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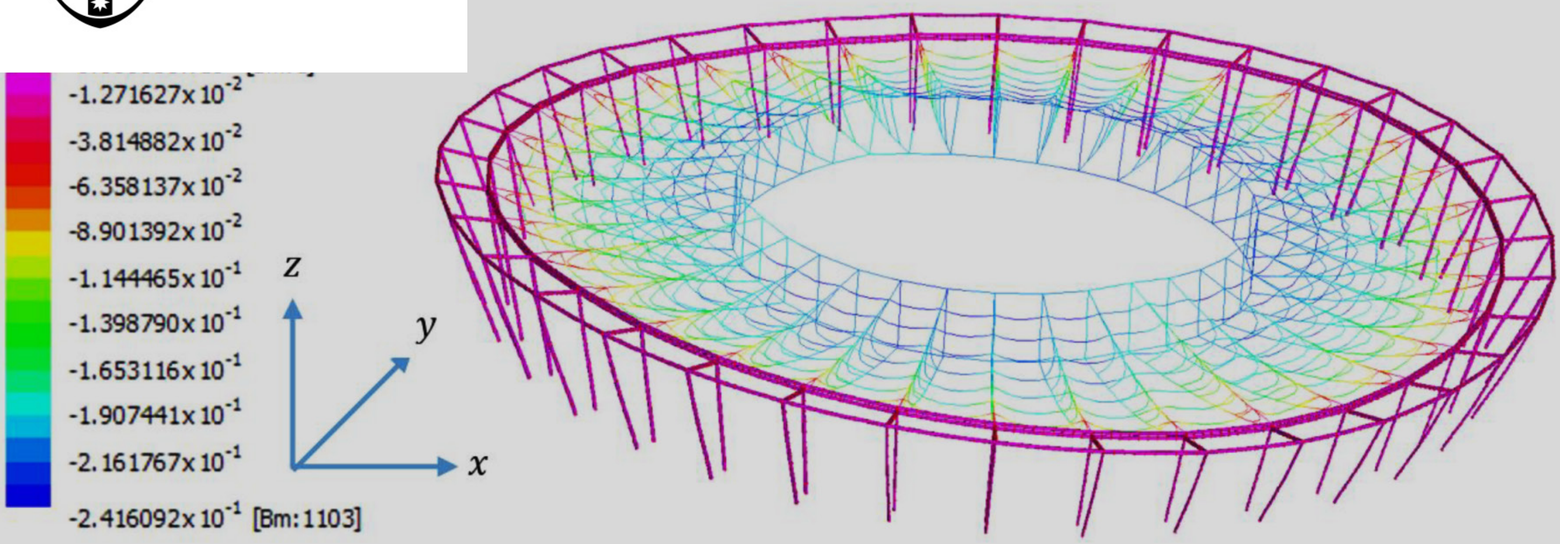

Numerical Methods in Civil Engineering

Landmark projects 2015

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Content

Landmark Projects

This catalogue 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

Student's contributors:

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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

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

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)**

Diagrid
- 310UC158 (beam element) Fy=300MPa
- Connection beam 250SHS12.5(beam element)

Beams & Columns
- Columns: ϕ800 mm circular section, beam elements, RC, 40MPa
- Floor Beams(internal &external):610 UB125, beam element, Fy=300MPa

Slabs &Shear Core
- Slab: 120mm depth, RC, 32MPa, Q8 Plate element
- Shear Core: 300mm thickness, RC, 40MPa, Q8 plate element

Facade
- Q8 elements, 10mm thickness, Glass

- **Applied Loads**

Dead Load 1.8 kPa (superimposed) Live Load 3 kPa (reduction factor 0.7)

Table 1: Wind Loads

Level	1	2	3	4	5	6	7	8	9	10	11
Windward (kPa)	0.50	0.52	0.57	0.62	0.66	0.69	0.73	75.00	0.77	0.79	0.81
Leeward (kPa)	0.31	0.32	0.36	0.39	0.41	0.43	0.45	0.47	0.48	0.49	0.51

Boundary Conditions Nodes at the ground level are all fixed

Results & Discussion

Lateral Deflection (0.9G+W)

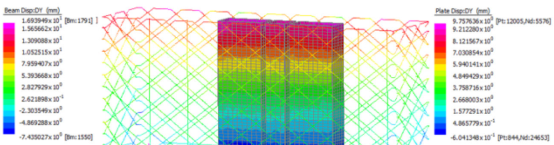

Figure 2: Lateral Deflection of Diagrid Structure under 0.9G+W

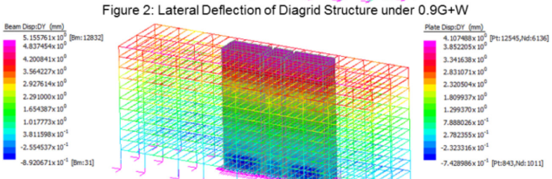

Figure 3: Lateral Deflection of Conventional Structure under 0.9G+W

1.2G+0.4Q+W	Diagrid Structure (mm)	Conventional Structure (mm)	Absolute difference (mm)
Beam Displacement	24.32	5.49	18.83
Plate Displacement	17.74	4.56	13.18

Table 2: Comparison of Lateral Deflection Under 1.2G+0.4Q+W

Conclusion

From the analysis of the Macquarie Bank model it was found that the diagrid is less effective and efficient compared to the conventional column beam structure. A diagrid system is selected mainly based on aesthetic purposes rather than performance.

Vertical Deflections(G+0.4Q)

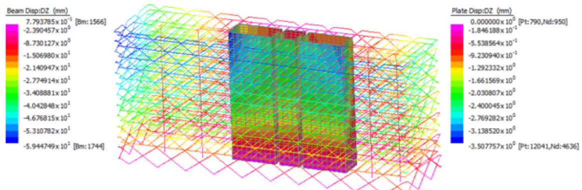

Figure 4 Vertical Deflections of Diagrid Structure under G+0.4Q

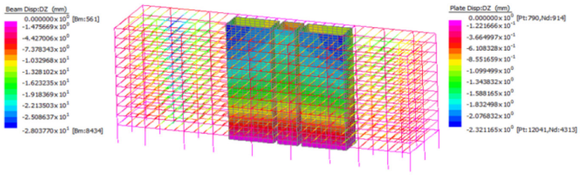

Figure 5 Vertical Deflections of Conventional Structure under G+0.4Q

Serviceability Check (Load Case: G+0.4Q)

	Max deflection in **conventional**(mm)	Max deflection in **diagrid**(mm)	Span (mm)	Deflection limit(mm)
Floor Slab	21.90	20.89	12800	span/300=42.7
Floor Beam	21.90	20.89	12200	span/250=48.8

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

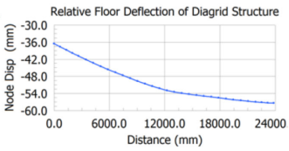

Figure 6 Relative Floor Deflection of Diagrid Structure

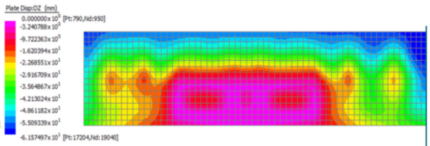

Figure 7 Top Floor Slab Deflection of Diagrid Structure

Stress Distribution(1.2G+1.5Q)

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

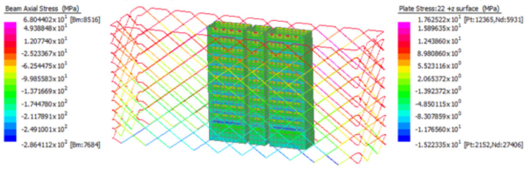

Figure 8 Stress Distribution of Diagrid Structure under 1.2G+1.5Q

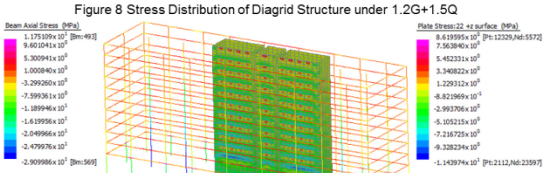

Figure 9 Stress Distribution of Conventional Structure under 1.2G+1.5Q

Cost Comparison

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

Structural System	Total Material Cost
Diagrid	$17.70million
Perimeter columns	$11.90million

Table 4: Comparison of Material Costs of Two Structures

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

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.

STATIC ANALYSIS OF 1 BLIGH ST (BUILDING)

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BACKGROUND

ELLIPTICAL 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

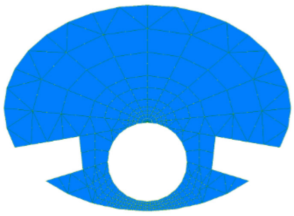

Figure 1 Mesh of the slab

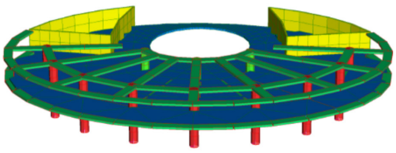

Figure 2 Columns and beams on a typical floor

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.

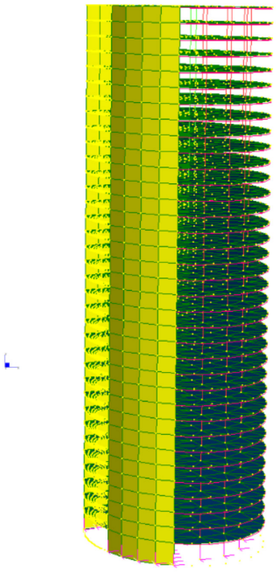

Figure 3 Model of the whole structure

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

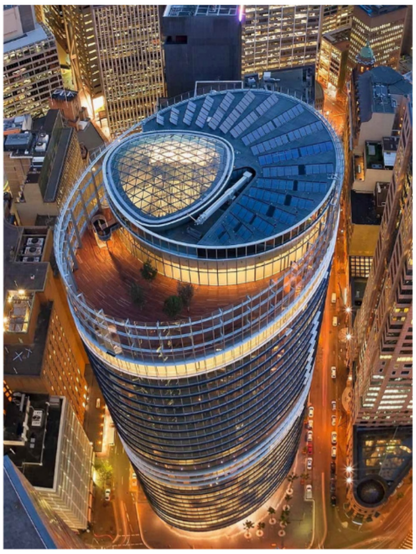

Figure 4 Photograph of 1 Bligh St

MAX	INTIAL STRUCTURE	DUE TO THE ADDITIONAL STRUCTURE
DX (mm) (top)	6.01	7.73
DY (mm) (top)	262.60	306.10
DZ (mm) (bottom)	97.77	109.21
BENDING MOMENT (kNm) (bottom)	5495.80	5976.90
AXIAL FORCE (kN) (bottom)	99270.00	106103.00
SHEAR FORCE (kN) (bottom)	2260.00	2454.60

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 $\Phi = 0.6$
- The initial percentage of steel reinforcement to each column = 2% (Area = 0.035 m^2)

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 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.
- Green Building Council Australia, 2011,Green Building Case Studies: 1 Bligh Street, Sydney. Viewed on 26 May 2015 https://www.gbca.org.au/green-star/green-building-case-studies/1-bligh-street/

Severin Bridgel, 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.

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Supervised by Faham Tahmasebinia, Peter Ansourian, and Fernando Alonso-Marroquín
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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 AS5100 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 1959 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.

Property No.	Description	Section
Beam Elements		
1	Main Girder	2m x 4.5m x 50mm RHS
2	Longitudinal Deck Stiffeners	310UB32.0
3	Transverse Deck Stiffeners	410UB43.7
4	Pedestrian Edge Girder	800mm x 20mm Plate
5	Transverse Major beam	500mm x 500mm x 50mm SHS
9	Cables	350mm Diameter Steel Cable
Plate Elements		
1	Main Deck	200mm Thick Concrete Slab, 50MPa
2	Pedestrian Deck	100mm Thick Concrete Slab, 50MPa
6	Pylon	4.5m x 5m x 100mm RHS

Table 1 – Section Properties

Figure 1 – Severin Bridge

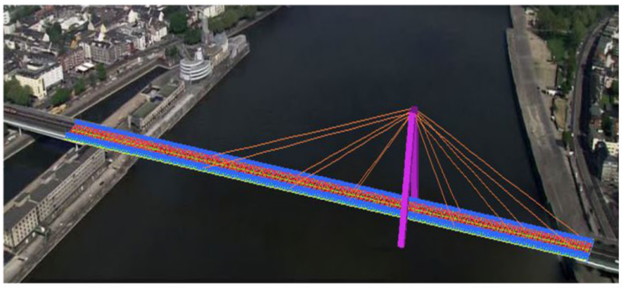

Figure 2 – Strand7 Model

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.

	Deflection (DY) (m)	Stress (VM) (MPa)	Stress (Axial) MPa)
Maximum	0.430	285	150

Table 2 - Results

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 AS5100.2 Cl. 6.11 (1/600 of the span).

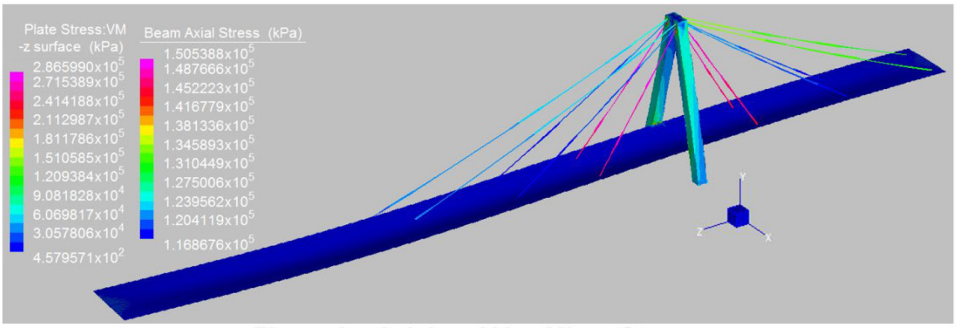

Figure 3 – Axial and Von Mises Stress

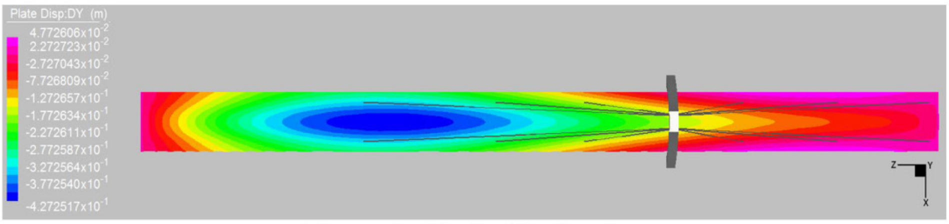

Figure 4 – Vertical Deflection

References:

www.structurae.net/structures/severin-bridge
Image: Rolf Heinrich, Cologne (Open Source)

http://www.stadt-koeln.de/leben-in-koeln/planen-bauen/bruecken/severinsbruecke

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

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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

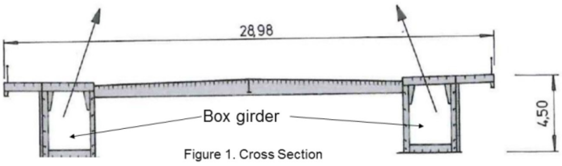

Figure 1. Cross Section

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

3D Model Construction

Table 1. Elements properties

Membes	Elements	Properties	Geometry	Dimension (m)
bridge deck	concrete plate	Type: Plate/Shell Material: Isotropic Modulus: 38002 Mpa Poisson' s Ratio: 0.2 Density: 2880 kg/m^3 Viscous Damping: 0kN.s/m/m^3 Damping Ratio: 0 Thermal Expansion: 1×10^{-5} /C	Solid Plate	Member Thickness: 0.3
beams	steel beam	Material: Structural Steel Type: Beam Modulus: 200000 Mpa Poisson' s Ratio: 0.25 Density: 8635.0 kg/m^3 Viscous Ratio: 0kN.s/m/m^3 Damping Ratio: 0 Thermal Expansion 1.17×10^{-5} /C		Flange Width: 0.6 Flange Thickness: 0.02 Total Depth: 2.7 Web Thickness: 0.02
box girders	steel beam			Width: 4.3 Depth: 4.5 Thickness: 0.3
tower	steel beam			Width: 8.0 Depth: 8.0 Thickness: 1.0
cables	pretensioned truss	Material: Structural Steel Type: Truss Modulus: 200000 Mpa Poisson' s Ratio: 0.25 Density: 8635.0 kg/m3 Viscous Ratio: 0kN.s/m/m3 Damping Ratio: 0 Thermal Expansion 1.17×10-5 /C		Diameter: 0.4

Load

- Dead load - self-weight
- Live Load calculation:

Normal Pressure act on the plate:

$$\text{Pressure} = \frac{Total\ load}{Related\ Area} = \frac{360*12*6}{20*10} = 0.1296\ MPa$$

- Pre-load act on the truss member:

$$Pre - load = \frac{Maximum\ steel\ stress}{2} * A_{cable} = 18.8\text{kN}$$

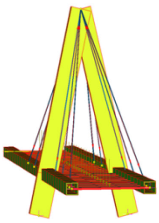

Figure 3. Model Cross Section

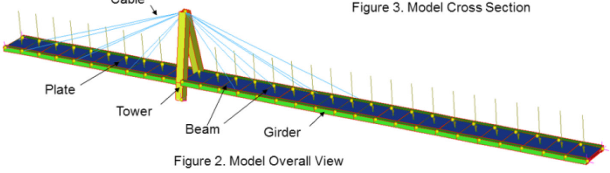

Figure 2. Model Overall View

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 spctrum was 1.25/x^(2/3). The period range was 0.35355-3
Sampling range was 2-20.

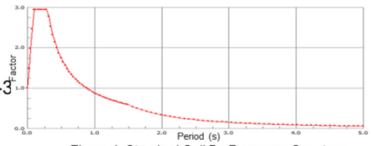

Figure 4. Standard Soil Be Response Spectrum

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/s^2 following a paper on seismic studies of the city of Cologne.

Results

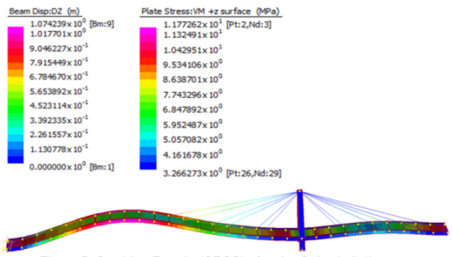

Figure 5. Combine Results (SRSS) of under Seismic Action

Figure 5 summerises 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.

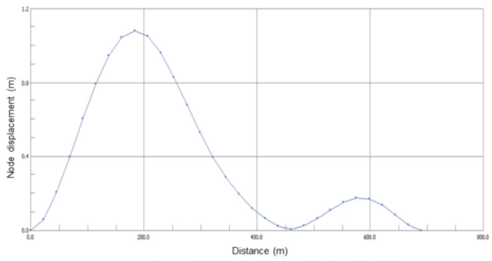

Figure 6. Node displacement along the bridge

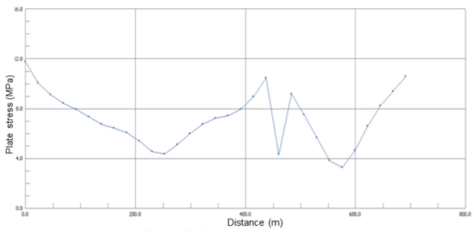

Figure 7. Plate stress along the bridge

The maximum plate stress the members of bridge experiencing is 11.78 MPa (figure 7). It occurs at the far end of the bridge.

Reference

Bridges - initial design. (2015, 6 3). Retrieved from SteelConstruction.info: http://www.steelconstruction.info/Bridges_-_initial_design
M. J. N. PriestleySeible, Gian Michele CalviF. (1996). Seismic Design and Retrofit of Bridges.
SCHWARZ, J., LANGHAMMER, T., MAIWALD, H., & SMOLKA, A. (2004). *COMPARATIVE SEISMIC RISK STUDIES FOR GERMAN EARTHQUAKE REGIONS.* Vancouver, B.C., Canada.
Severin Bridge. (2015): Struc turae: http://structurae.net/structures/severin-bridge

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.

THE UNIVERSITY OF SYDNEY

VERRAZANO NARROWS BRIDGE

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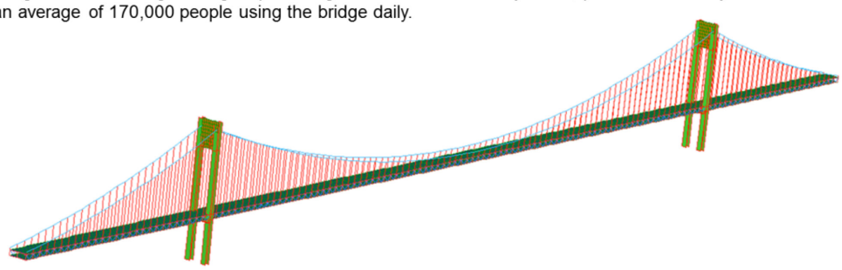
INTRODUCTION

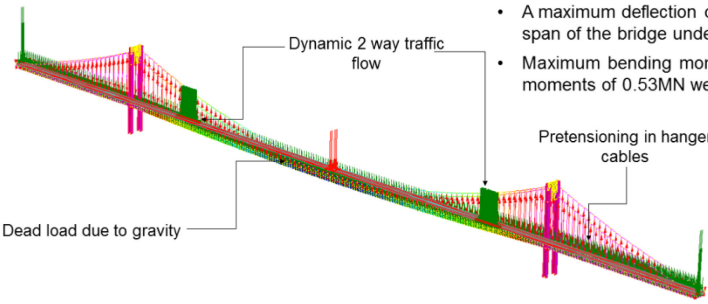
Aim
Undertake a simplified static and dynamic analysis of the Verrazano Narrows Bridge using Stand7 software. Both dead and live loads were considered. A linear solver was used in conjunction with influence lines to model the flow of traffic. Deflections and maximum stresses were reviewed.

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.



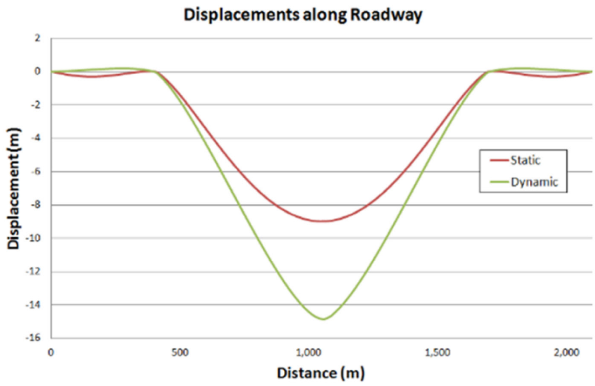
RESULTS

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 mid span 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

- Arnold W.T. 2009. A critical overview of the Verrazano-Narrows Bridge. Preceding of bridge engineering 2 conference. University of Bath
- 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

Façade Greening – Bosco Verticale (Static)

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INTRODUCTION

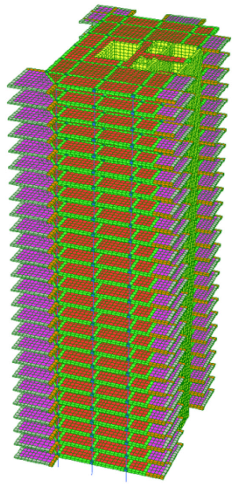
Background

Bosco Verticale, also known as the 'Vertical Forest' is a pair of residential towers in Milan, Italy. It features more than 900 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;

- 25x43m core structure
- 3.25m cantilevered balcony

Summary of material properties and loads:

	Member	Element type	Shell thickness (mm)	Material property
1	Column	Beam	N/A	AS3600 Concrete – Gr 32MPa [a]
2	Internal Slab	Plate (shell)	200	AS3600 Concrete – Gr 32MPa [a]
3	Internal beam	Plate (shell)	500	AS3600 Concrete – Gr 32MPa [a]
4	Shear wall	Plate (shell)	750	AS3600 Concrete – Gr 50MPa [a]
5	Balcony slab	Plate (shell)	280	AS3600 Concrete – Gr 40MPa [a]
6	Balcony slab perimeter	Plate (shell)	280	AS3600 Concrete – Gr 40MPa [a]
7	Continuous Cantilever beam	Plate (shell)	280	AS3600 Concrete – Gr 32MPa [a]

a. Material property subject to modification according to methodology adopted

	Member	Deal Load (kPa)	Live Load (kPa)	Wind Load (kPa)
1	Column	SW + Si [a]	2	1.956 [b]
2	Internal Slab	SW + Si [a]	2	1.956 [b]
3	Internal beam	SW + Si [a]	2	1.956 [b]
4	Shear wall	SW	N/A	1.956 [b]
5	Balcony slab	SW	3	N/A
6	Balcony slab perimeter	SW + 8.75 [c]	3	N/A
7	Continuous Cantilever beam	SW	3	N/A

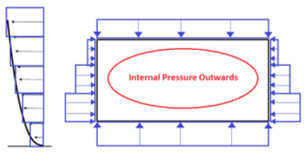
a. Si: Super-imposed loads of 1.5 kPa to account for M&E, walls, glass, ventilation etc

b. Wind load applied on internal members directly connected to the building perimeter (excluding balcony)

c. Total pressure from fully saturate soil, and mature trees & shrubs.

Wind load action with velocities of 30ms^{-1} - 43 ms^{-1}

Lateral deflection, winds loads are summarized as follow;



Strand7 Model

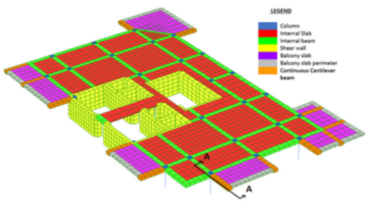
The structure was modelled using Quad 4 and Tri 3 elements as it provides sufficient accuracy as well as minimizing the execution time.

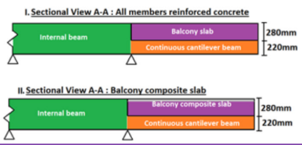
Design methods adopted;

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).

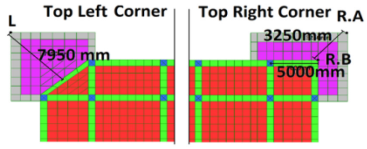
- Reinforced concrete: E values adjusted using steel to concrete ratio.
- Composite slab: Separate plane strain cantilever was modelled for a reinforced concrete member and a composite member, and $E_{composite}$ estimated using formula $\delta = \frac{FL^3}{3EI}$, assuming $I_{reinforced}$ = $I_{composite}$



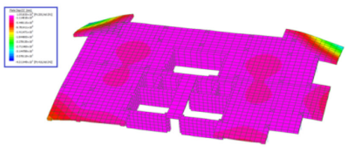


Limit & Strand7 output

Serviceability deflection limit, δ_{limit} = L/180 for the cantilevered balcony. As stated in Eurocode BS EN 1990.



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



Method	Deflection, δ (mm) L, Left corner	R.A, Right corner A	R.B, Right corner B	PASS /FAIL
	44.14 (δ_{limit})	27.78 (δ_{limit})	25.12 (δ_{limit})	
1 (Fully concrete)	70.61	17.22	34.9	FAIL
2 (Method 1 + 5% steel reinforcement)	60.64	14.87	29.76	FAIL
3 (Method 2 + continuous cantilever beam)	48.96	13.68	24.69	FAIL
4 (Method 3 + composite balcony slab)	40.11	10.49	24.08	PASS

The overall lateral displacement limit, Δ_{limit} =H/300 . As stated in the UK national Annex. Therefore the Δ_{limit} = 303 mm

Maximum lateral deflection of the building is 11.3 mm in the Y-axis and 2.78 in the X-axis.

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 neglecting the apparent increase in I, 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

Green Roofs, Walls & Facades. (2013). Victoria: The Growing Green Guide Partners.

Kim, K. (2014). Beyond Green: Growing Algae Facade. University of North Carolina.

Standards Australia. (2011). Australian Standard AS1170.2: Wind Loads, Standards Australia.

British Standards Institution. (1990). Eurocode 1: Actions on Structures; British standard, London,BSi.

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 think 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.

FAÇADE GREENING – Earthquake Analysis

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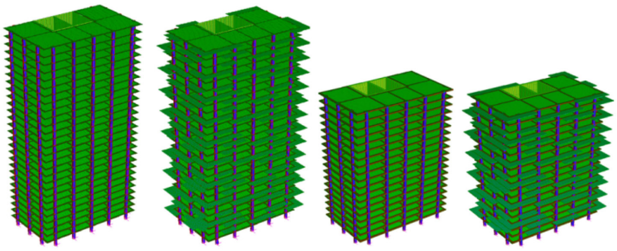
AIMS

Four models were developed so to fulfil the following aims:

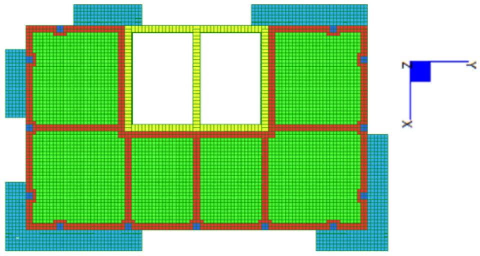
1. To simulate Models A & B under dead, live and earthquake loading.
2. To investigate the effect of the balconies and building heights by comparing Models A & B with (A.2 & B.2) and without (A.1 & B.1) cantilever balconies.
3. To compare STRAND7 results with theoretical results to ensure the accuracy of the models.



Model A.1 Model A.2 Model B.1 Model B.2

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.



Typical modelled floor plan with asymmetric balconies using Quad-4 plate elements

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

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

MAIN RESULTS

The four models generated the following results:

Strand7 Results	Model A.1	Model A.2	Model B.1	Model B.2
Fundamental Frequency (Hz)	0.684	0.685	1.38	1.368
Mass Participation (%)	75%	75%	60%	72%
Balcony z deflection (mm)	-	-11.4	-	-8.1
Column x Sway (mm)	67	77	30	31
Column y Sway (mm)	79	66	29	30
Base Horizontal Force (kN)	F_y = 414,400	F_y = 430,200	F_y = 387,600	F_y = 495,500
Base Moment (kN.m)	M_x = 31.41x10^6	M_x = 34.95x10^6	M_x = 21.14x10^6	M_x = 23.11x10^6
EQ Balcony Moment (kN.m)	N/A	372	N/A	157

Theoretical Results	112 m Building	76 m Building
Fundamental Frequency (Hz)	0.31	0.41
Mass Participation (%)	90	90
Balcony Deflection Limit (mm)	33.9	33.9
Sway Limit (mm)	1,620	1,140
Base Horizontal Force (kN)	155,333	105,404
Base Moment (kN.m)	11.15 x10^6	5.182 x10^6

CONCLUSIONS

- The fundamental frequency of the theoretical analysis is much lower than the results given by Strand7. This this could 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 it is noteworthy 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

Arup. 2014, 'Milan plants a new forest in the sky', Arup, viewed 8 June 2015, <http://www.arup.com/News/2014_05_May/13_May_Milan_plants_a_new_forest_in_the_sky>.
Boeri Studio. 2009, 'Vertical Forest', Stefano Boeri Architetti, viewed 8 June 2015, <http://www.stefanoboeriarchitetti.net/en/portfolios/bosco-verticale/>.
Maier, F. 2014, 'Complete Façade Greening: High-Rise Apartment Building in Milan', Detail Magazine, viewed 8 June 2015, < http://www.detail-online.com/architecture/topics/complete-facade-greening-high-rise-apartment-building-in-milan-022918.html>.
Pham, D. 2011, 'Bosco Verticale in Milan will be the world's first vertical forest', Inhabitat, viewed 8 June 2011, < http://inhabitat.com/bosco-verticale-in-milan-will-be-the-worlds-first-vertical-forest/>.
Standards Australia. 2009, 'Australia Standards – Concrete Structures', viewed 3 June 2015.
Standards Australia. 2007, 'Australia Standards – Structural Design Actions: Part 4 Earthquake Actions in Australia', viewed 3 June 2015.
Strand7 Reference Manual R2.4 2015, Strand7 Pty Ltd, Sydney, NSW, 2000, Australia.

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/(m^2.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

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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.

Passivehaus Triple Glazed Window

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

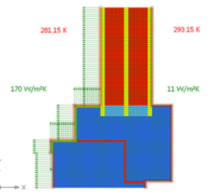
$$\left[\frac{\partial}{\partial x}\left(k\frac{\partial T}{\partial x}\right) + \frac{\partial}{\partial y}\left(k\frac{\partial T}{\partial y}\right) + \frac{\partial}{\partial z}\left(k\frac{\partial T}{\partial z}\right)\right] = 0$$

$$q = -U_w \Delta T$$

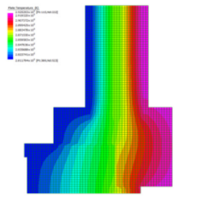
Results - Thermal Response

Triple Glazed Window

- 2980 Quad8 elements
- Average inside temp = 19.2°C
- U value = 0.734 W/m².K



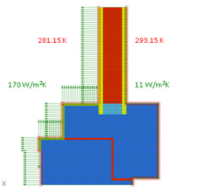
Strand7 Mesh – With Boundary Conditions



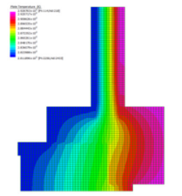
Strand7 Temperature Contour

Double Glazed Window

- 2485 Quad8 elements
- Average inside temp = 18.9°C
- U value = 1.1233 W/m².K



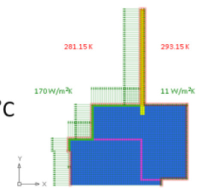
Strand7 Mesh – With Boundary Conditions



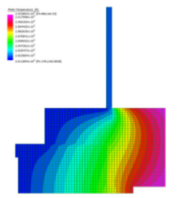
Strand7 Temperature Contour

Single Glazed Window

- 1990 Quad8 elements
- Average inside temp = 13.3°C
- U value = 14.937 W/m².K



Strand7 Mesh – With Boundary Conditions



Strand7 Temperature Contour

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

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

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

THE UNIVERSITY OF SYDNEY

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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: Tirschenreuth, 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

Photographs by ANNABAU

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 = 41.4 MPa

Bridge parapets

- Element: 8-node brick
- Material: 12% m.c pine, maritime

V-shaped support

- 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+0.8W)
    - AS 5100.2 (2004) Clause 22.3

SERVICEABILITY LIMITS

Vertical deflection

AS 5100.2 (2004) Clause 6.11:

- Sagging: δ < 42.5m / 600 = 0.07m = 70.83mm
- Hogging: δ < 42.5m / 300 = 0.1417m = 141.7mm

Horizontal deflection:

AS 5100.2 (2004) Clause 11.5:

- δ < 3.4525 m / 300 = 0.01142m = 11.42mm

LOAD FREQUENCIES

Pedestrian loading

- Walking: 2.0 Hz
- Jogging: 2.5 Hz
- Sprinting: 3.2 Hz

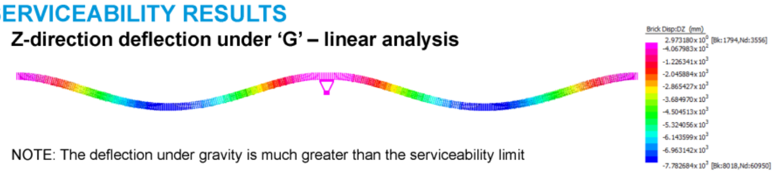
Maximum wind loading

- Re = $\rho vL/\mu$ = 1863566.434
- St = 0.198(1-19.7/Re) = 0.198
- ω = St*v/l = 4.8114 Hz

FINITE ELEMENT MODEL OF THE BRIDGE

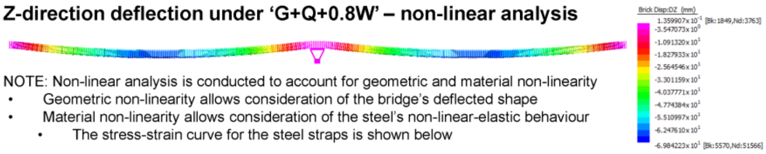
SERVICEABILITY RESULTS

Z-direction deflection under 'G' – linear analysis



NOTE: The deflection under gravity is much greater than the serviceability limit

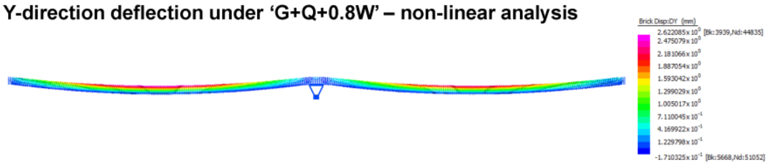
Z-direction deflection under 'G+Q+0.8W' – non-linear analysis



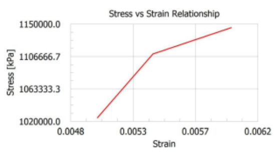
NOTE: Non-linear analysis is conducted to account for geometric and material non-linearity

- Geometric non-linearity allows consideration of the bridge's deflected shape
- Material non-linearity allows consideration of the steel's non-linear-elastic behaviour
    - The stress-strain curve for the steel straps is shown below

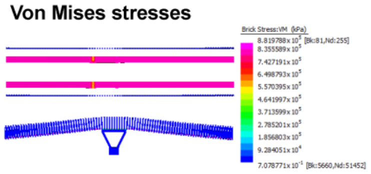
Y-direction deflection under 'G+Q+0.8W' – non-linear analysis



Stress-strain curve for AISI 4130



Von Mises stresses



Summary of deflection results

Parameter	FEM (mm)	Limit (mm)	
Max. lateral δ	2.622	11.42	✓
Max. sagging δ	69.84	70.83	✓
Max. hogging δ	0.136	141.7	✓

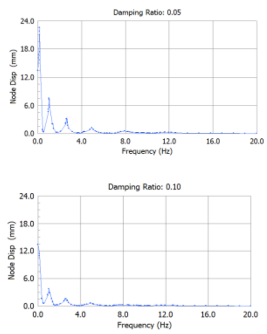
Summary of stress results

Material	FEM (MPa)	Yield (MPa)	
Timber - Pine	0.023	41.40	✓
Steel AISI 4130	882.0	1110	✓
Concrete	0.0007	32.00	✓

NATURAL FREQUENCY RESULTS

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

Mode	Hz	PF-Y	PF-Z
1	0.164	0.000	33.474
2	0.165	0.000	33.695
5	1.041	0.000	6.748
6	1.047	0.000	6.783
7	1.391	0.042	0.000
8	1.395	0.041	0.000
11	2.639	0.000	2.677
12	2.653	0.000	2.686
17	4.151	0.005	0.000
18	4.163	0.005	0.000
19	4.938	0.000	1.427
20	4.963	0.000	1.430
25	7.075	0.002	0.000
26	7.095	0.002	0.000
27	7.923	0.000	0.885
28	7.962	0.000	0.886
29	8.622	0.004	0.000
30	8.646	0.003	0.000
31	9.535	27.827	0.000
32	9.580	41.190	0.000
TOTAL MASS PF		69.121	90.691



Findings

- Pedestrian frequencies do not correspond with the dominant natural frequencies in the Z-direction
    - Jogging (2.5 Hz) *may* cause some excitation
- Wind load frequency does not correspond to the dominant natural frequencies in the Y-direction
- Increasing damping can be used to reduce the magnitude of nodal displacements due to excitation

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
    - Displacements remain within serviceability limits; can be further reduced with dampening

REFERENCES

- ANNABAU 2015, *Spannbandbrucke Tirschenreuth*. Retrieved from: <http://www.annabau.com/index.php?site=projects&subsite=project&id=37>
- Australian Standards 2004, *Bridge Design Set*, AS 5100.2-2004, Retrieved from: Australian Standards.
- MatWeb, *AISI 4130 Steel*. Retrieved from: <http://www.matweb.com/search/datasheet.aspx?matguid=284aa856470341acbf3c16bc90849ea1>
- MatWeb, *American Eastern Pine Wood*. Retrieved from: <http://www.matweb.com/search/datasheet.aspx?matguid=1bec7114d2524b63826044c3cc6c344c&ckck=1>
- Bachmann, H. and Ammann, W. (1987). *Vibrations in structures*. Zürich: International Association for Bridge and Structural Engineering.

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:

The New Your Times, Chernobyl: Capping a Catastrophe. By Henry Fountain. Photographs by William Daniels, April 27, 2014

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.

Chernobyl New Safe Confinement Thermal Analysis

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AIMS OF THE PROJECT

To analyse 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.

Figure 1: Chernobyl dome construction (Novarka, 2013)

ASSUMPTIONS

Assumption 1: Air conductivity

Input Values			Calculated Values		
$T_2\ (K)$	$T_{fluid}\ (K)$	$h\ (W/(m^2K))$	$k_{eff}\ (W/mK)$	$T_2\ (K)$	$T_{fluid}\ (K)$
458	573	1.9153	526.1	534	572
534	572	1.3997	517.4	534	572
534	572	1.3997	517.4	534	572

Table 1: Thermal conductivity iteration

$$k_{eff} = L(h + 4\sigma T_{fluid}^3)$$

(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

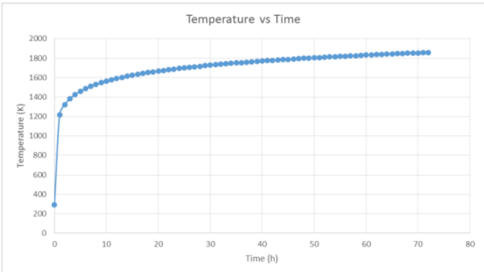

Figure 2: Fire curve

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

- Strand7. (2014, July 15). ST7-1.30.10.4 Approximating an AirGap in Thermal Analysis. Strand7
- Strand7. (2014, December 23). ST7-1.30.20.1 Nonlinear Weld Mechanical Analysis. Strand7
- Promat. (2002, July 1). *Fire curves*. Retrieved from Promat International: http://www.promat-tunnel.com/en/advices/fire-protection/fire%20curves
- Novarka. (2013, January 8). *Key Figures*. Retrieved from Novarka: http://www.novarka.com/en/key-figures/
- Excell, J. (2013, February 11). *Building Chernobyl's New Safe Confinement*. Retrieved from The Engineer: http://www.theengineer.co.uk/in-depth/the-big-story/building-chernobyls-new-safe-confinement/1015479.article

ANALYSES

The analysis was performed for 72 time steps each of one hour duration. A nonlinear analysis was used to account for the temperature-dependent properties.

Transient Heat Analysis

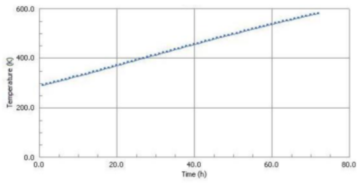

Figure 3: Temperature of the internal steel cladding

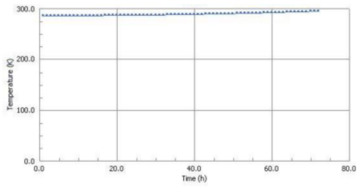

Figure 4: Temperature of the external steel cladding

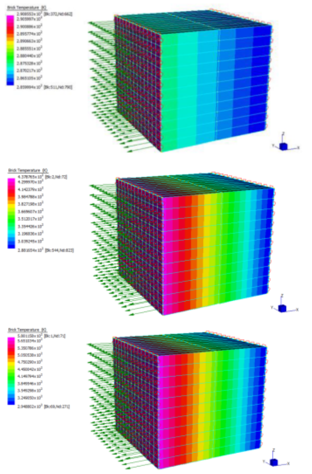

Figure 5: Time evolution of temperature in the cross section at 0, 36,72 hours

Quasi Static Analysis at 72 hours

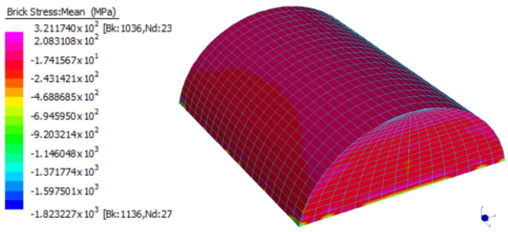

Figure 6: Mean stress of the internal steel cladding

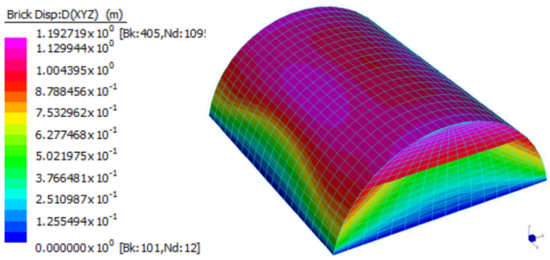

Figure 6: Deflection of the internal steel cladding

CONCLUSIONS

- The internal temperature changes from approximately 300K to 580K, while the external temperature changes from 275K 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.

THE UNIVERSITY OF SYDNEY

Bamboo Forest: Bao'An Stadium (Static)

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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.

Figure 1: Photograph of Bao'An Stadium (GMP Architekten)

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 70 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 stayed roof truss pre 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-trafficable 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.

Loading Type	Theoretical	Strand7
Flexure	$\varphi M_s = 4710\ kN.m$	$M^* = 223\ kN.m$
Axial Compression	$\varphi N_c = 2518\ kN$	$N_c^* = 753\ kN$
Axial Tension (Cables)	$\varphi N_t = 131\ kN$	$N_t^* = 71\ kN$
Axial Tension (Beams)	$\varphi N_t = 15{,}438\ kN$	$N_t^* = 365\ kN$
Tensile Stress (Cables)	$\varphi\sigma_t = 184\ 507\ kPa$	$\sigma_t^* = 9859\ kPa$
Tensile Stress (Beams)	$\varphi\sigma_t = 315\ 061\ kPa$	$\sigma_t^* = 7449\ kPa$
Compressive Stress	$\varphi\sigma_c = 124\ 472\ kPa$	$\sigma_c^* = 25\ 100\ kPa$
Deflection	$\delta_{limit} = 450\ mm$	$\delta_{max}^* = 242\ mm$
Critical Buckling Case	$P_{cr} = 3034\ kN$	$P^* = 793\ kN$

Table 1: Summary of Results

Strand7 Analysis

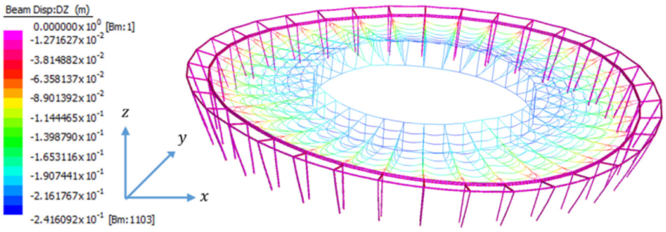

Figure 2: Beam Displacement in DZ (*m*)

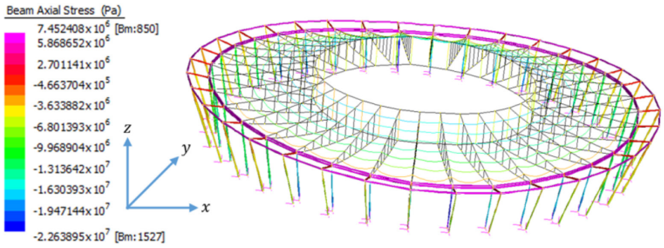

Figure 3: Axial Stress Distribution (*Pa*)

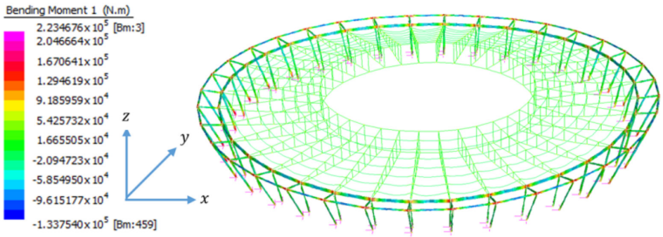

Figure 4: Bending Moment Distribution (*N.m*)

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* = $753kN$ ($< \varphi N_c = 2518\ kN$)
  - $\sigma_c^* = 25\ 100\ kPa$ ($< \varphi\sigma_c = 124\ 472\ kPa$)
- The structure is acceptable under tension
  - N* = $365kN$ ($< \varphi N_t = 15\ 438\ kN$)
  - $\sigma_t^* = 7449\ kPa$ ($< \varphi\sigma_t = 315\ 061\ kPa$)
- The structure is acceptable under flexure
  - M* = $223kN.m$ ($< \varphi M_s = 4710\ kN.m$)
- No buckling was experienced by any of the compressive members
  - The design buckling load = $793kN$ ($< P_{cr} = 3034\ kN$)

References

- Gerkan, M. & Marg, V., 2011. GMP Architekten. [Online] Available at: http://www.gmp-architekten.com/projects/baoan-stadium-universiade-2011.html
- Taiyo Kogyo, 2011. Taiyo Kogyo Corporation. [Online] Available at: http://www.makmax.com/news/2011/nw0513.html
- Australian Standards AS1170 and AS4100
- Online Editor: 'Modelling Tension-Only or Compression-Only Structural Members'

Cardboard cathedral, Christchurch, Static Analysis

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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School of Civil Engineering

FACULTY OF ENGINEERING & INFORMATION TECHNOLOGIES

BACKGROUND

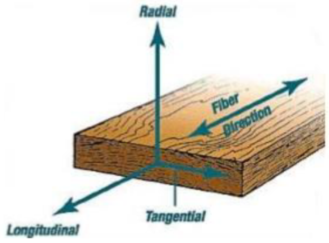
Components

- 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:

Load combinations	
1.35 G	1.2 G + W_{down}
1.2 G + 1.5 Q	0.9 G + W_{up}
1.2 G + Snow	

Main structural members:

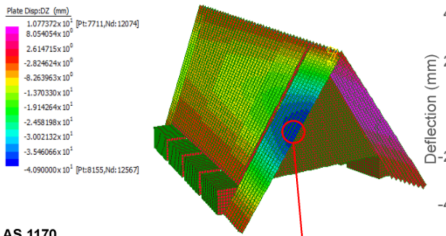
Radiata Pine LVL Timber

Geometry			
Section: 213 mm x 600 mm			
Length: 16 m to 20 m			
Material properties			
E_1	= 13,200 MPa	G_{12}	= 660 MPa
E_2	= 500 MPa	G_{23}	= 100 MPa
E_3	= 500 MPa	G_{31}	= 660 MPa
ν_{12}	= 0.34	ν_{23}	= 0.41
ν_{31}	= 0.014	ρ	= 405 kg/m^3
$f_{Tension}$	= 33 MPa		

RESULTS AND DISCUSSIONS

Check using Orthotropic Plates as Rafters

Serviceability Check (Critical Case: W_{up})



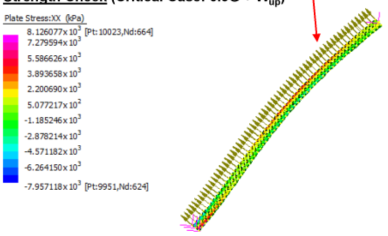
AS 1170

Serviceability Requirement

Main roof members: L/300 = 63 mm

40.9 mm < 63.0 mm

Strength Check (Critical Case: 0.9G + W_{up})

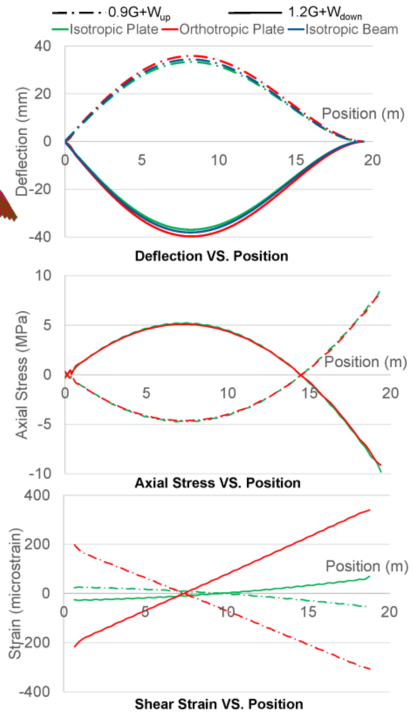


Maximum Tension = 8.13 MPa

Tensile Strength = 33 MPa

Comparison between 3 Models;

Isotropic Beam, Isotropic Plate and Orthotropic Plates as Individual Roof Members



Summary of Results

Worst case: 1.2 G + W_{down}	Isotropic plate (I)	Orthotropic plate (O)	O/I
Deflection	36.9 mm	39.7 mm	1.076
Axial Tension	8.71 MPa	8.13 MPa	0.933
Axial Compression	9.79 MPa	9.13 MPa	0.933
Shear Strain	69.9 με	340 με	4.864

Note: The above actions were chosen as they were the most suited 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_{up} 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

Strand7 Webnotes 2015, "Modelling Timber as an Orthotropic Material", Strand7 Pty Limited, Sydney, Australia.

Australian and New Zealand Standards AS/NZS1170.0:2002, "Structural design actions - General principles".

New Zealand Wood 2007, "Structural Materials: LVL Performance", Wellington, New Zealand.

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.

BHARATI ANTARTIC RESEARCH STATION

THE UNIVERSITY OF SYDNEY

Andrew Boland, Scott Grant, Mitchell Grech, Ludvig Arentz-Hansen

Supervised by Faham Tahmasebinia, Peter Ansourian, and Fernando Alonso-Marroquín

School of Civil Engineering

FACULTY OF ENGINEERING & INFORMATION TECHNOLOGIES

AIM

To analyse the static and thermo-mechanic stresses and displacements induced in the Bharati Antarctic research station under a set of load combinations.

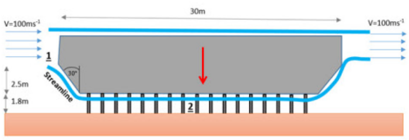
BACKGROUND

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 strutted 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/m^3. The resulting downforce was 1.2kPa.



MODEL AND ASSUMPTIONS

Member Sizes

By scaling from pictures, the member sizes were estimated and summarised below.

Member	Section
1.8m Columns	273.1x12.7 CHS
V Columns	250x150x9.0 RHS
Main front girder	800WB168
Frame beams	250UC89.5
Bay struts	125x75x4.0 RHS

The majority of the model was made from beam elements, with the foundations modelled 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/m^3 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.

	Linear Static Solver				Linear Buckling Solver
Load Case	DX (mm)	DY (mm)	DZ (mm)	Axial Stress (MPa)	Lowest Eigenvalue
Dead Load (G)	-0.1905	0.1458	-4.2464	-29.21	32.02
Live Load (Q)	-0.1587	0.1214	-3.5374	-24.33	38.44
Wind Load 1 (W)	-3.0303	-0.7117	3.2217	31.28	-32.17
Wind Load 2	-2.8050	-0.6695	2.3070	25.77	-46.66
Snow Load (S)	-0.1222	0.1045	-2.3618	-13.39	46.23
Temperature -40	7.7040	-16.0942	-6.5461	-88.17	2.68
Temperature 23	-3.0244	-3.3576	2.4237	-48.76	4.75
Temperature Combination (T)	7.8976	-14.9676	-8.2868	-135.06	1.72
1.2G+1.5Q	-0.4666	0.3570	-10.4017	-71.56	13.07
1.2G+W+0.6Q	-3.0236	0.6644	-7.5659	-52.74	26.6
1.2G+S+0.6Q	-0.4210	0.3519	-8.8192	-60.44	13.39
1.2G+1.5Q+T	7.4671	4.8483	-12.8684	-134.96	1.72
1.2G+W+0.6Q+T	-10.5931	-14.8088	-10.6960	-134.69	1.73
1.2G+S+0.6Q+T	7.4766	-14.7056	-12.7068	-134.97	1.72

Buckling Considerations

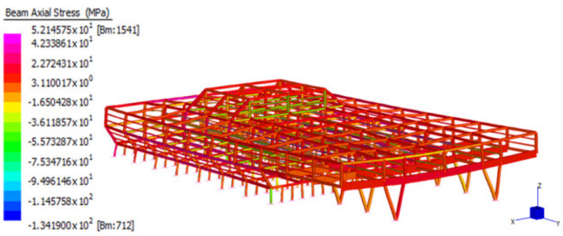
A buckling analysis was also performed on each case, giving the lowest first eigenvalue of 1.72, indicating that the applied loading would have to increase by 72% before buckling occurred in critical members.

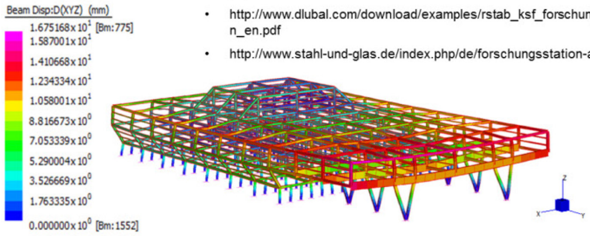
CONCLUSION

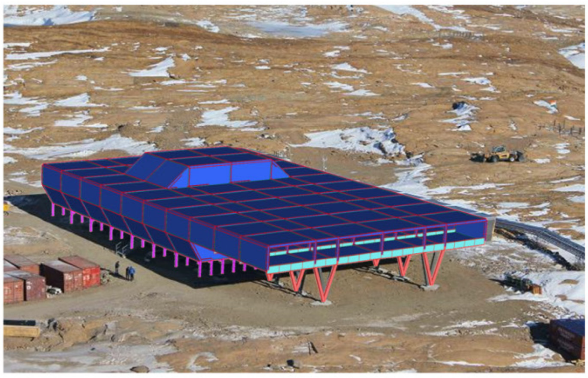
- Thermo-mechanical coupling has a significant effect on the structure
- No yielding or buckling occurs for the load combinations analysed
- Worst case axial stress is compression

REFERENCES

- www.engineeringtoolbox.com/air-properties-d_156.html
- http://www.antarctica.ac.uk/about_antarctica/geography/weather/snow.php
- http://www.dlubal.com/download/examples/rstab_ksf_forschungsstation_en.pdf
- http://www.stahl-und-glas.de/index.php/de/forschungsstation-antarktis







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

Adam Teoh, Rohit Selvaraj, Zhixin Deng, Ali Fawaz
Supervised by Faham Tahmasebinia, Peter Ansourian, and Fernando Alonso-Marroquín
School of Civil Engineering

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.

Figure 1: Image of Actual Structure

Method

Creation of the model

- Create each wall face of the main floor, roof and bottom floor using Hexa8 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

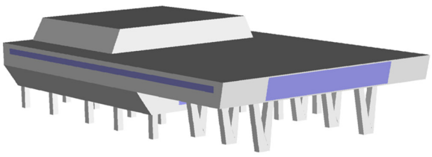


Figure 2: Image of Model

Strand 7 Inputs

Properties inputs

- Brick Hexa 8 Elements
- Thermal conductivity
- Triple insulated glass λ=0.8W/(m^{2}K)
- Composite aluminum insulting panels λ=25 W/(m^{2}K)

Results

1. 3D analysis of Thermal Loads

- Transient Thermal Analysis of structure.
- Result at a particular time step shown in Figure 4.

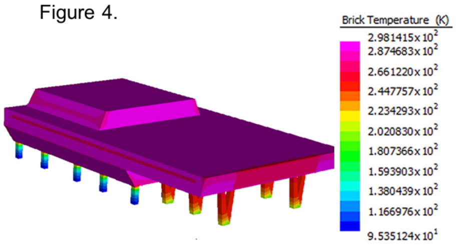


Figure 4: Typical temperature contour

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.

Material	Convection Coefficient W/(m^{2}K)	Thermal Conductivity W/(m*K)
Aluminium	25	151
Steel	25	51
Concrete	23.5	1.31

Materials	Total Heat Loss over 24 hours (J)
Set 1: Aluminium	4.53E+11
Set 2: Concrete	9.43E+10
Set 3: Steel	4.06E+11

Table 1: Energy Loss for different materials

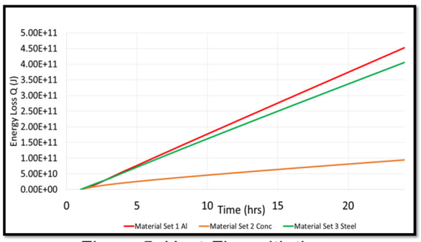


Figure 5: Heat Flux with time

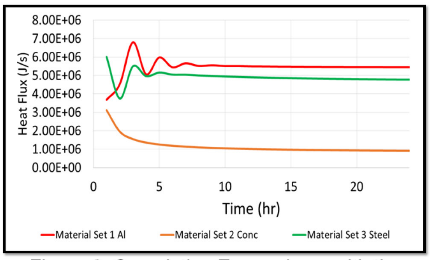


Figure 6: Cumulative Energy Loss with time

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 Heat Solver

$$\rho C \frac{dT}{dt} = k\nabla^2 T$$

- Concrete has better insulation properties than the alternatives
- Cost

Material	Cost ($ per m3)
Aluminium	11890
Steel	8530
Concrete	180

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

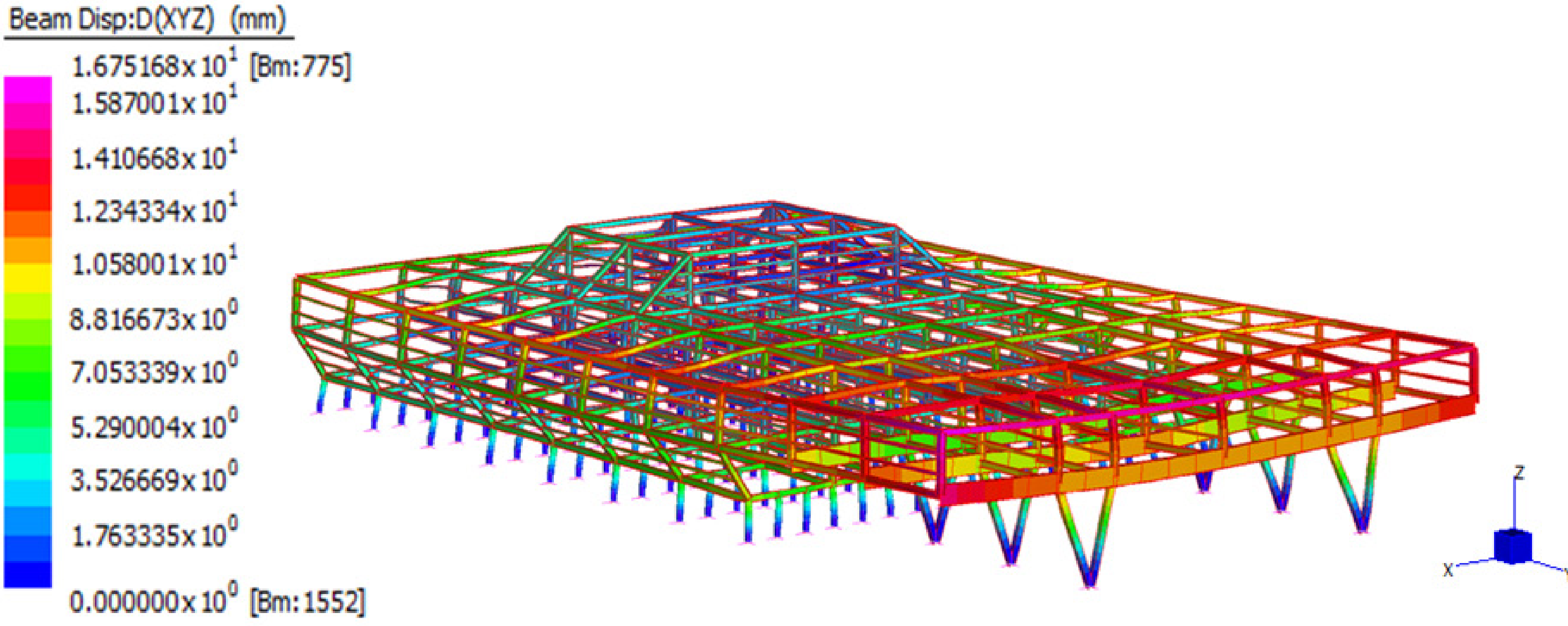
Conclusion

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

References

- Passive Buildings Techniques - Princess Elisabeth Antarctica Research Station. 2015. *Passive Buildings Techniques - Princess Elisabeth Antarctica Research Station*. [ONLINE] Available at: http://www.antarcticstation.org/station/passive_building.
- 5 Most Advanced Man-Made Structures In Antarctica. 2015. *5 Most Advanced Man-Made Structures In Antarctica*. [ONLINE] Available at: http://www.hexapolis.com/2014/11/28/gander-5-advanced-man-made-structures-antarctica-realm-science-fiction/.
- Overall Heat Transfer Coefficient. 2015. *Overall Heat Transfer Coefficient*. [ONLINE] Available at: http://www.engineeringtoolbox.com/overall-heat-transfer-coefficient-d_434.html.
- Passive Buildings Techniques - Princess Elisabeth Antarctica Research Station. 2015. *Passive Buildings Techniques - Princess Elisabeth Antarctica Research Station*. [ONLINE] Available at: http://www.antarcticstation.org/station/passive_building
- Convection coefficient for wooden barrels - Heat Transfer & Thermodynamics engineering - Eng-Tips. 2015. *Convection coefficient for wooden barrels - Heat Transfer & Thermodynamics engineering - Eng-Tips*. [ONLINE] Available at: http://www.engtips.com/viewthread.cfm?qid=125551.

Full technical reports and Strand7 files in USB drive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