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Structural Concepts

DETAILED ENGINEERING EVALUATION QUANTITATIVE REPORT

Cunningham House - Glasshouse

Botanical Gardens, Christchurch

Prepared For:

Christchurch City Council

Prepared By:

Garry Newton

BE(Civil), MIPENZ, CPEng, IntPE(NZ)

Ref: 1923-2245
2 April 2014 R2

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
PO Box 3315, Napier, 4142, New Zealand
P (06) 842 0111 F (06) 842 0113
E info@structuralconcepts.co.nz
www.structuralconcepts.co.nz

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2 April 2014

FOR:

CHRISTCHURCH CITY COUNCIL

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Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
PO Box 3315, Napier, 4142, New Zealand
P (06) 842 0111 F (06) 842 0113
E info@structuralconcepts.co.nz
www.structuralconcepts.co.nz

1.0 Preamble

We were requested by Insight Unlimited on behalf of the Christchurch City Council to investigate and report on the structural condition of the Cunningham Glasshouse building at the Botanical Gardens, Christchurch, following the September 2010 earthquake, plus magnitude 6.3 earthquakes on 22nd February 2011 and that of the 13th June 2011. Our assessment is based on a visual inspection of the outside and inside where it was deemed safe to enter. This was carried out on several occasions between July 2013 and February 2014 (as diarised)

This report describes the damage observed, and comments on remedial work options for both temporary securing of the building (if necessary), and long term repair where appropriate.

A Detailed Engineering Evaluation (DEE) process has recently been developed by CERA to provide consistent, comprehensive and auditable guidelines which help restore confidence in the remaining building stock in Canterbury. We have used these guidelines to form the basis for our Detailed Engineering Evaluation (DEE).

The Detailed Engineering Evaluation (DEE) follows a two step process, firstly a qualitative assessment and then a quantitative assessment, if necessary.

The qualitative assessment involves visual review of the structure and its conditions, in order to ascertain whether the structure does or does not fall within the required capacity limitations, without completing any complex analysis.

The quantitative assessment involves analytically calculating the capacity of the structure in terms of the Current Code requirements, i.e. to estimate the percentage of the New Building Strength available (%NBS).

The overall objective of this assessment is to determine if a strengthening solution is required or not.

More specifically, this report covers the following:

- Describes the existing building, its construction, and structural system.
- Outlines the level of investigation undertaken and where the information was obtained from.

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
PO Box 3315, Napier, 4142, New Zealand
P (06) 842 0111 F (06) 842 0113
Einfo@structuralconcepts.co.nz
www.structuralconcepts.co.nz

- Summarises earthquake damage caused by the recent Canterbury earthquakes.
- Reviews the building's performance in the recent Canterbury earthquakes.
- Identifies critical structural weaknesses.
- Assesses the building's seismic strength relative to the New Building Standard (NBS), commonly referred to as the "Current Code".
- Proposes earthquake strengthening work to bring the building as close as practically possible to 67% or 100% of the Current Code if found necessary.

On this occasion a quantitative report has been completed. It is notable that Opus International Consultants Ltd has completed a qualitative and quantative assessment prior to this report. This report expands on these by including additional intrusive investigations plus thorough assessment of the steel structure to draw more precise conclusions.

2.0 Scope of Investigation

On 7th May 2013 and again on 4th September 2013, we visually inspected the building including:

- The exterior from ground level.
- The interior.

Drawings were obtained which were of a limited nature.

No geotechnical report was available.

Intrusive investigations have been completed by Opus and Structural Concepts Ltd to determine reinforcing quantities and sizes in the reinforced concrete members.

The Opus investigations were extensive and included intrusive investigations of reinforcing in walls and slabs plus scanning of reinforced concrete members for reinforcing location and spacing. These included several investigations of the upper slab and slab-wall junctions, exterior wall-column junction and foundation depth to exterior buttresses. We were able to review these investigations on our visit on 7th May 2013.

Structural Concepts Ltd completed investigations on 4th September 2013 to determine reinforcing quantities and sizes in the concrete beams and columns including additional scanning.

The results of these investigations have been used to determine strength levels of the different reinforced concrete elements and were as follows:

Upper Mezzanine Slab	½" bars at12"ctrs
Walls	½" bars
Stirrups	½" bars
Column	4 ¾"bars
Beams	2 3/4 " to bottom

This information generally matches that indicated on the original cross section plans.

This report is based on our assessment of the building at the time stated. The assessment is for the seismic capacity only and does not include wind or snow or live load checks. Photos attached in **Appendix C** are indicative of the damage and findings. Any subsequent loading by aftershocks or high winds, may initiate further damage. No aftershocks of any significance have occurred since the time of the last inspection.

3.0 Building Description

General description:
Cunningham House was constructed in 1923 to serve as a tropical glasshouse for the Hagley Park Botanical Gardens. The glasshouse is listed as Category II by The Historic Places Trust (HPT) and Group 2 under the (Christchurch City Council) list for Protected Buildings.

Cunningham house is located within the Christchurch Botanical Gardens. The super-structure consists of two distinct parts; the lower reinforced concrete arched frames and the steel roof structure above. For the purposes of this report they will be referred to as the sub-structure and then roof structure respectively.

The sub structure comprises reinforced concrete frame supported on concrete pad footings. The roof is constructed from steel lattice trusses with glass supported off steel purlins which span between the roof trusses. There is a mid-height mezzanine floor which is 1.5m wide and runs around the perimeter of the glasshouse which is supported by the concrete frames under. The concrete frames are reinforced concrete with buttresses to the exterior.

The building is currently not occupied, and only accessible for assessment, investigation and maintenance. The main occupancy classification in NZS1170 is and Importance Level 2. Importance level 2 is chosen as the building is considered as a normal building that is unlikely to have more than 300 people in any one space.

The building areas are approximately as follows:

1. Ground floor 460m²

2. Mezzanine 210m²

Roof construction:
The roof consists of glass panes supported on steel purlins which are in turn supported by an elaborate steel truss structure. The trusses are supported of the reinforced concrete substructure.

Exterior walls:
The exterior walls consist of glass windows and insitu concrete panels between reinforced concrete columns and beams. The concrete walls consist of ½" bars at 12" centres.

Mezzanine floor:
The mezzanine consists of a two way reinforced concrete slab supported by reinforced concrete beams and columns acting as a two way frame system. The floor slab reinforcing consists of ½" bars at 12"-14" centres in both directions. The beams and columns have a minimum of 4 ¾" bars with one in each corner.

Ground floor slab:
The ground floor consists of a reinforced concrete slab over part of the area as paths with soil between.

Foundations:
Foundations consist of reinforced concrete pads beneath all columns with strip footings under walls.

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
PO Box 3315, Napier, 4142, New Zealand
P (06) 842 0111 F (06) 842 0113
Einfo@structuralconcepts.co.nz
www.structuralconcepts.co.nz

4.0 Structural System

Gravity Structural System:
The gravity structural system can be described as a simple roof beam to trusses transferring loads to a reinforced concrete structure supporting a concrete floor supported in turn by reinforced concrete columns then to foundations and into the ground.

Lateral System:

The lateral system in both directions can be described as a glass support system transferring lateral loads to tapered curved steel trusses acting as portal frame/space frame elements that in turn transfers loads to the top of a reinforced concrete slab and columns where lateral loads are distributed to a two way reinforced concrete beam and column portal structure.

5.0 Damage Description

5.1 Seismic Damage:

Damage caused by the February and June earthquakes to the Cunningham House Glasshouse is described below. Damage described is that observed on the day. Refer to **Appendix B** for marked-up drawings indicating damaged locations.

Interior:

General

i. Several minor non structural vertical cracks to lower level concrete walls.

ii. South Balcony
Non structural minor cracks to the base of the balusters.

iii. South Wall Upper Level Door Surround
Minor non structural cracks to door head at side of door.

6.0 Strength

The strength of the building has been determined as 59%NBS using methodologies provided by NZSEE. Refer **Appendix B** (reference plans) for the direction along and across. The following philosophies have been used:

6.1 Structural Size and Strength Assumptions

- Rivet size 12mm

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
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P (06) 842 0111 F (06) 842 0113
E info@structuralconcepts.co.nz
www.structuralconcepts.co.nz

- Column reinforcing ¾ inch bar (4 off minimum, 2 each face)
- Beam reinforcing ⅝ inch bar (4 off minimum, 2 each face)
- Reinforcing strength 300 MPa
- Concrete Strength 25 MPa
- Structural Steel Strength 293 MPa
- Steel members and connections are not corroded and able to achieve full capacity.

6.2 Ductility Factors Assumed

Roof Structure	μ max = 1.25
Concrete Structure	μ max = 1.25

6.3 Seismic Analysis Type

A three dimensional dynamic analysis has been utilised to assess the forces within the elements. The model approximately represents the existing building but is sufficiently accurate to be confident with the outputted results.

This method was chosen as opposed to a static analysis because of the three dimensional complexities of the roof structure plus the upper level deck around the internal perimeter being partial at this level and not full.

It is believed that higher mode affects may be governing the structure dynamic effects as opposed to single first mode static analysis.

6.4 Truss Member Stability

Assumptions around member stability is debatable with regard to the inner chord of the steel truss systems.

The following simple assumptions can be made:

(i) No rotational and only lateral restraint at each end ke = 1.0

(ii) Rotational/lateral restraint to one with lateral restraint but no rotational restraint to the other ke = 0.85

(iii) Rotational and lateral restraint to both ends ke = 0.7

The following observations are made of the structure and it's connections and the analysis results:

(a) The base of the T section of the inner chord is well fixed to the concrete structure with three bolts and thick cleats.

(b) The truss is tapered and of a curved nature.

(c) The diagonal ties and perpendicular struts are double members that mesh to each side of the T section web member that will provide twist restraint at each node.

(d) A pair of centre trusses located between the main truss structure provides lateral restraint to the inner chord near the top of the main truss either side of the apex.

(e) The curved nature of the inner chord along with the restraints near the apex and at the base plus the struts and ties are having an affect on the lateral restraint at this member.

6.4.1 Discussion

There is significant debate as to what the truss internal chord effective length should be. The following discussion is to elaborate on the facts and then to attempt to draw a logical conclusion on this.

The building is currently standing plus it withstood the seismic events of 2010 and 2011. Therefore the strength of the T section internal chord must reflect at least that of the ultimate limit state of the dead load only, otherwise the building collapses or at a minimum would show significant signs of distress. For the T section the following maximum ultimate critical loads can be obtained before buckling occurs at the effective lengths chosen in the simple approaches proposed to date:

(i)	ke = 1.0	Le = 8.4	Nc = 10.01 kN
(ii)	ke = 0.85	Le = 7.88	Nc = 14.02 kN
(iii)	ke = 0.7	Le = 5.88	Nc = 19.23 kN

Note these are ultimate allowable critical loads with a reduction factor of Ø = 1.0. Therefore the test is at a level of 1.35G although a check against 1.0G is also valid

For: 1.35G N* = 31 kN maximum

1.0G N = 23 kN maximum

Or alternatively and more likely the test should be against the average of the forces over the length of the element. The force being an average of the individual tributary lengths and as a second check the average of the number of elements.

This would mean the following:

		Ave - Max
Using: ke = 1.0	chord is overstressed at serviceability state by	184% - 230%
ke = 0.85	chord is overstressed at serviceability state by	131% - 164%
ke = 0.7	chord is overstressed at serviceability state by	96% - 120%

But at ultimate limit state (1.35G)

			Ave - Max
Using	ke = 1.0	overstressed	248% - 309%
	ke = 0.85	overstressed	176% - 219%
	ke = 0.7	overstressed	130% - 162%

When we consider these ratios it is not logical to accept the higher ke values as this would definitely mean the building should have collapsed as the chords are significantly overstressed at their ultimate allowable load compared to the serviceability state (unfactored load).

Furthermore considering that the seismic load is at maximum, 53.5kN it seems inconceivable, regardless of the direction of the EQ, at ultimate limit state or even 70% of ultimate seismic load at 43 kN that the building could have survived the events of February 2011 without significant signs of distress or even collapse, knowing that the forces in this vicinity were close to the old code load.

If we were to entertain the fact that ke = 1.0 or 0.85, then with even a seismic load at 25% of current code level percentages overstressing would be as follows:

ke	100% of Z = 0.3	100% of Z = 0.22	Across SLI Z = 0.3	Along SLI Z = 0.3
Axial force	53.5	43.2	29.6	25.9

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
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1.0	534%	432%	296%	259%
.85	381%	308%	211%	184%
0.7	278%	225%	154%	135%

At ultimate limit state

Average (sum of lengths)	24.83 kN
Average (number)	22.87 kN

At normal state (serviceability)

Average (sum of lengths)	18.43 kN
Average (number)	16.99 kN

Therefore, it is our opinion that at a maximum Le = 5.88m is most appropriate and it is also very likely that the sum of lengths average is more like the force in the member. Conservatively only the compression members are included in the average force.

Considering the lateral buckling force is only 0.775 kN (0.025 N*) for the maximum gravity condition and if we assume that we have some fixity with the riveted double angles then the effective length reduces significantly. If we ignore the 2.2m long double angles but assume the 1.4m long double angles are effective then the lateral deflection of the chord is only 11mm (allowing for some rotation of the outer chord) if full fixity is assumed with a maximum moment of 1.08kNm. The detail at the outer chord T section is quite capable of carrying this load and furthermore the outer chord is rotationally restrained by the steel purlins.

If we were then to take the effective length as 0.85L (fixed at floor and laterally restrained only at the 1.4m strut) then

Le = 0.85 x 5.32 = 4.52 and
Ø Nc = 32.2 kN

This is more probable, logical and consistent with the buildings current gravity state. The following table shows the % overstressing for each of the seismic loads discussed earlier.

	100% of Z = 0.3	100% of Z = 0.22	Across SLI Z = 0.3	Along SLI Z = 0.3
Le = 4.52m	166%	124%	92%	80%

Therefore it is our opinion that the effective length (Le) is equal to 4.52m. This will be the length to be used in the calculation check.

The check will be against the maximum load in the chord as opposed to the average.

6.5 Strength Assessment

Before September 2010:
The strength of the building before September 2010 is estimated as:

Pre Earthquake:		Critical Element
Across		
Hazard factor 0.22 (pre 19th May 2011)	88%NBS	(Internal Chord at central truss)
	95%NBS	(Concrete Columns)
Along		
Hazard factor 0.22 (pre 19th May 2011)	80%NBS	(Internal chord of middle end trusses each end)
	95%NBS	(Concrete Columns)
Post Earthquake:		
Across		
Hazard factor 0.3 (post 19th May 2011)	65%NBS	(Internal Chord at central truss)
	70% NBS	(Concrete Columns)
Along		
Hazard factor 0.3 (post 19th May 2011)	59%NBS	(Internal chord of middle end trusses each end)
	70%NBS	(Concrete Columns)

Building Current Strength
The current strength of the building as a single %NBS is as follows:

The Building as a Whole **59%NBS**

Discussion
It is noted that the assessed strengths above differ from the Qualitative report completed by Opus. The following indicates the main differences:

- *Truss Chord Strength* – Opus effective length of inner chord taken as 8.4m. Discussion above in 6.4 suggests this is not logical and further investigation of chord stability would suggest a significantly short effective length (hence higher assessed strength).

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
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P (06) 842 0111 F (06) 842 0113
Einfo@structuralconcepts.co.nz
www.structuralconcepts.co.nz

- *Concrete Arch Frames* – Opus assessed 1/2 inch reinforcing. New investigation shows to be ¾ inch (hence higher assessed strength).

- *Cantilever Columns* - Opus assessed 1/2 inch reinforcing. New investigation shows to be ¾ inch (hence higher assessed strength).

7.0 Areas of Structural Vulnerability

There are no areas of structural vulnerability.

Areas of non-structural vulnerability include the glazing which covers the whole of the building.

8.0 Long Term Repair

i. General repairs to concrete:
Seal all cracks larger than 0.2mm with a pressure injected epoxy (e.g Sikadur injectokit and Sikadur52), or similar. Seal smaller cracks by painting over with a brushable crack filler (e.g Resene brushable crack filler).

ii. Foundations general cracking:
Seal all cracks in the concrete foundation wall larger than 0.2mm with a pressure injected epoxy (e.g Sikadur injectokit and Sikadur52), or similar. Seal smaller cracks by painting over with a brushable crack filler (e.g Resene brushable crack filler).

9.0 Elements Not Inspected

The following is a list of elements not specifically inspected:

- Below ground foundations
- Soils
- High level steel
- Seating for glazing

10.0 Applicability

Recommendations and opinions in this report are based on data and records obtained from Christchurch City Council, plus the non-destructive and destructive visual inspections. Although there is nothing to suggest otherwise, the nature and continuity of the structure hidden from sight (e.g. reinforcing steel, bolt depths etc.) is inferred but it must be appreciated that actual conditions could vary.

Findings presented in this report are for the sole use of the client. The findings may not contain sufficient information for use by other parties, and as such should not be relied upon unless discussed with Structural Concepts Ltd. We have exercised our services in a professional manner using a degree of care and skill normally, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Prepared By:

Garry Newton
BE (Civil), MIPENZ(Civil, Structural), CPEng, IntPE

Managing Director
On Behalf of Structural Concepts Ltd

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
PO Box 3315, Napier, 4142, New Zealand
P (06) 842 0111 F (06) 842 0113
E info@structuralconcepts.co.nz
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APPENDIX A – Original Plans

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
PO Box 3315, Napier, 4142, New Zealand
P (06) 842 0111 F (06) 842 0113
Einfo@structuralconcepts.co.nz
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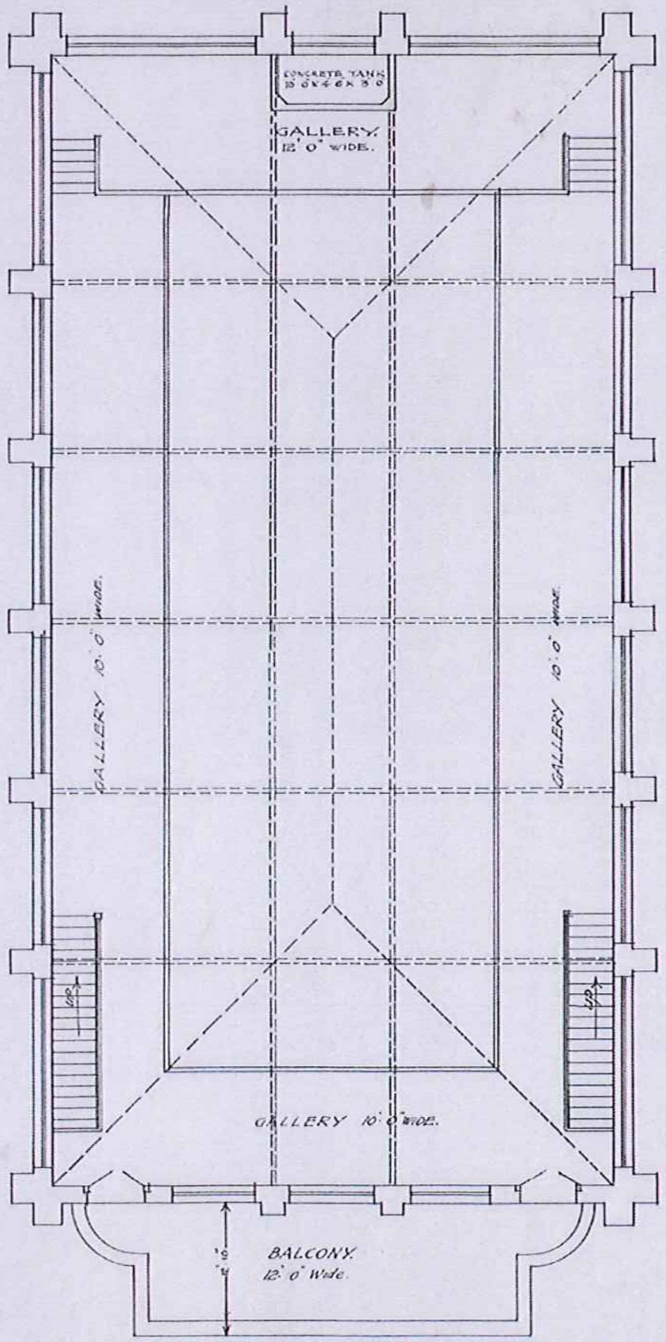
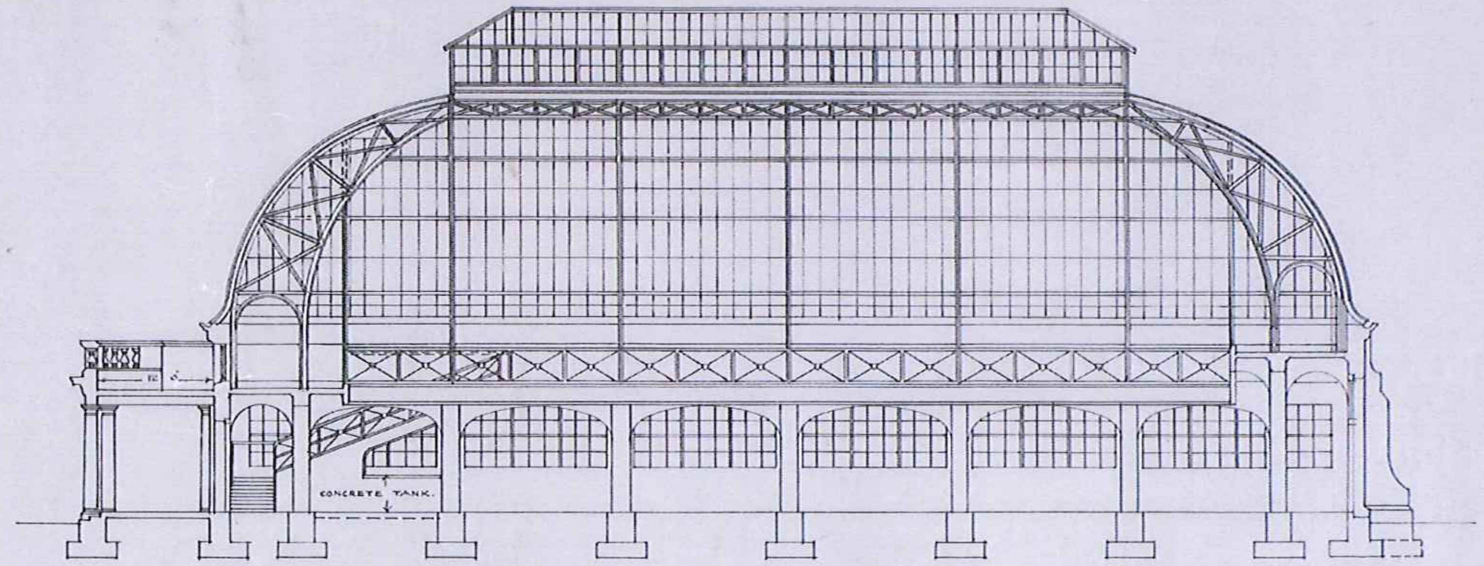
· CHRISTCHURCH DOMAINS BOARD ·
· WINTER GARDEN ·

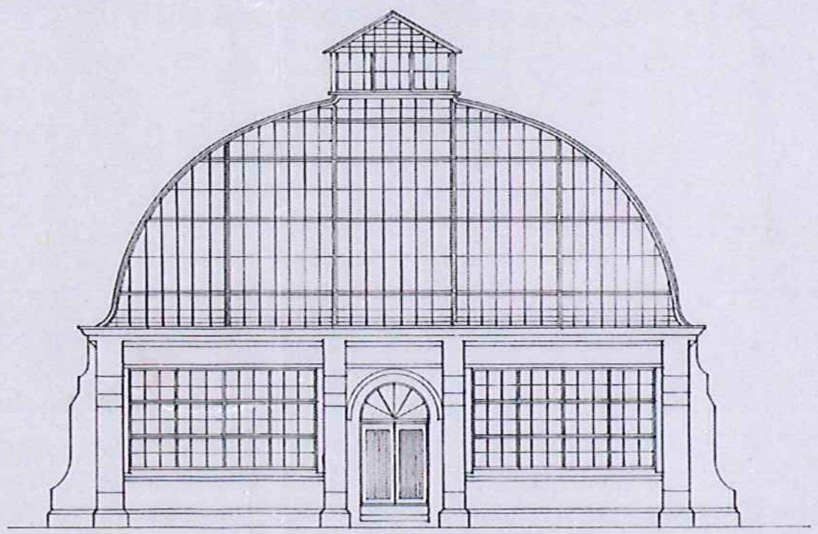
: Scale : 8 Feet to One Inch :



·· GALLERY PLAN ··



·· LONGITUDINAL SECTION ··



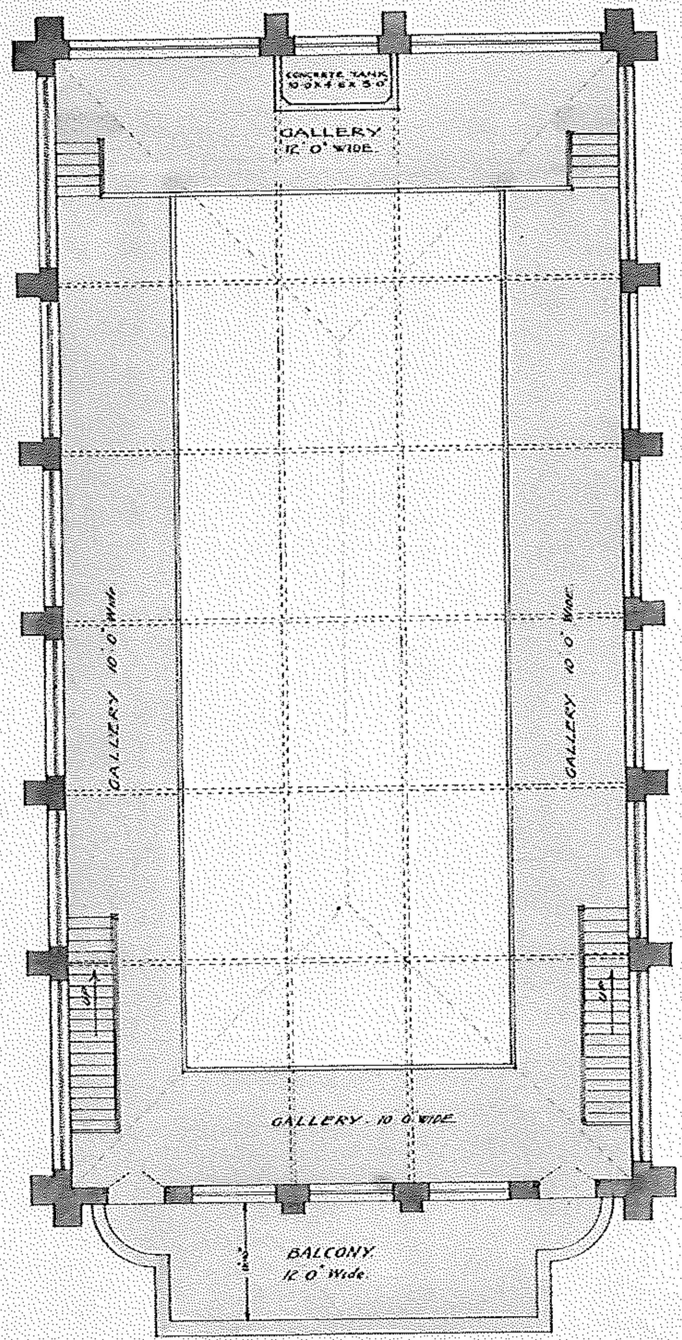
·· BACK ELEVATION

Collins & Harman
Registered Architects
81 Hereford Street.

CHRISTCHURCH DOMAINS BOARD.

WINTER GARDEN

Scale 8 Feet to One Inch



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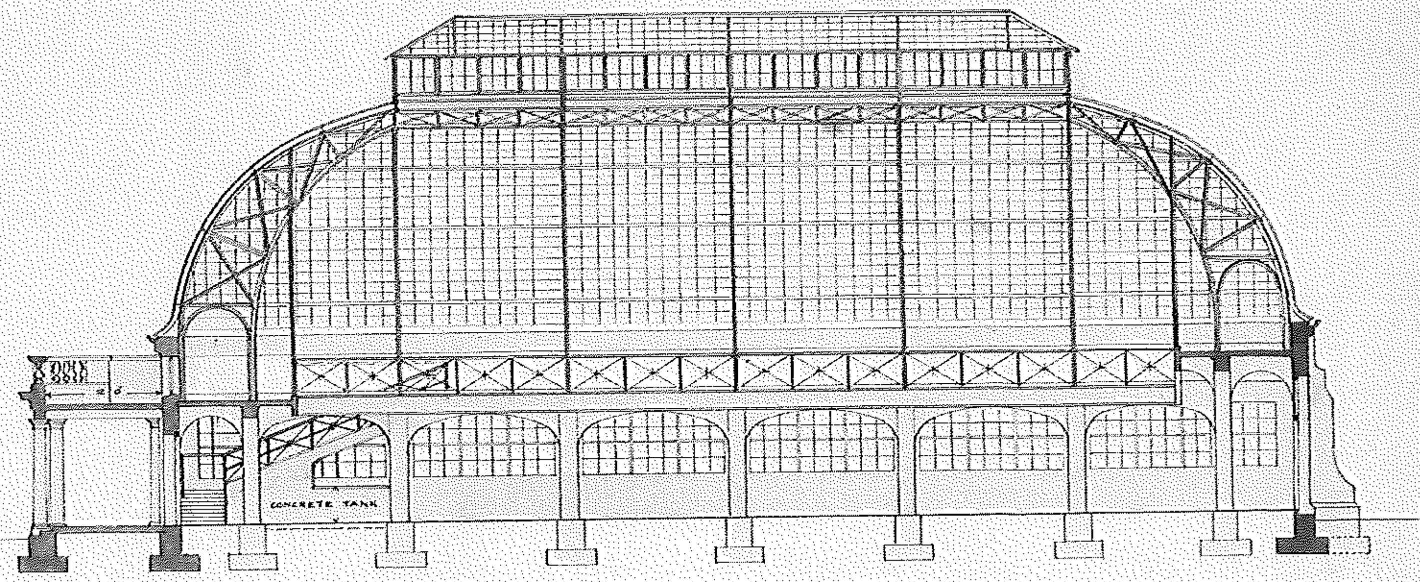

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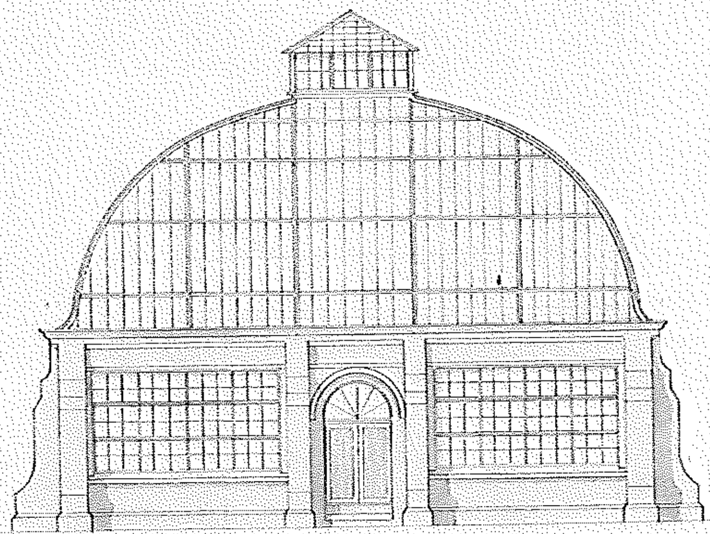
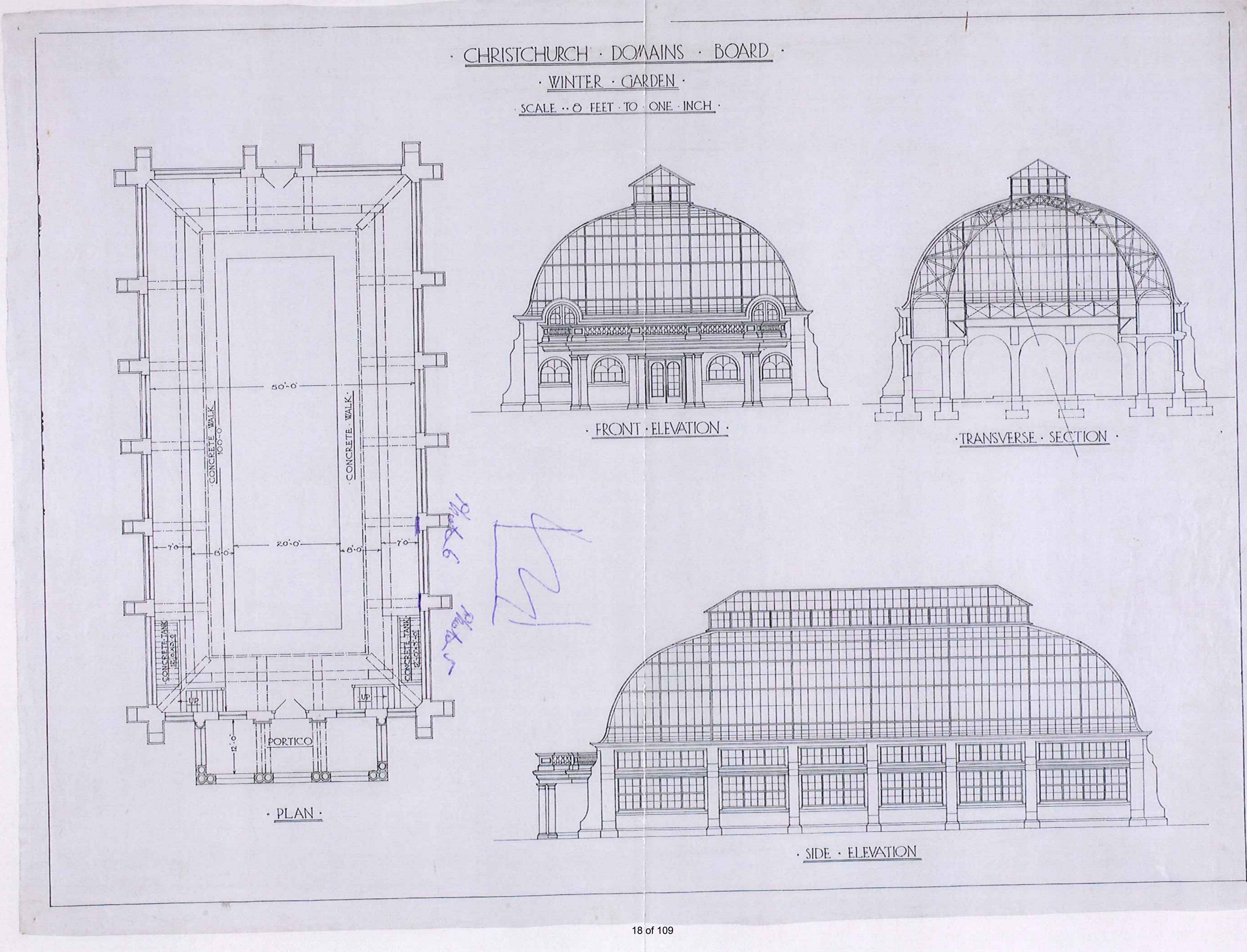


··BACK ELEVATION··

This is one of the Drawings referred to in the Contract made between the Christchurch Domains Board and Ourselves.
Dated Nov. 4 1922.

Contractor B. Munro Son
Witness J. G. Collins

Collins & Harman Architects Oct. 6. 1922.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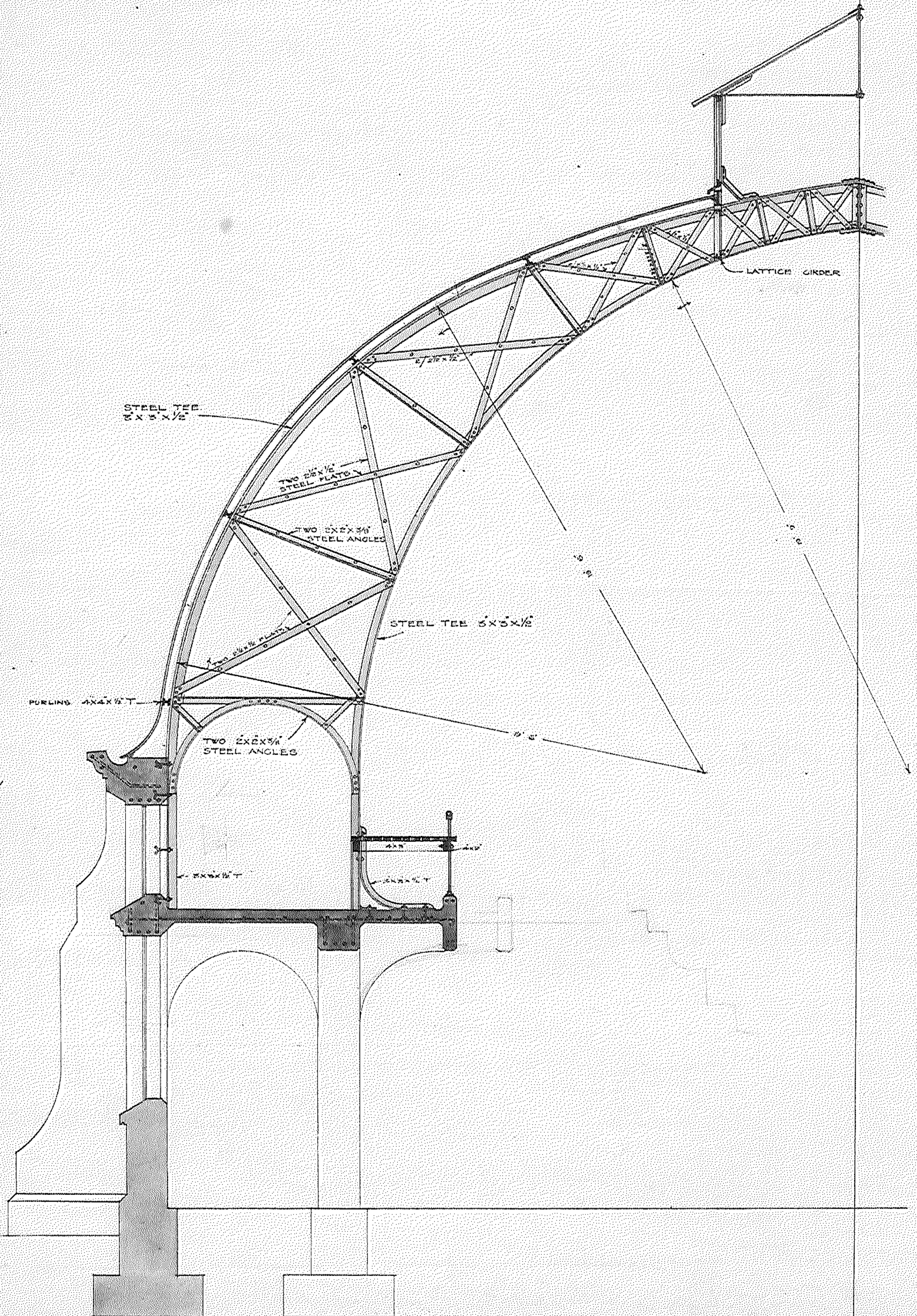
CHRISTCHURCH DOMAINS BOARD
WINTER GARDEN . DETAIL OF ROOF TRUSSES

SCALE – HALF AN INCH EQUALS ONE FOOT.



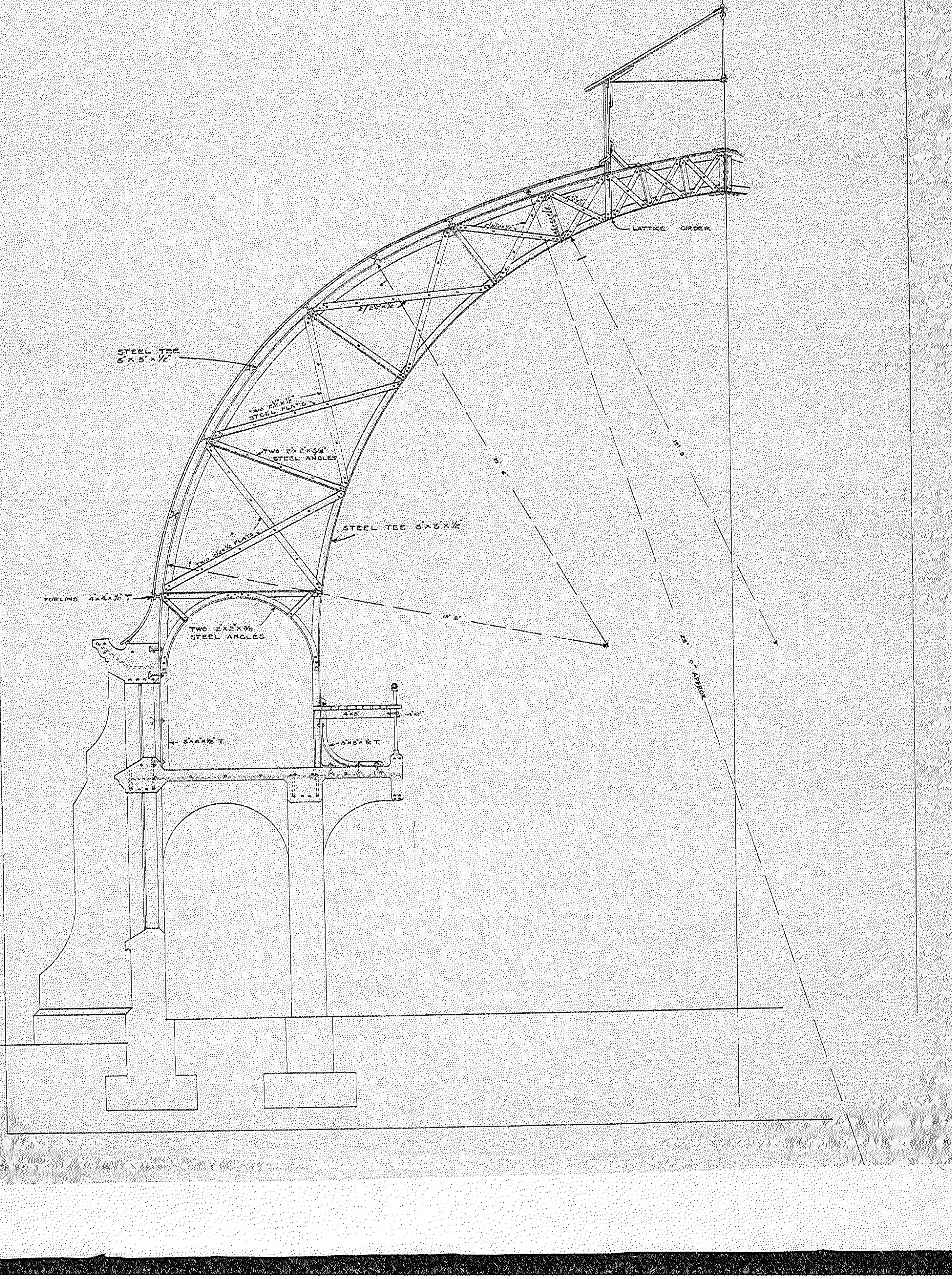
This is one of the Drawings referred to in the Contract made between the Christchurch Domains Board and [illegible] Dated Nov 4. 1922.
Contractor [illegible]
Witness J. G. Collins

Collins & Harman
Architects
Oct. 6. 1922

CHRISTCHURCH DOMAINS BOARD

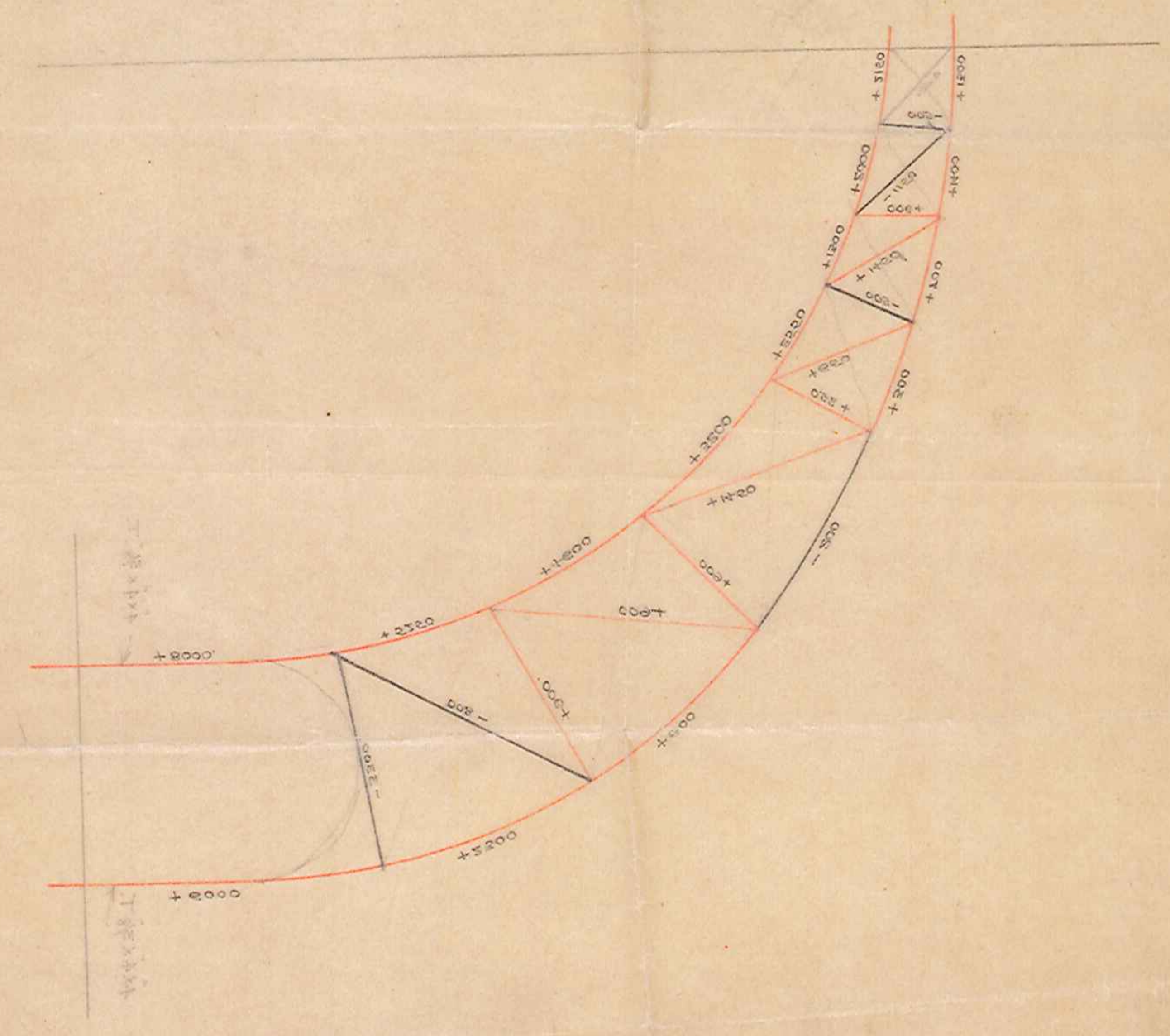
WINTER GARDEN DETAIL OF ROOF TRUSSES

SCALE ÷ HALF AN INCH EQUALS ONE FOOT.



Winter Garden

Cir. Ct. Domain



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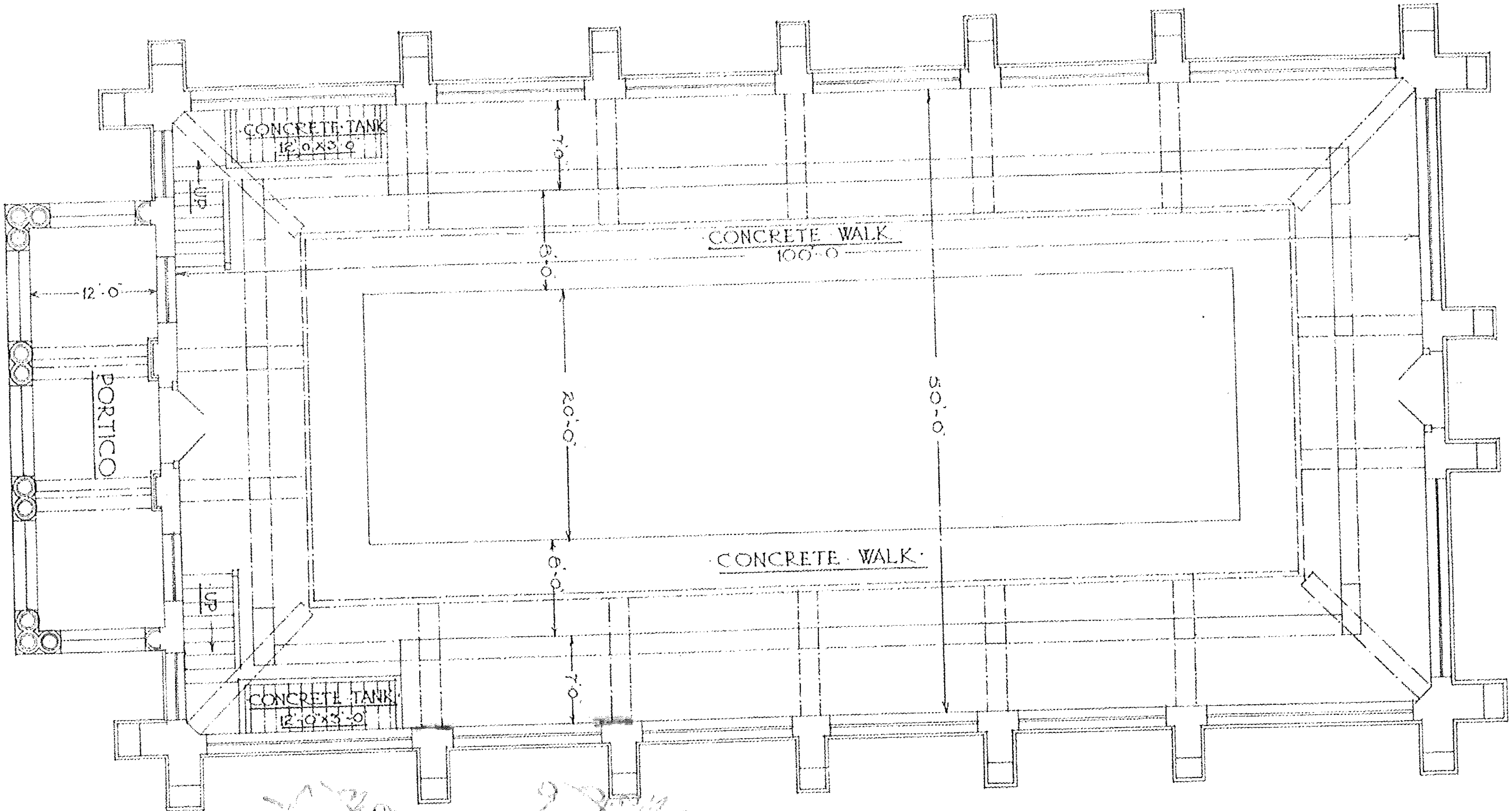

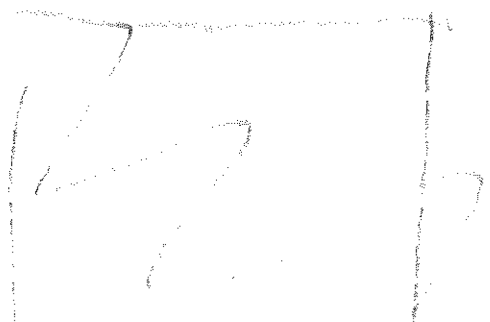

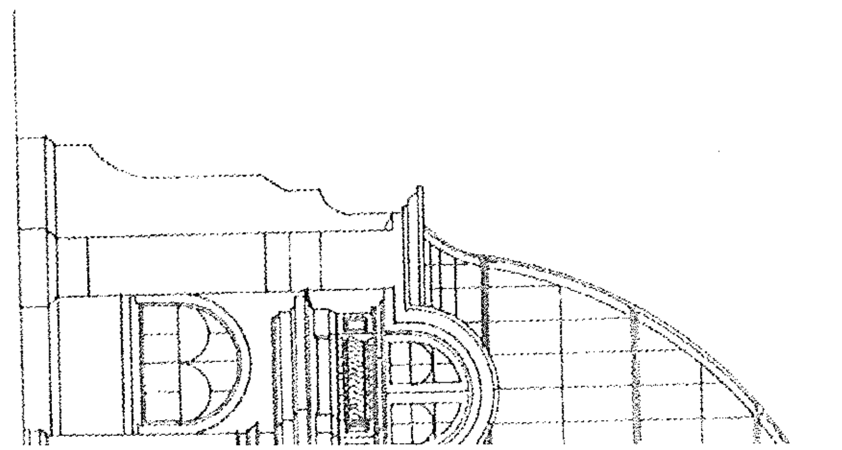

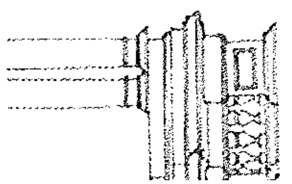

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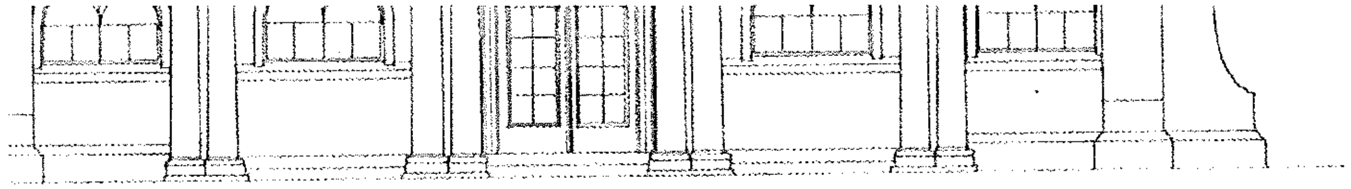

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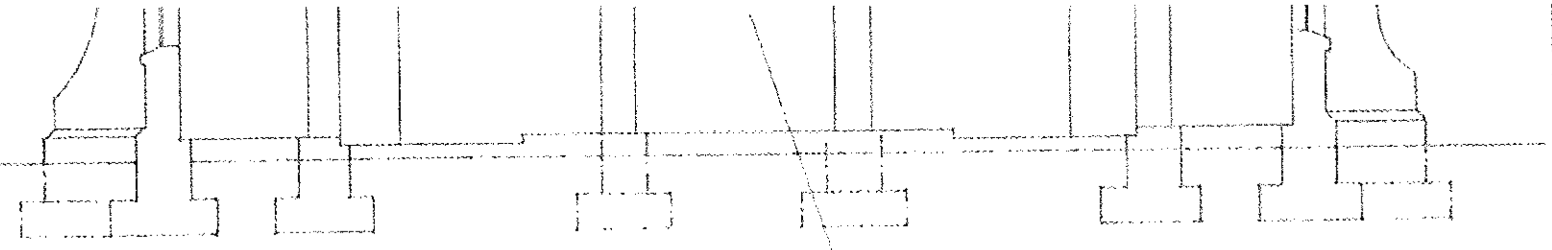

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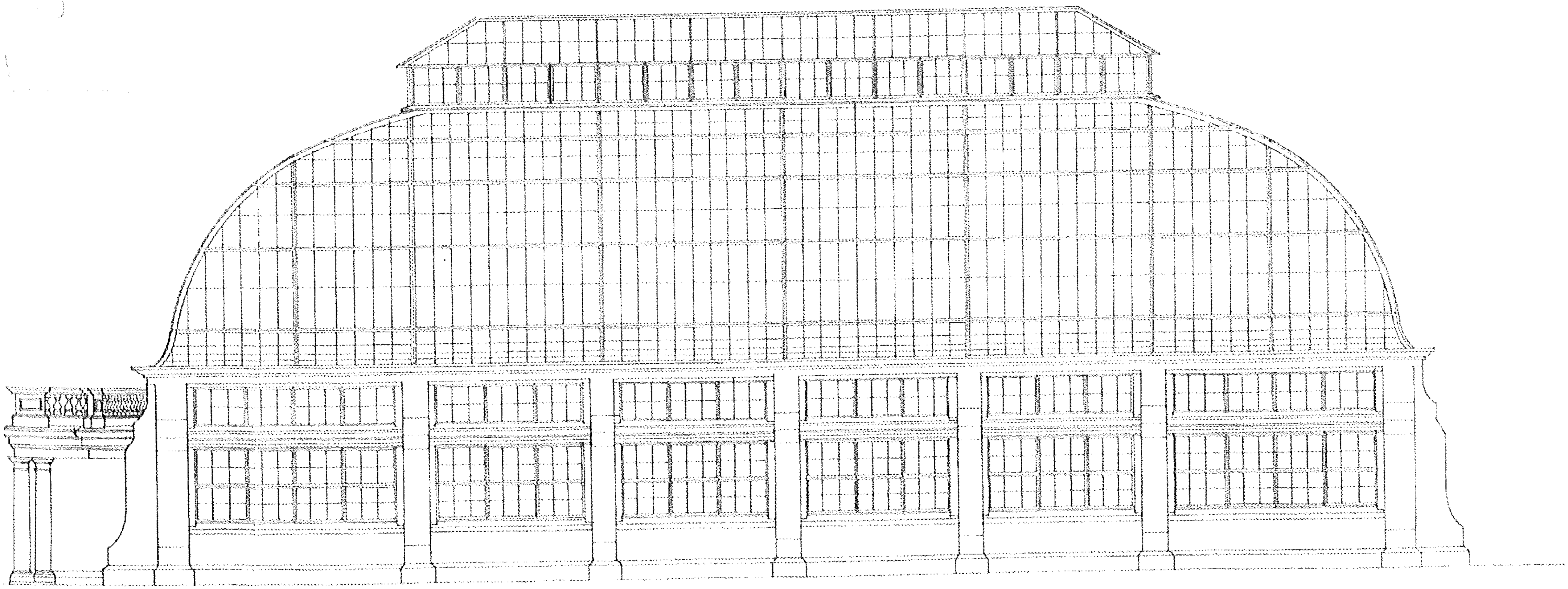
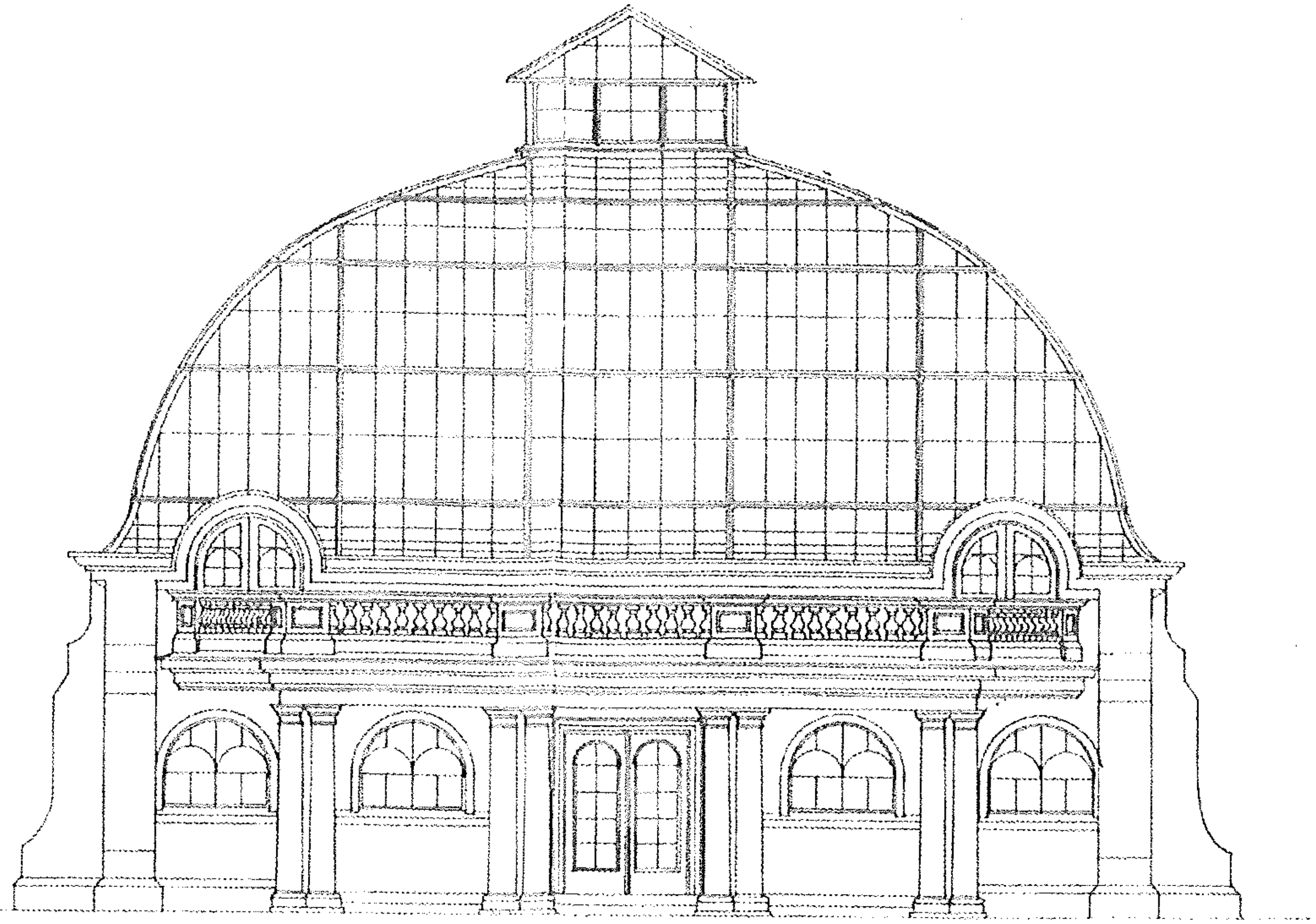


· SIDE · ELEVATION

· SCALE ·· 8 FEET · TO · ONE · INCH ·



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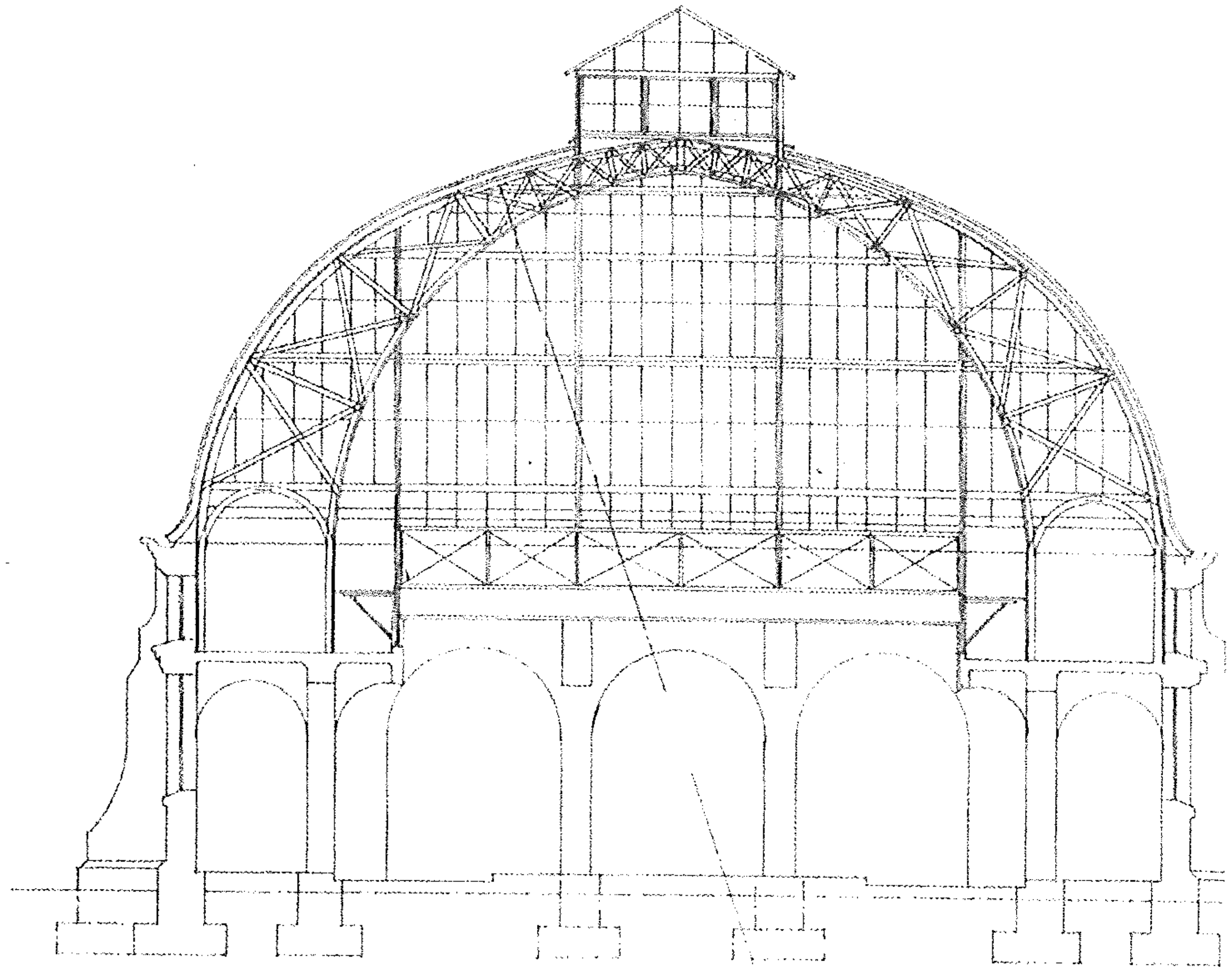
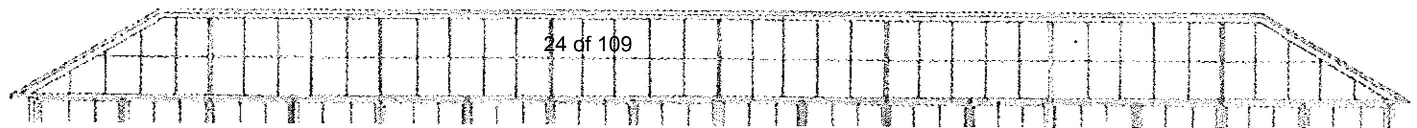


· TRANSVERSE · SECTION ·



APPENDIX B – MARKED-UP DRAWING INDICATING DAMAGE LOCATIONS

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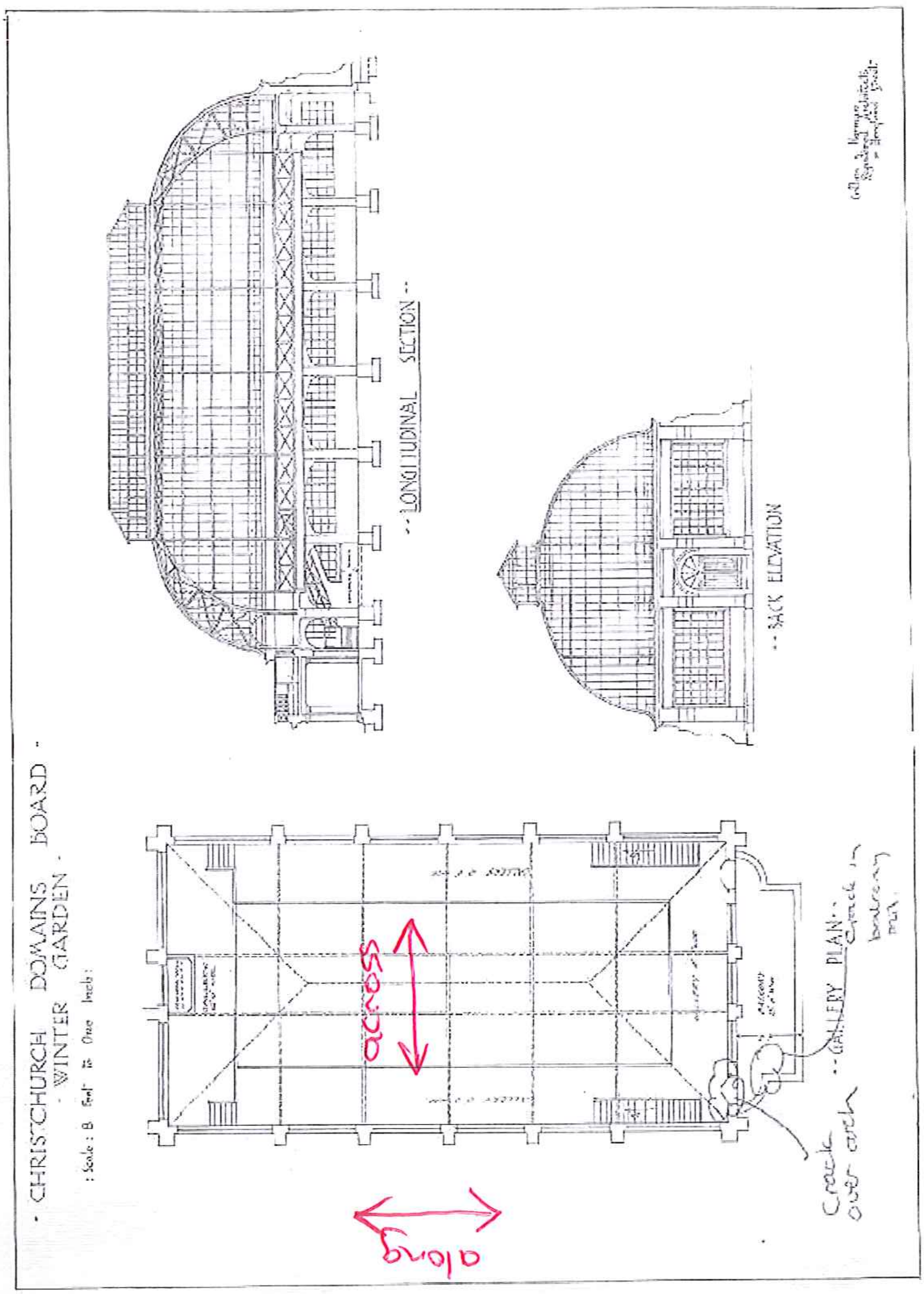
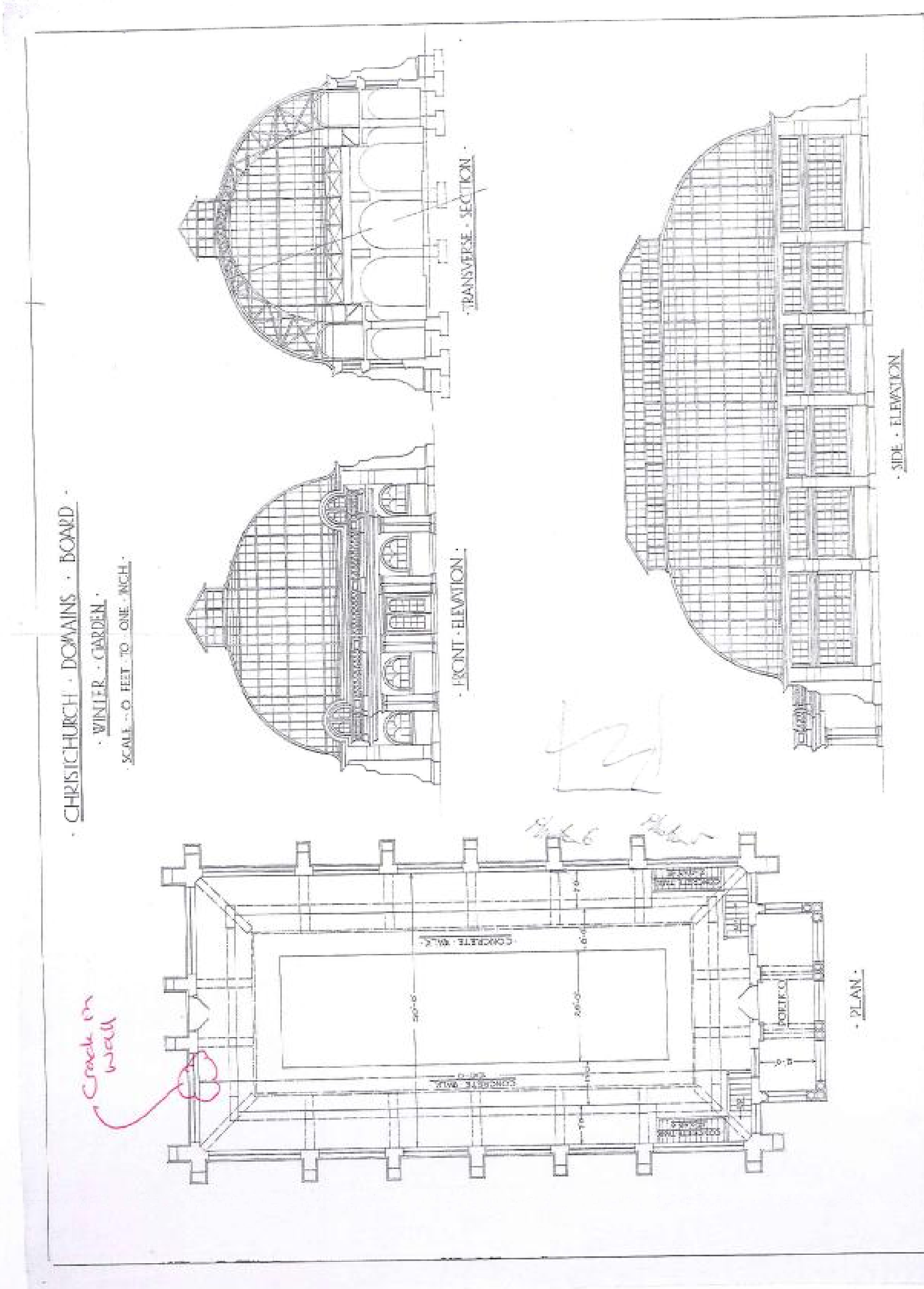

CHRISTCHURCH DOMAINS BOARD
WINTER GARDEN
Scale: 8 Feet to One Inch
LONGITUDINAL SECTION
BACK ELEVATION
GALLERY PLAN
across
along
Crack over arch


· CHRISTCHURCH · DOMAINS · BOARD ·
· WINTER · GARDEN ·
· TRANSVERSE · SECTION ·
· FRONT · ELEVATION ·
· SIDE · ELEVATION ·
· PLAN ·
CONCRETE WALK
PORTICO
Crack in wall

CRACK TO DOOR HEAD


CRACK TO WALL

BALCONY CRACKING

APPENDIX C – PHOTOGRAPHS

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P (06) 842 0111 F (06) 842 0113
Einfo@structuralconcepts.co.nz
www.structuralconcepts.co.nz

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PO Box 3315, Napier, 4142, New Zealand
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Einfo@structuralconcepts.co.nz
www.structuralconcepts.co.nz

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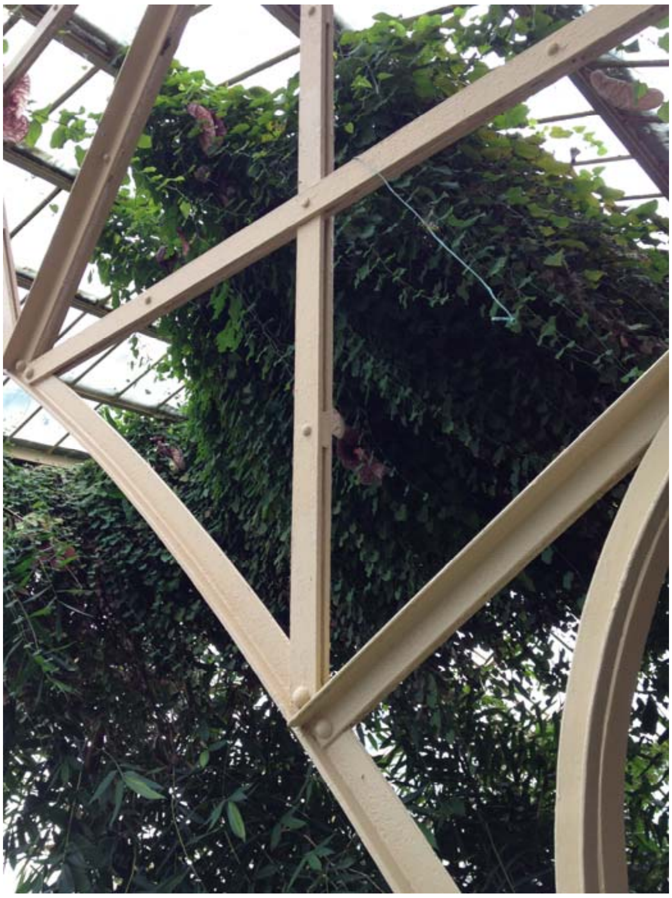

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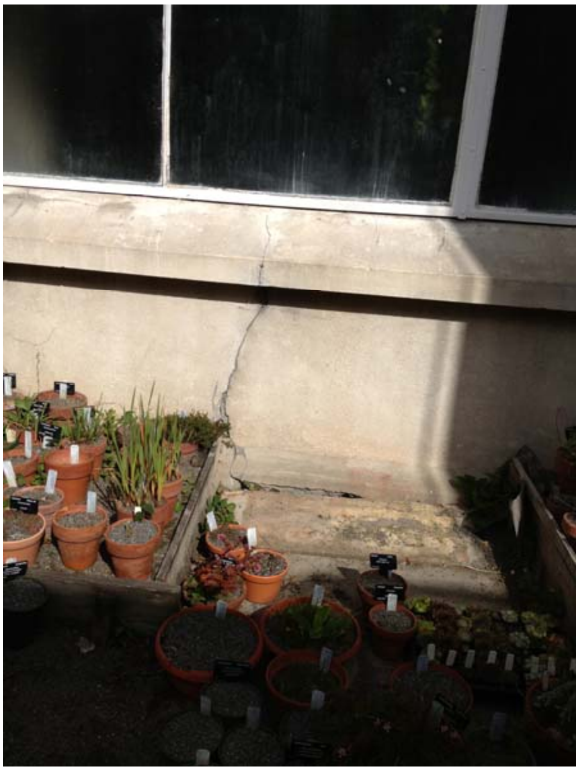

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Einfo@structuralconcepts.co.nz
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APPENDIX D – PRELIMINARY CALCULATIONS

Level 3, Dunvegan House, 215 Hastings Street, Napier 4110
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P (06) 842 0111 F (06) 842 0113
Einfo@structuralconcepts.co.nz
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emajineer

Structural Concepts

Client: Christchurch City Council

Project: Cunningham Glasshouse
Botanical Garden, Christchurch

1

Ref: 1923-2245

Date: 10-Feb-14

CALCULATIONS

BY GARRY NEWTON

BE (Civil) , MIPENZ, CPEng, IntPE(NZ), APEC Engineer

CONTENTS

Gravity Loads
Lump Sum Mass
EQ Static 1170.5 check
Dynamic Check Mass Case 2
Dynamic Check Mass Case 7
Buckling Check
Int Chord Effective Length
Lateral Restraint Assessment
Chord Assessment ke=1.0
Chord Assessment ke=0.85
Chord Assessment ke=0.7
Chord Assessment Le=4.52 (1.35G
Chord Assessment Le=4.52 (Edx)
Chord Assessment Le=4.52 (Edy)
Chord Load 1.35G & 1.0G
Trans Truss Chord Loading G+Edx
Long Truss Chord Loading G+Edy
Check Double Steel Flats
Check Double Steel Angles
Check Rivets
Table 6.3.3(2)
Table 6.3.3(2) cont.
Concrete Column Check Along
Concrete Column Check Across
Concrete Beam Check
Check Slab Diaphragm Forces

55 Dunlop Road, Onekawa, Napier
PO Box 3315, Napier, 4142, New Zealand
P (06) 842 0111 F (06) 842 0113
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Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Gravity Loads		

Sheet No.: 2

Loads

Roof		
6mm Glass	0.160	
Steel Structure	0.300	
	0.460	kPa
0.46 / Cos 12 =	0.470	kPa

Conservatory Structure		
0	0.000	
100 Concrete	2.400	
0	0.000	
	2.400	kPa

Live loads

R2 Roofs	0.25	kPa
C3 Balconies*	4.00	kPa

emajineer Structural Concepts

Client: Christchurch City Council | Ref: 1923-2245
Project: Cunningham Glasshouse, Botanical Garden, Christchurch | Date: 10/2/14
BY: GN
Subject: Lump Sum Mass

Seismic Loads to NZS 1170.5 — Sheet No.: 3

Ref: | Design | Output

Design working live: 50 Years

Importance level: 2

Annual Probability of exceedance (inverse) Ultimate: 500

Annual Probability of exceedance (inverse) Service: 25

Element			Area/length	Load KPa	Total kN
Roof			370.00	0.47	174.08
#N/A			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
#N/A	0.51	0.40	370.00	0.00	0.00
				Total kN	174.08 kN

Live load reduction

Total floor area 210.0

$$.3+\frac{3}{\sqrt{A}} = 0.507$$

But not less than .5

1.38 T per centre node; 2.28 T per end node

Element			Area/length	Load KPa	Total kN
Roof			270.00	0.47	127.03
Conservatory Structure			800.00	2.40	1920.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
#N/A	0.51	0.30	210.00	4.00	127.77
#N/A	0.51	0.40	0.00	0.00	0.00
				Total kN	2174.80

7.77 T per node

Element			Area/length	Load KPa	Total kN
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
#N/A	0.51	0.40	0.00	0.00	0.00
#N/A	0.51	0.40	0.00	0.00	0.00
				Total kN	0.00

Element			Area/length	Load KPa	Total kN
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
#N/A	0.51	0.40	0.00	0.00	0.00
#N/A	0.51	0.40	0.00	0.00	0.00

Total kN 0.00

Total building weight 2349 kN

emajineer Structural Concepts

Client: Christchurch City Council | Ref: 1923-2245
Project: Cunningham Glasshouse | Date: 10/2/14
Botanical Garden, Christchurch | BY: GN
Subject: EQ Static 1170.5 check

Seismic Loads to NZS 1170.5 | Sheet No.: 4

Ref: | Design | Output

Design working live 50 Years

Importance level 2

Annual Probability of exceedance (inverse) Ultimate 500

Annual Probability of exceedance (inverse) Service 25

Element			Area/length	Load Kpa	Total kN
Roof			370.00	0.47	174.08
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
	1.00	0.30	0.00	0.00	0.00
					174.08 kN

Live load reduction

Total floor area 210.0

$$.3+\frac{3}{\sqrt{A}} = 0.507$$

But not less than .5

Element			Area/length	Load Kpa	Total kN
Roof			270.00	0.47	127.03
Conservatory Structure			800.00	2.40	1920.00
0			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
C3 Balconies*	0.51	0.30	210.00	4.00	840.00
	0.51	0.40	0.00	0.00	0.00
					2174.80 kN

Total building weight
2348.88 kN

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	EQ Static 1170.5 check		

Sheet No.: 5

Ref:	Design				Output
	Soil type				
	D. Deep or soft soil				
	Across the building				
	Period of building across the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☐
	Structural ductility factor (Ultimate)	m =	1.25		
	Structural ductility factor (Service SLS1)	m =	1.25		
	Hazard Factor Christchurch	Z =	0.3		
	Return period factor	Ru =	1.00		
	Return period factor	Rs =	0.30		
	Structural Performance factor (Ultimate)	Sp =	0.925		
	Structural Performance factor (Service)	Sp =	0.70		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0	n/a	
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.27		
	Ultimate	km =	1.14		
	Service	km =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.73	But not less than 0.030Ru	
	Ultimate force across the building	Cd(T1) x Wi =	1711.01	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.17		
	Service force across the building	Cd(T1) x Wi =	388.45	kN Total	
	Along the building				
	Period of building along the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☐
	Structural ductility factor (Ultimate)	m =	1.25		
	Structural ductility factor (Service SLS1)	m =	1.25		
	Structural Performance factor (Ultimate)	Sp =	0.93		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0		
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.27		
	Ultimate	km =	1.14		
	Service	km =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.73	But not less than 0.030Ru	
	Ultimate force along the building	Cd(T1) x Wi =	1711.01	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.22		
	Service force across the building	Cd(T1) x Wi =	513.30	kN Total	

Ref:	Design	Output

DYNAMIC RESPONSE SPECTRUM (kN,T,sec,Hz)

Spectral case 12: Spectral loads on X direction

Mass load case: 2
Direction vector: Dx = 1.000, Dy = 0.000, Dz = 0.000
Loading code: NZS1170.5
Limit state: Ultimate
Structural ductility factor: 1.250 — PASS
Auto scaling of base shear: On
Vertical direction: Z-Axis
Base shear: Not less than 100% of total static force
Results scaled by factor: 1.386
Site subsoil class: (D)
Sign of the results: Mode shape 6 (Calculated)
Hazard factor: 0.300
Return period factor: 1.000
Near-fault factor: 1.000
Structural perf. factor: 0.925 (User defined)
Spectral curve multiplier: 0.2775
Mode combination method: SRSS (Square Root of the Sum of Squares)

Direction	Dominant Mode	Total Static Force	Total Mass	MPF for Dominant Mode	Total Mass Part Factor	Base Shear
X-Axis	6	1678.7300	235.0000	88.166%	100.006%	72.844%
Y-Axis	1	0.0000	0.0000	0.000%	0.000%	0.000%
Z-Axis	1	0.0000	0.0000	0.000%	0.000%	0.000%

PASS

Direction	Mode Shape	Spectral Curve	Damping Factor	Natural Period	Natural Frequency	Mass Part Factor
Vector	1	NZS1170.5D	5.0%	0.1730	5.780	0.003%
Vector	2	NZS1170.5D	5.0%	0.1686	5.933	9.248%
Vector	3	NZS1170.5D	5.0%	0.1570	6.369	2.544%
Vector	4	NZS1170.5D	5.0%	0.1523	6.564	0.000%
Vector	5	NZS1170.5D	5.0%	0.1362	7.343	0.046%
Vector	6	NZS1170.5D	5.0%	0.0699	14.311	88.166%
				Total		100.006%

PASS

Ref:	Design	Output

Spectral case 17: Spectral loads on Y direction

Mass load case: 7
Direction vector: Dx = 0.000, Dy = 1.000, Dz = 0.000
Loading code: NZS1170.5
Limit state: Ultimate
Structural ductility factor: 1.250
Auto scaling of base shear: On
Vertical direction: Z-Axis
Base shear: Not less than 100% of total static force — PASS
Results scaled by factor: 1.309
Site subsoil class: (D)
Sign of the results: Mode shape 3 (Calculated)
Hazard factor: 0.300
Return period factor: 1.000
Near-fault factor: 1.000
Structural perf. factor: 0.925 (User defined)
Spectral curve multiplier: 0.2775
Mode combination method: SRSS (Square Root of the Sum of Squares)

Direction	Dominant Mode	Total Static Force	Total Mass	MPF for Dominant Mode	Total Mass Part Factor	Base Shear	Output
X-Axis	1	0.0000	0.0000	0.000%	0.000%	0.000%	
Y-Axis	3	1678.7300	235.0000	89.904%	99.922%	72.844%	PASS
Z-Axis	1	0.0000	0.0000	0.000%	0.000%	0.000%	

Direction	Mode Shape	Spectral Curve	Damping Factor	Natural Period	Natural Frequency	Mass Part Factor	Output
Vector	1	NZS1170.5D	5.0%	0.2194	4.558	10.012%	
Vector	2	NZS1170.5D	5.0%	0.2157	4.635	0.006%	
Vector	3	NZS1170.5D	5.0%	0.0749	13.351	89.904%	
Vector	4	NZS1170.5D	5.0%	0.0365	27.370	0.000%	
Vector	5	NZS1170.5D	5.0%	0.0332	30.127	0.000%	
Vector	6	NZS1170.5D	5.0%	0.0241	41.435	0.000%	
					Total	99.922%	PASS

Structural Concepts

Client: Christchurch City Council
Project: Cunningham Glasshouse
Botanical Garden, Christchurch
Subject: Buckling Check

Ref: 1923-2245
Date: 10/2/14
BY: GN

Sheet No.: 8

Ref:	Design	Output

BUCKLING LOAD FACTORS   Mode 1 Governs

Load Case	Mode	Load Factor	Tolerance	Iterations	Node at Max Trans	Node at Max Rotn	
1	1	5.043	0.007812	13	111 (Y)	112 (X)	
3	1	8.879	0.007812	15	135 (Y)	136 (X)	
21	1	5.043	0.007812	13	111 (Y)	112 (X)	
22	1	5.043	0.007812	13	111 (Y)	112 (X)	
23	1	5.043	0.007812	13	111 (Y)	112 (X)	
24	1	5.043	0.007812	13	111 (Y)	112 (X)	
51	1	3.730	0.007812	11	135 (Y)	136 (X)	critical
52	1	8.879	0.007812	15	135 (Y)	136 (X)	
53	1	8.879	0.007812	15	135 (Y)	136 (X)	

BUCKLING EFFECTIVE LENGTHS (kN,m)

Load case 51 (Linear): 1.35G   Internal Chord Only

Mode	Memb	Pcr	Length	Ly	Lz
1	218	115.251	1.100	3.669	2.648
	219	118.050	1.346	3.626	2.617
	220	109.765	1.677	3.760	2.714
	221	89.808	1.697	4.157	3.000
	222	48.005	1.082	5.686	4.103
	223	2.460	0.671	25.118	18.128
	224	-33.666	0.608	Not in compression	
	225	-40.126	0.510	Not in compression	
	226	-42.375	0.412	Not in compression	
	270	115.765	1.000	3.661	2.642
	271	118.644	1.346	3.617	2.610
	272	110.338	1.677	3.750	2.707
	273	90.297	1.697	4.146	2.992
	274	48.006	1.082	5.686	4.103
	275	1.638	0.671	30.775	22.211
	276	-35.175	0.608	Not in compression	
	277	-41.117	0.510	Not in compression	
	278	-42.603	0.412	Not in compression	
	1597	113.087	1.200	3.704	2.674
	1599	113.644	1.200	3.695	2.667

To make sense of the current building stability it is fair to assume that the current effective lengths work. Therfore whatever is used for the assessment must reflect this.

at 1.35G the axial load a force of 31.5kN is acting in the internal chord.

for the building to be in equilibrium with this then the effective length must allow atleast this force to be acting.

so looking at the simple effective lengths between base and the longitudinal truss we have:

N*= 31.5 kN
L= 8.4 m

ke	Le	Nc	%of N*	
1	8.4	10.01	32%	FAILS
0.85	7.14	14.02	45%	FAILS
0.7	5.88	19.63	62%	FAILS

Therefore there must be other restraint available.
The double angle lateral members can provide this.

Assuming this is the case then

L= 5.32 m
ke 0.85
Le= 4.522 m

And Nc= 32.44 kN

Plus %of N*= 103% PASS

Therefore this is logical and fits the gravity model

Ref:	Design	Output

Restraint assumed at 1.4m double L lateral

Semi fixity in lateral restraint is provided by the lateral cantilevering of the outer T section which is restrained in rotation by the purlins.

The force to be retsrained at 1.53G is	P*=	31.7	kN
The force in the restraint is 0.025xP*	N*=	0.7925	kN
Deflection assuming full fixity	d=	5.5	mm
Bending in the double angle is N*xL	M*=	1.1095	kNm
Bending Stress in double angle is	fs=	83	Mpa

Combined with axial load in lateral member

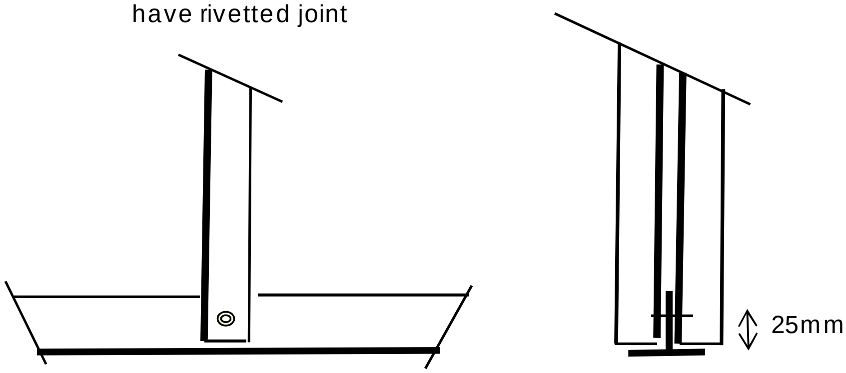
Nc*=	12	kN
Nn=	101	kN

83/293 + 12/101

0.40092 <1.0 OK

Check fixity at outer chord

have rivetted joint



Force on rivet in tension 44.38 kN OK

Ok to assume lateral restraint from Lateral member at 1400mm long.

Length between restraints	L=	5.32	m
	ke	0.85	
	Le=	4.52	m

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Chord Assessment ke=1.0		

AXIAL CAPACITY OF A STEEL COLUMN DESIGN TO NZS3404 — Sheet No.: 11

Ref:	Design				Output
	Dead load	G	0.0	kN	
	Imposed load	Q	26.0	kN	
	Dead load factor	DLF	1.0		
	Imposed load factor	LLF	1.0		
	Axial load		26.0	kN	26.0 kN
	Effecttive length x axis	Lex	2.0	m	
	Effective length y axis	Ley	8.4	m	
T3.3(1)	Strength reduction factor	ø	1		
	Yield stress of flanges	fyf	293	Mpa	
	Yield stress of web	fyw	293	Mpa	
	Yield stress of section		293	Mpa	
	Using steel section		75 T x 11		FAILED

Section properties

Section depth	D	75.0	mm	Form factor	kf	1.000	
Flange width	B	75.0	mm	Moment of inertia major	Ix	71.0	cm^4
Flange thickness	T	9.5	mm	Moment of inertia minor	Iy	34.0	cm^4
Web thickness	TW	9.5	mm	Plastic modulus	Sy	13.6	cm^3
Between flanges	DF	65.5	mm	Elastic modulus	Zy	8.9	cm^3
Ratios for local buckling				Radius of gyration	Rx	22.9	mm
Flange	b/t	6.9		Radius of gyration	Ry	16.4	mm
Web	b/t	6.9		Torsion Constant	J	8.1	
Section slenderness perameters				Warping Constant	Iw	0.1	
	lef	7.46		Youngs modulus	E	200000	MPa
	lew	7.46		Area of section		1367	mm^2

Ref	Design					Output
6.2.1	Section capacity		Ns	400.5	kN	
			ØNs	400.5	kN	
	Member Capacity					
6.6.3	Effective length ratios	$\left(\frac{Lex}{rx}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lnx	94.5		
		$\left(\frac{Ley}{ry}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lny	554.5		
			Ln	554.5		
T. 6.3.3(1)	Compression member constant		xb	0.5		
T. 6.3.3(3)	Slenderness reduction factor		xc	0.025		
	Member capacity	0.025 x 400.531 =	ØNc	10.01	kN	39%

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Chord Assessment ke=0.85		

AXIAL CAPACITY OF A STEEL COLUMN DESIGN TO NZS3404 — Sheet No.: 12

Ref:	Design				Output
	Dead load	G	0.0	kN	
	Imposed load	Q	39.0	kN	
	Dead load factor	DLF	1.0		
	Imposed load factor	LLF	1.0		
	Axial load		39.0	kN	39.0 kN
	Effecttive length x axis	Lex	2.00	m	
	Effective length y axis	Ley	7.10	m	
T3.3(1)	Strength reduction factor	ø	1		
	Yield stress of flanges	fyf	293	Mpa	
	Yield stress of web	fyw	293	Mpa	
	Yield stress of section		293	Mpa	
	Using steel section	75 T x 11			FAILED

Section properties

Section depth	D	75.0 mm	Form factor	kf	1.000	
Flange width	B	75.0 mm	Moment of inertia major	Ix	71.0	cm^4
Flange thickness	T	9.5 mm	Moment of inertia minor	Iy	34.0	cm^4
Web thickness	TW	9.5 mm	Plastic modulus	Sy	13.6	cm³
Between flanges	DF	65.5 mm	Elastic modulus	Zy	8.9	cm³
Ratios for local buckling			Radius of gyration	Rx	22.9	mm
Flange	b/t	6.9	Radius of gyration	Ry	16.4	mm
Web	b/t	6.9	Torsion Constant	J	8.1	
Section slenderness perameters			Warping Constant	Iw	0.1	
	lef	7.46	Youngs modulus	E	200000	MPa
	lew	7.46	Area of section		1367	mm²

Ref	Design					Output
6.2.1	Section capacity		Ns	400.5	kN	
			ØNs	400.5	kN	
	Member Capacity					
6.6.3	Effective length ratios	$\left(\frac{Lex}{rx}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lnx	94.5		
		$\left(\frac{Ley}{ry}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lny	468.7		
			Ln	468.7		
T. 6.3.3(1)	Compression member constant		xb	0.5		
T. 6.3.3(3)	Slenderness reduction factor		xc	0.035		
	Member capacity	0.035 x 400.531 =	ØNc	14.02	kN	36%

Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Chord Assessment ke=0.7		

AXIAL CAPACITY OF A STEEL COLUMN DESIGN TO NZS3404 — Sheet No.: 13

Ref:	Design				Output
	Dead load	G	0.0	kN	
	Imposed load	Q	39.0	kN	
	Dead load factor	DLF	1.0		
	Imposed load factor	LLF	1.0		
	Axial load		39.0	kN	39.0 kN
	Effecttive length x axis	Lex	2.0	m	
	Effective length y axis	Ley	5.9	m	
T3.3(1)	Strength reduction factor	ø	1		
	Yield stress of flanges	fyf	293	Mpa	
	Yield stress of web	fyw	293	Mpa	
	Yield stress of section		293	Mpa	
	Using steel section	75 T x 11			FAILED

Section properties

Section depth	D	75.0 mm	Form factor	kf	1.000	
Flange width	B	75.0 mm	Moment of inertia major	Ix	71.0	cm^4
Flange thickness	T	9.5 mm	Moment of inertia minor	Iy	34.0	cm^4
Web thickness	TW	9.5 mm	Plastic modulus	Sy	13.6	cm³
Between flanges	DF	65.5 mm	Elastic modulus	Zy	8.9	cm³
Ratios for local buckling			Radius of gyration	Rx	22.9	cm
Flange	b/t	6.9	Radius of gyration	Ry	16.4	cm
Web	b/t	6.9	Torsion Constant	J	8.1	
Section slenderness perameters			Warping Constant	Iw	0.1	
	lef	7.46	Youngs modulus	E	200000	MPa
	lew	7.46	Area of section		1367	mm²

Ref	Design					Output
6.2.1	Section capacity		Ns	400.5	kN	
			ØNs	400.5	kN	
	Member Capacity					
6.6.3	Effective length ratios	$\left(\frac{Lex}{rx}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lnx	94.5		
		$\left(\frac{Ley}{ry}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lny	388.1		
			Ln	388.1		
T. 6.3.3(1)	Compression member constant		xb	0.5		
T. 6.3.3(3)	Slenderness reduction factor		xc	0.049		
	Member capacity	0.049 x 400.531 =	ØNc	19.63	kN	50%

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Chord Assessment Le=4.52 (1.35G)		

AXIAL CAPACITY OF A STEEL COLUMN DESIGN TO NZS3404 — Sheet No.: 14

Ref:	Design				Output
	Dead load	G	0.0	kN	
	Imposed load	Q	31.7	kN	
	Dead load factor	DLF	1.0		
	Imposed load factor	LLF	1.0		
	Axial load		31.7	kN	31.7 kN
	Effecttive length x axis	Lex	2.0	m	
	Effective length y axis	Ley	4.52	m	
T3.3(1)	Strength reduction factor	ø	1		
	Yield stress of flanges	fyf	293	Mpa	
	Yield stress of web	fyw	293	Mpa	
	Yield stress of section		293	Mpa	
	Using steel section	75 T x 11			PASS

Section properties

Section depth	D	75.0	mm	Form factor	kf	1.000	
Flange width	B	75.0	mm	Moment of inertia major	Ix	71.0	cm^4
Flange thickness	T	9.5	mm	Moment of inertia minor	Iy	34.0	cm^4
Web thickness	TW	9.5	mm	Plastic modulus	Sy	13.6	cm³
Between flanges	DF	65.5	mm	Elastic modulus	Zy	8.9	cm³
Ratios for local buckling				Radius of gyration	Rx	22.9	cm
Flange	b/t	6.9		Radius of gyration	Ry	16.4	cm
Web	b/t	6.9		Torsion Constant	J	8.1	
Section slenderness perameters				Warping Constant	Iw	0.1	
	lef	7.46		Youngs modulus	E	200000	MPa
	lew	7.46		Area of section		1367	mm²

Ref	Design					Output
6.2.1	Section capacity		Ns	400.5	kN	
			ØNs	400.5	kN	
	Member Capacity					
6.6.3	Effective length ratios	$\left(\frac{Lex}{rx}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lnx	94.5		
		$\left(\frac{Ley}{ry}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lny	298.4		
			Ln	298.4		
T. 6.3.3(1)	Compression member constant		xb	0.5		
T. 6.3.3(3)	Slenderness reduction factor		xc	0.081		
	Member capacity	0.081 x 400.531 =	ØNc	32.44	kN	102% NBS

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Chord Assessment Le=4.52 (Edx)		

AXIAL CAPACITY OF A STEEL COLUMN DESIGN TO NZS3404 — Sheet No.: 15

Ref:	Design				Output
	Dead load	G	0.0	kN	
	Imposed load	Q	50.0	kN	
	Dead load factor	DLF	1.0		
	Imposed load factor	LLF	1.0		
	Axial load		50.0	kN	50.0 kN
	Effecttive length x axis	Lex	2.0	m	
	Effective length y axis	Ley	4.52	m	
T3.3(1)	Strength reduction factor	ø	1		
	Yield stress of flanges	fyf	293	Mpa	
	Yield stress of web	fyw	293	Mpa	
	Yield stress of section		293	Mpa	
	Using steel section	75 T x 11			FAILED

Section properties

Section depth	D	75.0	mm	Form factor	kf	1.000	
Flange width	B	75.0	mm	Moment of inertia major	Ix	71.0	cm^4
Flange thickness	T	9.5	mm	Moment of inertia minor	Iy	34.0	cm^4
Web thickness	TW	9.5	mm	Plastic modulus	Sy	13.6	cm³
Between flanges	DF	65.5	mm	Elastic modulus	Zy	8.9	cm³
Ratios for local buckling				Radius of gyration	Rx	22.9	cm
Flange	b/t	6.9		Radius of gyration	Ry	16.4	cm
Web	b/t	6.9		Torsion Constant	J	8.1	
Section slenderness perameters				Warping Constant	Iw	0.1	
	lef	7.46		Youngs modulus	E	200000	MPa
	lew	7.46		Area of section		1367	mm²

Ref	Design					Output
6.2.1	Section capacity		Ns	400.5	kN	
			øNs	400.5	kN	
	Member Capacity					
6.6.3	Effective length ratios	$\left(\frac{Lex}{rx}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lnx	94.5		
		$\left(\frac{Ley}{ry}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lny	298.4		
			Ln	298.4		
T. 6.3.3(1)	Compression member constant		xb	0.5		
T. 6.3.3(3)	Slenderness reduction factor		xc	0.081		
	Member capacity	0.081 x 400.531 =	øNc	32.44	kN	65% NBS

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Chord Assessment Le=4.52 (Edy)		

AXIAL CAPACITY OF A STEEL COLUMN DESIGN TO NZS3404 — Sheet No.: 16

Ref:	Design				Output
	Dead load	G	0.0	kN	
	Imposed load	Q	54.0	kN	
	Dead load factor	DLF	1.0		
	Imposed load factor	LLF	1.0		
	Axial load		54.0	kN	54.0 kN
	Effecttive length x axis	Lex	2.0	m	
	Effective length y axis	Ley	4.52	m	
T3.3(1)	Strength reduction factor	ø	1		
	Yield stress of flanges	fyf	293	Mpa	
	Yield stress of web	fyw	293	Mpa	
	Yield stress of section		293	Mpa	
	Using steel section	75 T x 11			FAILED

Section properties

Section depth	D	75.0 mm	Form factor	kf	1.000	
Flange width	B	75.0 mm	Moment of inertia major	Ix	71.0	cm^4
Flange thickness	T	9.5 mm	Moment of inertia minor	Iy	34.0	cm^4
Web thickness	TW	9.5 mm	Plastic modulus	Sy	13.6	cm³
Between flanges	DF	65.5 mm	Elastic modulus	Zy	8.9	cm³
Ratios for local buckling			Radius of gyration	Rx	22.9	cm
Flange	b/t	6.9	Radius of gyration	Ry	16.4	cm
Web	b/t	6.9	Torsion Constant	J	8.1	
Section slenderness perameters			Warping Constant	Iw	0.1	
	Ief	7.46	Youngs modulus	E	200000	MPa
	Iew	7.46	Area of section		1367	mm²

Ref	Design					Output
6.2.1	Section capacity		Ns	400.5	kN	
			ØNs	400.5	kN	
	Member Capacity					
6.6.3	Effective length ratios	$\left(\frac{Lex}{rx}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lnx	94.5		
		$\left(\frac{Ley}{ry}\right)\sqrt{kf}\sqrt{\left(\frac{fy}{250}\right)}$	Lny	298.4		
			Ln	298.4		
T. 6.3.3(1)	Compression member constant		xb	0.5		
T. 6.3.3(3)	Slenderness reduction factor		xc	0.081		
	Member capacity	0.081 x 400.531 =	ØNc	32.44	kN	60% NBS

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Chord Load 1.35G & 1.0G		

Sheet No.: 17

Ref:	Design	Output

1.35G

	F Chord Force kN	L Chord Length m	FxL
1	0.64	0.67	0.4288
2	12.87	1.08	13.8996
3	24.10	1.7	40.97
4	29.46	1.67	49.1982
5	31.69	1.35	42.7815
6	30.94	1	30.94
7	30.36	1.3	39.468
Sum ∑	160.06	8.77	217.686

∑(FxL)/∑L = 24.82 kN

∑F/7 = 22.87 kN

1.0G

	F Chord Force kN	L Chord Length m	FxL
1	0.47	0.67	0.31763
2	9.53	1.08	10.296
3	17.85	1.7	30.3481
4	21.82	1.67	36.4431
5	23.47	1.35	31.69
6	22.92	1	22.9185
7	22.49	1.3	29.2356
Sum ∑	118.563	8.77	161.249

∑(FxL)/∑L = 18.39 kN

∑F/7 = 16.94 kN

Ref: | Design | Output

G+Edx Lower portion

	F Chord Force kN	L Chord Length m	FxL	
1	0.00	0.67	0	
2	0.00	1.08	0	NOTE
3	0.00	1.7	0	Take all tension members
4	29.25	1.67	48.8475	out of the equation
5	33.14	1.35	44.739	sum only chord numbers 4,5,6,7
6	44.09	1	44.09	
7	49.54	1.3	64.402	
Sum ∑	156.02	5.32	202.079	

∑(FxL)/∑L = 37.98 kN

∑F/7 = 39.01 kN

G+Edx Upper portion

	F Chord Force kN	L Chord Length m	FxL	
1	35.48	0.67	23.7716	NOTE
2	39.29	1.08	42.4332	Take all tension members
3	37.51	1.7	63.767	out of the equation
4	14.60	1.67	24.382	sum only chord numbers 1,2,3,4,5,6
5	14.03	1.35	18.9405	
6	1.93	1	1.93	
7	0.00	1.3	0	
Sum ∑	142.84	7.47	175.224	

∑(FxL)/∑L = 23.46 kN

∑F/7 = 23.81 kN

Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Long Truss Chord Loading G+Edy	Sheet No.:	19

Ref: | Design | Output

G+Edy Lower portion

	F Chord Force kN	L Chord Length m	FxL	
1	0.00	0.67	0	
2	0.00	1.08	0	NOTE
3	12.93	1.7	21.981	Take all tension members
4	23.87	1.67	39.8629	out of the equation
5	39.79	1.35	53.7165	sum only chord numbers 3,4,5,6,7
6	50.18	1	50.18	
7	53.51	1.3	69.563	
Sum ∑	180.28	7.02	235.303	

∑(FxL)/∑L = 33.52 kN

∑F/7 = 36.06 kN

G+Edy Upper portion G+Edy

	F Chord Force kN	L Chord Length m	FxL	
1	28.22	0.67	18.9074	
2	20.23	1.08	21.8484	NOTE
3	8.47	1.7	14.399	Take all tension members
4	5.73	1.67	9.5691	out of the equation
5	0.00	1.35	0	sum only chord numbers 1,2,3,4
6	0.00	1	0	
7	0.00	1.3	0	
Sum ∑	62.65	5.12	64.7239	

∑(FxL)/∑L = 12.64 kN

∑F/7 = 15.66 kN

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Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Check Double Steel Flats		

Sheet No.: 20

Ref:	Design				Output
	<u>Plate tension capacity</u>				
	Force on plate		22.0	kN	Fin plate
	Yield strength of plate	Fy	350	Mpa	
	Tensile strength of plate	Fu	440	Mpa	
	Plate dimensions				
	Breadth	B	50		
	Thickness	t	6		
	Holes in plate				
	Number of holes in a single tensile plane		2	No.	
	Diameter of holes		12	mm	
	Total area area of holes	Ah	144.0	mm²	
	Net area of plate	An	300	mm²	
	Gross area of plate	Ag	156.0	mm²	
	Eccentricity correction factor	Kte	0.7		
	Nominal section capacity				
	Ag x Fy =	Nt	54.6	kN	
	OR				
	.85 x Kte x An x Fu =	Nt	78.5	kN	
	Therefore Nominal section capacity is				
		Nt	54.6	kN	
	Section capacity	ϕNt	54.6	kN	PASS

Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Check Double Steel Angles		

Sheet No.: 21

Ref:	Design				Output
	Check Double Angles	50x50x8 L	use	50x50x6 L	
	Maximum Compression Force				
	Force	N*=	12	kN	
	Effective Length	Le=	2.1	m	
		ke=	1		
	Area	A=	568	mm^2	
	Radius of Gyration	Ry=	15.1	mm	
	Steel Grade	Fy=	293	MPa	
		Le/ry=	139		
	Slenderness reduction factor	αc=	0.304		
		Nc=	101.2	kN	PASS

Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Check Rivets		

Design of bolts in shear, bearing & plate tearout — Sheet No.: 22

Ref:	Design				Output
	This calculation assumes that all rivets are equally loaded, as in the case of a tension only connection. It does not check for tension fracture of the connecting ply.				
	Thickness of plate	tp	8	mm	
	Grade of connecting plate	Fyp	250	Mpa	
	Tensile strength of plate	Fup	410	Mpa	
	Size of rivets in connection	df	12	mm	
	Grade of rivet		4.6		
	Tensile strength of rivet	fub	250		
	Number of rivets in group	n	1		
	Rivetted lap correction	Kr	1.0		
	Shear type		Double		
	Load type		Seismic		
	The seismic system is		(4) Elastic		
	Reduction factor C1	C1	1.0		
	<u>Rivet in shear</u>				
	Strength reduction factor	f	0.8		
	Shear plane through rivet is		Shank Ao		
	Area of rivet	A	113	mm	
	Nominal shear capacity of rivets				
	.62 x Fu x Kr x (n x Ac + n x Ao) = Vf		35.0	kN	
	Shear strength	fVf	28.0	kN	PASS
	<u>Rivets in Bearing</u>				
	<u>Plate</u>				
	Strength reduction factor	f	0.9		
	Nominal bearing capacity of rivets				
	3.2 x df x tp x fup x n = Vb		126.0	kN	
	bearing strength of rivets	C1 x f x Vb	113.4	kN	PASS
12.9.4.5.2	For category 1 members connected by snug tight bolt mode, holes shall be a maximum of .5mm oversized only.				
	<u>Plate tearout</u>				
	<u>plate</u>				
	Edge distance		30	mm	
	Force on each rive		22.0	kN	
	Nominal tearout capacity				
	ae x tp x Fup x n = Vb		98.4	kN	
	Tearout capacity		88.56	kN	PASS

Table 6.3.3(2) – Values of member slenderness reduction factor (α_c)

Modified member slenderness (λ_n)	Compression member section constant (α_b)				
	−1.0	−0.5	0	0.5	1.0
0	1.000	1.000	1.000	1.000	1.000
5	1.000	1.000	1.000	1.000	1.000
10	1.000	1.000	1.000	1.000	1.000
15	1.000	0.998	0.995	0.992	0.990
20	1.000	0.989	0.978	0.967	0.956
25	0.997	0.979	0.961	0.942	0.923
30	0.991	0.968	0.943	0.917	0.888
35	0.983	0.955	0.925	0.891	0.853
40	0.973	0.940	0.905	0.865	0.818
45	0.959	0.924	0.884	0.837	0.782
50	0.944	0.905	0.861	0.808	0.747
55	0.927	0.885	0.836	0.778	0.711
60	0.907	0.862	0.809	0.746	0.676
65	0.886	0.837	0.779	0.714	0.642
70	0.861	0.809	0.748	0.680	0.609
75	0.835	0.779	0.715	0.646	0.576
80	0.805	0.746	0.681	0.612	0.545
85	0.772	0.711	0.645	0.579	0.516
90	0.737	0.675	0.610	0.547	0.487
95	0.700	0.638	0.575	0.515	0.461
100	0.661	0.600	0.541	0.485	0.435
105	0.622	0.564	0.508	0.457	0.412
110	0.584	0.528	0.477	0.431	0.389
115	0.546	0.495	0.448	0.406	0.368
120	0.510	0.463	0.421	0.383	0.348
125	0.476	0.434	0.395	0.361	0.330
130	0.445	0.408	0.372	0.341	0.313
135	0.416	0.381	0.350	0.322	0.297
140	0.389	0.357	0.330	0.304	0.282
145	0.364	0.336	0.311	0.288	0.268
150	0.341	0.316	0.293	0.273	0.255
155	0.320	0.298	0.277	0.259	0.242
160	0.301	0.281	0.263	0.246	0.231
165	0.283	0.265	0.249	0.234	0.220
170	0.267	0.251	0.236	0.222	0.210
175	0.252	0.238	0.224	0.212	0.200

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Client: Christchurch City Council
Project: Cunningham Glasshouse
Botanical Garden, Christchurch
Subject: Table 6.3.3(2) cont.

Ref: 1923-2245
Date: 10/2/14
BY: GN

Sheet No.: 24

Table 6.3.3(2) – Values of member slenderness reduction factor (α_c) (continued)

Modified member slenderness (λ_n)	Compression member section constant (α_b)				
	−1.0	−0.5	0	0.5	1.0
180	0.239	0.225	0.213	0.202	0.192
185	0.226	0.214	0.203	0.193	0.183
190	0.214	0.203	0.193	0.184	0.175
195	0.204	0.194	0.185	0.176	0.168
200	0.194	0.185	0.176	0.168	0.161
205	0.184	0.176	0.168	0.161	0.154
210	0.176	0.168	0.161	0.154	0.148
215	0.167	0.161	0.154	0.148	0.142
220	0.160	0.154	0.148	0.142	0.137
225	0.153	0.147	0.142	0.137	0.132
230	0.146	0.141	0.136	0.131	0.127
235	0.140	0.135	0.131	0.126	0.122
240	0.134	0.130	0.126	0.122	0.118
245	0.129	0.125	0.121	0.117	0.114
250	0.124	0.120	0.116	0.113	0.110
255	0.119	0.116	0.112	0.109	0.106
260	0.115	0.111	0.108	0.105	0.102
265	0.110	0.107	0.104	0.102	0.099
270	0.106	0.103	0.101	0.098	0.096
275	0.102	0.100	0.097	0.095	0.092
280	0.099	0.096	0.094	0.092	0.089
285	0.095	0.093	0.091	0.089	0.087
290	0.092	0.090	0.088	0.086	0.084
295	0.089	0.087	0.085	0.083	0.081
300	0.086	0.084	0.082	0.081	0.079
305	0.083	0.082	0.080	0.078	0.077
310	0.081	0.079	0.077	0.076	0.074
315	0.078	0.077	0.075	0.074	0.072
320	0.076	0.074	0.073	0.071	0.070
340				0.064	
370				0.054	
400				0.047	
450				0.037	
500				0.031	
550				0.025	
600				0.021	

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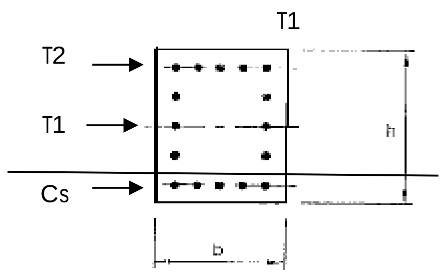
Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Concrete Column Check Along		

Design of elastic concrete column in bending NZS3101:2006 — Sheet No.: 25

Ref:	Design					Output
	Axial load		N*	0	kN	
	Design moment		M*	67	kNm	
	Design Shear force		V*	50	kN	
	Ductility factor used		m	1.25	<= 1.25	

7, 8, 9



The moment capacity is based on concrete theory, as found in any concrete text book, i.e. ccanz "Red Book"

Typical column steel configuration

Clear storey height			3000	mm
Depth of column		h	450	mm
Width column		b	450	mm
Cover			80	mm
Concrete grade		Fc'	30	Mpa
Steel reinforcement yield stress (Yeilding steel)		Fy	300	Mpa
Steel reinforcement yield stress (Shear steel)		fyt	300	Mpa
Tension steel For T1				
Number of bars		No.	0	
Diameter of bars		dia	16	mm
Area of bars at T1		As1	0	mm²
Tension capacity	As x Fy =	T1	0.0	kN
Tension steel For T2				
Number of bars		No.	2	
Diameter of bars		dia	19.05	mm
Area of bar: As x Fy =		As2	570	mm²
Tension capacity	As x Fy =	T2	171.0	kN
Compression steel For Cs				
Number of bars		No.	2	
Diameter of bars		dia	19	mm
Area of bars at T1		AsCs	570	mm²
Tension capacity	As x Fy =	Cs	171.0	kN
Axial load on wall				
Self weight of column 0.45 x 3 x 0.45 x 24 =			14.58	
Other dead load			0.00	
			14.58	
C = T1 + T2 + Nn - Cs = 0 x 171.04 + 14.58 - 171.04 =			15	

Ref:	Design				Output
	Depth of equivalent stress block a				
	a = C / (.85 x Fc' x b) = 1 mm				
	therefore c =	1 / 0.85 =	1	mm	
	With reference to centraidal axis of the column				
	Centroid of T1		0	mm	
	Centroid of T2		123	mm	
	Centriod of C		224	mm	
	Centriod of Cs		123	mm	
	Hence moment capacity of column is:-				
	Mn for T1 = AS x Fy x La x 10^-6		0.0		
	Mn for T2 = AS x Fy x La x 10^-6		21.1		
	Mn for C = C x La x 10^-3		3.3		
	Mn for Cs = Cs x La x 10^-3		21.1		
			45.5	kNm	
	1.00				
	øMn = 46 kNm > 67 kNm therefore OK				FAILED 68% NBS
	Shear Steel design				
	Shear force	V*wall	50.0	kN	
	Nominal shear stress				
11.3.10.3.3	Note d = 80% of actual length				
		vn = V*wall / bw x .8 x d =	0.31	Mpa	
7.5.2	Maximum shear stress				
		.2 Fc' =	6.00	Mpa	
		OR			
			8.00	Mpa	
11.3.10.3.5	Shear resistance provided by concrete				
	$\left(.27\sqrt{fc'}+\frac{N^*}{4Ag}\right)$	= vc	1.50	Mpa	
	OR				
	$.05\sqrt{fc'}+\frac{Lw\left(.1\sqrt{fc'}+.2\frac{N^*}{Ag}\right)}{\frac{M^*}{V^*}-\frac{Lw}{2}}$	= vc	N/A	Mpa	
7.5.1	Shear strength provided by concrete mechanisms				
	vc x Lw x .8 x b =	Vc	243	kN	
		øVc	182	kN	
	Only min shear steel to 11.3.10.3.8 b required				

Design of elastic concrete column in bending continued | Sheet No.: 27

Ref:	Design				Output
11.3.10.3.8	(a) Shear strength provided by shear reinforcement				
	Main bar diameter	DIA	12	mm> 10	
	Area of steel provided	Av	113	mm²/250	
	Bar spacing	S2	250	mm	
	Maximum bar spacing	Smax	450	mm	
	$Av.fyt\frac{d}{S_2}$	= Vs	49	kN	
		ØVs	37	kN	
	Total shear strength	ØVc + ØVs	219	kN	PASS
11.3.10.3.8	(b) Minimum shear steel				
	$\frac{.7bw.S2}{fyt}$	Av	263	mm²/250	NA

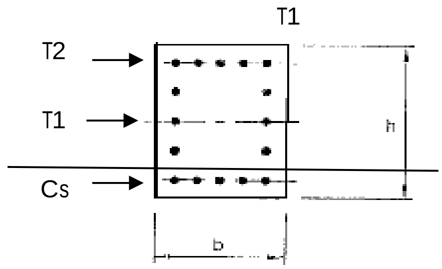
Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Concrete Column Check Across		

Design of elastic concrete column in bending NZS3101:2006 — Sheet No.: 28

Ref:	Design					Output
	Axial load		N*	0	kN	
	Design moment		M*	67	kNm	
	Design Shear force		V*	50	kN	
	Ductility factor used		m	1.25	<= 1.25	

7, 8, 9



The moment capacity is based on concrete theory, as found in any concrete text book, i.e. ccanz "Red Book"

Typical column steel configuration

Clear storey height			3000	mm
Depth of column		h	450	mm
Width column		b	450	mm
Cover			80	mm
Concrete grade		Fc'	30	Mpa
Steel reinforcement yield stress (Yeilding steel)		Fy	300	Mpa
Steel reinforcement yield stress (Shear steel)		fyt	300	Mpa
Tension steel For T1				
Number of bars		No.	0	
Diameter of bars		dia	16	mm
Area of bars at T1		As1	0	mm²
Tension capacity	As x Fy =	T1	0.0	kN
Tension steel For T2				
Number of bars		No.	2	
Diameter of bars		dia	19.05	mm
Area of bar: As x Fy =		As2	570	mm²
Tension capacity	As x Fy =	T2	171.0	kN
Compression steel For Cs				
Number of bars		No.	2	
Diameter of bars		dia	19	mm
Area of bars at T1		AsCs	570	mm²
Tension capacity	As x Fy =	Cs	171.0	kN
Axial load on wall				
Self weight of column 0.45 x 3 x 0.45 x 24 =			14.58	
Other dead load			0.00	
			14.58	
C = T1 + T2 + Nn - Cs = 0 x 171.04 + 14.58 - 171.04 =			15	

Client: Christchurch City Council | Ref: 1923-2245
Project: Cunningham Glasshouse | Date: 10/2/14
Botanical Garden, Christchurch | BY: GN
Subject: Concrete Column Check Across

Design of elastic concrete column in bending continued — Sheet No.: 29

Ref:	Design			Output
	Depth of equivalent stress block a			
	a = C / .85 x Fc' x b = 1		mm	
	therefore c = 1 / 0.85 =	1	mm	
	With reference to centraidal axis of the column			
	Centroid of T1	0	mm	
	Centroid of T2	123	mm	
	Centriod of C	224	mm	
	Centriod of Cs	123	mm	
	Hence moment capacity of column is:-			
	Mn for T1 = AS x Fy x La x 10^-6	0.0		
	Mn for T2 = AS x Fy x La x 10^-6	21.1		
	Mn for C = C x La x 10^-3	3.3		
	Mn for Cs = Cs x La x 10^-3	21.1		
		45.5	kNm	
	1.00			
	øMn = 46 kNm > 67 kNm therefore OK			FAILED
	Shear Steel design			68% NBS
	Shear force V*wall	50.0	kN	
	Nominal shear stress			
11.3.10.3.3	Note d = 80% of actual length			
	vn = V*wall / bw x .8 x d =	0.31	Mpa	
7.5.2	Maximum shear stress			
	.2 Fc' =	6.00	Mpa	
	OR			
		8.00	Mpa	
11.3.10.3.5	Shear resistance provided by concrete			
	$\left(.27\sqrt{fc'} + \frac{N*}{4Ag}\right)$ = vc	1.50	Mpa	
	OR			
	$.05\sqrt{fc'} + \frac{Lw\left(.1\sqrt{fc'} + .2\frac{N*}{Ag}\right)}{\frac{M*}{V*} - \frac{Lw}{2}}$ = vc	N/A	Mpa	
7.5.1	Shear strength provided by concrete mechanisms			
	vc x Lw x .8 x b = Vc	243	kN	
	øVc	182	kN	
	Only min shear steel to 11.3.10.3.8 b required			

Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	GN
Subject:	Concrete Column Check Across		

Design of elastic concrete column in bending continued — Sheet No.: 30

Ref:	Design				Output
11.3.10.3.8	(a) Shear strength provided by shear reinforcement				
	Main bar diameter	DIA	12	mm> 10	
	Area of steel provided	Av	113	mm²/250	
	Bar spacing	S2	250	mm	
	Maximum bar spacing	Smax	450	mm	
	$Av.fyt\frac{d}{S_2}$	= Vs	49	kN	
		ØVs	37	kN	
	Total shear strength	ØVc + ØVs	219	kN	PASS
11.3.10.3.8	(b) Minimum shear steel				
	$\frac{.7bw.S2}{fyt}$	Av	263	mm²/250	NA

emajineer Structural Concepts

Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	NS
Subject:	Concrete Beam Check		

REINFORCED CONCRETE RECTANGULAR BEAM DESIGNED TO NZS3101:PART 1:2006 — Sheet No.: 31

Ref:	Design					Output
	Design bending moment from analysis		M*	40	kNm	
	Shear from analysis		V*	10	kN	
	<u>Beam dimensions and materials</u>					
5.2.1	Concrete grade		Fc'	30	Mpa	
5.3.3	Steel reinforcement yield stress		Fy	300	Mpa	
	Shear steel yield stress		Fyt	300	Mpa	
	Cover to reinforcement		C	75	mm	
	Depth of beam		D	550	mm	
	Width of beam		bw	300	mm	
	Effective depth	H-C-DIA/2 =	d	467.063	mm	
2.3.2.2	Strength reduction factor flexural		f	1.00		
	Strength reduction factor Shear		f	0.85		
	Lever arm	$d - \frac{As.Fy}{1.7Fc'b} =$	Jd	459	mm	
	Main bar diameter		DIA	15.875	mm	
	Number of bars		N	2		
	Area of steel provided		As	396	mm²	
	<u>Minimum area of tension steel</u>					
9.3.8.2.1	Min. area of tension steel	$\frac{\sqrt{Fc'}}{4Fy} bw.d =$	As min	640	mm²	
	But equal to or greater than	1.4 bw.d/Fy =	As min	654	mm²	
9.3.8.2.3	Alternatively may be 1/3 greater than what is required by analysis					
	Moment capacity	ø x AS x Fy x Jd x 10^-6 =	øMn	54.5	kNm	> 40 kNm
						PASS
	<u>Shear Check</u>					136% NBS
7.5.1	Total nominal shear stress	V* / (bw.d) =	vn	0.071	Mpa	
7.5.2	Maximum shear stress vn shall be less than					
		.2Fc' or 8Mpa		6.0	Mpa	> 0.071 Mpa
						PASS
9.3.9.3.4	<u>Shear stress provided by concrete</u>					
	$Vc = vc.Acv$ Where $vc = kd.ka.vb$					
9.1	Ratio of tension reinfrocement	As / bw.d =	r	0.0028		
	vb = smaller of $(.07+10r)\sqrt{Fc'}$	OR $.2\sqrt{Fc'}$				
	But not less than .08 x Fc'^.5		vb	0.538	Mpa	
	Aggregate size factor		ka	1.0		
	Effective depth factor		kd	0.96		
		vb x ka x kd =	vc	0.518	Mpa	
	Nominal shear strength provided by concrete					
		vc.Acv =	Vc	72.5	kN	
			Shear steel not required			

emajineer Structural Concepts

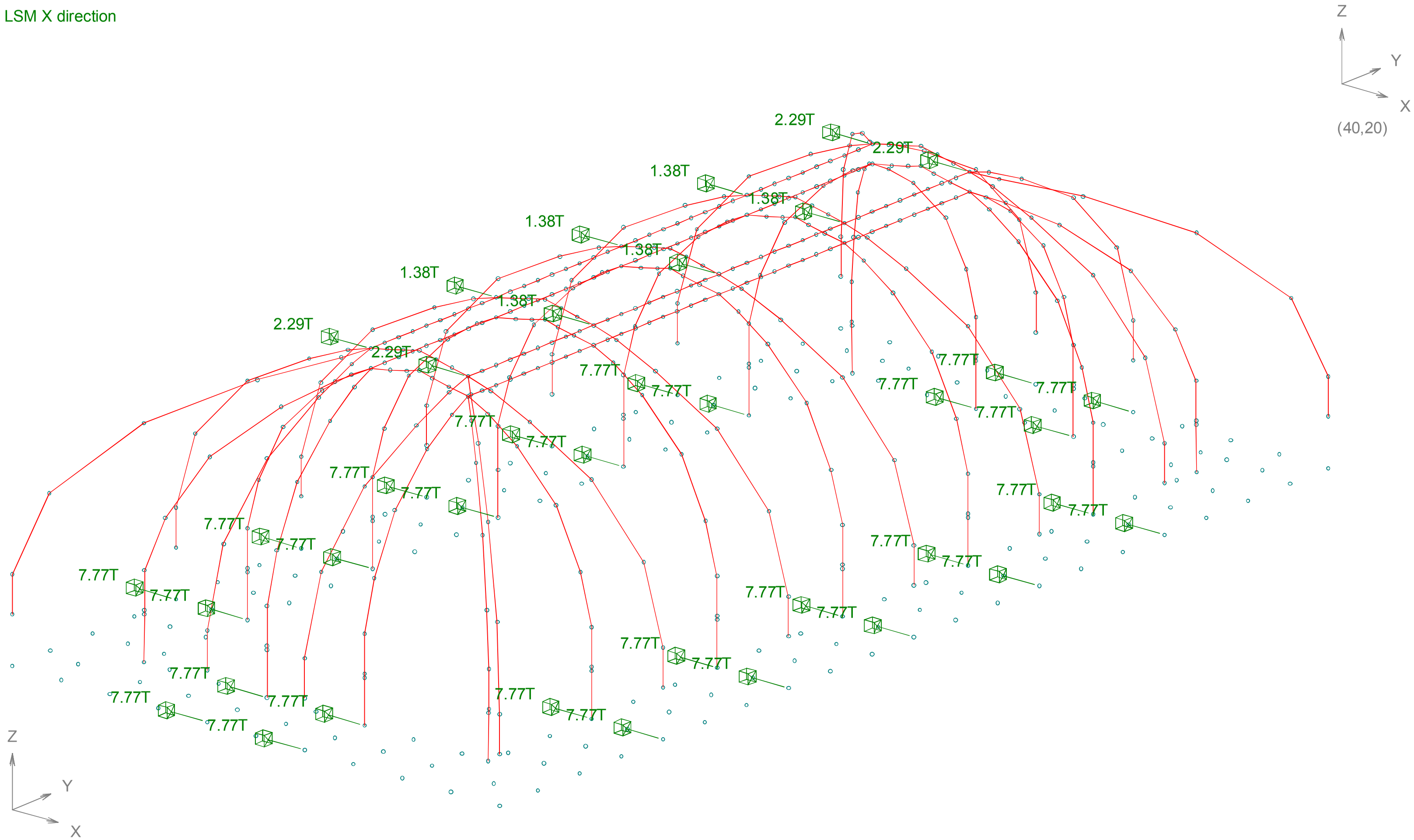
Client:	Christchurch City Council	Ref:	1923-2245
Project:	Cunningham Glasshouse	Date:	10/2/14
	Botanical Garden, Christchurch	BY:	NS
Subject:	Concrete Beam Check		

REINFORCED CONCRETE RECTANGULAR BEAM DESIGNED CONTINUED — Sheet No.: 32

Ref:	Design					Output
9.3.9.4.15	Minimum shear steel requirement					
		Bar dia	dia	10	mm	
		No. legs		2		
	$$Av = \frac{1}{16}\sqrt{f'c}\frac{bw.s}{fyt}$$	Spacing	S	300	mm	< 233.53125 mm
		Min Area	Av	102.7	mm²	FAILED
		Area provided		157.1	mm²	> 102.7 mm²
9.3.9.3.6	Shear reinforcement required when vn>vc					PASS
	$$Vs = Av.fyt.\frac{d}{S}$$		Vs	73.4	kN	
7.5.3	Shear strengt	(Vc + Vs) x f =	Vn	124.0	kN	> 10 kN
						PASS

Ref:	Design				Output
	Slab Thickness	t=	120	mm	
	Maximum Force	N*=	55	kN/m	
	Reinforcing Size	D=	12	mm^2	
	Centres	s=	450	mm	
		Area	273.1	mm^2/m	
	Steel Grade	Fy=	300	MPa	
	Allowable tension force	Fn	81.94	kN/m	PASS

Load cases:
2 LSM X direction



Materials:
1 STEEL
2 CONCRETE-32

Sections:
2 75 CT 12

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
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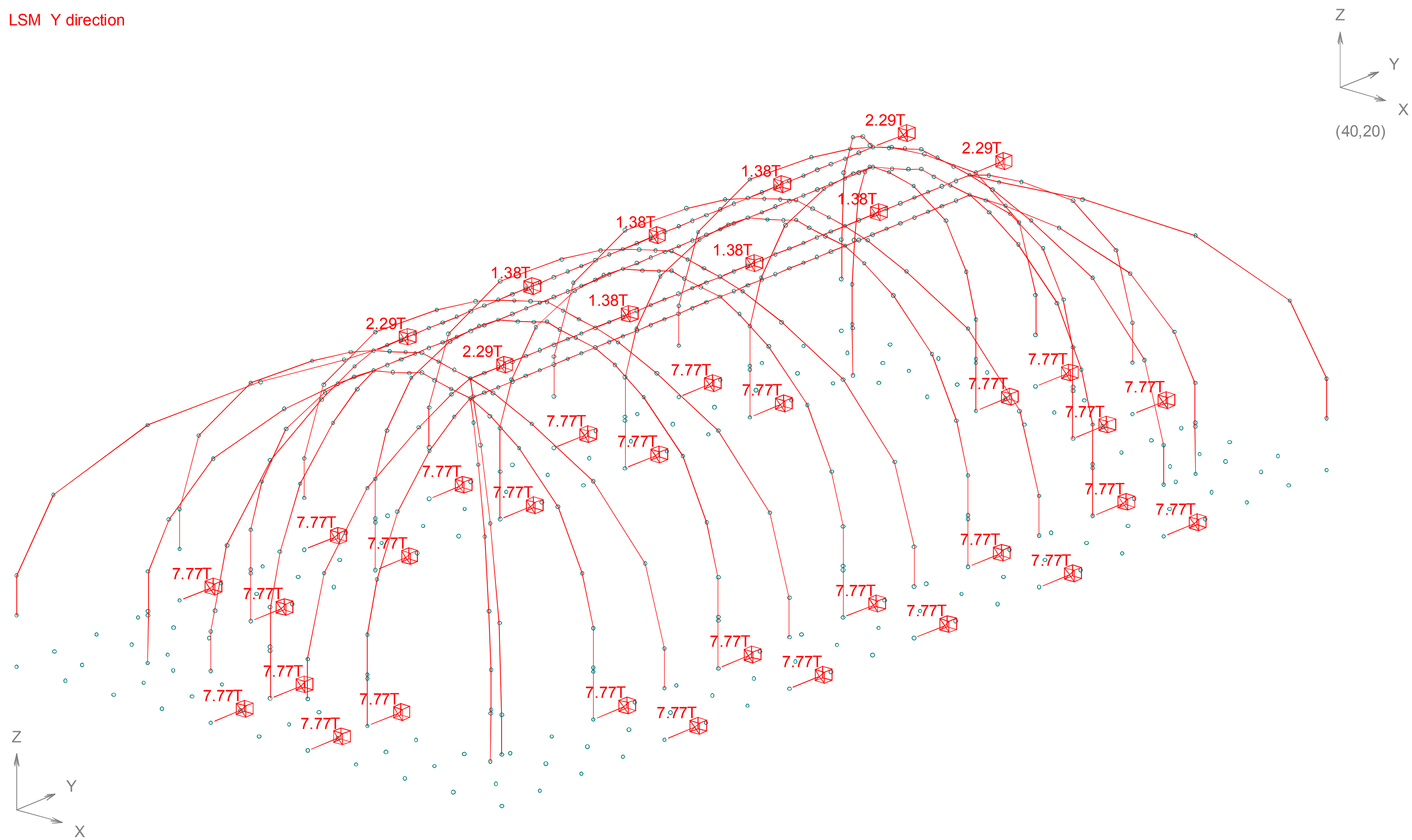
Load cases:

■ 7 ——— LSM Y direction



Materials:

■ 1 STEEL

■ 2 CONCRETE-32

Sections:

■ 2 75 CT 12

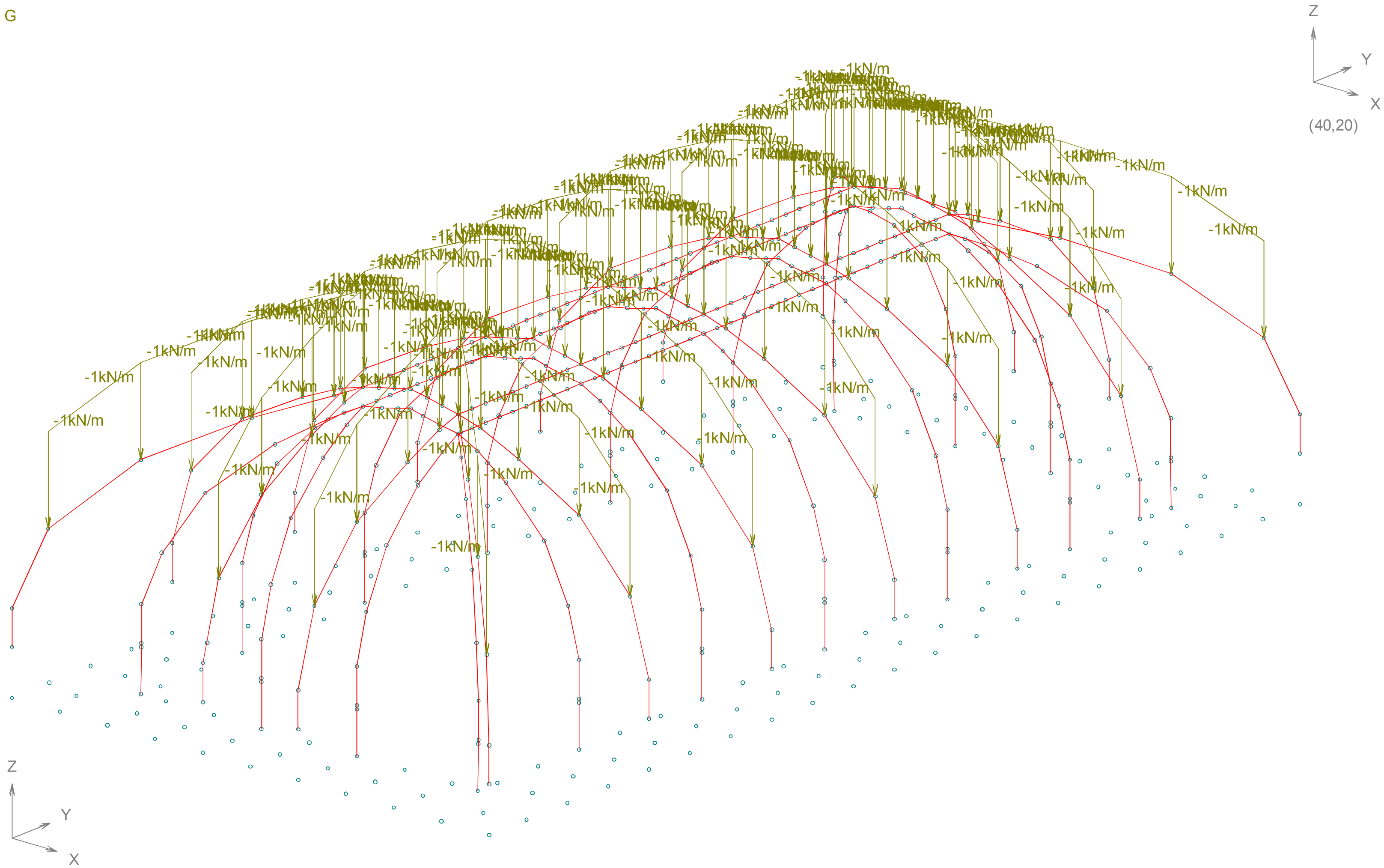
Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic

Filter: Steel frame

Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa

Scales - Frame: 1:140, Load: 0.056205, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

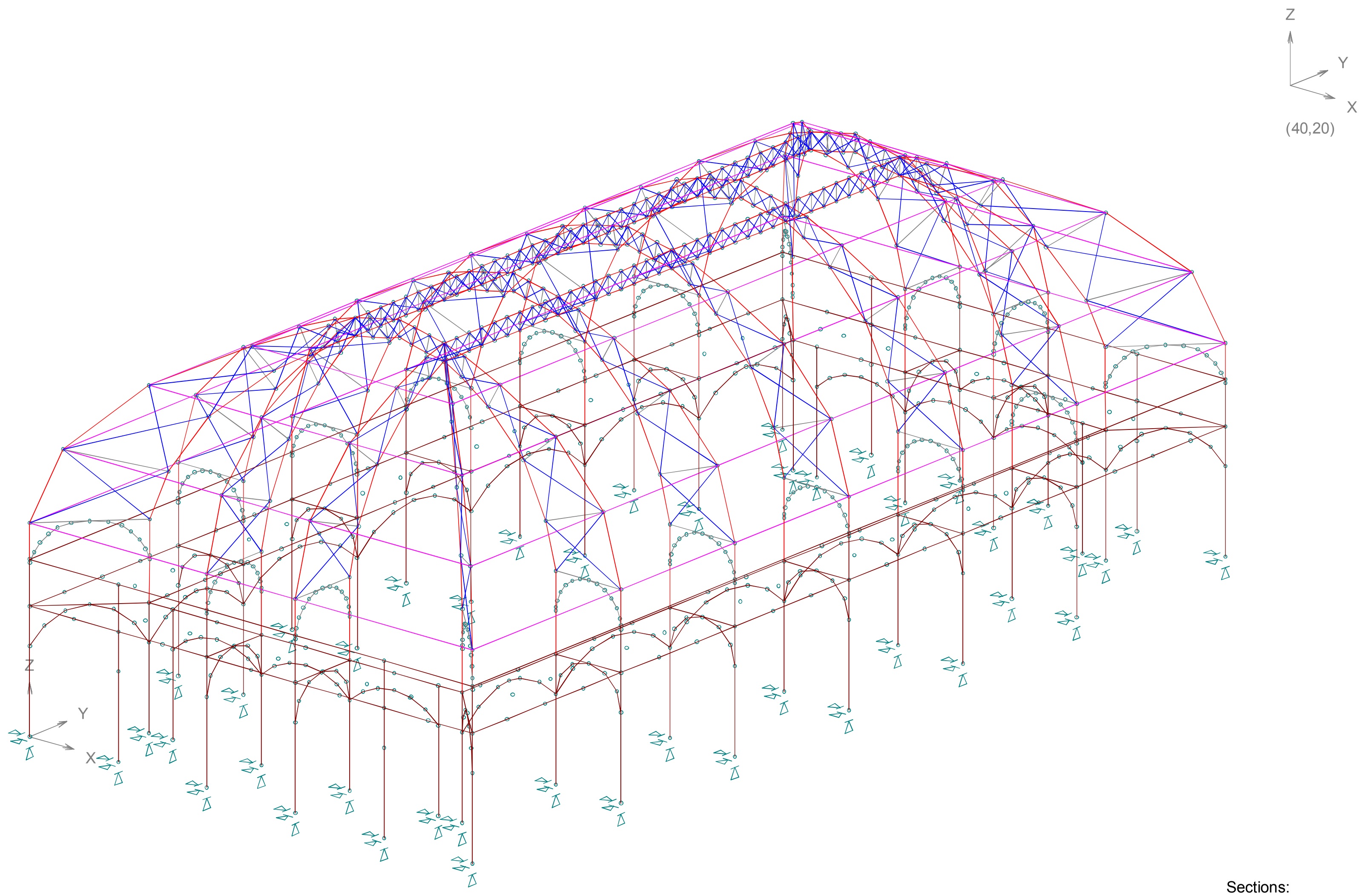
Load cases:
1 (SW) G



Materials:
1 STEEL
2 CONCRETE-32

Sections:
2 75 CT 12

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: Steel frame
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:140, Load: 0.056205, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None



Sections:
1 double 50*5 EA
2 75 CT 12
3 Steel flats
4 100 UC 15
5 Pillar 450x450

Materials:
1 STEEL
2 CONCRETE-32

No general restraint

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
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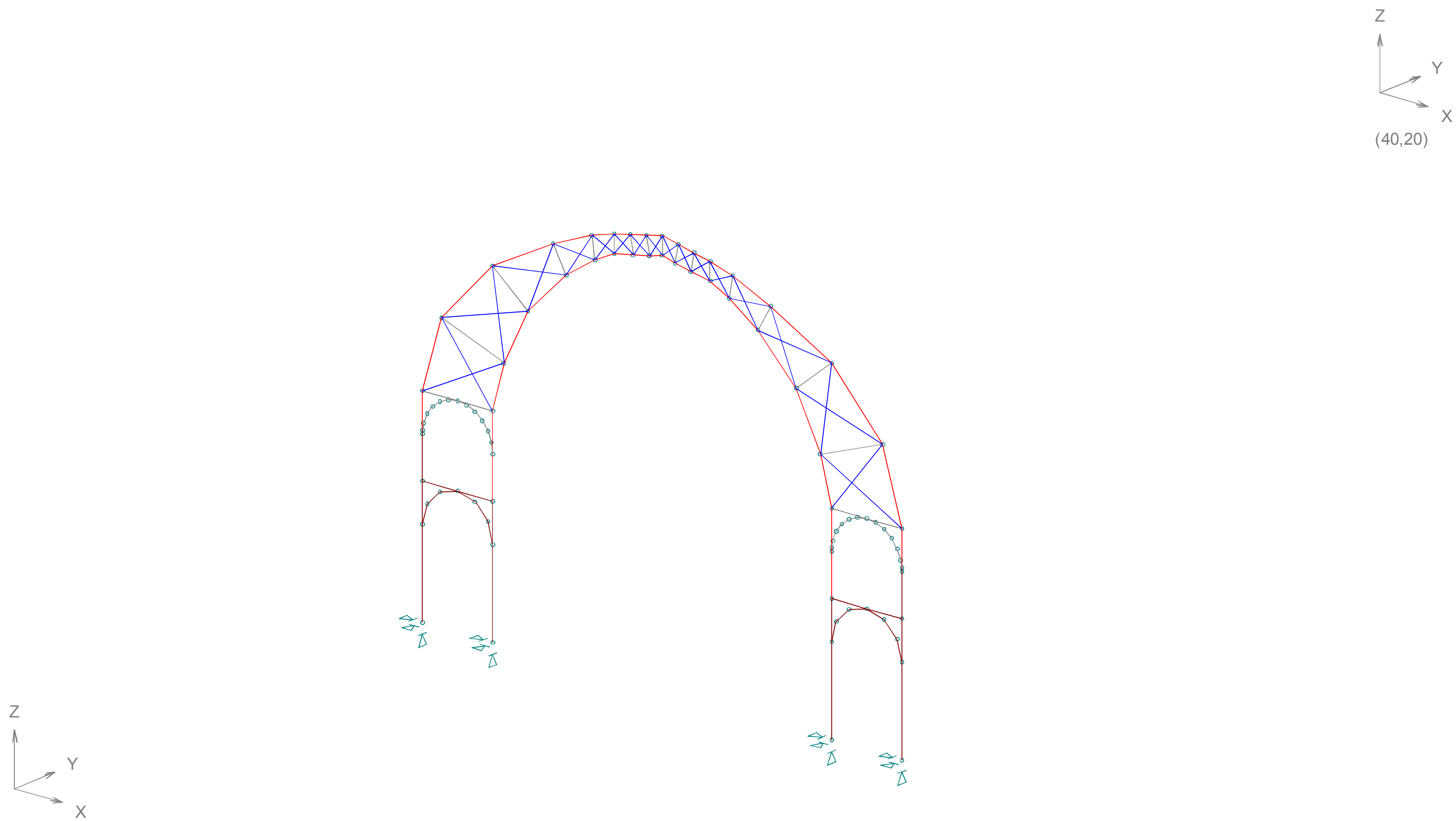

Sections:
1 double 50*5 EA
2 75 CT 12
3 Steel flats
5 Pillar 450x450

Materials:
1 STEEL
2 CONCRETE-32

No general restraint

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: frame
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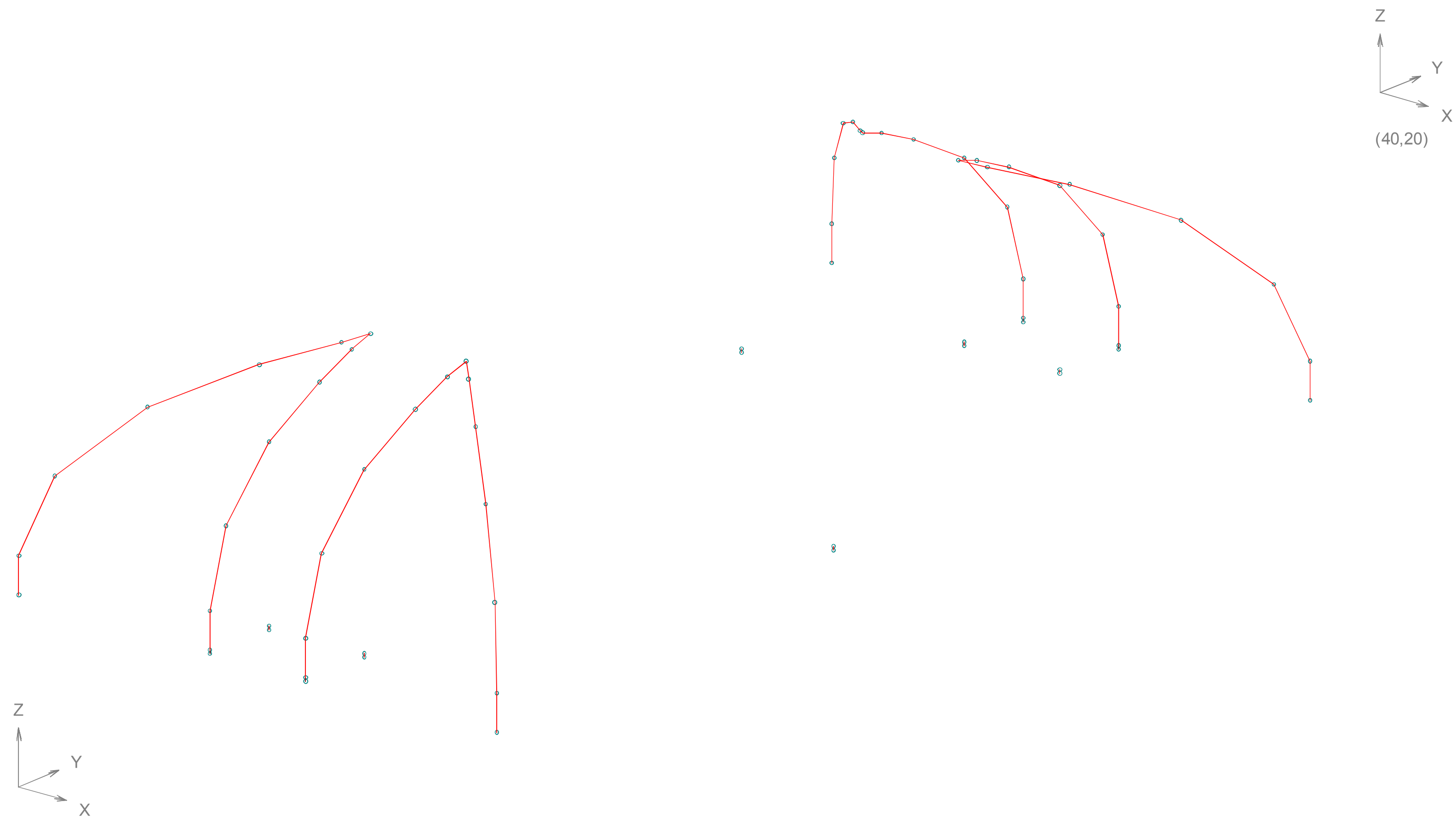

No general restraint

Materials:	
1	STEEL
2	CONCRETE-32

Sections:	
2	75 CT 12
5	Pillar 450x450

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: end Frame ext chords
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
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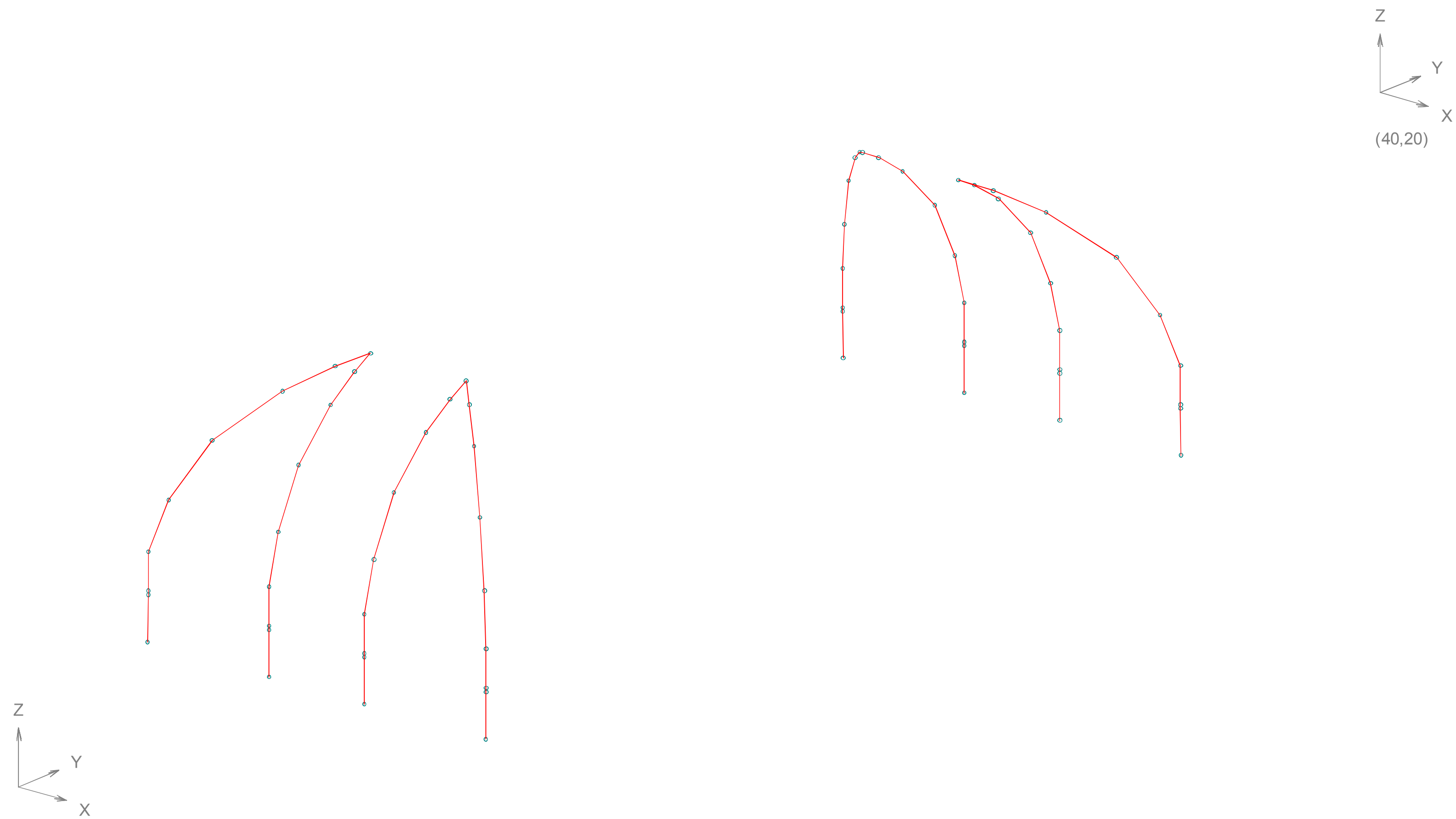
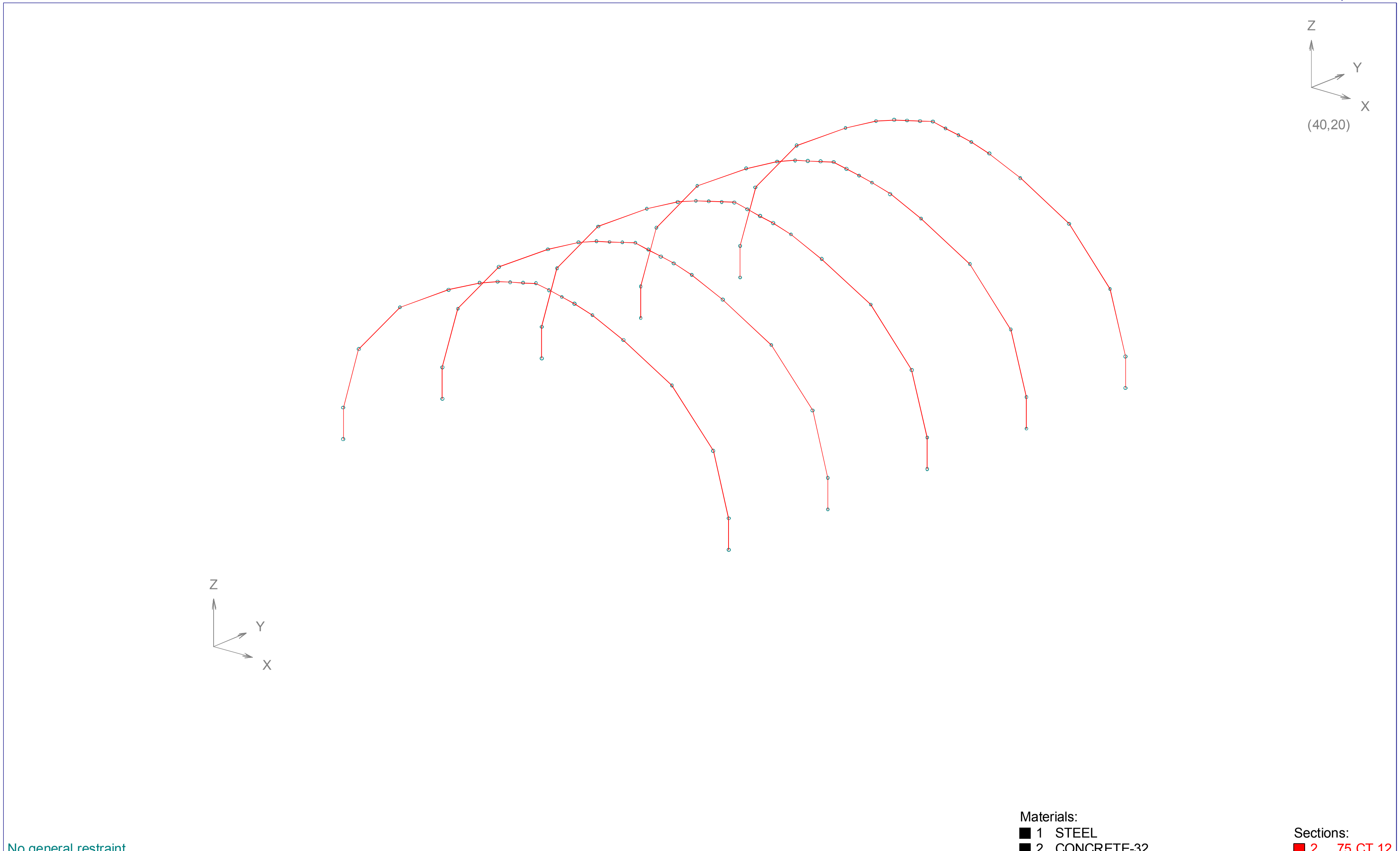


No general restraint

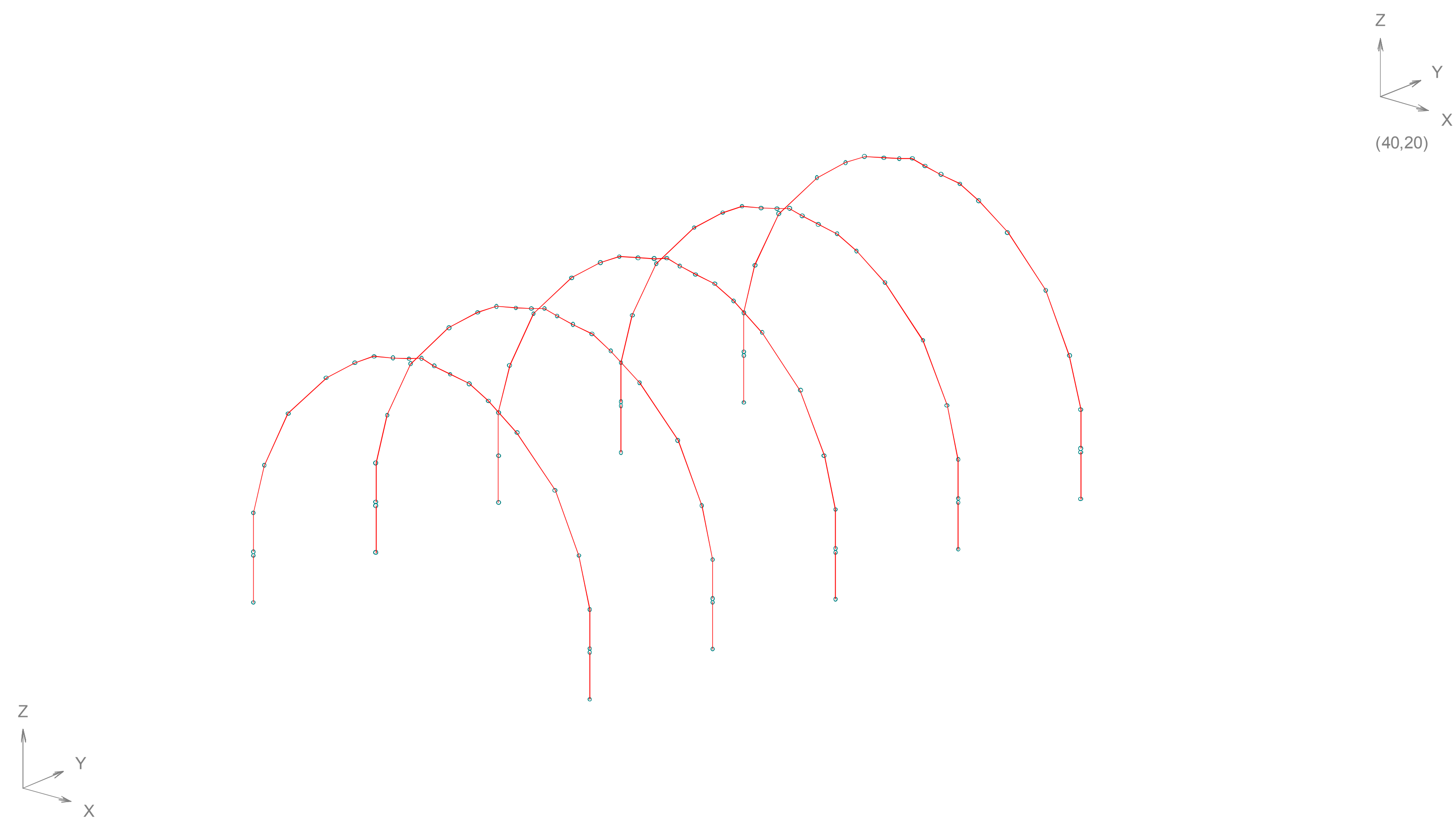
Materials:
1 STEEL
2 CONCRETE-32

Sections:
2 75 CT 12

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: End frame internal chords
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:150, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None



Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: main frame ext chord
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:150, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

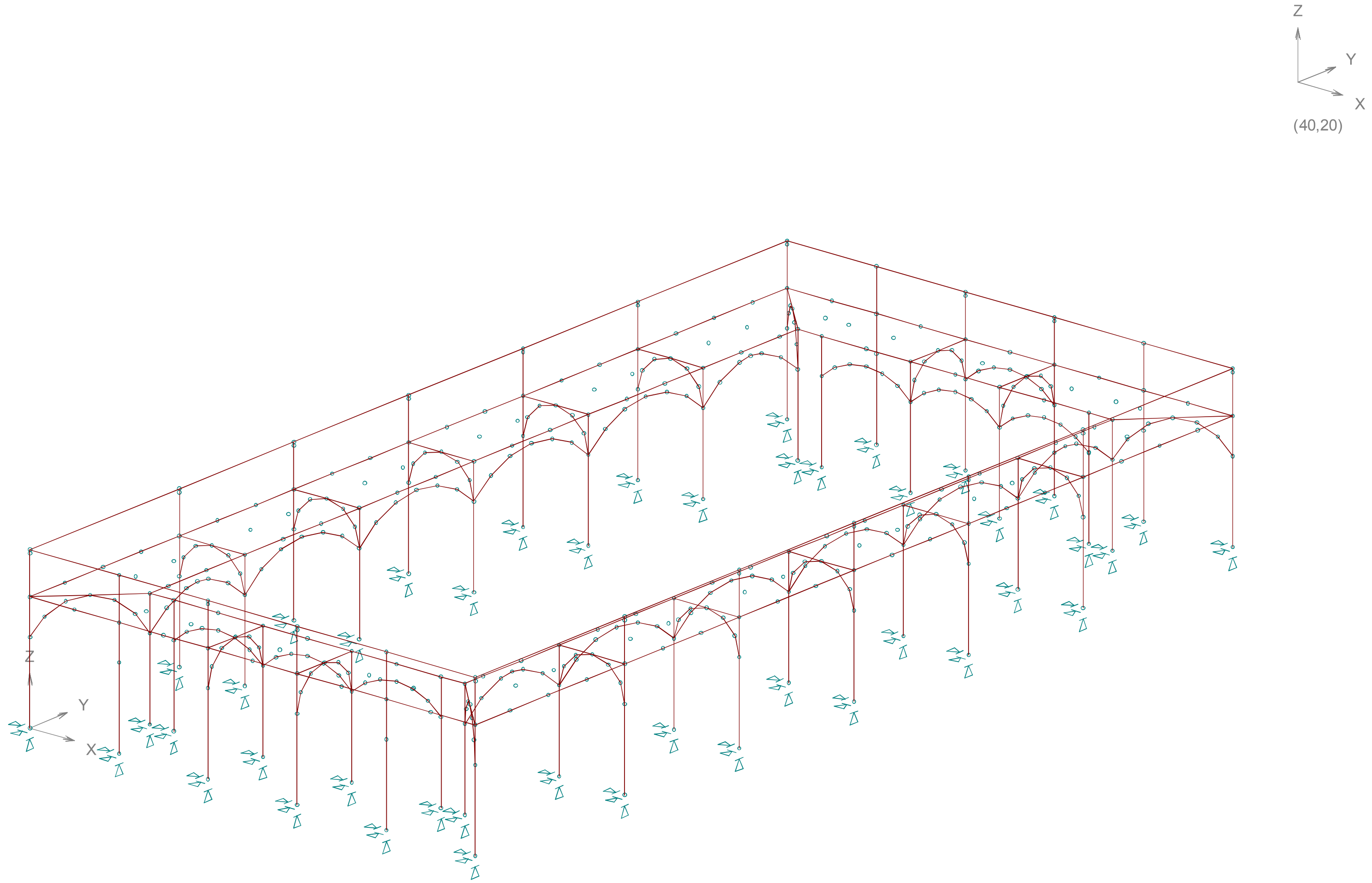


No general restraint

Materials:
■ 1 STEEL
■ 2 CONCRETE-32

Sections:
■ 2 75 CT 12

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: main frame int chords
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:150, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None

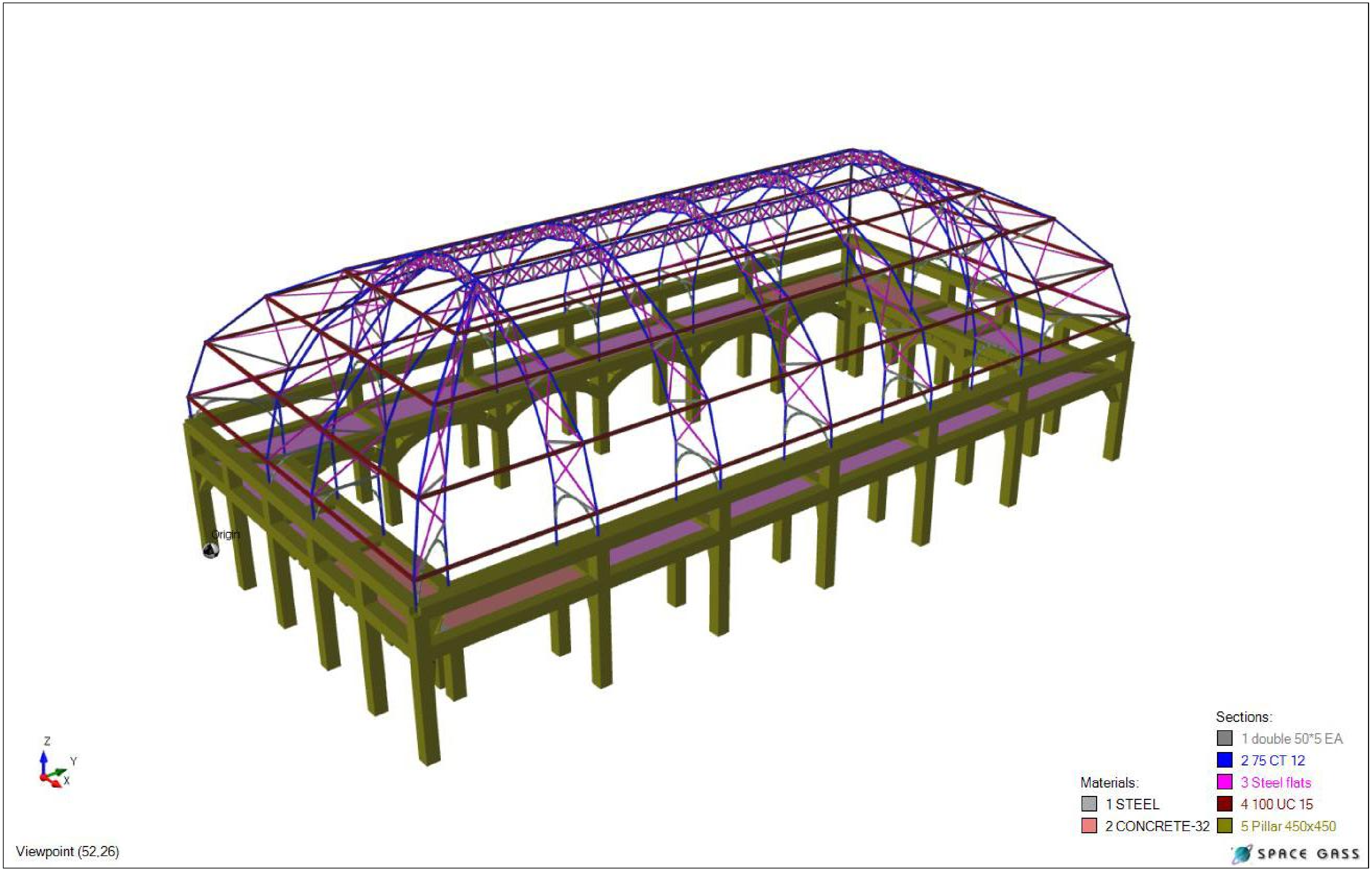


No general restraint

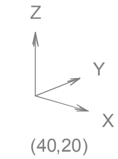
Materials:
■ 1 STEEL
■ 2 CONCRETE-32

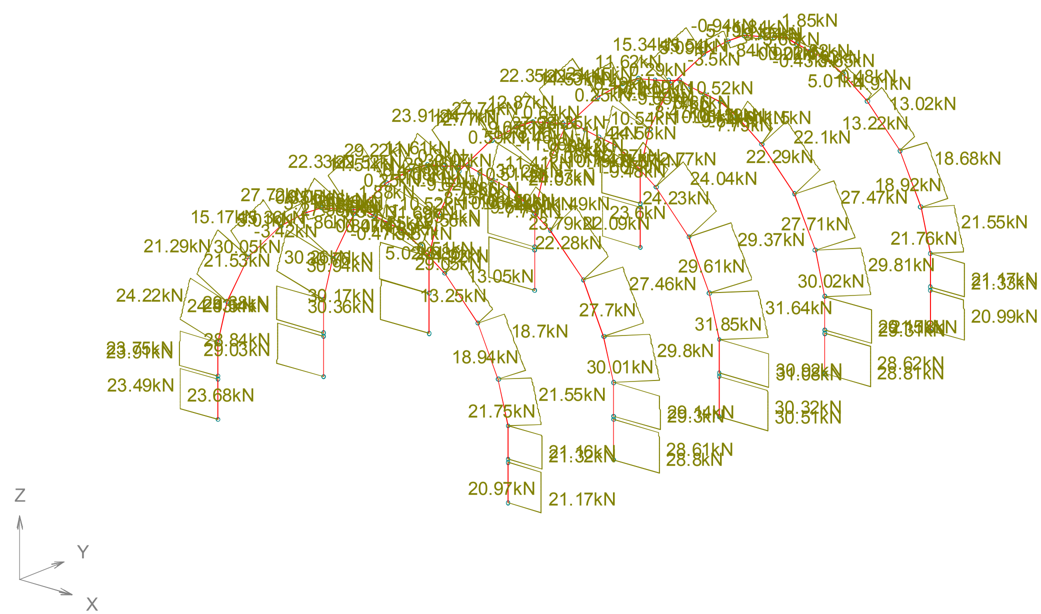
Sections:
■ 5 Pillar 450x450

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: CONCRETE
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:150, Load: None, Disp: None, Moment: None, Shear: None, Axial: None, Torsion: None


Origin
Z
Y
X
Viewpoint (52,26)
Materials:
1 STEEL
2 CONCRETE-32
Sections:
1 double 50*5 EA
2 75 CT 12
3 Steel flats
4 100 UC 15
5 Pillar 450x450
SPACE GASS

Load cases:
51 (SW) 1.35G





No general restraint

Materials:
1 STEEL
2 CONCRETE-32

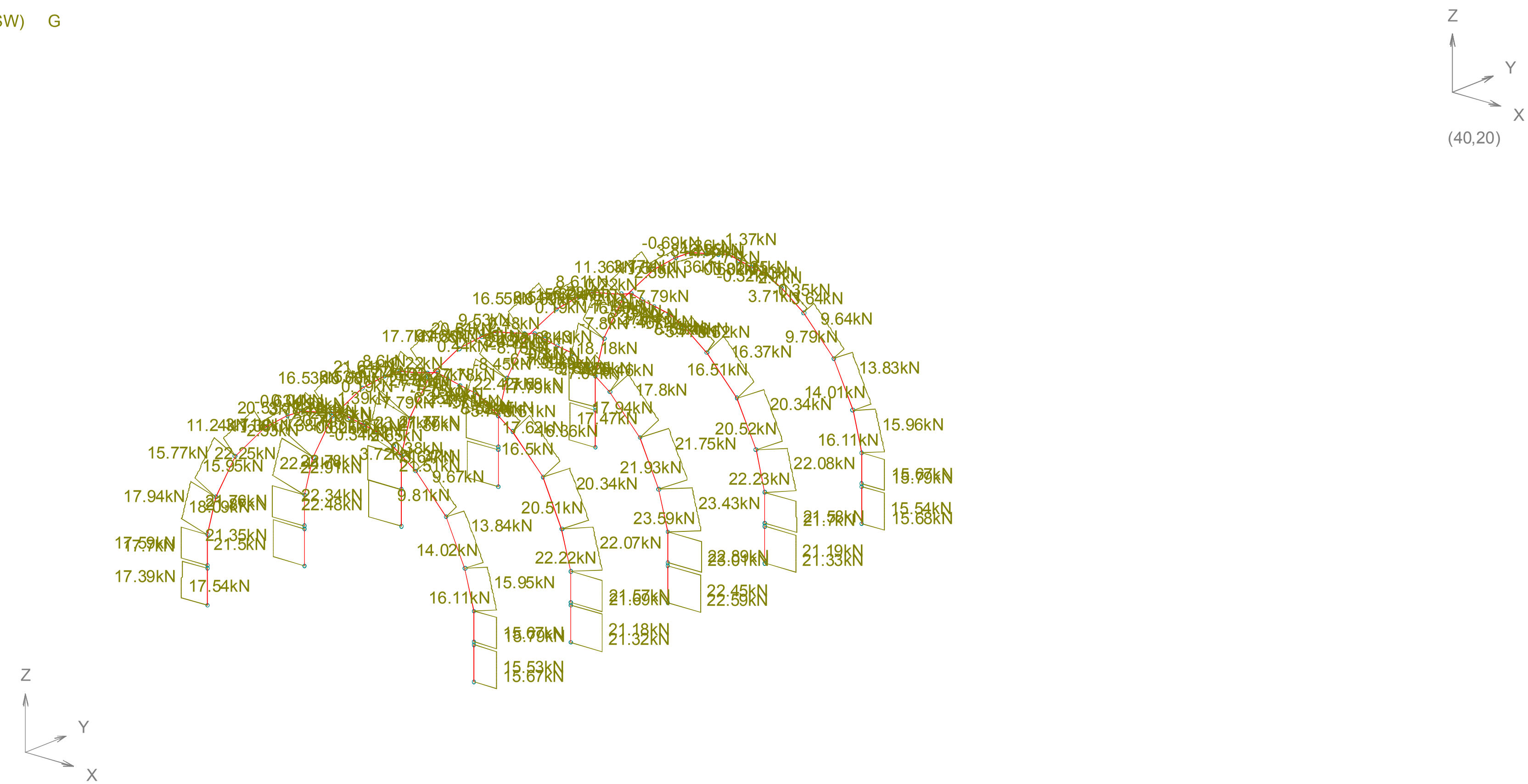
Sections:
2 75 CT 12

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: main frame int chords
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:190, Load: None, Disp: None, Moment: None, Shear: None, Axial: 3.264, Torsion: None

SPACE GASS 11.09 - STRUCTURAL CONCEPTS LTD

6 Mar 2014, 3:38 pm

Load cases:
1 ——— (SW) G



No general restraint

Materials:
■ 1 STEEL
■ 2 CONCRETE-32

Sections:
■ 2 75 CT 12

Job: \\Sclnapsbs001.scl.local\Data\Engin...\2245\1923-2245 140210 SGA3d dynamic
Filter: main frame int chords
Units - Len: m, Sec: mm, Mat: MPa, Dens: T/m^3, Temp: Celsius, Force: kN, Mom: kNm, Mass: T, Acc: g's, Trans: mm, Stress: MPa
Scales - Frame: 1:190, Load: None, Disp: None, Moment: None, Shear: None, Axial: 3.264, Torsion: None

APPENDIX E – CERA FORM

Detailed Engineering Evaluation Summary Data

V1.11

Location

Building Name:	Cunningham House Glasshouse	Reviewer:	Garry Newton
Unit / No: / Street		CPEng No:	65305
Building Address:	Botanical Gardens	Company:	Structural Concepts Ltd
Legal Description:		Company project number:	1923-2245
		Company phone number:	06 8420111
GPS south (Degrees / Min / Sec):		Date of submission:	28/02/2014
GPS east (Degrees / Min / Sec):		Inspection Date:	
		Revision:	
Building Unique Identifier (CCC):		Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	silty sand	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	Nil
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	6.00

Building

No. of storeys above ground:	2	single storey = 1	Ground floor elevation (Absolute) (m):	6.60
Ground floor split?	no		Ground floor elevation above ground (m):	0.60
Storeys below ground	0			
Foundation type:	pads with tie beams		if Foundation type is other, describe:	
Building height (m):	11.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	350			
Age of Building (years):	93		Date of design:	Pre 1935
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	commercial		Brief strengthening description:	
Use (upper floors):	commercial			
Use notes (if required):	Glasshouse			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:	steel truss	truss depth, purlin type and cladding	varies,steel T, glass
Floors:	concrete flat slab	slab thickness (mm)	120
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	600x450
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	450x450
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	non-ductile concrete moment frame		note typical bay length (m)	
Ductility assumed, μ:	1.25			
Period along:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	30		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	10		estimate or calculation?	estimated
Lateral system across:	non-ductile concrete moment frame		note typical bay length (m)	URM shear walls
Ductility assumed, μ:	1.25			
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	30		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	10		estimate or calculation?	estimated

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

	Stairs:	cast insitu	notes	
	Wall cladding:	other heavy	describe	concrete walls and glass
	Roof Cladding:	Other (specify)	describe	glass
	Glazing:	aluminium frames		
	Ceilings:	none		
	Services(list):	lights		

Available documentation

	Architectural	partial	original designer name/date	unknown
	Structural	partial	original designer name/date	unknown
	Mechanical	none	original designer name/date	
	Electrical	none	original designer name/date	
	Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)	Site performance:	average	Describe damage:	no liqefaction near building
	Settlement:	none observed	notes (if applicable):	
	Differential settlement:	none observed	notes (if applicable):	
	Liquefaction:	none apparent	notes (if applicable):	
	Lateral Spread:	none apparent	notes (if applicable):	
	Differential lateral spread:	none apparent	notes (if applicable):	
	Ground cracks:	none apparent	notes (if applicable):	
	Damage to area:	moderate to substantial (1 in 5)	notes (if applicable):	

Building:	Current Placard Status:	yellow		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	assumed level of cracking and loss of chimneys
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	minor cracks to walls

$$Damage\_Ratio = \frac{(\%NBS\,(before) - \%NBS\,(after))}{\%NBS\,(before)}$$

Recommendations

	Level of repair/strengthening required:	minor non-structural		Describe:	cracking to walls
	Building Consent required:	no		Describe:	
	Interim occupancy recommendations:	full occupancy		Describe:	
Along	Assessed %NBS before:	59%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	dynamic analysis and spreadsheet calculation
	Assessed %NBS after:	59%			
Across	Assessed %NBS before:	59%	##### %NBS from IEP below		
	Assessed %NBS after:	59%			