends on the truss system being used, along with overall grid size, site access and component size. Below are common construction methods of space trusses as provided by Tata Steel’s resources (2013):

1. Assembly of the individual space truss modules on a temporary scaffold support – this is expensive and is used when no other method is possible, but it may also be used for large trusses so as to stabilize the area for later connection of pre-assembled modules;
2) Connection in the air, in which space truss elements or modules are lifted individually by crane for connection to areas of the truss that have already been installed – this is for heavy modular systems or when the site cannot be obstructed by assembly of the truss at ground level;

3) Assembly of truss elements or modules into panels before lifting by crane and connecting in the air – this is for when the whole space truss is too heavy to be lifted as one piece or when there is not enough space to assemble the whole truss on the ground;

4) Assembly of the whole grid on the ground before lifting onto the permanent supports by a crane in one lift;

5) Assembly of a part or the whole truss on the ground before jacking or winching into position over temporary or permanent supports – this is for trusses installed at a lower height.

If cladding and installing of services need to be done, an important advantage may be gained from assembling the grid at or slightly above ground level prior to lifting it to its final position, eliminating the need for temporary access scaffolding. In general, methods (3) and (4) are the most cost-effective ones for most situations.
3 METHODOLOGY

Familiarisation with SAP2000, an integrated software for structural analysis and design, is necessary. While awaiting correspondence with both supervisors, four weeks were needed for familiarisation with the software’s user interface via the use of tutorials and online videos.

3.1 Standardisation of Experimental Factors

Section 2.5 reasoned out that circular hollow steel sections that were cold-formed has some slight advantages when used for this experiment. The design standard used is thus EN10219-1. It also showed the jointing advantages of unalloyed steels. As such steel grades S235H, S275H and S355H are being used throughout this project. Being the three lowest steel grades, this conservative view means that if a structure can hold using the lower steel grades, it follows that higher steel grades may allow for a more efficient deck. The use of these three grades thus serves as a gauge on the structure’s minimum capacity.

Section 2.7 stated that structural and buckling checks are based on EC3-2005, but manual checks have to be conducted on deflection due to unfactored live loads, with a limit of span/500. The checks were done using the EC3-based load combination of 1.35 DL + 1.5 LL + 0.5 WL. The DL is the structure’s self-weight, computed by SAP2000 while analyses run. The WL is calculated by SAP2000 based on EC1-1-3 and certain user-inputs. The laterally-applied wind loads were at a speed of 20 m/s, based on Weather Studies (National Environmental Agency, 2013), and on terrain category 0 (sea/exposed to sea), corresponding to a WL of about 0.6 kN/m². This is also the minimum WL for the design of flats up to 36.4m, based on Singapore’s Housing Development Board (2012). 36.4m was set as the minimum due to a requirement in the JTC brief provided by Mr Ng, which suggested that structures built over ports need to exceed 30m in order to clear the gantry cranes used in handling containers.

As for the steel sections, SAP2000 runs an optimisation between analyses and design checks, selecting from a user-defined list the combination that utilises the least steel mass without compromising the structural strength.

Appendix B thus shows the summarised process taken on SAP2000 before subjecting any structure to all analysis and design checks.
3.2 Selection of Topology

An analysis was required to determine one grid topology that will be used for the analyses throughout the rest of the project. The four qualifying topologies are Type 2 from Fig. 2.2.1, types (f), (g) and (h) from Fig. 2.2.2.

These were the factors standardised for all four grid types:

1) Steel grade – S235H;
2) LL – 7.5 kN/m², the minimum value for typical industrial LL as provided by Mr Ng.
3) Grid configuration – 7m by 7m with the depth, module and angle seen in Fig. 3.1.

Fig. 3.1. Grid Configuration (Source: own)

In order to mathematically simplify this first analysis, 1m modules were chosen. Based on Fig. 2.3.2, the average module number is 7, resulting in a 7m span. Based on Eq. 1, a span-depth ratio of 14 is acceptable, resulting in a depth of 0.5m. The angle between chord and brace would then be 45°, acceptable according to Tien (2005).

The analysis was then conducted in the following manner:

1) Construct the structure with the chosen topology on SAP2000, keeping in mind the above standardisations.
2) Run the analyses on SAP2000.
3) If structure passes checks, find its maximum deflection.
4) Repeat steps (1) to (3) for all four grid topologies.
5) If all the structures fail, lower the LL by 1.5 kN/m² and repeat steps (1) to (4).
6) Repeat step (5) until at least one of the grids can hold the given LL.

If only one grid can hold the highest given LL, that grid is the most effective one. If all the grids can hold the same LL, the most effective topology will be determined by the grid with the least deflection.
3.3 Determining the Configuration

The configurations of the space trusses depend on the span, the number of modules and the depth. All three are responsible in determining the trusses’ span-depth (L/d) ratio and chord-module angle, which are correlated (increase in angle results in decrease in ratio). There are many possible configurations, but the aim is to end up with values of span, depth and number of modules that are practical enough for manufacturing, and easy to systematically analyse.

The analysis in Section 3.2 had provided the reasons for starting off with a 7x7 grid. To simplify the options and procedures, the grids will have spans in multiples of 7 (e.g. 7x7, 14x14, 21x21 etc.). These are the steps taken to determine each truss’ configuration:

1) Determine span, starting with 7x7 and increasing in multiples of 7.
2) Based on Eq. 1, calculate range of L/d ratio.
3) Based on Fig. 2.3.2, decide on number of modules that will result in a module length (span divided by number of modules) that is practical to manufacture.
4) Given that the chord-module angle, \( \theta \), must lie between 30˚ to 60˚, find the depth that will result in the range found in (2) and that range of angles using
   \[
   \tan \theta = \frac{\text{depth}}{(\text{module}/2)}
   \]  
   (Eq. 2)
5) Repeat steps (1) to (4) for other truss spans.

3.4 Load-dependent Analyses

For this section, the objective is to find the maximum deflection due to a range of live loadings. Suppose the truss at a certain steel grade cannot handle the live loading (due to failed design, buckling or deflection checks), a higher steel grade will be used. As mentioned in section 3.1, three grades are available for testing – S235H, S275H and S355H. The variable LL – 7.5, 10, 12.5, 15, 20 and 25 kN/m² – are according to industrial standards and were provided by Mr Ng.

The methodology is as follows:

1) Specify structure’s configuration and steel grade. Check the DL and WL settings.
2) For LL = 7.5 kN/m², run the analysis and design check until the optimised steel sections have been selected. Note the maximum live deflection (in the middle of the structure).
3) Run the buckling analysis and note the factor for the first mode of buckling.
4) Repeat steps (2) and (3) for LL values of 10, 12.5, 15, 20 and 25 kN/m².
5) If the deck fails at a particular loading, change to a higher steel grade before carrying out steps (2) and (3).
6) Continue with subsequent LL values using that changed/higher steel grade.

3.5 Steel-dependent Analyses

This section is to determine the robustness of each truss deck of specified configuration, using the steel grades as a variable. Trial-and-error is carried out to determine the maximum live loading the structure can carry before failing due to design, buckling or deflection failures.

The methodology is as follows:
1) Determine the truss deck’s configurations.
2) Starting with steel grade S235H, use trial-and-error to find the maximum live loading that the deck can carry before it fails in any mode.
3) Having found the robustness of the structure, check its maximum deflection value and buckling factor.
4) Repeat steps (2) and (3) for steel grades S275H and S355H.

3.6 Selection of Construction Method

Cost and speed of assembly are the main factors in determining the type of joint connection method to use. In order to understand the technologies available in Singapore’s construction industry, an online search for construction companies was conducted and a few were emailed or called up for quotations and procedures. Based on Section 2.6, jointing of the space truss is best done via the MERO connectors, bolting or welding of conventional steel. Section 2.8 requires sourcing for companies that have cranes capable of erecting the structure. These are the steps taken to decide between the two jointing methods:
1) Conduct online search of company.
2) Explain, via email or telephone, the scope of this FYP and how the approached company can help.
3) Obtain a catalogue or quotation on materials, equipment and/or manpower.
4) Understand the company’s fabrication and/or construction procedures.
5) Estimate the overall cost.
4 RESULTS

4.1 Selection of Topology

Based on the literature review, four grid types were chosen in order to analyse, using SAP2000, their efficiency and cost-effectiveness. Type 2 from Fig. 2.2.1 and types (f), (g) and (h) from Fig. 2.2.2 were respectively relabelled as Types A, B, C and D. Appendix C shows the 3-D and plan views of the four types when constructed on SAP2000. The four grid types were tested as per Section 3.2 and yielded the following results:

Table 4.1.1. Selection I of Grid Type

<table>
<thead>
<tr>
<th>No.</th>
<th>Grid Type</th>
<th>Live Load (kN/m²)</th>
<th>Passed design checks? (Y/N)</th>
<th>Maximum Deflection (mm)</th>
<th>Total Weight of Structure (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>7.5</td>
<td>Y</td>
<td>2.6</td>
<td>315.592</td>
</tr>
<tr>
<td>2</td>
<td>B</td>
<td>7.5</td>
<td>Y</td>
<td>5.0</td>
<td>305.787</td>
</tr>
<tr>
<td>3</td>
<td>C</td>
<td>7.5</td>
<td>Y</td>
<td>4.4</td>
<td>306.484</td>
</tr>
<tr>
<td>4</td>
<td>D</td>
<td>7.5</td>
<td>Y</td>
<td>4.3</td>
<td>307.460</td>
</tr>
</tbody>
</table>

Using a deflection limit of L/500 on a structure with a 7m span, Type B is the closest to the limit of 14mm. Thus, despite being the lightest (and cheapest) structure, it is eliminated. Types A, C and D were not as straightforward to determine. Types C and D have greater deflections than Type A, but Type A is the heaviest (and costliest) of the three. Thus, another analysis was completed using an increased LL of 10 kN/m², another common industrial live loading as suggested by Mr Ng. In Table 4.1.2, we also gave an estimate of the structure’s cost based on MEPS’s Asian Carbon Steel Prices (2012) of USD735/tonne for structural sections in October 2012, with a conversion rate of 1 USD = 1.2303 SGD

Table 4.1.2. Selection II of Grid Type

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<th>No.</th>
<th>Grid Type</th>
<th>Live Load (kN/m²)</th>
<th>Passed design checks? (Y/N)</th>
<th>Maximum Deflection (mm)</th>
<th>Total Weight of Structure (kN)</th>
<th>Total Mass of Structure (tonne)</th>
<th>Estimated Cost of Steel (SGD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>10</td>
<td>Y</td>
<td>3.2</td>
<td>315.781</td>
<td>32.20</td>
<td>29,117.51</td>
</tr>
<tr>
<td>2</td>
<td>C</td>
<td>10</td>
<td>Y</td>
<td>5.5</td>
<td>306.696</td>
<td>31.27</td>
<td>28,276.54</td>
</tr>
<tr>
<td>3</td>
<td>D</td>
<td>10</td>
<td>Y</td>
<td>5.5</td>
<td>307.731</td>
<td>31.38</td>
<td>28,376.01</td>
</tr>
</tbody>
</table>
Stronger sections are needed to support increased loadings, resulting in an increase in the grid’s structural weight. This self-weight and the live loading thus contribute to an increase in vertical deflection. Type A increased in weight by 0.0599% and deflection by 23.07%. Type C increased in weight by 0.0692% and deflection by 25%. Type D increased in weight by 0.0881% and deflection by 27.91%.

From the second analysis, Type A proves to be the most expensive to produce for a 7m by 7m grid, but it has the minimum absolute value for maximum deflection, no matter the loading. Upon increasing the loading, it experienced the least increase in deflection, showing its ability to sustain heavier loadings without compromising its structural integrity. Also, Type A has the least weight increase when loading is increased. Its increase on 0.0599% was less than that of Type C and D. The value might be minimal, but at greater loads, Type A’s minute weight increase can save costs when it comes to purchasing steel sections.

### 4.2 Determining the Configuration

Throughout this section, the grade of circular hollow section used is S235H. The module lengths and depths chosen were kept to 2 decimal places to reduce inaccuracies in manufacturing the steel sections. Each table below shows the configurations of three different angles – 45°, highest and lowest possible angles.

**Table 4.2.1. Configuration of 7x7m deck**

<table>
<thead>
<tr>
<th>No.</th>
<th>Range of L/d Ratio (based on Eq. 1)</th>
<th>Number of Modules</th>
<th>Module Length (m)</th>
<th>Depth, d (m)</th>
<th>Angle (°)</th>
<th>Actual L/d Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.79 – 16.79</td>
<td>7</td>
<td>1</td>
<td>0.50</td>
<td>45</td>
<td>14</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>7</td>
<td>1</td>
<td>0.42</td>
<td>40.03</td>
<td>16.67</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>7</td>
<td>1</td>
<td>0.54</td>
<td>47.20</td>
<td>12.96</td>
</tr>
</tbody>
</table>

For the 7x7m deck, no. 1 in Table 4.2.1 was chosen due to the simplicity of its module length and depth. However, out of experimental curiosity, an analysis was done on three different configurations of the 14x14m deck, showing that the configuration with the greatest depth (and smallest L/d ratio) results in the least deflection-prone structure.
Table 4.2.2. Deflection of Configuration of 14x14m deck

<table>
<thead>
<tr>
<th>No.</th>
<th>Range of L/d Ratio (based on Eq. 1)</th>
<th>No. of Modules</th>
<th>Module Length (m)</th>
<th>Depth, d (m)</th>
<th>Angle (˚)</th>
<th>Actual L/d Ratio</th>
<th>Deflection Max. (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.59 – 16.59</td>
<td>7</td>
<td>2</td>
<td>1</td>
<td>45</td>
<td>14</td>
<td>0.0128</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>2</td>
<td>1.11</td>
<td>47.98</td>
<td>12.61</td>
<td>0.0106</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>2</td>
<td>0.85</td>
<td>40.36</td>
<td>16.47</td>
<td>_</td>
<td></td>
</tr>
</tbody>
</table>

Configuration no. 3 failed as a structure, showing that for structural trusses of this topology, the chord-module angle cannot be less than 45˚. Also, the most effective structure is one with an easy-to-manufacture depth whose corresponding L/d ratio is low and within range. Thus, no. 2 is the selected configuration.

Based on the above observation, there is no need to test out the different configurations. The most efficient deck given a specific number of modules would be that with the largest possible depth (only 2 decimal places). Table 4.2.3 thus shows the configuration needed for the 21x21m deck.

Table 4.2.3. Configuration of 21x21m deck

<table>
<thead>
<tr>
<th>No.</th>
<th>Range of L/d Ratio (based on Eq. 1)</th>
<th>Number of Modules</th>
<th>Module Length (m)</th>
<th>Depth, d (m)</th>
<th>Angle (˚)</th>
<th>Actual L/d Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.38 – 16.38</td>
<td>7</td>
<td>3</td>
<td>1.69</td>
<td>48.41</td>
<td>12.43</td>
</tr>
</tbody>
</table>

4.3 Load-dependent Analyses

Table 4.3 shows the configurations of the analysed truss decks, the structural self-weights, maximum deflections and first-mode buckling factors. The steel sections used and the buckling factors for six modes can be found in Appendix D.

The results show that an increase in span causes increased maximum deflection. For LL = 7.5 kN/m², deflection for 7x7m is only 19.3% of its limit. For the same load at 14x14m, the value increased to 37.9%. At 21x21m, it goes higher to 45.7%. This observation is also shown in the 21x21m grid, where each LL increase corresponds to a higher steel grade. The buckling factor is larger for the shorter-spanned decks, meaning that they are less likely to fail due to buckling. For total structural weight, comparing steel grade S235H, a 21x21m deck uses less steel mass (2744.18 kN) compared to nine 7x7m decks arranged into a square (2840.85 kN).
Table 4.3. Load-dependent Analyses

<table>
<thead>
<tr>
<th>Span (m)</th>
<th>Steel Grade</th>
<th>Module Length (m)</th>
<th>Depth (m)</th>
<th>Loads (kN/m²)</th>
<th>Maximum Deflection (mm)</th>
<th>Buckling Factor</th>
<th>Total Structural Weight (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Dead</td>
<td>Live</td>
<td>Wind</td>
<td>7.5</td>
</tr>
<tr>
<td>7</td>
<td>S235H</td>
<td>1</td>
<td>0.5</td>
<td>Self</td>
<td>6</td>
<td>0.6</td>
<td>2.6 (&lt;14)</td>
</tr>
<tr>
<td>14</td>
<td>S235H</td>
<td>2</td>
<td>1.11</td>
<td>Self</td>
<td>6</td>
<td>0.6</td>
<td>14.1</td>
</tr>
<tr>
<td>21</td>
<td>S235H</td>
<td>3</td>
<td>1.69</td>
<td>Self</td>
<td>6</td>
<td>0.6</td>
<td>25.0</td>
</tr>
</tbody>
</table>
### 4.4 Steel-dependent Analyses

The results can be seen in Table 4.4 and it is obvious that the longer the span, the lower the robustness of the truss deck. A higher steel grade will result in greater robustness. These live loadings are valuable in understanding the limits of the structure. This enables careful planning in terms of specifying a loading that is well within the structure’s limits and safety factor.

**Table 4.4. Steel-dependent Analyses**

<table>
<thead>
<tr>
<th>Span (m)</th>
<th>Steel Grade</th>
<th>Module Length (m)</th>
<th>Depth (m)</th>
<th>Loads (kN/m²)</th>
<th>Maximum Deflection (mm)</th>
<th>Buckling Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Dead</td>
<td>Wind</td>
<td>Live</td>
</tr>
<tr>
<td>7</td>
<td>S235H</td>
<td>1</td>
<td>0.5</td>
<td>Self</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S275H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>S355H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>S235H</td>
<td>2</td>
<td>1.11</td>
<td>Self</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S275H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>S355H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>S235H</td>
<td>3</td>
<td>1.69</td>
<td>Self</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S275H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>S355H</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 4.5 Selection of Construction Method

First, the nodular systems were considered. However, Octatube did not have a Singapore office, making MERO the only viable nodular connection for now. This was the outcome after a few personal communications with their Business Development Manager, Mr Adrian Ang:

1) MERO focuses more on space frames, taking more conservative calculations compared to space trusses as moments in the structure are being accounted for;
2) Maximum LL was 3 kN/m² on a span larger than 21x21m at the height of 30m;
3) LL values as provided by Mr Ng Kian Wee are too large for MERO’s nodular system as the decreased rigidity due to the nodes will cause a greater degree of deflection and the bolts between sections and nodes may not be strong enough.

As such, MERO will not be used for this study. Meanwhile, Mr Ang is contacting the German headquarter to see if MERO nodes can maintain structural integrity under industrial-level LL.
There are several factors to determine if the structure should be bolted or welded. It is even possible to weld the structure and bolt reinforcement plates at the joints, but this will cost more thus it is not being considered. However, for stacked end-flattened bolting, Mr Ang’s advice comes into mind – this bolting technique is the less elegant version of a MERO node, thus structure will not be as rigid and it depend on the bolts’ strength. In order to preserve the deflection values and rigidity of the structures analysed in SAP2000, the joint connection shall be welding.

As seen in Table 4.3, there are three groups of structural steel weight. In order to obtain a range of “per square meter” costs and find the most economical deck, construction costs will be calculated according to Table 4.5, with:

Table 4.5. Estimated Costs

<table>
<thead>
<tr>
<th>Procedures/Details</th>
<th>7x7m</th>
<th>14x14m</th>
<th>21x21m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Raw Materials</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depends on structural steel weight in tonnes.</td>
<td>32 ton</td>
<td>125 ton</td>
<td>275 ton</td>
</tr>
<tr>
<td></td>
<td>=&gt; 28.94</td>
<td>=&gt; 113.03</td>
<td>=&gt; 248.67</td>
</tr>
<tr>
<td>Subsection Fabrication</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Plant Assembly)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6G welders (Assume 5 welders working for $2200/mth).</td>
<td>1 subsection</td>
<td>4 subsections</td>
<td>9 subsections</td>
</tr>
<tr>
<td>Maximum size of subsections is 7x7m, for ease of transport.</td>
<td>=&gt; 2 mths</td>
<td>=&gt; 8 mths</td>
<td>=&gt; 18 mths</td>
</tr>
<tr>
<td></td>
<td>=&gt; 22</td>
<td>=&gt; 88</td>
<td>=&gt; 198</td>
</tr>
<tr>
<td>Transport</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rental cost of 1 detachable gooseneck trailer is $135/hour. Assume a 4-hour rental.</td>
<td>1 trailer</td>
<td>4 trailers</td>
<td>9 trailers</td>
</tr>
<tr>
<td></td>
<td>=&gt; 0.54</td>
<td>=&gt; 2.16</td>
<td>=&gt; 4.86</td>
</tr>
<tr>
<td>On-site Assembly</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6G welders (Assume 5 welders working for $2200/mth)</td>
<td>-</td>
<td>1 mth</td>
<td>3 mths</td>
</tr>
<tr>
<td></td>
<td></td>
<td>=&gt; 11</td>
<td>=&gt; 33</td>
</tr>
<tr>
<td>Erection</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rental cost of mobile crane (45 ton, 40m tall) is $820/day</td>
<td>1 crane</td>
<td>4 cranes</td>
<td>9 cranes</td>
</tr>
<tr>
<td></td>
<td>=&gt; 24.6</td>
<td>=&gt; 98.4</td>
<td>=&gt; 221.4</td>
</tr>
<tr>
<td>Total</td>
<td>76.08</td>
<td>312.59</td>
<td>705.93</td>
</tr>
<tr>
<td>Per-square-meter</td>
<td>1.55</td>
<td>1.59</td>
<td>1.60</td>
</tr>
</tbody>
</table>

A few companies were approached to provide exact quotations. Unfortunately, all were involved in constructing space frames, but not space trusses. They were unable to provide quotations for something they were unfamiliar with, so rough estimations had to be made instead.
Values obtained are from those working in the industry contributing to a few Planning Planet forums and Singapore-based jobsite, Careerjet.sg. The cost of the structural steel is the same as that used in Table 4.1.2. Crane and trailer rates were obtained from a survey of rental rates in Cranes and Access (2012), based on the average rates of a few countries, including Singapore.

Many assumptions were taken in calculating the costs – the duration of the project, the productivity rate of the welders and the rate of fabrication and construction of the structure which is highly subject to a contractor’s available machineries. Each of these require complex calculations, for example, welding costs consists of base metal, preparation, labour, filler metal, gases, equipment, energy, overhead, inspecting and finishing costs, according to Levi (2013). These costs also do not consider unforeseen circumstances that may delay the construction and increase the costs.

In all, the cost of constructing this structural deck with welded joints will cost more than the prices projected in Table 4.5. However, from the per-square-meter calculation, we can see that for a space truss with welded joints, the structure with 7x7m span proves to possibly be the most cost-effective one. Also, if asked for an area of 21x21m, it would be cheaper to use nine 7x7m structures, instead of a 21x21m structure. This is proven by the following calculations:

\[
\text{Total cost} = 76\,080 \times 9 = 684\,720 \\
\text{Cost per-square-meter} = \frac{684\,720}{49m^2} = 1\,552.65/m^2
\]
CONCLUSION AND RECOMMENDATIONS

The completed analyses show that the cost of construction of a structural truss deck is over a very wide range, where the most expensive is ten times that of the cheapest. It is apparently more cost-saving to have nine 7x7m decks arranged in a square compared to one 21x21m deck. If a longer column-free span is needed, then the latter needs to be adopted. But if columns are not an issue, a combination of smaller decks would result in an overall more efficient structure, due to the observation that such decks are less prone to deflection and more buckling-resistant.

It is still difficult to decide on the most economical structural truss deck based on this simplified project. Further studies would be required in order to find the best possible structure. A time-lapse analysis of the structures would be required to see how they perform in, for example, 50 years to come. Using higher steel grades of fine-grained steel would increase the strength of the structures, and a study should be done to understand the extent of that increase.

Grid topology was the first thing determined in the project, showing its significance. Changing it can bring very significant changes in the structure’s behaviour. This double-layered grid is the minimum truss deck possible of holding industrial loadings. Using a triple-layered grid would increase the strength and loading capabilities of the structure, even though it will be costlier.

Another future recommendation is to find out the maximum loading and span that the structure can hold if it were bolted instead of welded. The bolting can be in the form of patented connectors or conventional bolting, or even a combination of welding and bolting. When determining the bolt strength, it is necessary to calculate based on the maximum force on a joint so as to have a conservative value.

Another possible change is that perhaps we could build a reinforced concrete structure. Modelling the structural truss deck after bridges, for example, can allow for longer spans, especially if following the cantilever bridges, which can easily be constructed via form traveller method or balanced cantilever method.

If we still want to stick to a structure much like the space truss, it is more advisable to construct and analysis how space frames would work in this situation. This will account for the moments experienced in the structure and its steel sections. Besides, construction industries are more inclined to provide for space frames as compared to space trusses.
Ultimately, cost-effectiveness of a structure is well-represented in per-square-meter calculations. It is not only important to consider the design of the mega-deck, but it is just as important to understand its construction and financial feasibilities. However, it is important to note that structural integrity should not be compromised just to cut the construction costs. With the right structure, it is not impossible to picture developments being built over existing developments, making better use of Singapore’s land/air space.
REFERENCES


APPENDIX A: DESIGN CHECKS

**Fig. A.1. Member Design**

**Fig. A.2. Design Axial Resistance**
Fig. A.3. Design Axial Buckling Resistance

Fig. A.4. Design Shear Resistance
APPENDIX B: PRE-ANALYSES PROCEDURES

1) Define > Materials > Add new materials. This is how steel grade S235H, S275H and S355H are introduced into the software.
2) Define > Section Properties > Frame Sections > Import new property > Pipe > EURO.PRO > Define > Section Properties > Frame Sections > Add new property. This is how an automatic list (for analysis-design optimisation) is created using the specified steel grades in (1) and the steel sections found in the EuroCode.
3) Define > Section Properties > Area Sections > Add new section > Membrane. This is to ensure that load acts directly onto truss members.
4) Define > Load Patterns (then Load Cases and Load Combinations). This is to ensure that loadings are applied according to the Eurocode.
5) Define > Coordinate Systems/Grids > Modify/Show Systems. This is for keying in the number of gridlines and grid spacing that will help in the next step.
6) Draw the joints and truss members onto the grid.
7) Assign > Joint > Restraints. This is to ensure that all members, including the supports, are pin-connected.
8) Select all > Assign > Frame > Releases/Partial Fixity. This is to ensure that there are no moments in the members – the structure is treated as a space truss.
9) Select area > Assign > Area > Automatic Area Mesh. This is to better portray the deflection experienced by the structure.
10) Select area > Assign > Area Loads > Uniform to Frame (Shell). This is where you key in your DL, LL and WL.
11) Analyze > Set Analysis Options (then Create Analysis Model, Set Load Cases to Run and Run Analysis).
12) Design > Steel Frame Design > View Revise Preferences. This is to make sure analyses and design checks adhere to EuroCode.

After all these are set, the analyses and design checks can be conducted.