Numerical Methods in Civil Engineering

Landmark Projects 2015

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Landmark Projects

This book encloses a selection of numerical analysis projects developed in Strand7 by civil engineering students of the University of Sydney. These students, generally in their 4th and final year of study, worked under supervision and guidance from academic staff of the School.

Strand7 is a general-purpose finite element analysis capable of modelling a broad range of structures that may consist of beams, trusses, frames, cables, as well as continua in 2 and 3 dimensions. The materials composing these structures may be elastic isotropic, anisotropic, and generally non-linear such as metals and soils. These structures may be loaded by their own weight, live loads, wind and earthquake loads. Loadings may be more general, and include thermal actions. The software also has strong dynamic analysis capabilities.

The University is grateful for the access to Strand7 for educational purposes.

The purpose of these student projects outlined below was to complete a full analysis of a landmark structure. The first step was to model the structure using beams, cables, spring-dashpots, plates, and bricks elements. The creation of the models was followed by a finite element analysis of the structure when subjected to live, dead, earthquake, wind or thermal loading.

Contributors

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Acknowledgments:

Ian Bowie and Malcolm Boyd
1 Shelley Street, Sydney,

Static Analysis

Description

1 Shelley Street, Sydney consists of two towers. Ten levels of commercial office space and one level of retail, eleven levels in all, make up Tower 1. Tower Two has six levels of commercial office space. A ten-storey atrium stands between the towers. As Tower 1 is subject to dead, live, and wind loads that, due to its higher stature, are more significant. Therefore, it is the subject of the following analysis. The diagrid is a steel structural system that replaces perimeter columns on the outside the façade. It was designed to provide substantial support to the structure. The omission of perimeter columns was necessary to maximised internal floor space. In addition, the triangular structure of the diagrid itself provides greater structural integrity, thus, higher lateral strength.

Source:

ARUP project

Aim

The aim of the project was to compare the strength and the costs of the conventional column-beam structure to the Macquarie Bank building by using a 3-D model in Strand7. The comparison involved analysing the strength, deflection and stress distribution of the structure. The total material costs of the structural members were also compared to assess which of the two buildings is more economical. The costs were determined using market prices and the weight of the material used.
Comparison of Diagrid and Conventional Structure  
(Ying) David Huang, Winston Phan, Shuo Han & Baoyi Tan  
Supervised by Faham Tahmasebinia, Peter Anssourian, and Fernando Alonso-Marqlqui  
School of Civil Engineering  
FACULTY OF ENGINEERING & INFORMATION TECHNOLOGIES

Aim
The aim of the project is to compare the strength and the costs of the conventional column-beam structure to the Macquarie Bank building by using a 3-D model in Strand7. The comparison involves analysing the strength, deflection and stress distribution of the structure. The costs will be determined using market prices and the weight of the material used.

Methods
- The Online Editor was used to input the node coordinates for the diagrid and other structural members of a typical floor plan.
- A standard diagrid and a standard level (including the beams, columns, slabs, shear core and the facade) were modelled by groups and duplicated to form the remaining levels.
- Wind loads were applied as face load on the facade for each level.
- Linear static analysis was used to generate the results for the following load cases: 1.2G+1.5Q(ultimate) G+0.4Q(serviceability) 0.9G+W (wind).

Applied Loads
- Dead Load 1.5 kPa (superimposed) Live Load 3 kPa (reduction factor 0.7)

<table>
<thead>
<tr>
<th>Level</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind</td>
<td>0.50</td>
<td>0.52</td>
<td>0.57</td>
<td>0.62</td>
<td>0.66</td>
<td>0.69</td>
<td>0.73</td>
<td>75.00</td>
<td>0.77</td>
<td>0.79</td>
<td>0.81</td>
</tr>
<tr>
<td>Load</td>
<td>0.31</td>
<td>0.32</td>
<td>0.36</td>
<td>0.39</td>
<td>0.41</td>
<td>0.43</td>
<td>0.45</td>
<td>0.47</td>
<td>0.48</td>
<td>0.49</td>
<td>0.51</td>
</tr>
</tbody>
</table>

Boundary Conditions
- Nodes at the ground level are all fixed.

Results & Discussion

Lateral Deflection (0.9G+W)

Vertical Deflections (G+0.4Q)

Serviceability Check (Load Case: G+0.4Q)

<table>
<thead>
<tr>
<th>Component</th>
<th>Max deflection in conventional (mm)</th>
<th>Max deflection in diagrid (mm)</th>
<th>Span (mm)</th>
<th>Deflection limit (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor Slab</td>
<td>21.90</td>
<td>20.89</td>
<td>12800</td>
<td>350/42.7</td>
</tr>
<tr>
<td>Beam Slab</td>
<td>21.90</td>
<td>20.89</td>
<td>12200</td>
<td>250/48.8</td>
</tr>
</tbody>
</table>

Stress Distribution (1.2G+1.5Q)

The maximum principal stress at shear core is: 15.2 MPa (1.2G+1.5Q)

Cost Comparison
The cost of external structure of two buildings (diagrid and external columns) are compared by using weight/volume of the materials.

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Total Material Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diagrid</td>
<td>$177,000</td>
</tr>
<tr>
<td>Perimeter columns</td>
<td>$15,000</td>
</tr>
</tbody>
</table>

The material costs of the conventional column to beam structure is only about 67% of the external diagrid structure.

Limitations and Improvements
- The western tower and the atrium were ignored in constructing the model, but they may sustain some loads and provide some lateral restraints to the eastern tower.
- The foundation of the building was originally inclined.
- Standard floor plan was applied to the entire structure.
- The deflections of the slab can be reduced by utilising pre-stressed concrete and adding more internal columns.

Table 2: Comparison of Lateral Deflection Under G+0.4Q+W

Table 3: Comparison of Serviceability Check Under G+0.4Q

Figure 1: Actual Building vs Strand7 Model of One Shelley Street

Figure 2: Lateral Deflection of Diagrid Structure under 0.9G+W

Figure 3: Lateral Deflection of Conventional Structure under 0.9G+W

Figure 4: Vertical Deflections of Diagrid Structure under G+0.4Q

Figure 5: Vertical Deflections of Conventional Structure under G+0.4Q

Figure 6: Relative Floor Deflection of Diagrid Structure

Figure 7: Top Floor Slab Deflection of Diagrid Structure

Figure 8: Stress Distribution of Diagrid Structure under 1.2G+1.5Q

Figure 9: Stress Distribution of Conventional Structure under 1.2G+1.5Q

5
1 Bligh Sydney, Static Analysis

Description

1 Bligh, Sydney, stands at 139 metres high. Ingenhoven architects designed this significant element in Sydney's skyline. Builders completed construction of the office building in 2011. Its unique design, utilizing compact geometry and rotation affords each office area a clear view of the harbour. A public plaza necessitated the elevation of the ground floor area. Efficient use of space and an ecological concept make this “green star” building the only one of its kind in all of Australia.

Source:

Nominated for DETAIL Prize 2012

Photograph: Hans Georg Esch

Aim

The aim of this project was to perform a static analysis of the structure under different combinations of loads. These load combinations included dead load, live load and wind load. The analysis also considered the effect of adding additional structure to the columns capacity and to strengthen these if required.
BACKGROUND

ELLIPICAL STRUCTURE IN 1 BLIGH ST

• 29 Office floors
• Height = 139 m
• Ventilated sky-lit atrium
• 6 Star Green Star Award
• High strength concrete
• Double skin facade
• 12 Columns with diameter of 1.5m
• Two core-walls

AIM

The aim of this project is to analyse the static behaviour of the structure on 1 Bligh St under different load combinations. The analysis also considered the effect of increasing the column strength where required.

METHOD

STRAND7 MODEL

The STRAND7 model was effected by creating the floor slab using plate elements, subdivided manually using Quad8 and Tri6 elements. The columns and beams were created using beam elements, while the core walls were created by extruding beam elements into plate elements. The circular hole on the southern side of the column was a representation of the full height atrium.

The first floor was taken to be 6m high, while the other floors were assumed at an average height of 4m, and were copied by increment. The columns and core-walls were assumed fully fixed at foundation level.

LOAD CONSIDERATION

• Dead Load (G + SDL)
• Live Load (3 kPa)
• Wind Load

The wind pressure varied with height and was applied to the façade. The façade was simplified as 30mm thick glass. Three wind cases were considered as coming from the north, south and west.

LOAD ANALYSIS

• Linear Static

COMPARISON

The construction of additional floors resulted in higher bending moments and axial forces, particularly on the bottom of the structure. Therefore, the columns were required to have higher strength capacity. The steel reinforcement calculation of the columns was done by making use of the dimensionless column chart. The calculation was based on several assumptions, including:

• Concrete cover to columns: 225mm.
• Yield stress of the steel reinforcement: 400 MPa, reduction factor Φ = 0.6
• The initial percentage of steel reinforcement to each column = 2% (Area = 0.035 m²)

The installation of additional floors required 1.5% steel reinforcement in the columns. Therefore the originally designed columns were sufficient to carry the additional loads. In reality however, the final steel reinforcement is likely to be higher than 2% due to safety and other external factors.

CONCLUSION

From this analysis, the displacement, bending moment, axial force and shear force was obtained. The maximum bending moment and axial force were found at the bottom of the structure. The maximum displacement was found at the top of the structure. From these results, it can be concluded that the initial column reinforcement was theoretically sufficient to carry 2 additional floors.

REFERENCES

• Beletich, A.S, Hymas, I.D, Reid, S.G & Uno, P.J., 2013, Reinforced Concrete: The Designers Handbook, Cement and Concrete Services, NSW.
• Standards Australia, 2002, Australian/New Zealand Standard 1170: Structural Design Actions, Sydney, NSW.
Severin Bridge, Rhine River,
Static Analysis

Description

The Severin Bridge, over the Rhine River, spans 691 metres, with the longest unsupported span, measuring 121 metres, from the last cable to the end support. The bridge is 29.5 metres wide with a three metre wide walkway, allowing for pedestrian traffic. Vehicle lanes are 3.2 metres and can support 360kN at 6.25 metre centre axle loads, with wheel groups at 2 metre centres. The limit state load factor for M1600 traffic load is 1.80. The dead load factors are as follows: steel 1.1, concrete 1.2. The bridge features steel towers totalling 77 metres in height, and a box girder depth of 4.6 metres at mid-span. This depth is reduced to 3.2 metres at the ends.

Aim

We aimed to create a realistic model of the bridge. The scope was to conduct both linear and non-linear static analysis to determine the deflection and stress in various members under loading equivalent to AS5100 (Australian bridge code). Using the available element types, the pylon, cables, supporting structure, deck and sub-deck structure were included in the model. These primarily consisted of beam and plate elements.
Numerical Investigation Stress and Deflection in the Severin Bridge - Cologne, Germany.

Jim Chen, Andreas Pelosi, Alexander Smith and Ganan Yin
Supervised by Faham Tahmassebinia, Peter Ansourian, and Fernando Alonso-Marroquin
School of Civil Engineering

Aim:
To create a realistic model of the Severin Bridge including the sub-deck structure, followed by linear and non-linear static analysis, to determine deflections and stresses in various members under loading specified by ASS100 Bridge Design.

Background Information:
The Severin Bridge (Severinsbrücke) was the first of five bridges built over the Rhine River in Cologne, Germany after World War 2. This fan style cable-stayed girder bridge was commissioned in 1956 and finished in 1969 on an overall span of 691m and width 29.5m. The design of the bridge took into account the cityscape including the Cathedral, along with the ports and functionality of the river. Hence an asymmetric span system was adopted, the longest of 302m. The asymmetrically placed A-shaped pylon stands 77.2m above the foundations. At the time the bridge had the longest span of any cable-stayed bridge and the first to contain an 'A-shaped' pylon. The bridge itself holds 4 lanes of traffic along with 2 rail lines and a pedestrian and cycling pathway on each side.

Figure 1 – Severin Bridge

Figure 2 – Strand7 Model

Table 1 – Section Properties

<table>
<thead>
<tr>
<th>Property No.</th>
<th>Description</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam Elements</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Main Girder</td>
<td>2m x 4.5m x 50mm RHS</td>
</tr>
<tr>
<td>2</td>
<td>Longitudinal Deck Stiffeners</td>
<td>310UB32.0</td>
</tr>
<tr>
<td>3</td>
<td>Transverse Deck Stiffeners</td>
<td>410UB43.7</td>
</tr>
<tr>
<td>4</td>
<td>Pedestrian Edge Girder</td>
<td>800mm x 20mm Plate</td>
</tr>
<tr>
<td>5</td>
<td>Transverse Major beam</td>
<td>500mm x 500mm x 50mm SHS</td>
</tr>
<tr>
<td>6</td>
<td>Cables</td>
<td>350mm Diameter Steel Cable</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Plate Elements</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Main Deck</td>
</tr>
<tr>
<td>2</td>
<td>Pedestrian Deck</td>
</tr>
<tr>
<td>3</td>
<td>Pylon</td>
</tr>
</tbody>
</table>

Results:
Stress and deflection results from the Strand7 model are best shown in the contour diagrams of Figures 4 and 5. The maximum values are shown in Table 2.

Table 2 - Results

<table>
<thead>
<tr>
<th></th>
<th>Deflection (DY) (m)</th>
<th>Stress (VM) (MPa)</th>
<th>Stress (Axial) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>0.430</td>
<td>285</td>
<td>150</td>
</tr>
</tbody>
</table>

Conclusion:
The Strand7 model produced stresses in which no members were yielding, including areas of connection which had high stress.

The deflection produced by Strand7 gives a span/deflection value of 700 (302/0.430), higher than that required by ASS100.2 Cl. 8.11 (1/600 of the span).

References:
www.structure-net/structure/s/severin-bridge
Image: Rolf Heinrich, Cologne
(Open Source)

Figure 3 – Axial and Von Mises Stress

Figure 4 – Vertical Deflection
Severin Bridge, Rhine River,
Earthquake Analysis

Description

The Severin Bridge, over the Rhine River, spans 691 metres, with the longest unsupported span, measuring 121 metres, from the last cable to the end support. The bridge is 29.5 metres wide with a three metre wide walkway, allowing for pedestrian traffic. Vehicle lanes are 3.2 metres and can support 360kN at 6.25 metre centre axle loads, with wheel groups at 2 metre centres. The limit state load factor for M1600 traffic load is 1.80. The dead load factors are as follows: steel 1.1, concrete 1.2. The bridge features steel towers totalling 77 metres in height, and a box girder depth of 4.6 metres at mid-span. This depth is reduced to 3.2 metres at the ends.

Aim

The objectives of the project were to build a 3D model of the bridge and to analyse it under earthquake loading; then to identify the deflections and stresses under different frequencies by using the natural frequency solver, followed by spectral response solver. Finally, an analysis of different conditions was used to determine the maximum deflection and largest stresses in the plate elements.
Severin Bridge
Chaozheng ZHANG, Qin LI, Pu WANG, Mengxi CHEN
Supervised by Faham Tahmasebinia, Peter Ansourian, and Fernando Alonso-Marroquin
School of Civil Engineering
FACULTY OF ENGINEERING & INFORMATION TECHNOLOGIES

Aim
- Using Strand7 to build the 3D model of Severin Bridge
- Earthquake analysis of the Strand 7 3D model
- Identify the stability of the bridge under local worst earthquake condition

Background Information
Severin Bridge, in Cologne, Germany, built in 1960
- Bridge dimensions:
  - Total span 691m; Width 28.98m
  - Steel towers total height 77m
  - Box girder depth 4.5m
- Dead Load factor (AS1170): Steel 1.1, concrete 1.2
- Live load factor (AS5100.2): Standard design lane 3.2m
  - Axle loads, 360kN at 6.25m centers, wheel groups at 2m centers

Earthquake Analysis
Natural Frequency
45 seismic modes were selected to analyze the natural frequency of the bridge model, resulting in a mass participation factor is 73%. This step determined the range of frequencies that the system would use to simulate the earthquake. It was the fundamental data used for spectral response analysis.

Sand soil Be Response Spectrum
From Period 0 to 5.0.
The equation used for the spectrum was 1.25×(f/23). The period range was 0.35356-3. Sampling range was 2-20.

Spectral Response
Spectral response analysis generated the final results for seismic effects. It was based on the frequencies obtained in the natural frequency analysis. Base acceleration was set at 2.2 m/s² following a paper on seismic studies of the city of Cologne.

Results
Figure 5 summarises the response under seismic actions. The maximum displacement of 1.18m occurred at the bridge deck (purple contour).

The maximum nodal displacement was 1.18 m (figure 6). This is less than the standard design value, and therefore the bridge design met the stability requirement under earthquake loading.

Figure 7. Plate stress along the bridge

Reference
Bridge - initial design. (2015, 8 3). Retrieved from SteelConstruction.info:
http://www.steelconstruction.info/Bridges_initial_design
Verrazano Narrow Bridge, New York, Static Analysis

Description

Three million rivets and one million bolts hold together the bridge’s towers, which stand at an impressive 221 metres high. The 1300 metre span of the bridge is complete with an 8m deep stiffening truss. Once the longest suspension span in the world, the length necessitated compensation for the curvature of the Earth. This means the towers are 4cm further apart at their apexes than at their bases. Each of these monolithic structures weighs more than 24,000 tonnes. The two-deck, 29.5m-wide roadway is 69.5 metres off the water in the winter and 65.9 metres off the water in the summer. The variance in clearance is due to seasonal expansion and contraction of the cables. As with the Severin Bridge, vehicle lanes are 3.2 metres and can support 360kN at 6.25 metre centre axle loads, with wheel groups at 2 metre centres. The limit state load factor for M1600 traffic load is 1.80. The dead load factors are as follows: steel 1.1, concrete 1.2.

Aim

We aimed to analyse a simplified model of the Verrazano Narrows Bridge using both static and dynamic analysis. Limitations and assumptions were imposed to simplify the problem of modelling the full structure in Strand7. Static loading consisted of the dead load of the self-weight of the structure and live loads applied uniformly across the bridge. A Non-linear Static solver was used to obtain a static analysis of the bridge. Dynamic loading included load paths to simulate the flow of two-way traffic. A Quasi Static solver was used to assess the bridge behaviour over time with the changing load path. Deflections and maximum stresses were reviewed in both Non-linear static and Quasi Static analyses.
INTRODUCTION

Aim
Undertake a simplified static and dynamic analysis of the Verrazano Narrows Bridge using Strand7 software. Both dead and live loads were considered. A linear solver was used in conjunction with influence lines to model the flow of traffic. Deflections and maximum stresses were reviewed.

Background
The construction of the bridge between Brooklyn and Staten Island began in 1959. The project took 5 years to complete and cost $320m. The bridge is the 11th longest single span bridge in the world. Used by traffic, pedestrian and cyclists with an average of 170,000 people using the bridge daily.

MODELLING

- Bridge element sizes were obtained from a paper by W.T. Arnold. Elements not mentioned in this paper were scaled from Google Maps.
- Cut off bars were used to model the hanger cables under pre-tensioning stresses.
- Main cables assumed parabolic.
- Dynamic traffic load on the bridge was applied according to AS5100 Bridge Design.
- Additional restraints were applied to the model to aid Strand7 in a functioning and converging analysis.

RESULTS

Static analysis
- A maximum deflection of 10.5m was found in the linear analysis, occurring at midspan of the bridge under ultimate limit state conditions of 1.2G+1.5Q. These deflections should be ‘linearly’ reduced to serviceability loading, as ultimate limit state is unrealistic.
- A maximum deflection of 8.9m was found in the non-linear analysis, occurring at the midspan of the bridge under ultimate limit state of 1.2G+1.5Q.
- Maximum bending moment of 0.13MN was found in the bridge deck. Maximum hogging moments of 0.53MN were conversely found in the same region of the bridge.
  - Maximum shear forces were found to be 13.9MN in the bridge deck concentrated around the near the bridge towers.
  - Maximum compression and tension stresses found in the truss elements on the deck of the bridge were 446MPa and 462MPa respectively.
  - Bridge hanger cables and main cables were found to have a maximum tensile stress of 95.8MPa and 394MPa respectively.

Dynamic Analysis
- Quasi static analysis found a peak deflection at the 35.0s time step reaching a maximum deflection of 14.9m.

DISCUSSION

Static analysis
- Displacements in the model exceed allowable limits outlined in Australian standards. This can be due to the simplifications made in the model, such as the exclusion of the physical deck curvature.
- The deflected shape of the bridge resembles that of a uniformly distributed load on a beam with roller supports at each and fixed support conditions at the towers.

Dynamic
- Quasi static solver was used which applies both non-linear transient dynamic solver and nonlinear static solver to produce results.
- Load paths were positioned in opposite directions in order to simulate two lanes of traffic.
- Differences in displacement with additional dynamic load would lead to increased passenger discomfort of bridge users.

CONCLUSION

This model has been simplified to represent the basic function of the Verrazano Narrows Bridge. Assumptions made hinder the accuracy and reliability of the results obtained from this numerical analysis. Further investigations into the effect of the curvature of the bridge could also made.

REFERENCES

- AS5100. 2007. Australian Standards for bridge design
Complete Facade Greening: High-Rise Apartment Building in Milan, Static Analysis

Description

Forty different types of trees were carefully chosen and integrated into the construction of the high-rise apartment buildings at Porta Nuova. Each of the 11,000 ground covers, 5000 bushes, and 730 trees incorporated into the construction was carefully selected for each storey, with the fruit trees anchored with stable rigging belts into the substrate layer of the balconies. The green façade provides a number of features, including a better atmospheric microclimate and protection from UV rays in the homes, as well as a solution to urban sprawl that can be applied in the future. It also results in what is, at least for now, an innovative and unique appearance.

Source:
Detail Online Magazine

Aim

With the gaining popularity of façade greening in buildings, static analysis must be conducted to ensure the feasibility of the building design to support the façade. A major concern with the large balcony designs supporting a façade is serviceability. Static finite element analysis using Strand7 was conducted to cover the main focus of the project:

I. Realistically model one level of the Bosco vertical

II. Study the short term serviceability of the cantilevered balconies

III. Investigate alternative designs for the balconies

IV. To replicate the single level model over 26 storeys to study the serviceability lateral deflection of the entire structure
INTRODUCTION

Background
Bosco Verticale, also known as the "Vertical Forest" is a pair of residential towers in Milan, Italy. It features more than 500 trees on the large balconies creating a green, environment friendly building. The green façade has a combination of its sophisticated plant selection and the structural design to represent the uniqueness of the building in recent memory of all existing infrastructures.

Project Focus
With the gaining popularity of façade greening in buildings, static analysis must be conducted to ensure the feasibility of the building design. A major concern with the large balconies design supporting a façade greening is associated with its serviceability. Static finite element analysis using Strand7 is conducted to cover the main foci of the project:

1. Realistically model one level of the Bosco vertical
2. Study the serviceability of the cantilevered balconies
3. Investigate alternative designs for the cantilevered balconies
4. To replicate the single level model over 26 storeys to study the sideways deflection of the entire structure.

METHODOLOGY & RESULTS

Data input
Dimensions are estimated from schematic drawings:
- 204x24m core structures
- 3.25m cantilevered balcony

Summary of material properties and loads:

<table>
<thead>
<tr>
<th>Member</th>
<th>Material Type</th>
<th>Material Property</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>Reinforced Concrete</td>
<td>50,000 psi</td>
<td>300</td>
</tr>
<tr>
<td>Internal beam</td>
<td>Reinforced Concrete</td>
<td>50,000 psi</td>
<td>300</td>
</tr>
<tr>
<td>Internal wall</td>
<td>Reinforced Concrete</td>
<td>50,000 psi</td>
<td>300</td>
</tr>
<tr>
<td>Cantilever</td>
<td>Reinforced Concrete</td>
<td>50,000 psi</td>
<td>300</td>
</tr>
<tr>
<td>Balcony slab</td>
<td>Reinforced Concrete</td>
<td>50,000 psi</td>
<td>300</td>
</tr>
<tr>
<td>Balcony slab perimeter</td>
<td>Reinforced Concrete</td>
<td>50,000 psi</td>
<td>300</td>
</tr>
<tr>
<td>Cantilevered beam</td>
<td>Reinforced Concrete</td>
<td>50,000 psi</td>
<td>300</td>
</tr>
</tbody>
</table>

Wind load action with velocities of 30ms^-1 - 43ms^-1
Lateral deflection, winds loads are summarized as follow;

<table>
<thead>
<tr>
<th>Member</th>
<th>Dead Load (kN/m)</th>
<th>Live Load (kN/m)</th>
<th>Wind Load (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Internal beam</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Internal wall</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Cantilever</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Balcony slab</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Balcony slab perimeter</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Cantilevered beam</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

Limit & Strand7 output
Serviceability deflection limit, \( \Delta_{\text{limit}} = 1/100 \) for the cantilevered balcony. As stated in Eurocode 8 EN 1990.

Maximum deflection is from load case DL+LL-WL, although the LL had minimal influence on deflection.

METHOD & RESULTS

Strand7 Model
The structure was modelled using Quad 4 and Tri 3 elements as it provides sufficient accuracy as well as minimizing the execution time.

Design methods adopted:
1. Entire structure concrete
2. Method 1: 5% steel reinforcement
3. Method 2 + continuous cantilever beams
4. Method 3 + composite balcony slab (steel concentrated in tension zone)

In order to simulate the designs, material properties were adjusted accordingly, which was done by estimating the appropriate Young’s Modulus (E):
- Reinforced concrete: E values adjusted using steel to concrete ratio.
- Composite slab: Separate plane strain cantilever was modelled for a reinforced concrete member and a composite member, and \( E_{\text{composite}} \) estimated using formula \( E_{\text{composite}} = E_{\text{concrete}} + E_{\text{steel}} \), assuming the former \( E_{\text{concrete}} \) is Composite.

DISCUSSION & CONCLUSION

The balcony serviceability and the overall lateral deflection of the entire structure was successfully investigated and compared with limits stated in the standards. It was established that design method 4 is the optimum design as it was the only design to have satisfied the serviceability limit. In addition, the lateral deflection was found to be well within the limit, which is likely attributable to the large shear core wall structure. It should be noted that the reinforced concrete properties were modified by only adjusting the E value, and negating the apparent increase in \( l \), which leads to a conservative design analysis.

In conclusion, it was found that an ordinary reinforced concrete balcony slab is inadequate and alternative balcony design methods must be explored. It is recommended and a composite balcony slab design be further investigated to validate its adequacy in the ultimate design state.

REFERENCES
Complete Facade Greening:
High-Rise Apartment
Building in Milan,
Earthquake Analysis

Description

Boeri Studio and Arup developed a pair of high-rise apartment buildings known as the Bosco Verticale (Vertical Forest) in Milan, Italy. The taller tower stands at 112m high, while the shorter tower stands at 76m. The cantilevering balconies surrounding the structures contain 730 trees, 5000 bushes and 11,000 ground covering plants. If the area were a forest on flat land it would be equivalent to 7000m$^2$. It has created a microclimate, which absorbs CO$_2$ and produces oxygen (Boeri Studio, 2009). The internal temperature is naturally moderated by the deciduous trees that provide shade to apartments housed in the building and allow sunlight to enter in winter. The trees also provide protection from radiation, mitigate smog, and reduce noise pollution. Additionally, the buildings incorporate a photovoltaic energy system to increase their self-sufficiency. The balconies extend outward with 280 mm thick reinforced concrete balconies supporting the vegetation.

Source:
Detail Online Magazine

Aim

This project aimed to perform an earthquake analysis of both towers. This analysis included two parts: modelling the towers in strand7, and investigating the buildings’ response under dead, live, and earthquake loads.

The impact of the building height on earthquake response was examined by comparing the results between the tall and short towers. The effect of the cantilever balconies was also examined by comparing Strand7 models with and without balconies for both the 110 m and 76 m high cases.
AIMS
Four models were developed so to fulfil the following aims:
1. To simulate Models A & B under dead, live and earthquake loading.
2. To investigate the effect of the balconies and building heights by comparing Models A & B with (A.2 & B.2) and without (A.1 & B.1) cantilever balconies.
3. To compare STRAND7 results with theoretical results to ensure the accuracy of the models.

BACKGROUND
- "Bosco Verticale", or vertical forest, are two high-rise residential towers in Milan, Italy.
- Structure A and Structure B are 112 m and 76 m tall respectively.
- Building is formed by a simplified core, columns, beams and slabs with 6 unique cantilevered balcony arrangements repeated up the building.

LOADING
- Dead load: Gravity load due to the reinforced concrete structure.
- Live load: Live load was based on normal residential loading 4 kPa and an additional live load representing the green façade of 16 kPa, applied on the balconies only.
- Earthquake Load: Loads were applied uniformly, according to AS1170.4 for earthquake load cases.

MAIN RESULTS
The four models generated the following results:

<table>
<thead>
<tr>
<th>STRAND7 Results</th>
<th>Model A.1</th>
<th>Model A.2</th>
<th>Model B.1</th>
<th>Model B.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fundamental Frequency (Hz)</td>
<td>0.884</td>
<td>0.885</td>
<td>1.38</td>
<td>1.368</td>
</tr>
<tr>
<td>Mass Participation (%)</td>
<td>75%</td>
<td>75%</td>
<td>60%</td>
<td>72%</td>
</tr>
<tr>
<td>Column x Swy (mm)</td>
<td>67</td>
<td>77</td>
<td>30</td>
<td>31</td>
</tr>
<tr>
<td>Column y Swy (mm)</td>
<td>79</td>
<td>66</td>
<td>29</td>
<td>30</td>
</tr>
<tr>
<td>Base Horizontal Force (kN)</td>
<td>Fx = 414.400, Fy = 403.200, Fz = 387.600, F = 495.500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Moment (kN.m)</td>
<td>Mx = 31.41x10^6, My = 34.95x10^6, Mz = 21.14x10^6, M = 23.11x10^6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E2 Balcony Moment (kN.m)</td>
<td>N/A</td>
<td>372</td>
<td>N/A</td>
<td>157</td>
</tr>
</tbody>
</table>

METHOD
- Natural Frequency Analysis: We considered up to 18 modes to ensure our mass participation factors were close to 90%.
- Spectral Response Solver: Used the spectral curve from AS 1170.4 for a building founded on hard rock.
- Earthquake Cases: Earthquake combinations in both 0.3X + Y + 0.5Z and 0.3Y + X + 0.5Z directions were considered, to find the worst case.

CONCLUSIONS
- The fundamental frequency of the theoretical analysis is much lower than the results given by STRAND7. This might be related to the theoretical analysis not incorporating the shear core and its stiffness contribution to the building’s frame.
- A lack of modes can lead to low participation factors and could explain why the mass participation factor is below 90% for all building models. It should be noted that model B.1 had a mass participation factor of 60%, much lower than the others.
- The sway values are well below the acceptable limit. The low values are most likely a result of the large core in the building, as the shear walls are 1m thick which would make the building very rigid.
- The balcony deflection results were also safely beneath the acceptable deflection thresholds set out by AS3600 – Concrete Structures.
- Base moments and horizontal forces were not consistent between the STRAND 7 and Theoretical results. The vast differences in these calculations could be put down to the simplified theoretical models, low Mass Participation Factors in Strand 7 models, and differing Spectral Shape Factors.

REFERENCES
The Passive House, Thermal Analysis

Description

Passive House (or Passivhaus) is a concept that creates a comfortable, energy efficient environment. The standard is applicable by anyone, and has been tried and tested. Only windows manufactured with exceptionally high R-values (low U-values, typically 0.85 to 0.70 W/(m².K) for the entire window, including the frame) meet the requirements of the Passivhaus standard. These typically employ air seals, thermally broken frames, and triple-pane insulated glazing with: a high solar heat-gain coefficient; sealed, gas-filled inter-pane voids; insulating glass spacers; and low-emission coatings. Heat gains from the sun average higher than heat losses in parts of Europe and the United States for southward Passivhaus windows, provided the windows are unobstructed, even in the dead of winter.

Source:
Detail Online Magazine

Aim

This study aimed to model the two Passivhaus window designs, triple glazed and double glazed, using Strand7, and to investigate the thermal efficiency of the designs. These two models were then compared to a standard window design of single glazing. The comparison of each model was based on the heat loss coefficient $U$, found for each window design. This value was then cross-referenced to the values advertised by the building standard, and used to do a costing analysis to determine the cost savings of the Passivhaus designs compared to a standard window.
The Passive House
Sophie Karayannis, Danny Moses, Hoda Alameddine, Aman Shamsin
Supervised by Faham Tahmasebinia, Peter Ansourian, and Fernando Alonso-Marroquin
School of Civil Engineering
FACULTY OF ENGINEERING & INFORMATION TECHNOLOGIES

Introduction
Passive House
- Building standard used to increase thermal efficiency.
- High insulation used to minimise summer heat gain, and winter heat loss.
- Aims to maintain a constant temperature inside a house subject to a range of temperatures.

Passive Windows
- Most energy losses occur through windows.
- Designed using materials that minimise energy losses through the window and frame.
- Eliminate thermal bridging.
- Materials exhibit low heat loss coefficients U and thermal conductivities K.

Aim
To analyse the thermal response and compare the cost savings of a triple glazed, double glazed & single glazed windows in Strand7.

Model
- Quad8 Plate Elements
- 2D Plane Strain
- Air modeled using plate elements
- Symmetrical about the top
- 2mm x 2mm plates

Material Properties
Thermal conductivity values:
- Glass (Yellow) = 0.95 W/mK
- Argon (Red) = 0.016 W/mK
- Atmospheric Air (Pink) = 0.024 W/mK
- Aluminium (Green) = 205 W/mK
- Hardwood (Dark Blue) = 0.16 W/mK
- Warm Edge Spacer (Light Blue) = 0.05 W/mK

Boundary Conditions
- Inside temperature = 20°C
- Outside temperature = 8°C
- Forced Convection (outside edge) = 170 W/m²K
- Natural Convection (inside edge) = 11 W/m²K
- No radiation

Solver
- Steady State Heat formula
  \[ \frac{\partial}{\partial x} \left( k \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left( k \frac{\partial T}{\partial y} \right) + \frac{\partial}{\partial z} \left( k \frac{\partial T}{\partial z} \right) = 0 \]
  \[ q = -U_p \Delta T \]

Results - Thermal Response
Triple Glazed Window
- 2980 Quad8 elements
- Average inside temp = 19.2°C
- U value = 0.734 W/m²K

Double Glazed Window
- 2485 Quad8 elements
- Average inside temp = 18.9°C
- U value = 1.1233 W/m²K

Single Glazed Window
- 1990 Quad8 elements
- Average inside temp = 13.3°C
- U value = 14.937 W/m²K

Costing Analysis
The table below indicates the quarterly electricity costs required to keep the house at a constant temperature all day.
- 100m² heating/cooling area was assumed.
- $0.2609/kWh was assumed using the average of peak and off-peak rates

<table>
<thead>
<tr>
<th></th>
<th>Triple Glazed Window</th>
<th>Double Glazed Window</th>
<th>Single Glazed Window</th>
<th>Researched Single Glazed Window</th>
</tr>
</thead>
<tbody>
<tr>
<td>U - Value (W/m²K)</td>
<td>0.734</td>
<td>1.1233</td>
<td>14.937</td>
<td>5.6</td>
</tr>
<tr>
<td>Cost Per Quarter ($)</td>
<td>55.24</td>
<td>84.42</td>
<td>1119.77</td>
<td>420.78</td>
</tr>
<tr>
<td>Percentage Increase in cost above the triple glazed window (%)</td>
<td>-</td>
<td>+52.82% (0.53 times greater)</td>
<td>+1927% (19.27 times greater)</td>
<td>+763% (76.3 times greater)</td>
</tr>
</tbody>
</table>

Costing Analysis Summary Table

Conclusion
- Successful Strand7 models of each window were generated.
- Realistic U values were achieved.
- Triple glazed and double glazed experienced low thermal exchanges.
- Single glazed window demonstrated high thermal bridging.
- Triple glazed window offered half the heat transfer of the double glazed solution.
- Triple glazed window offered 19 times greater thermal efficiency than the single glazed alternative.
- Cost savings were directly proportional to the U-value.
Long-span Pedestrian Bridge in Tirschenreuth, Germany, Static Analysis

Description

The construction of this bridge in Tirschenreuth, Germany, is barely visible. The pedestrian bridge is 3.9m wide and spans 85 metres. It is primarily constructed of wood, with two 25mm thick steel straps running along the length of the bridge that allow for great deflection under self-weight. A pier provides support for the middle of the bridge, while steel plates fasten each end at the abutments. These two supports are constructed first, then the aforementioned straps are welded on. After the frame is complete, prefabricated railings and wooden planks are fit together over the structure.

Source:
Detail Online Magazine

Photograph:
Hanns Joosten, ANNABAU

Aim

In this analysis, three load cases were considered: dead load due to self-weight of structural members, live load due to pedestrian use, and wind load specific to the geographic region. The determination of these loads is outlined in the technical report.

Under these loads, and combination loads, deflections in the lateral and vertical direction was compared to maximum allowable deflections defined by AS 5100 – Bridge Design. Due to the length of the spans, oscillation of the bridge under loading was also considered. This was done by comparing the natural frequencies of the bridge with the oscillation frequency of wind and the oscillation frequency of pedestrian movement.
LONG-SPAN PEDESTRIAN BRIDGE

Henry Lam, Victor Ngo, Mufrat Noor, and John Tu

Supervised by Faham Tahmassebi, Peter Ansourian, and Fernando Alonso-Marroquin

School of Civil Engineering

FACULTY OF ENGINEERING & INFORMATION TECHNOLOGIES

AIMS OF THE PROJECT
- To compare linear and non-linear methods of finite element analysis
- To check the serviceability of the bridge
- To investigate the resonance frequency of the bridge under loading

BACKGROUND INFORMATION
- Location: Tischensucht, Germany
- Total length of the bridge is 85m
- 2 spans – each span is 42.5m
- V-shaped support in the middle
- 2 end abutments
- 2 steel straps tensioned and attached to plates on the end abutments
- Timber planks and pre-fabricated railing attached to the steel frame

FINITE ELEMENT MODEL OF THE BRIDGE

MODEL SPECIFICATIONS
- Steel straps
  - Element: 8-node brick
  - Material: Steel, AISI 4130
  - Elastic modulus = 205 GPa
  - Ultimate strength = 1145 MPa
  - Yield strength = 1110 MPa
  - Pre-stress = 800 MPa

Bridge deck
- Element: Q4 plate
- Material: 12% m c pine, maritime
- Elastic modulus = 10.5 GPa
- Ultimate strength = 4.1 MPa

Bridge parapets
- Element: 8-node brick
- Material: Steel, AISI 4130
- Concrete foundation
  - Element: 8-node brick
  - Material: AS 3600 (1994) Concrete
  - Elastic modulus = 31 GPa
  - Compressive strength = 32 MPa

LOADS CONSIDERED
- Dead load: self-weight (G)
- Live load: pedestrians (Q)
- Wind load: specific to the region (W)
- Combination loads (G+Q+W)

SERVICEABILITY LIMITS
Vertical deflection
- AS 5100.2 (2004) Clause 8.11:
  - Sagging: 6 < 42.5 m / 600 = 0.07 m = 70.83mm
  - Hoggling: 5 < 42.5 m / 300 = 0.1417 m = 141.7mm

Horizontal deflection:
- AS 5100.2 (2004) Clause 11.5:
  - δ < 3.4625 m / 300 = 0.01142 m = 11.42mm

LOAD FREQUENCIES
- Pedestrian loading
  - Walking: 2.0 Hz
  - Jogging: 2.5 Hz
  - Sprinting: 3.2 Hz
- Maximum wind loading
  - Re = κ/ν/ν = 1863565.434
  - St = 0.195 / (1 - 19.7% Re) = 0.195
  - Ω = St/ν = 4.611 Hz

NATURAL FREQUENCY RESULTS
- LEFT: Natural frequencies of structure and mass participation factors for selected modes
- RIGHT: Nodal displacements versus frequencies for increasing damping ratios

CONCLUSION
- Non-linear analysis allows for consideration of geometric and material non-linearity
- All serviceability requirements are satisfied
- Structural components do not fail under loading
- Load frequencies do not correspond with dominant frequencies, some matching with minor frequencies
- Resonance is negligible in the structure
- Increasing damping can be used to reduce the magnitude of nodal displacements due to excitation

REFERENCES
- "Advances in Civil Engineering, 2021, Article ID 4567891.
Chernobyl Dome
Construction, Thermo-nuclear
Analysis

Description

On the site of the Chernobyl power plant, a new confinement structure is being built atop the concrete poured in the aftermath of the 1986 explosion. The futuristic dome features stainless steel spread over a massive area. The goal is to reduce the risk of further atmospheric contamination, and guarantee the safety of all who live near the affected areas.

Source:

Aim

This project considered the situation of a simulated nuclear incident occurring inside the structure. Under such conditions a complete thermal analysis was performed to determine the mechanical response of the structure. Such an analysis on a model would provide valuable information on the performance of not only the building materials but also the structure as a whole when subject to high temperatures over a certain period of time. The possibility of the structure’s failure could also yield valuable information on the time available for evacuation purposes in the face of a nuclear mishap.
AIMS OF THE PROJECT
To analyze the structure under a nuclear explosion, specifically the structural response induced by a fire. This analysis will be completed in two stages:
Stage 1) Thermal analysis using a transient heat solver
Stage 2) Mechanical analyses of displacement using a Quasi Static solver

BACKGROUND
The Chernobyl New Safe Confinement is a permanent solution to the spread of radiation after the nuclear explosion, it is currently under construction.

ASSUMPTIONS

<table>
<thead>
<tr>
<th>Assumption 1: Air conductivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input Values</td>
</tr>
<tr>
<td>$T_2$ (K)</td>
</tr>
<tr>
<td>458</td>
</tr>
<tr>
<td>534</td>
</tr>
<tr>
<td>534</td>
</tr>
</tbody>
</table>

Table 1: Thermal conductivity iteration
$k_{eff} = L(h + 4AT_{fluid})$

(STRAND7, ST7-1.30.10.4 Approximating an AirGap in Thermal Analysis)
An iterative steady state heat analysis was used to find the thermal conductivity of the air at 623 K.

$k_{eff}=517.4$ W/mK at 623 K

Assumption 2: Temperature due to fire

The temperature of the fire applied to the inner layer was assumed to vary according to the Cellulosic building fire curve defined in various national standards. (Promat, 2002).

Assumption 3: Steel material properties
The following steel properties were assumed to vary with temperature:
(STRAND7, ST7-1.30.20.1 Nonlinear Weld Mechanical Analysis)
- Elastic Modulus
- Yield Stress
- Coefficient of thermal expansion
- Thermal conductivity
- Specific heat of steel

The variations where included using Factor vs Temperature tables

REFERENCES

CONCLUSIONS
- The internal temperature changes from approximately 300K to 580K, while the external temperature changes from 279K to 290K.
- The largest deformation in the structure is .58m at 72 hours
- While the majority of the structure did not yield, there was localized yielding at the base of the structure.
- Due to the comparatively low temperature and deformation of the structure under the thermal load of a fire over 72 hours we can assume the structure will remain intact.
Bamboo Forest: Bao‘An Stadion in Shenzhen, Static Analysis

Description

In an effort to capture the majesty of southern China’s bamboo forests, the designers of the stadium employed the use of steel supports, fashioned to resemble the recognizable reed, to create the façade. The interplay of light and shadow and the impression of natural bamboo is a consistent architectural theme throughout the structure. The roof and the frame are supported by many slender supports in keeping with the overall concept.

Source:
Detail Online Magazine

Aim

We aimed to provide a complete static analysis of the structure. This analysis included a full model of the structural components, as well as application of dead loads, live loads, and wind loads, in accordance with AS1170. Both linear static and linear buckling solvers were used to gain a detailed understanding of the static loading of the structure. In the report we provide a concise description on the theory behind the finite element analysis, the use of an axisymmetric coordinate system, and the use of online editing tool in Strand7. We also outlined the assumptions that are needed to produce a detailed and accurate analysis.
Bamboo Forest: Bao‘An Stadium (Static)

Samuel Lane, Hashan Subasinghe, Dennis Chan, Gabriel Garayalde
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Aims
- This project plans to provide a complete static analysis of the structure using Strand7.
- This analysis will model the structural components, dead loads, live loads, and wind loads, in accordance with AS1170.
- Linear static and linear buckling solver modules will be run using software STRAND7.
- An axisymmetric coordinate system and online editing tool have been utilized to deliver a detailed and accurate analysis.

Background
Inspired by the bamboo forests of southern China, the Bao'An stadium is known for its creative 'bamboo' design of the structural members surrounding the outside of the building. The stadium complements both the intrinsic Chinese bamboo forest design, as well holding an iconic place in the cityscape of southern China. A photograph of the real Bao'An structure is shown below in Figure 1.

Modelling
Columns
- CHS - 800mm diam. tall columns and 550mm diam. short columns around perimeter.
- CHS - 150mm diam. skeleton truss for concentric circles around roof.
- Perimeter columns come out at 90 degree angle to the horizontal as per architectural drawings and fixed for rotation and translational at the ground.

Cables Stayed Roof Truss and Plate
- Cables and roof truss are tensioned with 2000kN and 4000kN for upper and lower truss cables respectively.
- The pre-tensioning is achieved selecting beam properties > point contact > Tension contact and inputting 515000kN for maximum tension (yield stress).
- Approximate square plate elements sufficiently accurate as Quad4 elements in analysis.

AS1170 Wind Load and Live Load
- 1000N force applied at top nodes in structures for worst case wind scenario. Uplift was not considered because the structure is fully open.
- 0.24kPa live load applied for non-traficable roof.

Results
The Australian Standards design capacities for the members subject to flexure, axial compression/tension and buckling as well as the corresponding Strand7 results are summarised below in Table 1.

<table>
<thead>
<tr>
<th>Loading Type</th>
<th>Theoretical</th>
<th>Strand7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure</td>
<td>$M = 4710 \text{ kN.m}$</td>
<td>$M^* = 223 \text{ kN.m}$</td>
</tr>
<tr>
<td>Axial Compression</td>
<td>$N = 2510 \text{ kN}$</td>
<td>$N^* = 753 \text{ kN}$</td>
</tr>
<tr>
<td>Axial Tension (Cables)</td>
<td>$V = 131 \text{ kN}$</td>
<td>$V^* = 71 \text{ kN}$</td>
</tr>
<tr>
<td>Compressive Stress</td>
<td>$N = 15438 \text{ kPa}$</td>
<td>$N^* = 365 \text{ kPa}$</td>
</tr>
<tr>
<td>Tensile Stress (Cables)</td>
<td>$S = 184.5 \text{ kPa}$</td>
<td>$S^* = 985.9 \text{ kPa}$</td>
</tr>
<tr>
<td>Tensile Stress (Beam)</td>
<td>$S = 315.06 \text{ kPa}$</td>
<td>$S^* = 744.9 \text{ kPa}$</td>
</tr>
<tr>
<td>Compressive Stress</td>
<td>$P = 124.72 \text{ kPa}$</td>
<td>$P^* = 25.10 \text{ kPa}$</td>
</tr>
<tr>
<td>Deflection</td>
<td>$\delta_{\text{limit}} = 450 \text{ mm}$</td>
<td>$\delta_{\text{max}} = 242 \text{ mm}$</td>
</tr>
<tr>
<td>Critical Buckling Case</td>
<td>$P_{\text{cr}} = 3034 \text{ kN}$</td>
<td>$P^* = 793 \text{ kN}$</td>
</tr>
</tbody>
</table>

Table 1: Summary of Results

Strand7 Analysis

Figure 2: Beam Displacement in DZ (m)

Figure 3: Axial Stress Distribution (Pa)

Figure 4: Bending Moment Distribution (N.m)

Conclusions
- All design actions were calculated using Strand7, and were compared with design capacities as specified by Australian Standards (AS4100 Steel structures).
- A calculated deflection of 242mm for the cantilevered roof is well within the limit of (<= 450mm).
- The structure is acceptable under compression
  - $N = 753 \text{ kN} (< \phi N = 2510 \text{ kN})$
  - $\sigma^* = 25.10 \text{ kPa} (< \phi \sigma = 124.72 \text{ kPa})$
- The structure is acceptable under tension
  - $N = 365 \text{ kN} (< \phi N = 15438 \text{ kN})$
  - $\sigma^* = 744.9 \text{ kPa} (< \phi \sigma = 315.06 \text{ kPa})$
- The structure is acceptable under flexure
  - $M = 223 \text{ kN.m} (< \phi M = 4710 \text{ kN.m})$
- No buckling was experienced by any of the compressive members
  - The design buckling load = 793 \text{ kN} ($P^* = 3034 \text{ kN})$

References
- Australian Standards AS1170 and AS4100
- Online Editor: ‘Modelling Tension-Only or Compression-Only Structural Members’
Description

The Cardboard Cathedral was designed by Shigeru Ban, winner of the 2014 Pritzker Architecture Prize. The structure was intended to be a temporary replacement for Christchurch’s Anglican Cathedral, which was destroyed by the tragic 2011 earthquake. The Cathedral incorporates traditional materials, such as timber and steel, as well as unconventional elements, like 60cm diameter cardboard tubes and eight shipping containers that form the roof and walls, respectively. The cardboard tubes are coated in polycarbon, and the whole building rests on a concrete slab.

Source:

2014 Pritzker Architecture Prize

Aim

I. Investigate the structure’s performance under static loading.
II. Investigate how the orthotropic nature of timber may affect the structural behaviour compared with an isotropic assumption.
III. Assess the suitability of different element types (isotropic beams, isotropic plates, and orthotropic plates) for modelling the roof members.
CARDBOARD CATHEDRAL (Static Analysis)

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BACKGROUND
Components
- Timber-reinforced cardboard tube
- Concrete slab
- Shipping containers
- Steel frames

Structural overview

AIM
- Investigate the structure's performance under static loading.
- Investigate how the orthotropic nature of timber may affect the structural behavior compared with an isotropic assumption.
- Assess the suitability of different element types (isotropic beams, isotropic plates, and orthotropic plates) for modelling the roof members.

FEM MODELLING: Linear static

Loading:
<table>
<thead>
<tr>
<th>Load combinations</th>
<th>1.2G</th>
<th>1.2G + W_down</th>
<th>1.2G + 1.5Q</th>
<th>0.9G + W_up</th>
<th>1.2G + Snow</th>
</tr>
</thead>
</table>

Main structural members:
Radiata Pine LVL Timber

Geometry
Section: 210 mm x 600 mm
Length: 16 m to 20 m

Material properties
- E1 = 13,200 MPa
- G12 = 660 MPa
- E2 = 500 MPa
- G23 = 100 MPa
- G13 = 500 MPa
- G12 = 660 MPa
- V12 = 0.34
- V23 = 0.41
- V13 = 0.014
- ρ = 405 kg/m³
- f_tension = 33 MPa

RESULTS AND DISCUSSIONS

Comparison between 3 Models:
Isotropic Beam, Isotropic Plate and Orthotropic Plates as Individual Roof Members

Summary of Results

<table>
<thead>
<tr>
<th>Worst case: 1.2G + W_down</th>
<th>Isotropic plate (I)</th>
<th>Orthotropic plate (O)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deflection</td>
<td>39.7 mm</td>
<td>39.7 mm</td>
</tr>
<tr>
<td>Axial Tension</td>
<td>9.13 MPa</td>
<td>9.13 MPa</td>
</tr>
<tr>
<td>Axial Compression</td>
<td>9.76 MPa</td>
<td>9.13 MPa</td>
</tr>
<tr>
<td>Shear Strain</td>
<td>340 με</td>
<td>4864 με</td>
</tr>
</tbody>
</table>

Note: The above actions were chosen as they were the most suitable to highlight the differences between the isotropic and orthotropic nature of the elements.

Main Findings
- Wind was the most dominant load action, making 1.2G+W_down and 0.9G+W_e the most critical load cases for design purposes.
- The structural members are adequate for both serviceability and strength limit states.
- Deflection does not exhibit any significant sensitivity to the type of element or properties used.
- The main disparity was evident in shear strains in isotropic and orthotropic plate. This can be explained by the smaller moduli in the radial and tangential directions for the orthotropic plate.

CONCLUSIONS

Due to the proximity of results between the three main element types used (orthotropic and isotropic plate, and isotropic beam), along with the simplicity in both modelling and post-processing of isotropic beam results, it is suggested to use isotropic beam for the modelling and analysis of timber structures.

REFERENCES
Australian and New Zealand Standards AS/NZS1170.0:2002, “Structural design actions - General principles”.
Antarctica Building, Static Analysis

Description

The new research station situated off the coast of Antarctica must withstand blizzards with large quantities of snow, extreme wind speeds and temperatures lower than -40°C, resulting in extreme mechanical and thermal strain. To prevent and counter claustrophobia, a view of the landscape is provided to the researchers by large glazed portions of the façade.

Source:
Detail Online Magazine

Aim

To analyse the static and thermo-mechanic stresses and displacements induced in the Bharati Antarctic research station under a set of load combinations.
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BACKGROUND
The structure was modelled in Strand7 from a series of images of the project (prototypes, construction and finished building). The basic structure consists of frames and strutting bays, and offices were made from recycled shipping containers which for the purposes of this model were converted to static pressures on the floors. The floor plan is 30x50m.

The structure is subjected to extremes of temperature (-40°C) and wind speeds of up to 100ms⁻¹, in addition to gravity and live loads. A wind analysis to AS1170.2 was conducted to calculate the pressure distribution over the surfaces from the worst-case wind loading. An office live load of 3.0kPa was used.

Downforce
As the building is elevated above ground, the inverse ‘wing’ shape of the rear of the building results in a differential pressure resulting in downforce, as shown below. Due to the cold air, the density was assumed at 1.55kg/m³. The resulting downforce was 1.25kPa.

MODEL AND ASSUMPTIONS
Member Sizes
By scaling from pictures, the member sizes were estimated and summarised below.

<table>
<thead>
<tr>
<th>Member</th>
<th>Section</th>
<th>Linear Static Solver</th>
<th>Linear Buckling Solver</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.8m Columns</td>
<td>273.1×12.7 CHS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>V Columns</td>
<td>250×150×9.9 RHS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Main front girder</td>
<td>800WB168</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frame beams</td>
<td>250UC99.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bay slits</td>
<td>125×75×4.0 RHS</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The majority of the model was made from beam elements, with the foundations modeled as rigid supports. Plate elements were used as exterior cladding in order to transfer wind pressures and snow loads.

Thermo-mechanical coupling – Temperature Load Case
Ignoring any insulating properties from the aluminium cladding, exterior nodes were set to a temperature of -40°C whilst interior nodes were set to 23°C indicating a standard room temperature.

Snow Load
Snow density was assumed to be 5kN/m³ with a depth of 500mm, giving a conservatively high 2.5kPa roof load.

RESULTS
The ‘worst case’ load combination varies depending on the member in question. The most sensitive part of the structure is the front section supported by the v-columns, as the main girders have much larger spans.

Downward forces are most detrimental to this section, however wind uplift balances this in some load combinations. The thermo-mechanical coupling due to the temperature extremes clearly increased the stresses and displacements.

REFERENCES
- [www.engineeringtoolbox.com/air-properties-s_156.html](http://www.engineeringtoolbox.com/air-properties-s_156.html)
- [http://www.antarctica.ac.uk/about_antarctica/geography/weather/snow.php](http://www.antarctica.ac.uk/about_antarctica/geography/weather/snow.php)
Antarctica Building, Thermal Analysis

Description

A new station situated on the Antarctic coast is intended for research purposes. The structure has to withstand abnormally high thermal and mechanical strains caused by blizzards with huge quantities of snow, enormous wind speeds and temperatures of -40°C and under. A large proportion of the façade is glazed to give the researchers a generous view of the polar region and to counteract any feelings of claustrophobia.

Source:
Detail Online Magazine

Aim

I. To complete the 3D analysis of thermal load effects in energy loss over time between the building and the environment.
II. To improve the insulation in the structure to reduce the energy loss.
Antarctica Buildings Thermal – The Bharati Research Station

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Aim
• To complete the 3D analysis of thermal load effects in energy loss over time between the building and the environment.
• To improve the insulation in the structure to reduce the energy loss.

Method
Creation of the model
• Create each wall face of the main floor, roof and bottom floor using Hexit8 brick elements
• Subdivide the walls, input node temperatures and heat transfer properties
• Add glass property representing windows on the walls.
• Footings were created last by extruding the bricks at the bottom and copied incrementally

Strand 7 Inputs
Properties inputs
• Brick Hexit8 Elements
• Thermal conductivity λ=0.8W/mK
• Composite aluminium Insulating panels λ=25 W/mK

Results
1. 3D analysis of Thermal Loads
• Transient Thermal Analysis of structure.
• Result at a particular time step shown in Figure 4.

2. Calculating the Energy Loss over time
• The energy dissipated from the structure was calculated using the following steps.
  1. Calculation of Sum of Node Flux on exterior surfaces
  2. Approximation of area underneath a Flux vs Time graph to obtain total energy loss from the structure.

3. Improving the current design
• Different materials used to model insulation efficiency.

<table>
<thead>
<tr>
<th>Material</th>
<th>Convective Coefficient W/m²K</th>
<th>Thermal Conductivity W/m²K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminum</td>
<td>25</td>
<td>151</td>
</tr>
<tr>
<td>Steel</td>
<td>25</td>
<td>51</td>
</tr>
<tr>
<td>Concrete</td>
<td>23.5</td>
<td>1.31</td>
</tr>
</tbody>
</table>

Table 1: Energy Loss for different materials

Figure 1: Image of Actual Structure
Figure 2: Image of Model

Figure 3: Heat Flux with time

Figure 4: Typical temperature contour

Figure 5: Heat Flux with time

Figure 6: Cumulative Energy Loss with time

Assumptions
• Radiation is negligible
• Most of the dimensions is estimated on the real length and width of the structure found. No structural plans were available.
• The building consist only of Aluminium as material property 1 and glass as property 2
• The total energy loss is approximated by calculated the area underneath the heat flux plot. The heat flux plot is not continuous
• Mechanical Strains due to thermal loading are not modelled only the thermal loss.

Improvements
• Retrieve actual dimensions from structural plans to improve the accuracy.
• The mechanical strains can be analysed to determine whether material is appropriate for the current environment thermal loading.
• The mesh created is very fine, the number of elements can be reduced to save time in running analysis.
• Modelling the insulating foam in walls
• Obtaining ducting plan, to see how the research station is heated

Discussion
• Transient Solver \( \frac{dC}{dt} = kT^2 \)
• Concrete has better insulation properties than the alternatives
• Cost

<table>
<thead>
<tr>
<th>Material</th>
<th>Cost ($ per m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminum</td>
<td>11550</td>
</tr>
<tr>
<td>Steel</td>
<td>8530</td>
</tr>
<tr>
<td>Concrete</td>
<td>158</td>
</tr>
</tbody>
</table>

Precast concrete is cheaper, however Transportation issues may arise. Issues with concrete cracking.

Conclusion
• A quick overview on how the model was made
• From the comparison of the alternative materials, concrete insulation is superior to alternatives.

References