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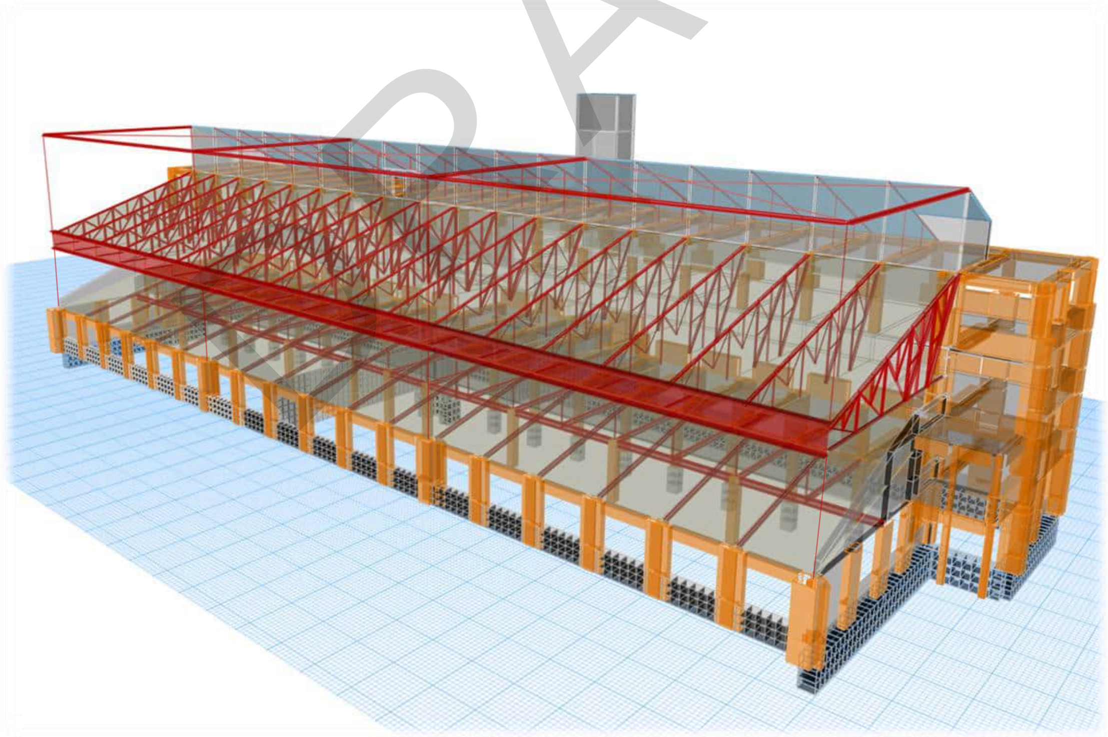
Grand National Stand - Detailed Damage Evaluation

3D Response Spectrum, 3D Non-linear Pushover & Vertical analysis



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Client: Canterbury Jockey Club

ABN: N/A

Prepared by

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			Name/Position	Signature
Draft	27-Jan-2016	Draft for comments	Mark Ferfolja Associate Director	

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Executive Summary

AECOM was engaged by the Canterbury Jockey Club (CJC) to undertake a Detailed Damage Evaluation (DDE) of the Grand National Stand (GNS) also known as the "Public Stand" at Riccarton Racecourse in Christchurch.

This report was prepared as an extension to AECOM's 2D structural assessment of the building (completed in July 2015) and is the third published quantitative assessment of the building.

This report was completed in close consultation with the Insurer's Engineer, Thornton Tomasetti (TT). During the pre-analysis phase, assumptions and design approach were discussed and documented. In the event that consensus could not be reached, AECOM adopted the preferences of TT (a condition of AECOM's engagement). During the analysis phase, weekly meetings were conducted to discuss results, and work packages were submitted on a weekly basis for TT review.

This DDE report documents the pre-earthquake and current seismic capacity of the building in terms of the New Building Standard (NBS) as defined by NZS 1170.5:2004 – Earthquake Actions. As agreed between CJC and their insurer, this report also includes a gravity assessment of the primary structural elements of the building undertaken to gain an appreciation of the building's capacity for typical "in-service" loads (e.g. gravity, wind and snow). This report does not consider strengthening or retrofit options, as these were outside the scope of AECOM's engagement.

AECOM is of the opinion that the results from this DDE should supersede previous historical assessments, as this study includes a non-linear, 3D analysis of the entire building and, to date, most accurately reflects the buildings' response to seismic excitation.

As no structural drawings of the building exist, this report also captures the outcomes of all intrusive investigations conducted on site. It should be noted that whilst these intrusive investigations refine a number of structural and geometric assumptions, it is impractical, and in some instances impossible, to entirely eliminate many assumptions. A limited number of elements were investigated intrusively, and these investigations generally revealed a higher degree of variability in detailing than previously assumed, leading to a more significant margin of uncertainty for many elements, and therefore the subsequent analysis.

A detailed 3D ETABS model was constructed to evaluate the building's seismic performance. AECOM used this model to complete two types of analysis; a 3D modal response spectrum analysis (RSA) and a 3D non-linear pushover analysis (NLPO). Gravity, wind and snow assessments for the building were completed using simple 2D sub assemblies of the building.

Being comparatively crude but efficient, the RSA was used to initially evaluate the building prior to the commencement of the NLPO. The RSA assisted in:

- Developing an appreciation of the overall behaviour of the building including its torsional response,
- Determining the period of vibration of the building,
- Providing a lower-bound capacity of selected structural elements.

The RSA revealed that the seismic capacity of the building is approximately 2%NBS to 5%NBS, with flexure of the beams governing failure.

The NLPO was used to evaluate post-elastic behaviour of the building and to determine capacity in terms of %NBS. The NLPO technique provides a more accurate tool for the assessment of capacity, as it better mimics actual building behaviour. To fully understand the building's seismic performance, multiple pushover analyses were performed with bi-directional loads applied orthogonally.

Based on the NLPO, the seismic capacity of the building is governed by brittle shear failure of the beams connecting the elevator core to the main structure. This mode of failure was observed in all of the four NLPO analyses. The pre-earthquake capacity of the building is governed by the "push" in the south direction, and is estimated to be approximately 8%NBS. It is noted that the results of this analysis do not materially change the findings of our work completed in July 2015, which concluded a %NBS between 11%NBS and 18%NBS.

It should be noted that the seismic %NBS values noted above are based on potentially non-conservative assumptions, and an optimistic position has been taken by AECOM on matters relating to bond slip behaviour, adequacy of lap lengths, splices of embedded steel sections, adequacy of confinement reinforcement and concrete strength.

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In our opinion, the building is likely to collapse in a moderate earthquake as there are a number of Critical Structural Weaknesses (CSW's) which include an inverted shear wall arrangement on gridline C, the use of plain round bar reinforcement throughout the building (with uncertain / irregular lap lengths) and extremely low reinforcement ratios in concrete elements promoting rapid degradation when subjected to cyclic loading (noting that the lowest bound failure mechanism is a brittle shear failure of primary beams).

AECOM is of the opinion that the building is "earthquake prone" when considered within the context of the NZ Building Act 2004 based on the assessed seismic %NBS being less than 34% and our opinion that the building would be likely to collapse in a moderate earthquake.

The capacity of the circular steel columns supporting the upper stand is approximately 54%NBS based on the RSA. Investigation in the NLPO indicated that the columns do not fail at the maximum displacement achieved in the nonlinear analyses. As a target displacement corresponding to 100%NBS has not been reached, the %NBS of these columns cannot be more meaningfully determined. It is recommended that the capacity of these columns be considered in any potential retrofit / strengthening scheme.

A gravity, wind and snow assessment considering only the strength performance of the building was also undertaken. For the purpose of undertaking this assessment, the structure was divided into a number of sub-assemblies including the primary frame, roof, upper stand, lower stand and internal stairs. In summary this analysis revealed that:

- The majority of the building frame meets current code requirement, with the exception of the primary beams which appear to have been designed for 2kPa – 3kPa (modern codes require the live load capacity to be 5kPa),
- The roof framing has several deficiencies including the bottom chord of the girder truss which is unrestrained and is unstable and a number of elements do not achieve minimum strength criteria including typical roof trusses (20%NBS), purlins (15%NBS), and the girder truss to steel columns connections (55%NBS),
- The upper stand retains approximately 90%NBS.
- Generally the lower stand did not satisfy code defined gravity loads with capacities between 60%NBS and 80%NBS. The framing does however satisfy a "credible lower bound" live load of 2.5kPa,
- The internal stairs and platforms are generally satisfactory with the exception of the stair between Lvl 2 to Lvl 3 which has 70%NBS capacity,

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1.0 Introduction

1.1 Overview

AECOM New Zealand Ltd (AECOM) has been engaged by the Canterbury Jockey Club (CJC) to undertake a three dimensional detailed quantitative seismic analysis and a wind and gravity assessment of the Grand National Stand (GNS) for the Club. The facility is located at Riccarton Park Raceway, 165 Racecourse Road, Christchurch. This report will henceforth be referred to as a Detailed Damage Evaluation (DDE).

This report has been prepared as an extension to AECOM's 2D assessment and detailed damage evaluation of the building subsequent to the 4 September 2010, 22 February and 13 June 2011 earthquakes and subsequent aftershocks. This sequence of earthquakes will henceforth be referred to as the "Canterbury earthquakes" in this report.

1.2 Scope

Scope meetings were conducted prior to commencement of analysis. The details of the scope were largely agreed prior to commencement and partially refined during the analysis process as data became available. Refer to Appendix D for initial minutes of scope meetings and clarifications (dated; 16.09.15, 18.09.15, 24.09.15, 01.10.15 & 19.10.15).

1.3 Related reports

This report should be read in conjunction with the following related reports for the building:

- Damage Assessment Report (DAR), dated 14 July 2015 prepared by AECOM
- Design Features Report (DFR) – 3D – Grand National (Public) Stand, dated 29 July 2015 prepared by AECOM (refer to Appendix A)

Refer to the DAR for the following information:

- site description,
- site seismic records,
- detailed damage assessment of the building,
- floor level and verticality surveys,
- material sampling and testing,
- detailed photographical record.

Refer to the DFR for the following information:

- scope of the analysis,
- detailed building description,
- structural layout and load paths,
- soil properties,
- geometric assumptions,
- loading assumptions,
- analysis methodology,
- material properties.

When considered appropriate, some of the information contained in the above mentioned reports has been reproduced in this report.

Contained within Appendix B are the site memoranda, which detail the intrusive investigations undertaken on site, and the findings of these investigations.

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1.4 Purpose of this report

The purpose of this DDE is to:

- Evaluate the pre and post-earthquake seismic capacity of the building in terms of percentage of new building standard (%NBS, i.e. NZS1170.5:2004-Earthquake Actions),
- Locate the hierarchy of failure that could be used (if desired) to focus progressive strengthening efforts,
- Assess the wind and gravity performance of the building against modern building codes.

This report does not include indicative repair solutions nor any conceptual strengthening schemes.

1.5 Building Code requirements

1.5.1 New buildings

The Building Code specifies the current loading code NZS 1170:2002-Structural Design Actions as a means of compliance with the Building Act in terms of the structural strength required for new buildings. Accordingly, the earthquake loading component of this loading code, NZS 1170.5:2004-Earthquake Actions has been used to define the New Building Standard (NBS) in this investigation.

1.5.2 Increase of Christchurch Earthquake Standard

As a result of the recent earthquakes in Canterbury, the seismic hazard factor in the NZ loadings code NZS1170.5 has been increased from 0.22 to 0.3. This change effectively increased the design ultimate seismic loads applied to buildings by 36%. This means that a building designed to meet 100% of NZS1170.5 before this change was effected, would now meet approximately 73%NBS.

1.5.3 Earthquake-Prone Building

The Building Act 2004 and associated regulations define any building which has a seismic capacity of less than or equal to one third of that required for a similar new building (i.e. <34%NBS) and would be likely to collapse in a moderate earthquake causing injury or death to persons in the building or to persons on any other property; and or damage to any other property as an “Earthquake Prone” building.

1.5.4 Earthquake-Risk Building

The New Zealand Society for Earthquake Engineering considers that any building meeting a seismic capacity of at least two thirds of that required for a new building (i e. > 67%NBS) has reached an adequate standard and does not need to be considered as an earthquake risk. Buildings with seismic capacity less than 67%NBS are deemed an “Earthquake Risk” building. The NZSEE strongly recommends every effort be made to achieve improvement to at least 67% NBS. Strengthening a building from 34% NBS to 67% NBS will reduce the relative risk of the building from around 20 times to 3 times that of a new building.

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2.0 Building description

A brief summary of the building is provided below and in Table 2-1 and Table 2-2 and illustrated in Figure 2-1. Refer to the DFR in Appendix A for a detailed description of the Grand National Stand. The as-built drawings and damage status form part of the DAR.

The Grand-National Stand is a four storey reinforced concrete structure with timber grandstands, built circa 1920. The lateral load-transfer systems are predominantly moment frames with some shear walls also present throughout the building.

The Grand National Stand is a heritage building and is listed as Group 4 in the Christchurch City Council (CCC) South-West Christchurch Area Plan: Phase 1 Report – European Cultural Heritage.

Table 2-1: Building Summary

Grand National Stand	
Total Length	~ 82 m
Total Width	~ 25 m
Total Height	~ 18.6 m
Importance Level (IL)	3
Number of Stories	5 floor levels 2 grandstands
Total Plan Area (Approximate)	7700m^2

Table 2-2: Level-by-level Building Information

Level	Occupancy	Area	Storey Height
Ground	Workshop & Storage Public Access	1170 m^2 565m^2	0 m (reference level)
First	Public Access	1230 m^2	4 m
Lower Stand	Public Access	825 m^2	4 m – 7.7 m
Second	Public Access	1000 m^2	7.7 m
Third	Public Access	1065 m^2	11.5 m
Upper Stand	Public Access	1080 m^2	12.1 m – 16.4 m
Fourth	Maintenance Access Only	765 m^2	15.6 m
Roof	No Access	~ 2873 m^2	18.6 m

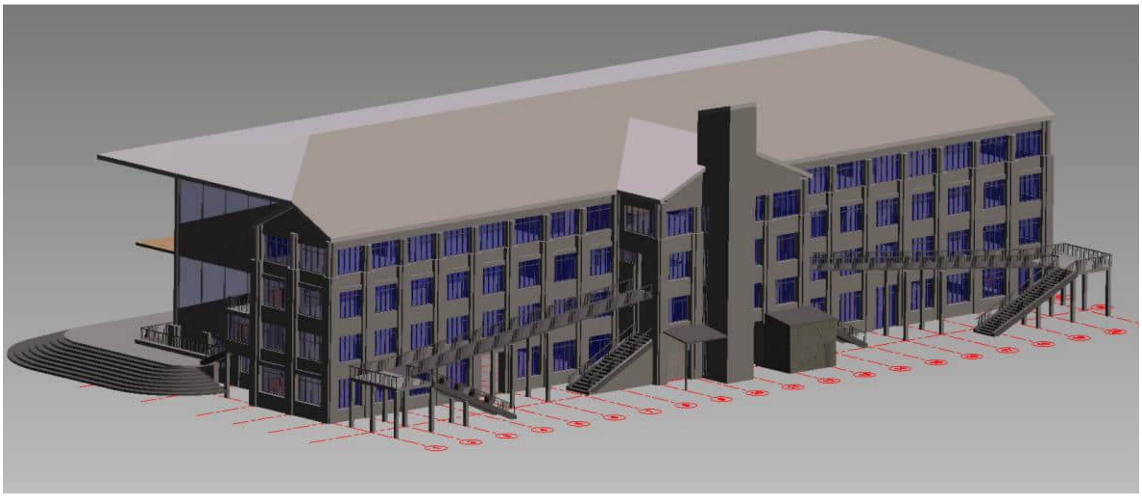

Figure 2-1: Grand National Stand layout

3.0 Assumptions

In the absence of the original construction drawings or specifications and in order to adopt realistic material and section properties, a programme of intrusive investigations and dimensional surveying has been completed for the Grand National Stand. The scanning of reinforcement, localized removal of concrete cover and selective material testing allowed AECOM to make calculated assumptions with regard to material properties and sections' reinforcement patterns.

It should be appreciated, that while these intrusive investigations refined a number of assumptions, it is impossible to entirely eliminate assumptions which are inherent for this type of assessment. For practical reasons only a limited number of elements could be investigated intrusively (beams, columns, beam-column joints, walls etc.) and these investigations generally indicated a high degree of variability in detailing. It has been assumed, for the purpose of this assessment, that the results from investigations could be used to infer the typical detailing of multiple elements. However, it should be noted that the level of uncertainty associated with these assumptions remains high.

The following sections outline some of the assumptions made in the analysis which have been broken down into two categories with some of the parameters adopted being potentially conservative while others being potentially optimistic.

For key parameters (e.g. geometry, material strengths, typical sections and reinforcement layouts) adopted in the analysis refer to DFR in Appendix A.

3.1 Potentially non-conservative assumptions

The following assumptions adopted are considered to be possibly non-conservative and may contribute to an overestimation of the %NBS seismic capacity of the building:

- It is assumed that existing lap lengths in reinforced concrete columns and beams can develop full capacity of reinforcement bars or steel sections encased in concrete elements (e.g. steel angles encased in concrete columns are assumed to have splices capable to develop full tensile capacity of the angle),
- Effects of bond slip due to round bars being used in reinforced concrete sections have not been considered,
- The assumed spacing, arrangement and sizes of reinforcement used in the assessment were based on the results of intrusive investigations with the "most typical" arrangements being adopted,
- Strength and stiffness degradation due to sustained, cyclic seismic loading has not been considered in the analysis,
- The concrete compressive strength used in the analysis is based on the limited concrete core tests and ignores the observed defects such as segregation and oversized aggregates.

3.2 Potentially conservative assumptions

The following assumptions adopted are considered to be potentially conservative and may contribute to an underestimation of the %NBS seismic capacity of the building:

- The shear capacity of elements with concrete encased steel sections has been calculated based on the shear capacity of the steel section only (i.e. concrete contribution ignored)

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4.0 Seismic assessment

4.1 Previous seismic assessments

AECOM is aware of two historical quantitative seismic assessments of the Grand National Stand which were carried out subsequent to the 2010 and 2011Canterbury Earthquake sequence:

- Detailed Engineering Evaluation (DEE) by Airey Consultants Ltd. (Airey), dated 20th August 2012 and subsequent e-mail correspondence between Airey and Canterbury Earthquake Recovery Authority (CERA)
- Detailed Damage Evaluation (DDE) by AECOM, dated 30 July 2015 (Draft)

Table 4-1briefly summarizes findings of these reports and provides additional commentary on the type of analysis performed and the level of intrusive investigation carried out to inform the analysis.

Table 4-1 Previous seismic assessments of the Grand National Stand

Report	%NBS	Type of analysis	Intrusive works
Airey DEE Report (20/8/2012)	37.8%	A single non-linear pushover (2d) on a typical frame in transverse direction	No intrusive works carried out.
E-mail 2/10/2012	25%		Limited scanning of reinforcement for the internal columns performed.
E-mail 15/10/2012	37.8%		
AECOM DDE Report (7/2015)	11-18%	Multiple (7) non-linear pushover (2d) analyses on frames in two orthogonal directions	Programme of intrusive works carried out and involved: - removal of linings in selected locations - breaking out of concrete in selected locations, - laboratory testing of materials (concrete and reinforcement bars), - foundation exposure - scanning of reinforcement.

AECOM considers that the analysis in this report supersedes the above assessments as it involves a 3d model of the entire building and most accurately reflects its response to seismic excitation. The report also captures outcomes of the additional intrusive works carried out subsequently to the above reports (see Appendix B).

4.2 Methodology of assessment

Two types of seismic analyses have been performed on the building:

- 3D modal response spectrum analysis (RSA),
- 3D non-linear pushover analysis (NLPO).

The RSA has been performed to gain appreciation of the overall behaviour of the building, its torsional response, to evaluate the period of vibration and assess the lower-bound capacity in terms of the current building code. Refer to section 5.1 of Appendix A for a detailed description of this procedure.

The NLPO is an analysis technique used to estimate the capacity of a structure beyond its elastic limit up to its ultimate strength in the post-elastic range. It is used to determine how progressive failure is likely to occur in buildings, and can identify the final failure mechanism. Refer to section 5.2 of Appendix A for a detailed description of this procedure.

A single 3D ETABS model has been utilized for all analyses of the building (see Figure 4-1 and Figure 4-2).

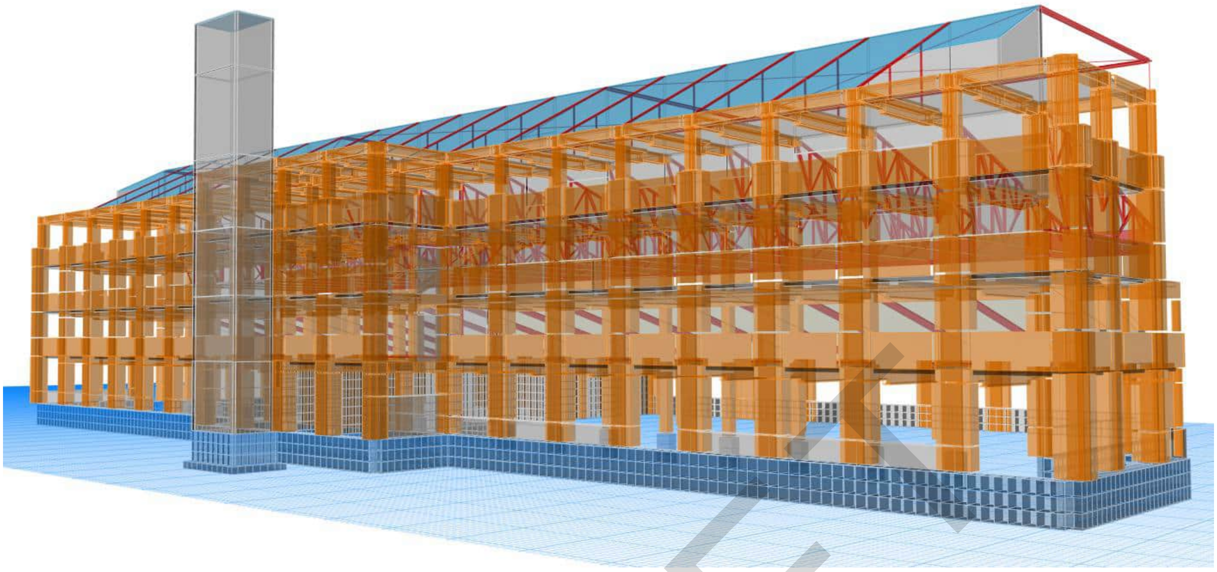

Figure 4-1 View of the ETABS model (south elevation)

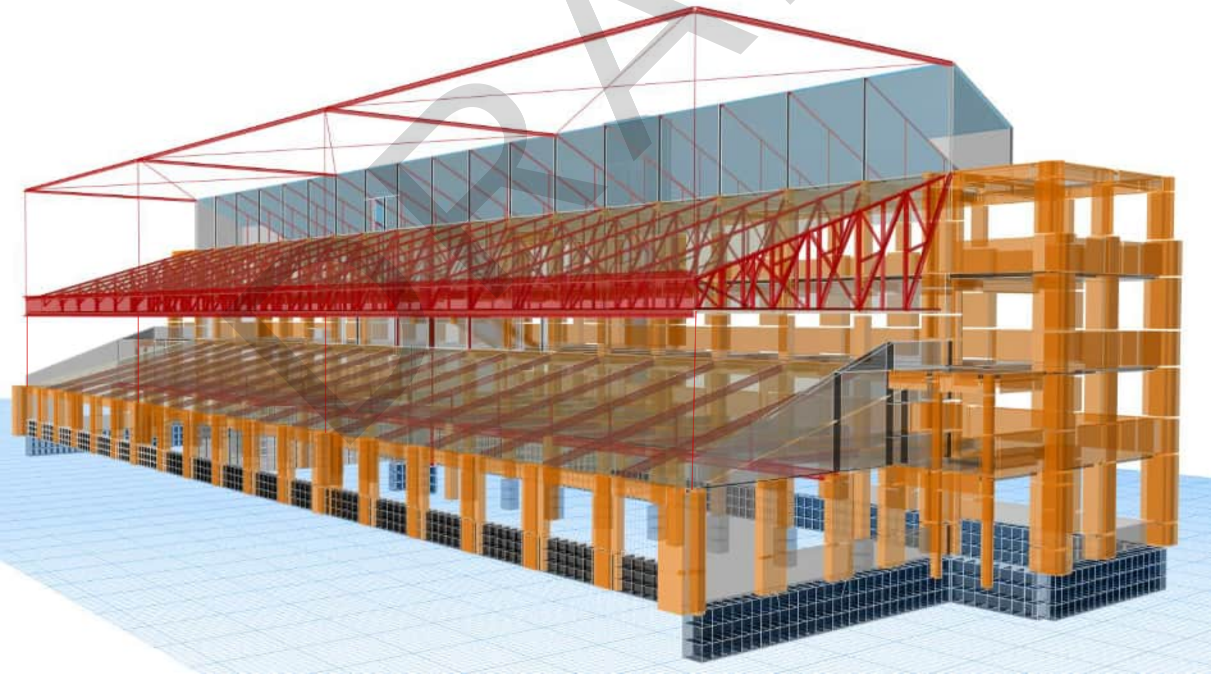

Figure 4-2 View of the ETABS model (north elevation)

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4.3 3D modal response spectrum analysis

4.3.1 Introduction

As indicated in section 4.2 the response spectrum analysis (RSA) was performed to gain general appreciation of the response of the buildings to seismic excitation.

The advantage of the RSA is its relative simplicity and small computational effort when compared to a pushover analysis. The main disadvantage is that the method is purely elastic and does not capture any post-elastic behaviour of the structure leading to potentially conservative results. Nevertheless, the analysis helped in the identification of the potential “hot-spots” and provided a baseline model for the more realistic pushover analysis.

The following items were investigated as a part of the RSA:

- Mode shapes and the period of the building,
- Initial investigation of the displacement demand and capacity for the circular steel columns supporting upper stand and the roof (along grid A),
- Lower-bound demand-capacity ratios for selected structural members in terms of current building standard (%NBS).

4.3.2 Modal analysis

The periods shown below in Table 4-2 relate to the first four modes of the structure. It should be noted that mode 1 as shown below does not relate to one of the main translational or torsional modes and its mass participation is very low. The deflected shape of this mode is the edge of the upper stand translating in the longitudinal direction.

Table 4-2 Periods of the building and corresponding mass participations

Mode	Period (sec)	Ux (%) [1]	Uy (%) [2]	Rz (%) [3]	Comment
1	0.647	14.06	0.01	4.86	Grid A of the upper stand moving in the longitudinal direction
2	0.545	0.09	66.64	0.38	Main translational mode in the transverse direction
3	0.434	45.36	0.001	7.95	Main translational mode in the longitudinal direction with minor torsional effect
4	0.399	19.4	0.27	49.02	Main torsional mode with translation in the longitudinal direction

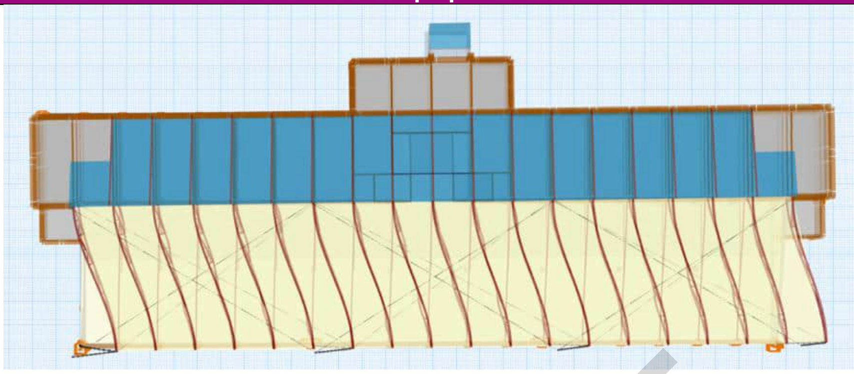
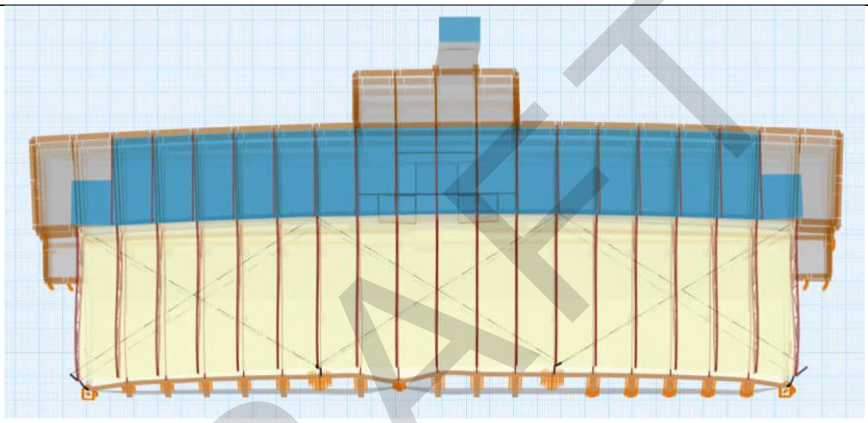
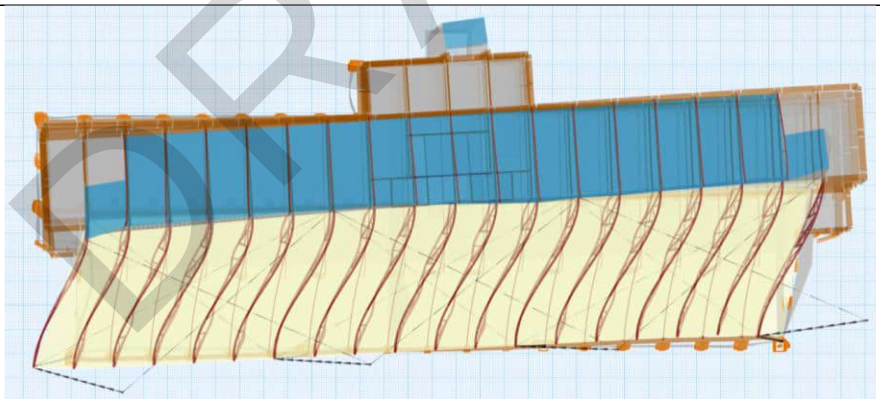
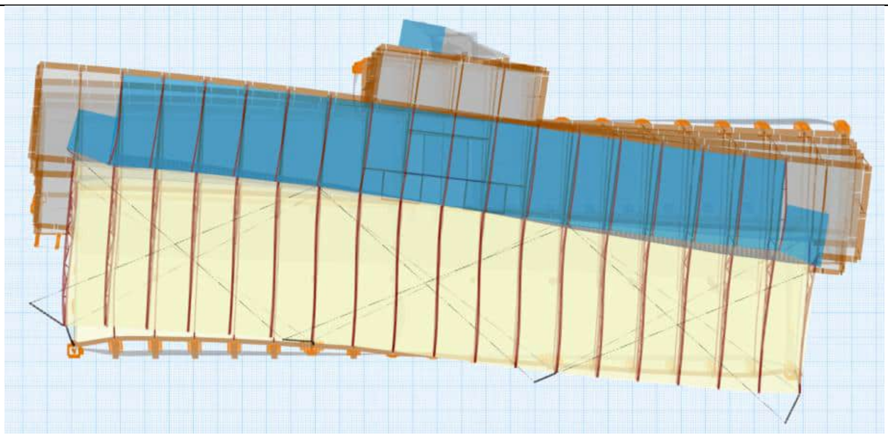
[1] Percentage of mass contributing in the x direction

[2] Percentage of mass contributing in the y direction

[3] Percentage of mass contributing to rotation

In the response spectrum analysis sufficient number of modes were used to satisfy the code requirement that 90% of the mass contributes in two orthogonal directions.

Table 4-3 Plan view of mode shape displacements

Mode	Mode shape plan view
1	
2	
3	
4	

4.3.3 Circular steel column capacity

The circular steel columns are located on grid A at the intersection of grids 2, 8, 14 and 20 and consist of a lower and upper column. The lower column is 235mm in diameter and was modelled as fixed at the base and pinned at the top. The upper column is 215mm in diameter and was modelled as pinned at both ends. The columns have been identified as requiring specific structural assessment due to their critical role within the building. Failure of any of these columns would result in a collapse with likely catastrophic consequences.

The resultant displacements of the columns from the RSA are shown in Table 4-4 (note that Ux is the displacement in the longitudinal direction or east-west while Uy is the displacement in the transverse direction or north-south). In summary the maximum inter-storey drifts are 2.1% for the lower stand columns and up to 0.76% for the columns located at the upper stand. These drifts are within the drift limit of 2.5% described in AS/NZS 1170.5:2004.

Table 4-4 Circular steel column RSA displacements

Grid 2					
Lower			Upper		
	Base (mm)	Top (mm)		Base (mm)	Top (mm)
Ux	6	124	Ux	126	119
Uy	44	99	Uy	99	145
Resultant		129.7	Resultant		46
Drift		2.1%	Drift		0.76%
Grid 8					
Lower			Upper		
	Base (mm)	Top (mm)		Base (mm)	Top (mm)
Ux	2	126	Ux	125	119
Uy	79	118	Uy	118	139
Resultant		130.0	Resultant		22
Drift		2.1%	Drift		0.36%
Grid 14					
Lower			Upper		
	Base (mm)	Top (mm)		Base (mm)	Top (mm)
Ux	2	125	Ux	125	119
Uy	74	113	Uy	113	132
Resultant		129.5	Resultant		20
Drift		2.1%	Drift		0.33%
Grid 20					
Lower			Upper		
	Base (mm)	Top (mm)		Base (mm)	Top (mm)
Ux	7	126	Ux	124	119
Uy	35	87	Uy	88	130
Resultant		129.9	Resultant		43
Drift		2.1%	Drift		0.76%

The axial and moment demands of the circular steel columns was determined, scaled, then compared to the axial and moment capacities to estimate the %NBS. The results are displayed in Table 4-5.

Table 4-5 Circular steel column RSA capacities

Grid Line		%NBS
2	Lower	83%
	Upper	>100%
8	Lower	54%
	Upper	>100%
14	Lower	56%

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Grid Line		%NBS
	Upper	>100%
20	Lower	82%
	Upper	>100%

The response spectrum analysis estimates the seismic capacity of the columns (supporting lower stand) as:

- 82-83%NBS for the external columns (grid 2 and 20)
- 54-56%NBS for the internal columns (grid 8 and 14)

The difference is in seismic capacity due to the axial demand on the columns, which is approximately half on the external columns compared to the internal columns.

4.3.4 Capacity check for selected members

As part of the RSA selected beams and columns were checked for their capacity in terms of current Building Standard. The maximum independent moment, shear and axial demands were collected, scaled and compared to the beam and column capacities. The %NBS relating to bending moment, shear and axial force are shown in Figure 4-3, Figure 4-4 and Figure 4-5 respectively.

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4.3.4.1 Bending moment

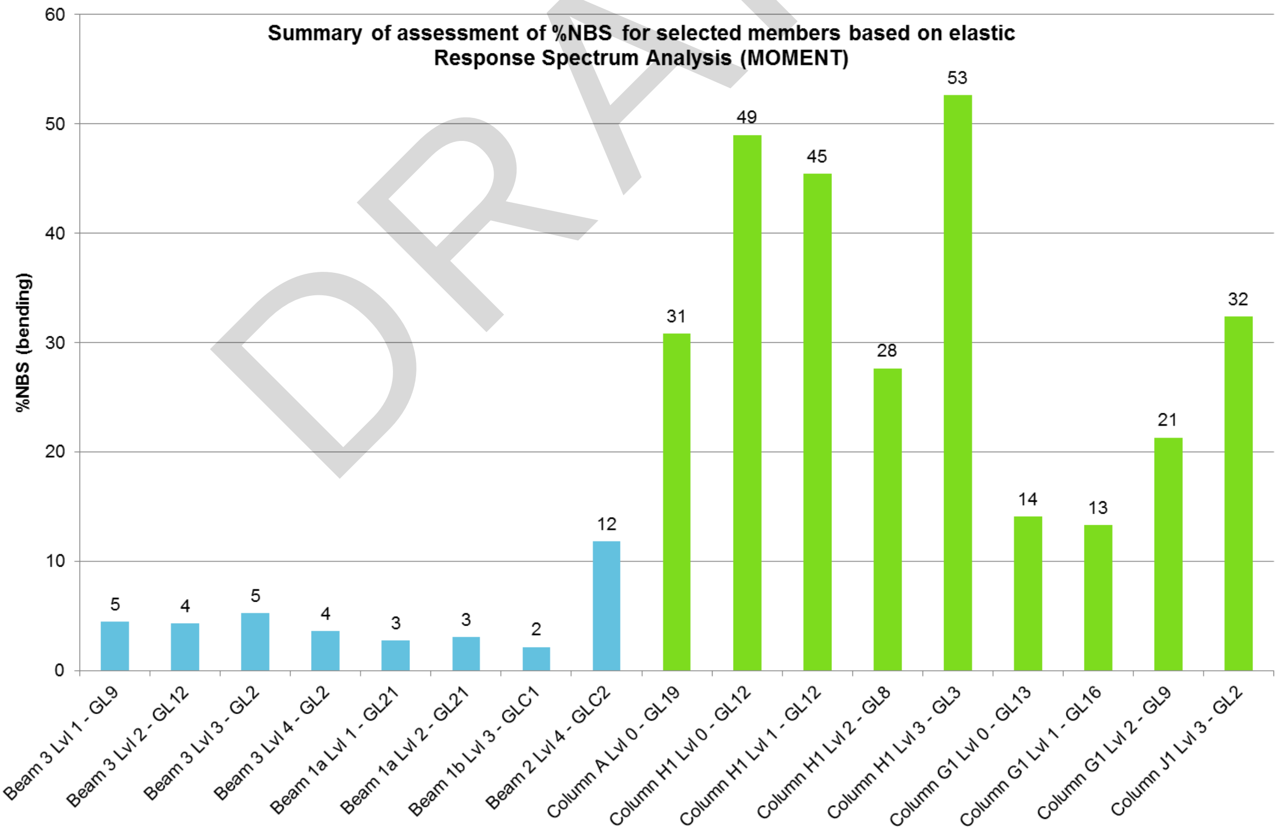

Figure 4-3 RSA selected member moment capacities

It should be noted that the transverse concrete encased steel beams (BEAM 3) have their moment capacity limited by the moment transferring ability of the beam-column joint. The exterior spandrel beams (BEAM 1A, 1B and 2) have a large depth but have only nominal top longitudinal reinforcement, resulting in a low moment capacity and %NBS when seismic conditions are considered (these perform satisfactorily under gravity conditions).

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The columns perform better than the beams in bending apart from the lower level exterior columns (COL G1). Larger inter-storey displacements are experienced at the lower levels of the structure and the type G1 columns have a single embedded angle compared to the type H1 columns which have double angles.

4.3.4.2 Shear force

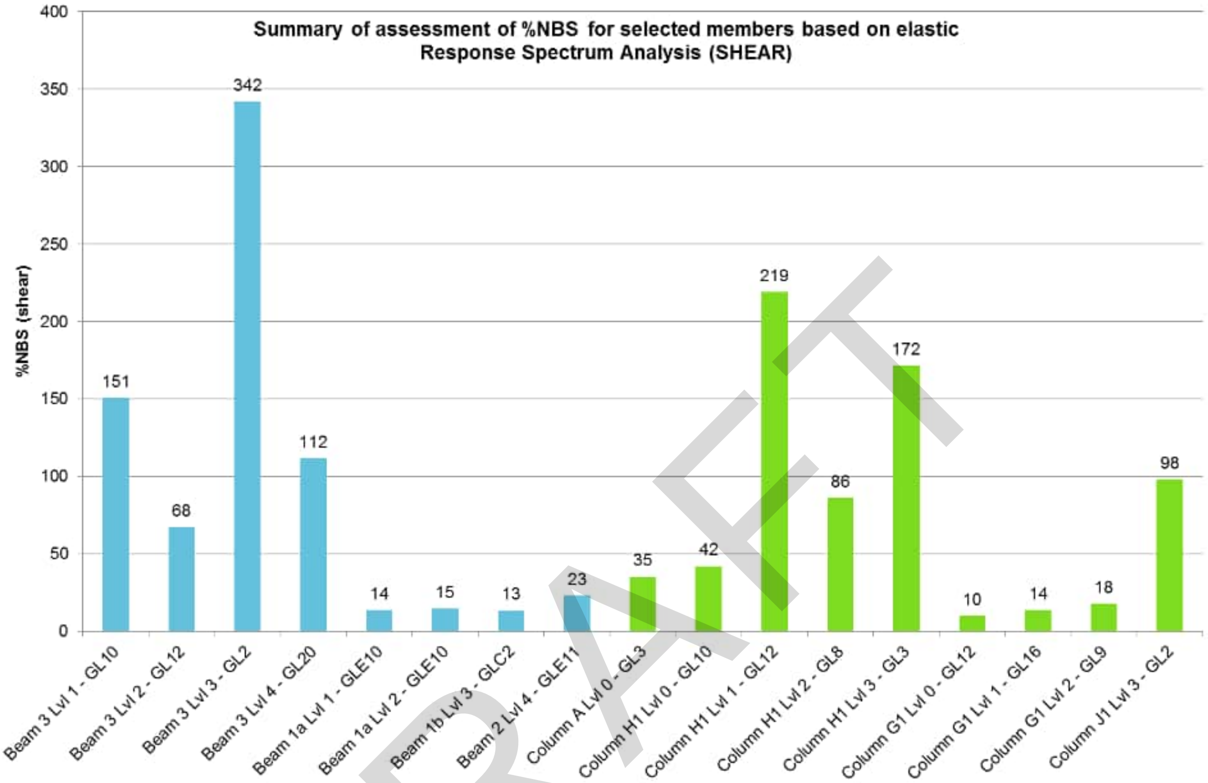

Figure 4-4 RSA selected member shear capacities

The transverse concrete encased steel beams (BEAM 3) performed well in shear because of the embedded steel beam. The exterior spandrel beams (BEAM 1A, 1B and 2) have nominal amounts of transverse reinforcement, resulting in a low shear capacity and %NBS.

The columns have nominal amounts of transverse reinforcement and rely on the embedded steel angles and longitudinal reinforcement for shear capacity. On the exterior type G1 columns the deep spandrel beams reduce the effective height of the columns, increasing the shear, resulting in a low %NBS.

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4.3.4.3 Axial force

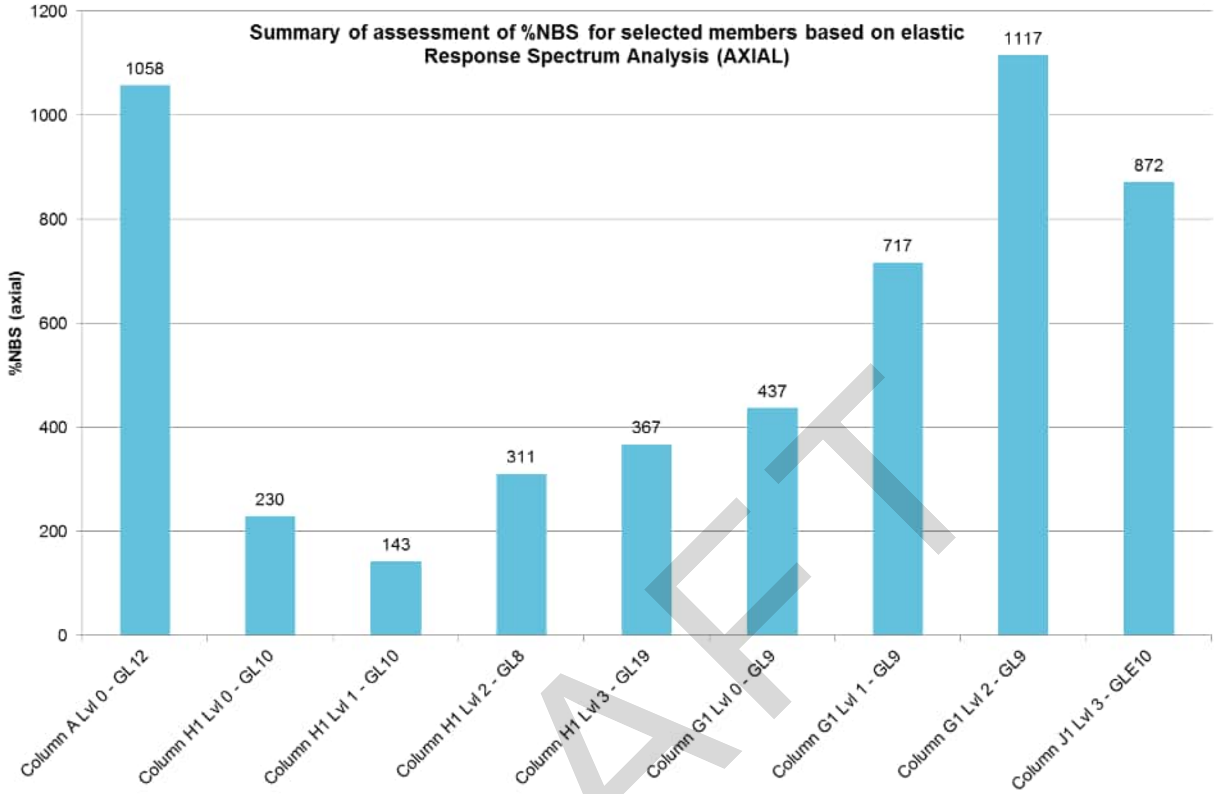

Figure 4-5 RSA selected member axial capacities

Cursory check of axial capacity-demand ratios on the selected columns indicated no problems with their axial strength.

4.4 3D non-linear pushover analysis

4.4.1 Introduction

Subsequently to the 3D response spectrum analysis, multiple nonlinear pushover (NLPO) analyses were performed on the structure. The aim of these analyses was to capture the post-elastic behaviour and identify the likely collapse mechanisms.

NLPO analyses have been completed with the entire building being “pushed” in four orthogonal directions along the buildings main axes. The following sections discuss some of the salient features of the analysis. Also refer to the DFR in Appendix A for a description of the procedure used in the assessment.

The results from the pushover analyses have been used as the basis of the seismic capacity of the building in terms of New Building Standard (%NBS).

4.4.2 Non-linear links

The material nonlinearity within the structure has been modelled using ETABS multi-linear link elements which were assigned to ends of beams and columns. In principle, the properties of the links have been based on the moment-curvature analysis of various sections and represented by a bilinear moment-rotation curve in the analysis package. See Figure 4-6 for an example link definition.

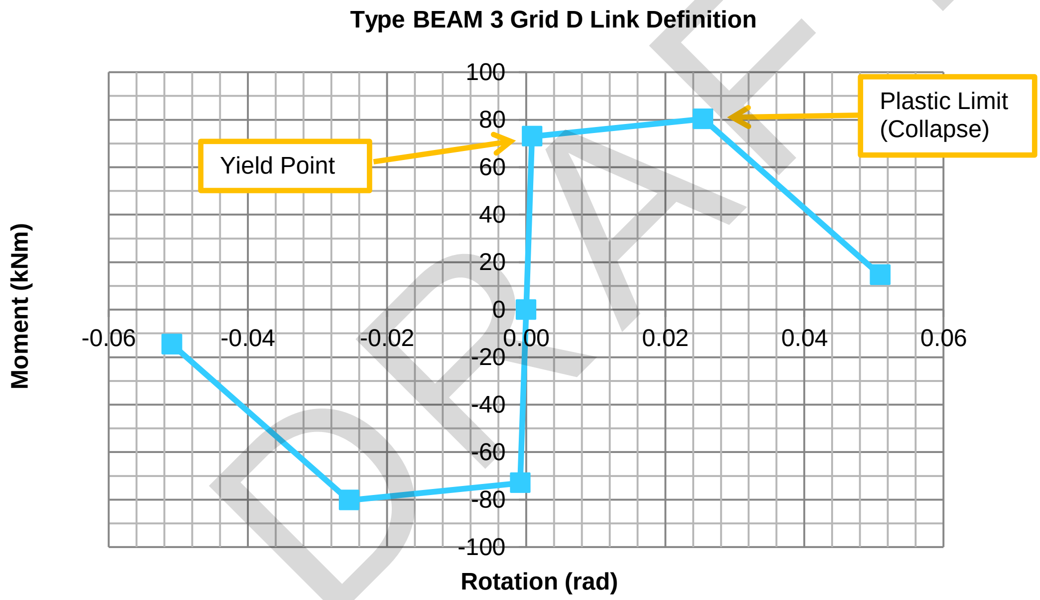

Figure 4-6 BEAM 3 Grid D M3 link definition

Where considered appropriate shear links have been introduced to allow for monitoring of shear behaviour of various structural elements.

4.4.3 Gravity load pre-load

Prior to application of incremental lateral load (i.e. “push”) the structure is preloaded with gravity. The gravity load consists of 100% of the dead load and 30% of the live load.

It should be noted that the analysis indicates that 142 of the links have gone beyond the elastic range under gravity load. The breakdown of which links have yielded is shown in Table 4-6. Also refer to Appendix C for a graphical representation of links yielding when subjected to gravity load.

Table 4-6 Links yielding under gravity load

Link Type	Yielded
BEAM 3 Grid C	72
BEAM 3 Grid D	58
BEAM 7	2
BEAM 9 Grid C	10
Sum	142

The large number of links yielding (BEAM 3 and BEAM 9) is closely associated with the limited moment capacity of the beam-column joints. The behaviour of these beams is close to the one exhibited by simply supported beams with nearly pinned connection.

The link type BEAM 7 is associated with beams located near the elevator core. The reason the beam yields is because there is insufficient reinforcement in the top of the section and therefore cannot accommodate the negative moment developed due to gravity.

Yielding of links does not represent failure of the element (which is limited by the maximum plastic rotation) but indicates that the non-linear behaviour in the structure would occur early in the analysis.

4.4.4 Pushover lateral load pattern

The NLPO lateral force is applied to the structure using the AS/NZS 1170.5:2004 lateral load pattern. The AS/NZS 1170.5:2004 lateral load pattern is proportional to the distribution of mass throughout the structure. Figure 4-7 shows the lateral load distribution used in the pushover analysis.

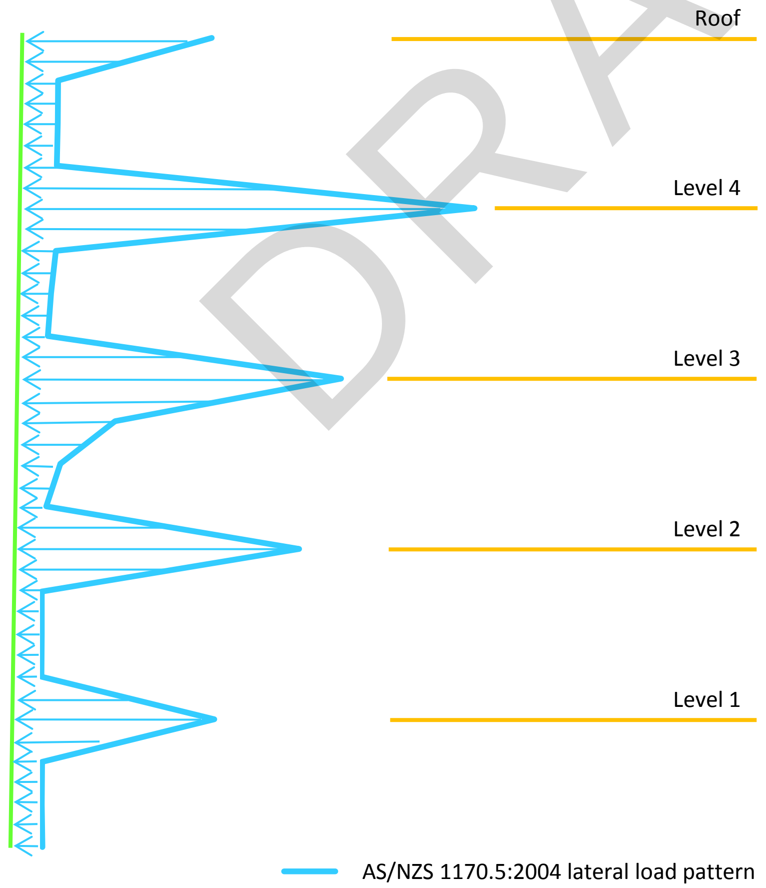

Figure 4-7 AS/NZS 1170.5:2004 lateral load pattern adopted in the pushover analysis

4.4.5 Target displacement

A target displacement is an estimate of the global displacement experienced by the structure in a design earthquake associated with a specified performance level. The internal forces and deformations computed at the target displacement levels are estimates of the strength and deformation demands, which need to be compared to available capacities.

A target displacement for each direction has been estimated based on section 7.4.3.3 of ASCE 41-13.

Refer to Table 4-7 for calculated target displacements.

Table 4-7 Target displacements to ASCE 41-13

Direction	Target displacement (mm)
PUSH X D1 (east)	125
PUSH X D2 (west)	114
PUSH Y D1 (north)	333
PUSH Y D2 (south)	187

4.4.6 Pushover curves and ADRS plots

It has been attempted to carry out the analyses to at least 150% of the target displacement (in line with C7.4.3.2.1 of the ASCE 41-13). In practice the analyses has been carried out until numerical instability was reached and analysis terminated.

It is important to note that the analysis has been continued after shear failure (refer to section 4.4.8) occurred in some of the elements. This was done to determine likely subsequent failure mechanisms within the structure.

The pushover curve from the analysis is replaced with an idealized bilinear approximation in accordance with clause 7.4.3.2.4 of the ASCE 41-13 (refer to Figure 4-8, reproduced from ASCE 41-13). The idealization is required to calculate the effective lateral stiffness and effective yield strength of the building.

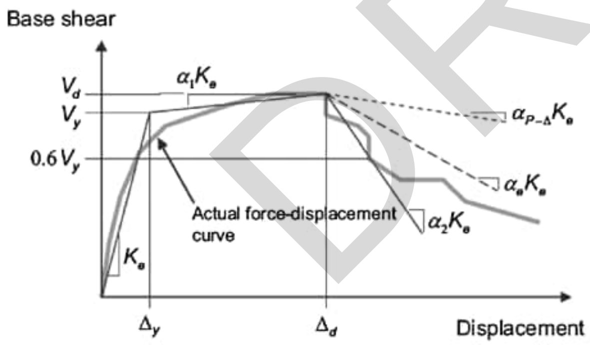

Figure 4-8 Idealized force – displacement curves

The idealized pushover curve is then transformed to an Acceleration Displacement Response Spectra (ADRS) representation. This allows for the comparison of the capacity curve with the demand spectrum and assessment of the seismic capacity of the building with respect to the New Building Standard (%NBS).

4.4.7 Damping

The assessment of equivalent viscous damping was determined using method recommended by NZSEE "Red Book" in section 6.3, Equation 6(4).

4.4.8 Shear failure

It was recognized in the course of the initial analyses that some elements of the structure fail in shear at low levels of drifts. This occurs along grid C in the locations where a number of walls have been removed which resulted in "short column effects" and along the interface between the elevator core and main structure.

To investigate these phenomena a number of shear links have been introduced into the structure in the locations where excessive shear was observed. These shear links allow for easy identification of failure and allow for controlled continuation of the analysis beyond shear failures.

It should be noted that brittle shear failure restricts seismic capacity of the structure. However, continuation of the analysis allows for investigation of the potential subsequent failure mechanism and gives better insight to the performance of the building.

4.4.9 Calculation of %NBS capacity of the building

The key purpose of the analysis was to establish the likely seismic capacity of the building, expressed in terms of the New Building Standard (%NBS). For the purpose of this assessment the %NBS was calculated as the minimum of the following:

- Ratio of displacement achieved at maximum base shear to target displacement (displacement-based assessment)
- Factor by which the demand spectrum needs to be scaled, to intersect with the capacity curve in the ADRS representation (force-based assessment). This is required in cases where the performance point does not exist (i.e. the capacity curve does not intersect demand spectrum).

The seismic capacity is also limited by shear failures if they occur at either, lower drifts or lower base shears than those established using the methodology described above.

4.4.10 Pushover in north direction (X D1)

The analysis in the east direction "pushes" the structure towards the Club Stand.

The lateral load resisting system in this direction comprises five major gridlines consisting of concrete moment frames and a shear wall located along grid C.

The pushover curve and its bilinear idealization are presented in Figure 4-9. The figure also shows the effective yield strength of the building (V_y) as calculated in accordance with ASCE 41-13. It should be noted that the pushover curve is relatively linear with no characteristic plateau observed.

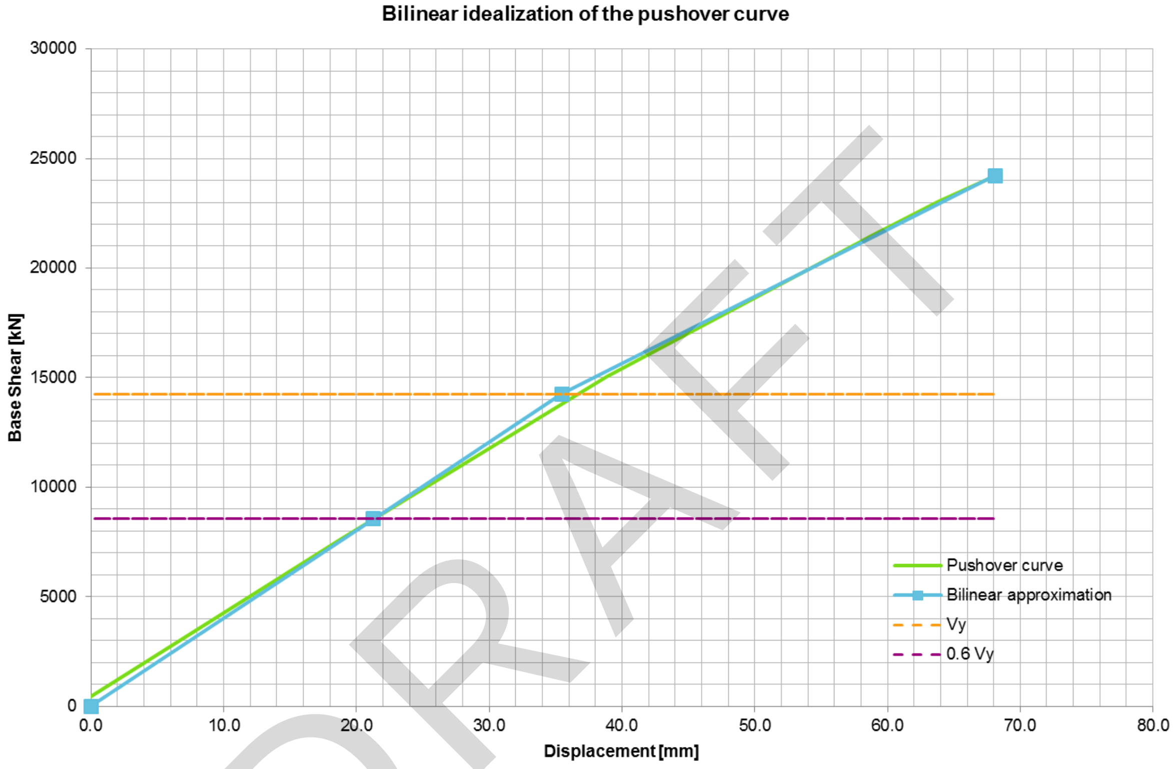

Figure 4-9 Pushover and bilinear idealization plots for push in east direction

The progression of the inelastic behaviour within the structure is demonstrated in Table 4-8 which shows the development of links and their status at different drift levels. As the structure is analysed a total number of 246 links yielded and 26 exceeded their ultimate capacity.

The key steps of the pushover analysis can be summarized as follows:

- Numerous links are yielding at step 0 (gravity) as discussed in section 4.4.3.
- As the structure is initially laterally loaded ("pushed") three (BEAM 3 Grid D), three (BEAM 7) and one (BEAM 9 Grid C) links yield,
- At drift levels between 10.9mm and 22.9mm displacement; one (BEAM 9 Grid C), one (BEAM 6), three (BEAM 4) and one (BEAM 12) links yield,
- Up to 34.9mm displacement; one (BEAM 9 Grid C), one (BEAM 6), 13 (BEAM 4), 11 (BEAM 1A) hinges develops. One (BEAM 6), one (BEAM 7), one (BEAM 2 SHEAR), one (BEAM 1B SHEAR) and two (BEAM 1A SHEAR) links reach their ultimate capacity,
- At approximately 42mm displacement a shear failure occurs at the interface between the elevator core and main structure. The shear failure occurs in the western side beams connecting the elevator core to the structure as shown in Figure 4-10.

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Table 4-8 Link results push in east direction

X D1	Displacement (mm)	Base Shear (kN)	Number of Links Yielding	Number of Links at Collapse
Full	68.1	24208	246	26
Step 4	45.6	17311	224	11
Step 3	34.9	13623	179	6
Step 2	22.9	9266	155	0
Step 1	10.9	4648	149	0
Gravity (step 0)	-1.14	0	142	0

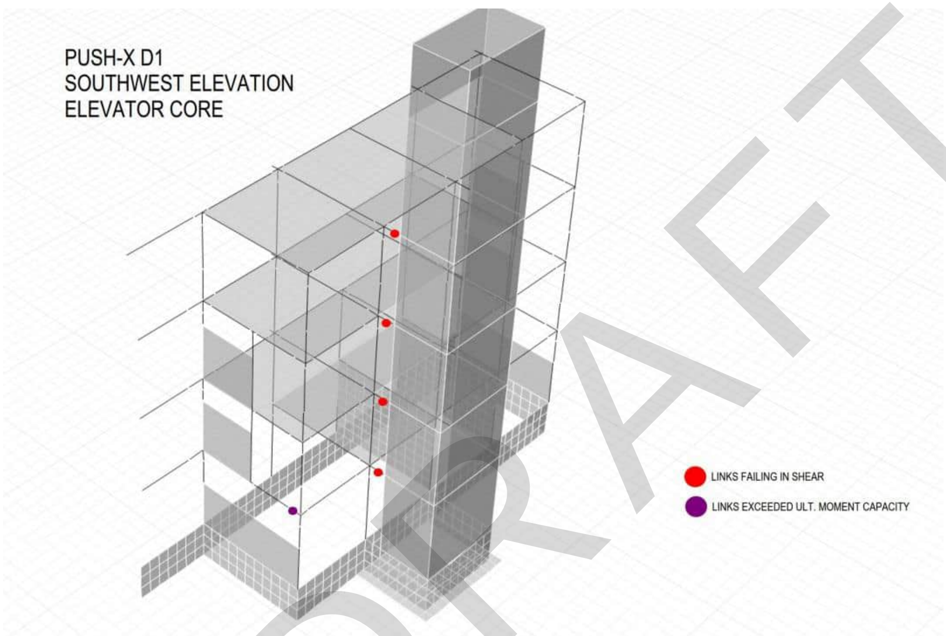

Figure 4-10 Links failing in shear push in east direction

Figure 4-11 presents the ADRS representation of the pushover analysis with the demand spectrum and capacity curves shown.

It is evident that there is a substantial shortfall between the two curves indicating a deficit in capacity. The plot also demonstrates a significant gap between the displacement achieved by the structure in the analysis and the target displacement.

As discussed above the first shear failure occurs at relatively low drifts and limits the capacity of the structure to approximately 25%NBS.

For the tabulated results from the analysis and the resulting %NBS refer to Table 4-9 below.

Table 4-9 Summary of results push in east direction

Load case	Pushover curve			First shear failure		%NBS		
	Max base shear (kN)	Max displacement (mm)	Target displacement (mm)	Main building/elevator core Base shear (kN)	Main building/elevator core Displacement (mm)	Displacement based assessment (%NBS)	Force based assessment (%NBS)	Shear failure (%NBS)
PUSH-X D1	24208	68	125	16132	42	54%	38%	25%

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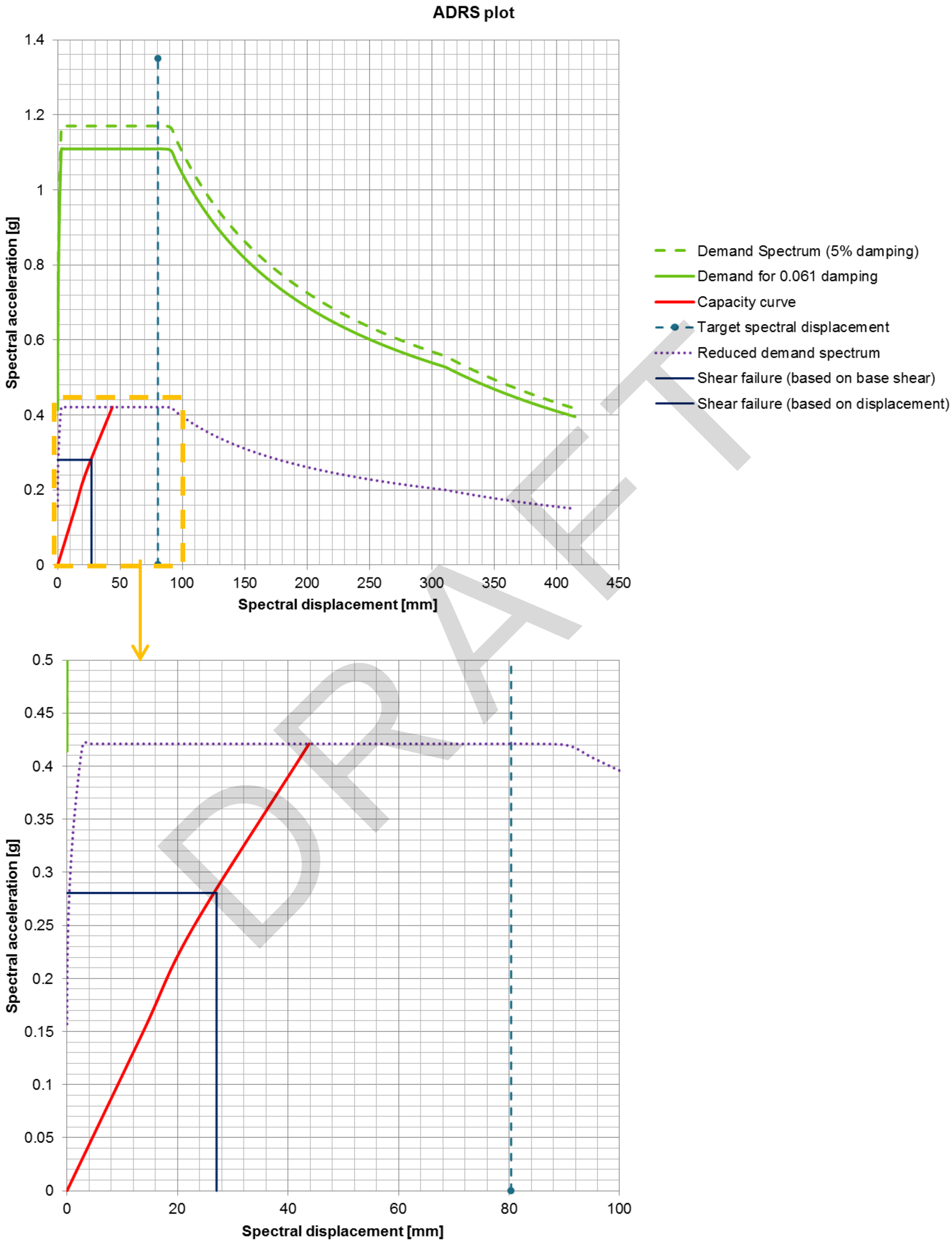

Figure 4-11 ADRS plot push in east direction

4.4.11 Pushover in north direction (X D2)

The analysis in the west direction “pushes” the structure away from the Club Stand.

The lateral load resisting system in this direction comprises five major gridlines consisting of concrete moment frames and a shear wall located along grid C.

The pushover curve and its bilinear idealization are presented in Figure 4-12. The figure also shows the effective yield strength of the building (V_y) as calculated in accordance with ASCE 41-13. It should be noted that the pushover curve is relatively linear with no characteristic plateau observed.

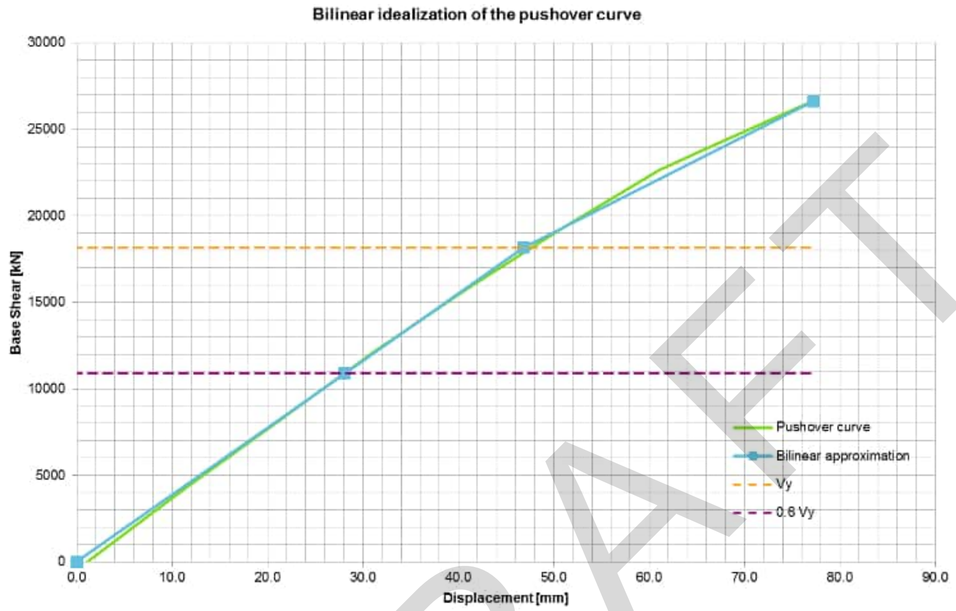

Figure 4-12 Pushover and bilinear idealization curves for push in west direction

The progression of the inelastic behaviour within the structure is demonstrated in Table 4-10 which shows the development of links and their status at different drift levels. As the structure is analysed a total number of 255 links yielded and 49 exceeded their ultimate capacity.

The key steps of the pushover analysis can be summarized as follows:

- Numerous links are yielding at step 0 (gravity) as discussed in section 4.4.3.
- As the structure is initially laterally loaded (“pushed”) two (BEAM 3 Grid D), one (BEAM 7) and one (BEAM 9 Grid C) links yield,
- At drift levels between 9.1mm and 17.1mm displacement; one (BEAM 9 Grid C), two (BEAM 7), one (BEAM 6), one (BEAM 4) and four (BEAM 3 Grid C) links yield,
- At drift levels between 17.1mm and 25.1mm displacement; one (BEAM 6) and one (BEAM 4) links yield,
- Up to 33.1mm displacement; eight (BEAM 4) and 19 (BEAM 1A) hinges develops. One (BEAM 2 SHEAR) one (BEAM 1B SHEAR) and two (BEAM 1A SHEAR) links reach their ultimate capacity,
- At approximately 32mm displacement a shear failure occurs at the interface between the elevator core and main structure. The shear failure occurs in the western side beams connecting the elevator core to the structure as shown in Figure 4-13.

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Table 4-10 Link Results push in west direction

X D2	Displacement (mm)	Base Shear (kN)	Number of Links Yielding	Number of Links at Collapse
Full	-77.2	26618	255	49
Step 5	-43.1	16550	223	6
Step 4	-33.1	12895	184	4
Step 3	-25.1	9757	157	0
Step 2	-17.1	6625	155	0
Step 1	-9.1	3312	146	0
Gravity (step 0)	-1.1	0	142	0

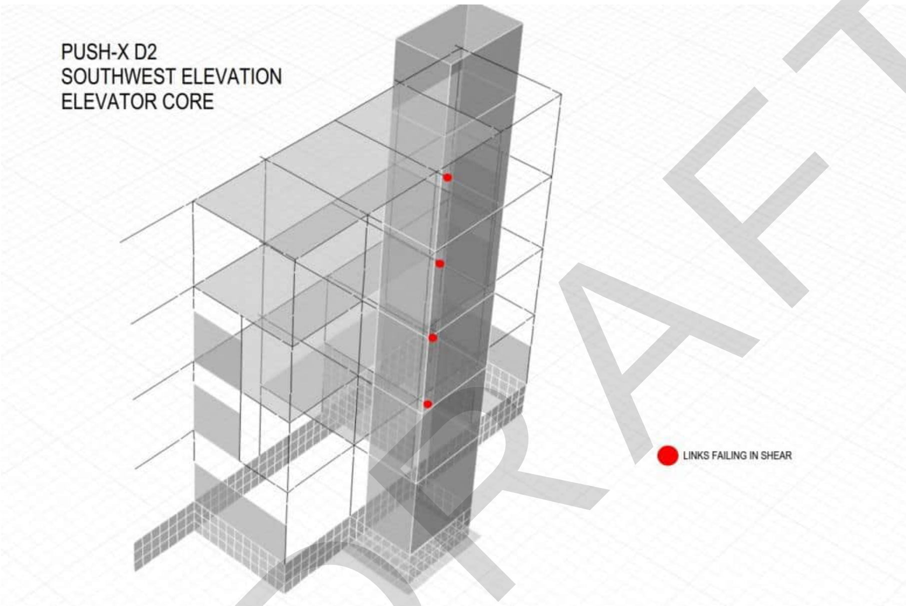

Figure 4-13 Links failing in shear push in west direction

Figure 4-14 presents the ADRS representation of the pushover analysis with the demand spectrum and capacity curves shown.

It is evident that there is a substantial shortfall between the two curves indicating a deficit in capacity. The plot also demonstrates a significant gap between the displacement achieved by the structure in the analysis and the target displacement.

As discussed above the first shear failure occurs at relatively low drifts and limits the capacity of the structure to approximately 20%NBS.

For the tabulated results from the analysis and the resulting %NBS refer to Table 4-11 below.

Table 4-11 Summary of results push in west direction

Load case	Pushover curve			First shear failure		%NBS		
	Max base shear (kN)	Max displacement (mm)	Target displacement (mm)	Main building/elevator core Base shear (kN)	Main building/elevator core Displacement (mm)	Displacement based assessment (%NBS)	Force based assessment (%NBS)	Shear failure (%NBS)
PUSH-X D2	26618	77	114	12507	32	68%	42%	20%

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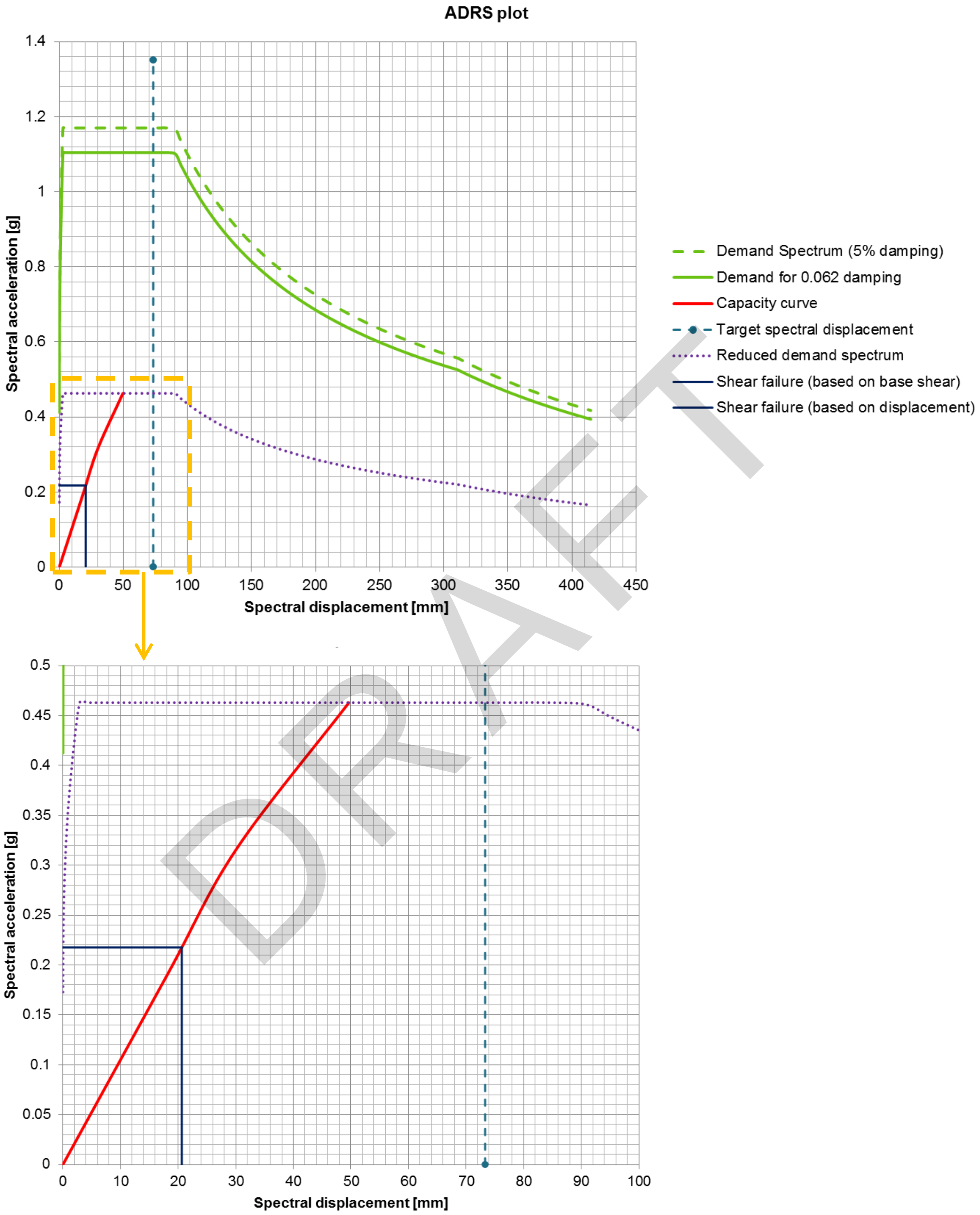

Figure 4-14 ADRS plot push in west direction

4.4.12 Pushover in north direction (Y D1)

The analysis in the north direction “pushes” the structure towards the racecourse track.

The lateral load resisting system in this direction comprises moment frames on grids 1 to 21. There are also internal concrete walls located on the ground floor and concrete wing walls on grids 2 and 20 (level 0 and 4).

The pushover curve and its bilinear idealization are presented in Figure 4-15. The figure also shows the effective yield strength of the building (V_y) as calculated in accordance with ASCE 41-13. It should be noted that the pushover curve is relatively linear with no characteristic plateau observed.

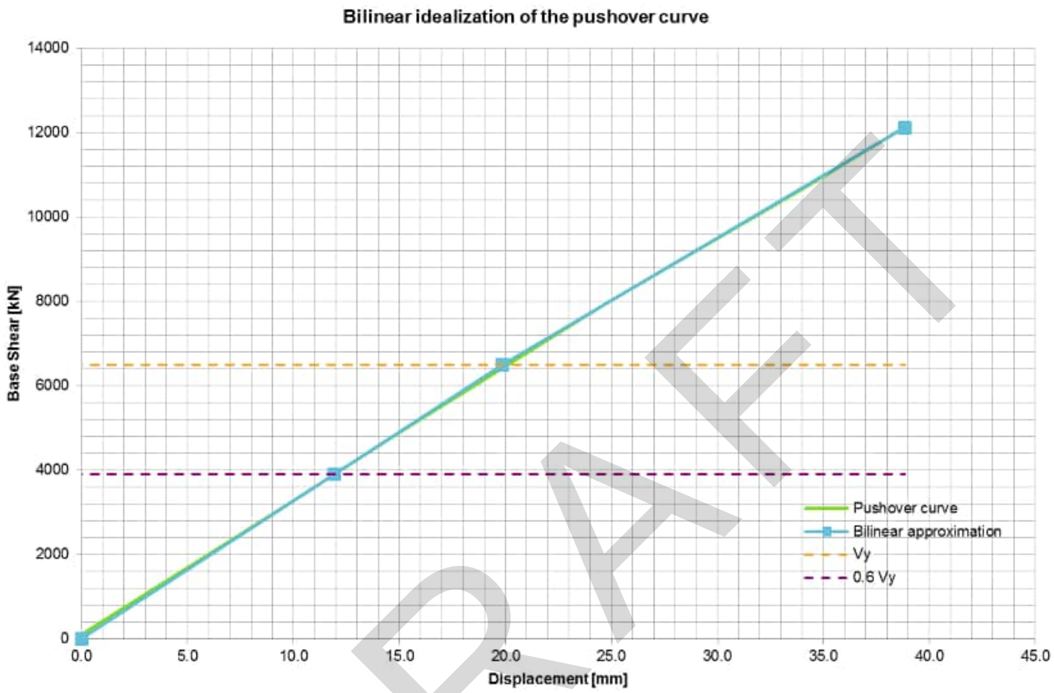

Figure 4-15 Pushover and bilinear idealization curves for pushover analysis in north direction

The progression of the inelastic behaviour within the structure is demonstrated in Table 4-12 which shows the development of links and their status at different drift levels. As the structure is analysed a total number of 167 links yielded and 5 exceeded their ultimate capacity.

The key steps of the pushover analysis can be summarized as follows:

- Numerous links are yielding at step 0 (gravity) as discussed in section 4.4.3.
- As the structure is initially laterally loaded (“pushed”) three hinges develop in the transverse concrete encased steel beams (type BEAM 3 Grid C).
- At drift levels between 14.7mm and 22.7mm further nine hinges develop (“BEAM 7’ and “BEAM 3 Grid C” type hinges). The BEAM 7 hinges are located adjacent to the elevator core.
- Up to 29.7mm displacement three more links are yielding (BEAM 3 Grid D) and three shear hinges (BEAM 1A SHEAR) reach their ultimate capacity.
- At approximately 31mm displacement a shear failure occurs at the interface between the elevator core and main structure on level 1 and 2. The shear failure occurs in the type 1A beams connecting the elevator core to the structure as illustrated in Figure 4-16.

Table 4-12 Link results for the pushover analysis in north direction

Push Y D1	Displacement (mm)	Base Shear (kN)	Number of Links Yielding	Number of Links at Collapse
Full	38.8	12116	167	5
Step 5	37.7	11767	167	4
Step 4	29.7	9367	157	3
Step 3	22.7	7417	154	0
Step 2	14.7	4792	145	0
Step 1	7.7	2534	145	0
Gravity (step 0)	-0.3	0	142	0

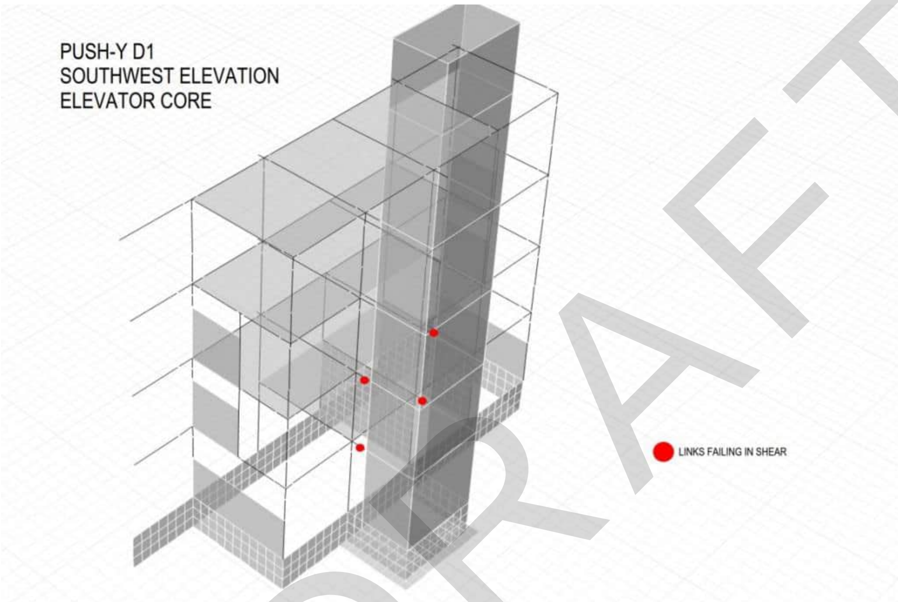

Figure 4-16 Links failing in shear – analysis in north direction

Figure 4-17 presents the ADRS representation of the pushover analysis with the demand spectrum and capacity curves shown.

It is evident that there is a substantial shortfall between the two curves indicating deficiency in capacity. The plot also demonstrates a significant gap between the displacement achieved by the structure in the analysis and the target displacement.

As discussed above the first shear failure occurs at relatively low drifts and limits the capacity of the structure to approximately 9%NBS.

For the tabulated results from the analysis and the resulting %NBS refer to Table 4-13 below.

Table 4-13 Summary of results - analysis in north direction

Load case	Pushover curve			First shear failure		%NBS		
	Max base shear (kN)	Max displacement (mm)	Target displacement (mm)	Main building/elevator core Base shear (kN)	Main building/elevator core Displacement (mm)	Displacement based assessment (%NBS)	Force based assessment (%NBS)	Shear failure (%NBS)
PUSHY D1	12116	39	333	9673	31	12%	18%	9%

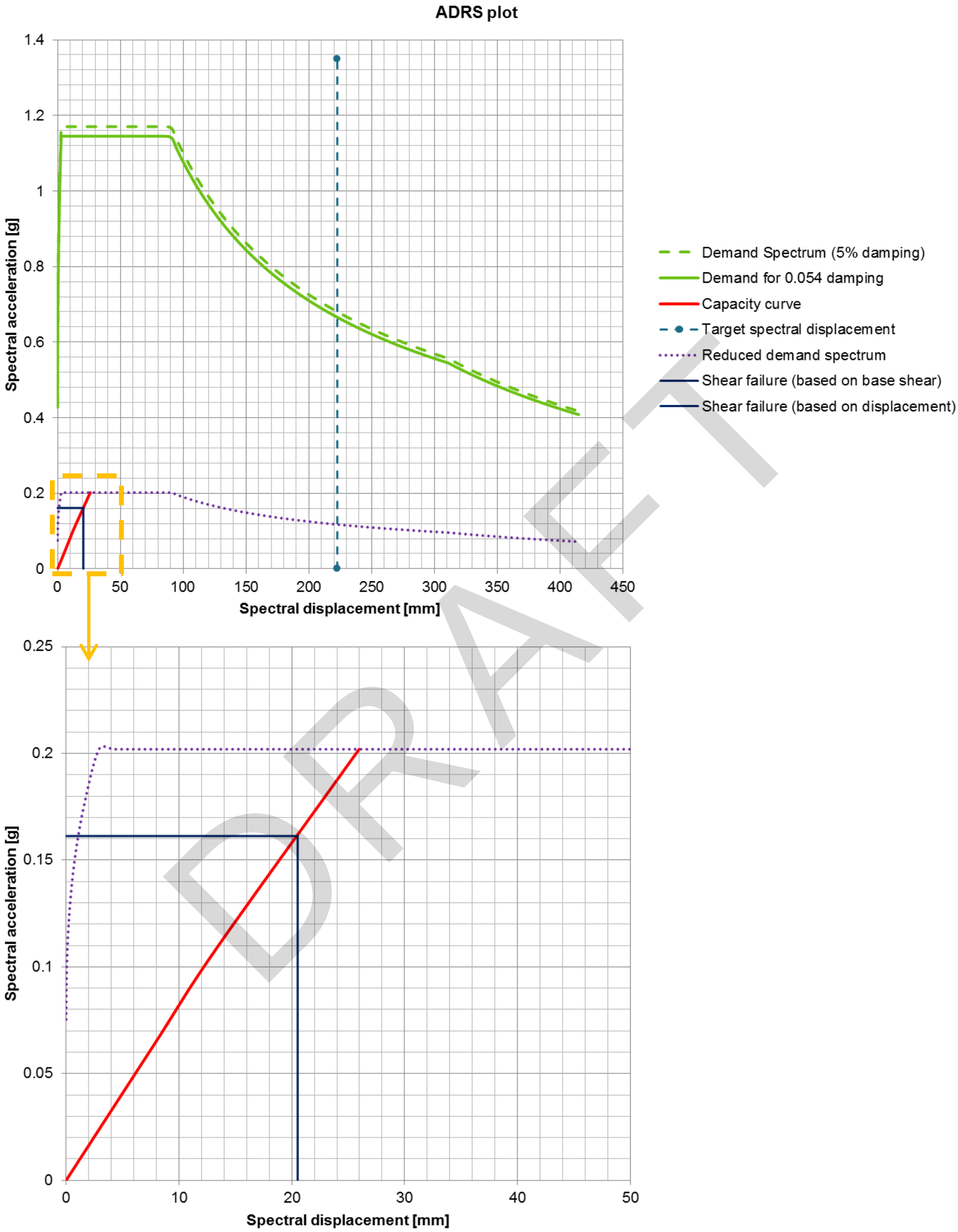

Figure 4-17 ADRS plot – analysis in north direction

4.4.13 Pushover in south direction (Y D2)

The analysis in the south direction "pushes" the structure away from the racecourse track.

The lateral load resisting system in this direction comprises moment frames on grids 1 to 21. There are also internal concrete walls located on the ground floor and concrete wing walls on grids 2 and 20 (level 0 and 4).

The pushover curve and its bilinear idealization are presented in Figure 4-18. The figure also shows the effective yield strength of the building (V_y) as calculated in accordance with ASCE 41-13. It should be noted that the pushover curve is relatively linear with a minimal plateau observed.

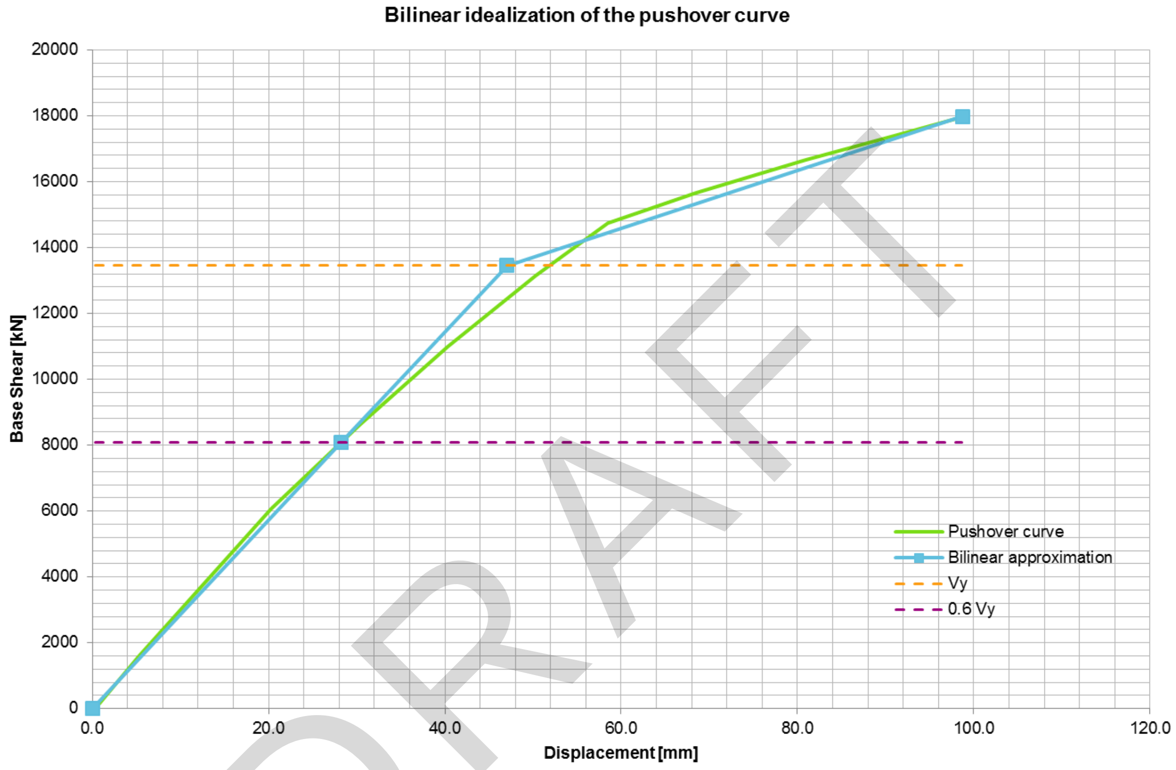

Figure 4-18 Pushover and bilinear idealization curves for push in south direction

The progression of the inelastic behaviour within the structure is demonstrated in Table 4-14 which shows the development of links and their status at different drift levels. As the structure is analysed a total number of 251 links yielded and 30 exceeded their ultimate capacity.

The key steps of the pushover analysis can be summarized as follows:

- Numerous links are yielding at step 0 (gravity) as discussed in section 4.4.3.
- As the structure is initially laterally loaded ("pushed") three hinge developed in the transverse concrete encased steel beams (BEAM 3 Grid D), six (BEAM 7) which are adjacent to the elevator core and four (BEAM 9 Grid C) develop,
- At drift levels between 6.3mm and 13.3mm displacement; two (BEAM 9 Grid C), one (BEAM 7), five (BEAM 3 Grid D) links yield and two (BEAM 7) and 1 (BEAM 1A SHEAR) links reach their ultimate capacity,
- Up to 19.3mm displacement; one (BEAM 3 Grid D) hinge develops. Six (BEAM 7) and three (BEAM 1A SHEAR) links reach their ultimate capacity,
- At approximately 16mm displacement a shear failure occurs at the interface between the elevator core and main structure on level 1 and 2. The shear failure occurs in the type 1A beams connecting the elevator core to the structure as shown in Figure 4-19.

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Table 4-14 Link results push in south direction

Y D2	Displacement (mm)	Base Shear (kN)	Number of Links Yielding	Number of Links at Collapse
Full	-98.7	17980	251	30
Step 5	-32.3	9106	166	14
Step 4	-26.3	7570	162	14
Step 3	-19.3	5782	154	12
Step 2	-13.3	4152	159	3
Step 1	-6.3	1925	153	0
Gravity (step 0)	-0.3	0	142	0

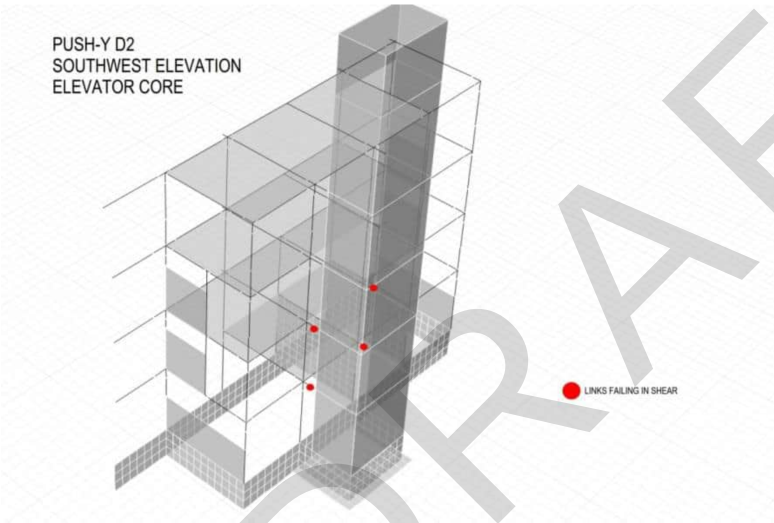

Figure 4-19 Links failing in shear push in the south direction

Figure 4-20 presents the ADRS representation of the pushover analysis with the demand spectrum and capacity curves shown.

It is evident that there is a substantial shortfall between the two curves indicating a deficit in capacity. The plot also demonstrates a significant gap between the displacement achieved by the structure in the analysis and the target displacement.

As discussed above the first shear failure occurs at relatively low drifts and limits the capacity of the structure to approximately 8%NBS.

For the tabulated results from the analysis and the resulting %NBS refer to Table 4-15

Table 4-15 Summary of results push in south direction

Load case	Pushover curve			First shear failure		%NBS		
	Max base shear (kN)	Max displacement (mm)	Target displacement (mm)	Main building/elevator core Base shear (kN)	Displacement (mm)	Displacement based assessment (%NBS)	Force based assessment (%NBS)	Shear failure (%NBS)
PUSH-Y D2	17980	99	187	5013	16	53%	30%	8%

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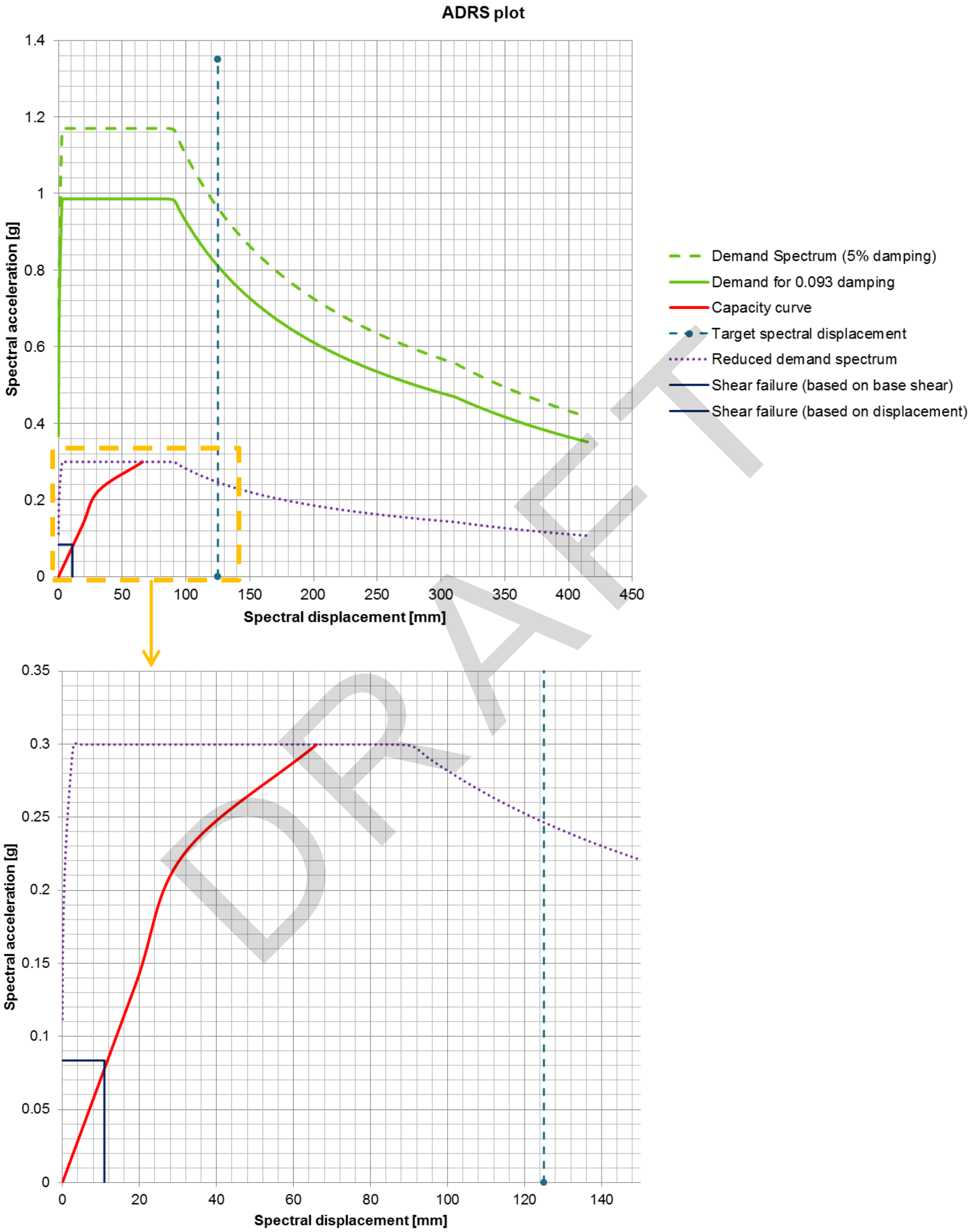

Figure 4-20 ADRS plot push in south direction

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4.4.14 Circular steel columns

The initial assessment of the circular columns along grid A was carried out using the elastic RSA method (refer to section 4.3.3) which resulted in the seismic capacity of 54%NBS at the worst case scenario. These columns were further investigated in the nonlinear pushover analyses discussed in the sections 4.4.10 to 4.4.13.

The capacity of the columns was not exceeded in any of the above analyses, which indicates that the columns do not fail at the maximum displacement reached in the nonlinear analyses. It should be noted that the target displacement, corresponding to 100%NBS, was not achieved in any of the pushover cases. Therefore, the assessment only confirms that other parts of the structure fail before the columns on grid A. As such the %NBS of the columns cannot be determined in the nonlinear pushover analysis.

The capacity of these columns could be considered in the potential retrofit scheme. In case the retrofit / strengthening solution warrants that the drifts are kept within the displacements observed in the pushover analyses the columns would achieve 100%NBS.

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4.5 Summary of results and estimate of pre-earthquake capacity

In all four of the NLPO analyses the seismic capacity of the building is restricted by the brittle shear failure of the beams connecting the elevator core to the main structure. Analyses were continued beyond the shear failure of these beams which identified the potential subsequent failure (either due to shear or excessive rotation) of the beams in the vicinity of the core. It is important to note that while the analysis was continued after the initial shear failure the certainty of subsequent failure mechanisms is reduced due to the potential onset of partial collapse of the structure and the associated unpredictability of the load redistribution.

The pre-earthquake capacity of the building is governed by the first shear failure of the load bearing element in the structure. This failure occurs in the pushover analysis in the south direction (denoted Y D2) and corresponds to approximately 8% of the New Building Standard (%NBS).

A summary of the results from the nonlinear pushover analyses is shown in Table 4-16 and illustrated in Figure 4-21.

Table 4-16 Overall summary of results pushes in all directions

Load case	Pushover curve			First shear failure		%NBS		
	Max base shear (kN)	Max displacement (mm)	Target displacement (mm)	Main building/elevator core Base shear (kN)	Main building/elevator core Displacement (mm)	Displacement based assessment (%NBS)	Force based assessment (%NBS)	Shear failure (%NBS)
X D1	24208	68	125	16132	42	54%	38%	25%
X D2	26618	77	114	12507	32	68%	42%	20%
Y D1	12116	39	333	9673	31	12%	18%	9%
Y D2	17980	99	187	5013	16	53%	30%	8%

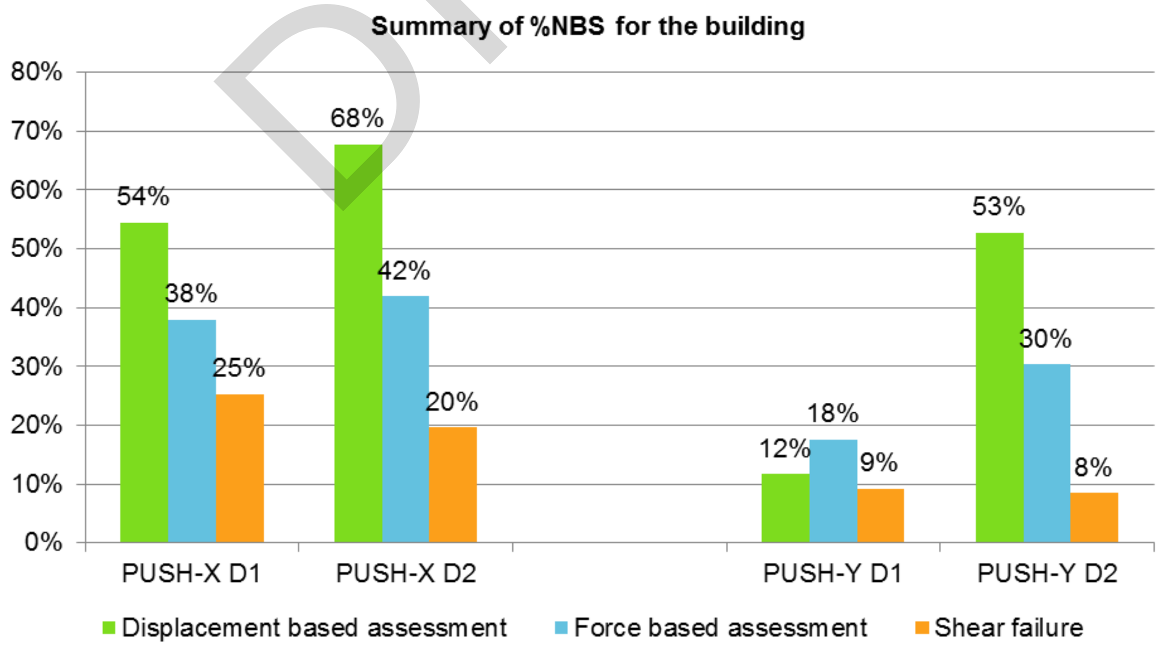

Figure 4-21 Graphical summary of results

4.6 Estimate of post-earthquake capacity

It should be noted that the building has sustained relatively significant earthquake attributed damage given the level of shaking it was subjected to in the Canterbury earthquakes (in the order of 40 to 50% of the design level earthquake; refer to DAR, section 2.3). As the building had a significant deficiency in seismic capacity based on its pre-earthquake estimate, the post-earthquake capacity would, naturally be less. We note that it would be difficult to quantify the post-earthquake capacity of the building with a sufficient degree of certainty and it would not change the status of the building.

In our opinion the building is likely to collapse in a moderate earthquake for the following reasons:

- The building displays a substantial deficiency in a lateral resisting system in particular in the transverse direction,
- The analysis indicates brittle shear failure to be the primary failure mechanism for the structure in all four directions,
- The building suffered fairly substantial damage from relatively low levels of shaking during the “Canterbury earthquakes”,
- The structural system exhibits a severe vertical irregularity in the form of an “inverted shear wall” arrangement on gridline C,
- The reinforcement ratios in all reinforced elements are extremely low and do not meet minima required by current building codes,
- The spacing of confining reinforcement is inconsistent with a high degree of variability in spacing observed,
- The use of round reinforcement bars throughout the building and high degree of uncertainty with regards to lap lengths and their effectiveness,

As a result the building is deemed to be an “earthquake prone” building according to the NZ Building Act 2004 based on the assessed current %NBS seismic capacity (less than 34%NBS and our opinion that it would be likely to collapse in a moderate earthquake).

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5.0 Gravity assessment

The gravity assessment has been performed to gain an appreciation of the building capacity compared to the ultimate limit state demand from AS/NZS 1170.0:2002 permanent, imposed, wind and snow load combinations. Refer to section 5.3.1 of Appendix A for a detailed description of this procedure.

5.1 Methodology

Refer to section 5.3.2 of Appendix A for the analysis methodology and detailed analysis procedure.

5.2 Loads

The basic loading requirements are given below in Table 5-1. The load combinations used during the analysis have been derived from AS/NZs 1170.0:2002.

Table 5-1 Basic Design Criteria

Description	Criteria
Design working life of building	50 years
Importance category	3
Annual probability of exceedance (ULS)	1/250 (snow) 1/1000 (wind)
Annual probability of exceedance (SLS)	1/25 (snow) 1/25 (wind)

The design life of 50yrs was used to determine the appropriate loading for the building. AECOM makes no warranty of the actual residual life of this building.

5.2.1 Gravity

The gravity loads consist of permanent (dead) and imposed (live) loads and have been derived from AS/NZS 1170.1:2002. Permanent loads include the self-weight of all permanently fixed materials. Imposed loads consist of a blanket 5 kPa for all floors and 5 kPa for the stairs. Refer to section 6.2 and 6.3 of Appendix A for a detailed breakdown of the gravity loads.

5.2.2 Wind

The vertical and horizontal wind actions have been derived in accordance with AS/NZS 1170.2:2011, with a design wind speed of 49.2 m/s used. The factors used in the calculations and the derivation of the loads are shown in section 6.4 of Appendix A.

5.2.3 Snow

The snow load actions have been derived in accordance with AS/NZS 1170.3:2003 and the site is classified as region N4 subalpine. The snow actions are shown in section 6.5 of Appendix A.

As the region is N4 subalpine ice actions have not been considered.

5.3 Models

5.3.1 Primary building frame

Simple beam models were used to calculate the capacity of the primary structural elements of the main building. Hand calculations were used to assess moments and shear (where appropriate).

5.3.2 Roof trusses

A typical roof truss was modelled in 2D using Spacegas version 12.00. The model geometry and member section sizes were based on a limited intrusive investigation and compared with available architectural record drawings. The chord members were analysed as continuous while the vertical and diagonal members were analysed as pin ended. The modelled supports for the truss consisted of a pinned roller support at the roof girder truss (grid A)

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and pinned supports at the wall on grid C and level 4 slab on grid D. Horizontal out of plane restraints at 900mm centres were used to model the purlins.

The assessment was limited to the typical roof trusses and the hip girder trusses. Jack trusses and creeper trusses forming the hip ends of the stadium roof were not assessed. Similarly, the roof trusses over the level 4 area bound by grids 9-13/C-E have not been assessed.

5.3.3 Roof girders with bracing

The roof girder trusses which support the roof trusses along grids A, 2 and 20 were modelled in 3D using Spacegas version 12.00. The model included the roof plane cross bracing but did not include the individual roof trusses. The model geometry and member section sizes were based on a limited intrusive investigation and compared with available architectural record drawings. The chord members were analysed as continuous while the vertical and diagonal members were analysed as pin ended. The modelled supports for the girder truss consisted of pinned roller supports to the bottom chord at the locations of the circular columns on grids A/2, A/8, A/14 and A/20 and pinned supports to the top and bottom chord at the ends of the trusses on grids C/2 and C/20. Note that the bottom chord of the actual girder truss is laterally unrestrained. In order to produce a stable model bottom chord, lateral restraints needed to be introduced at the column support locations on grids A/8 and A/14. An unrestrained model was used to carry out a buckling check to determine the buckling load factors.

5.3.4 Upper stand trusses

A typical Upper Stand truss was modelled in 2D using ETABs. The model geometry and member section sizes were based on a limited intrusive investigation and compared with available architectural record drawings. The chord members were analysed as continuous while the vertical and diagonal members were analysed as pin ended. The modelled supports for the truss consisted of a pinned roller support on the bottom and top chords at grid A and pinned supports at the wall on grid C. Lateral restraints were modelled at 1500mm centres along the top chord to represent the timber joists supporting the supper stand deck.

5.3.5 Upper stand cross bracing

The Upper Stand Level horizontal tension only cross bracing was modelled in 2D using ETABs. The model geometry and member section sizes were based on a limited intrusive investigation and compared with available architectural record drawings. The model included the Upper Stand plate girder along grid A and typical Upper Stand trusses on grids 2, 8, 14 and 20, at the node locations of the braces, to resist the compression forces. Vertical only supports were provided on grids A/2, A/8, A/14, and A/20. Pinned supports were provided at grids C/2, C/8, C/14, and C/20.

5.3.6 Upper stand plate girder

The Upper Stand plate girder along grid A was modelled in 2D using Spacegas version 12.00. The model geometry and member section sizes were based on a limited intrusive investigation and compared with available architectural record drawings. The plate girder was analysed as a continuous member from grid A/2 to grid A/20 with vertical supports provided at grids A/2, A/8, A/14, and A/20 representing the steel columns. Lateral restraints to the top and bottom chord were modelled at 4.1m centres to represent the restraint provided by Upper Stand trusses. The Upper Stand trusses were not included in this model.

5.3.7 Lower stand steel beams

The Lower Stand frames were modelled in 3D using Spacegas version 12.00. The model geometry and member section sizes were based on a limited intrusive investigation and compared with available architectural record drawings. Additional investigative effort was focussed on determining the structural arrangement for the connection between the raking steel beams (running north / south) supporting the bleachers and the columns on Grid A. This connection was found to differ from that used elsewhere in the main building with no embedded structural steelwork found in the concrete columns. The strength of the steel beam to concrete column connection was assessed to be approximately 10kNm.

5.3.8 Steel Columns

The north edges of the upper stand and the roof along are supported by two rows of four columns. The lowest row of columns span from the lower stand level to the bottom flange of the upper stand plate girder. These columns are 235mm in diameter and are considered to be effectively fixed at the base and pinned at the top. The upper row of columns are located concentrically on top of the lower row and span from the top flange of the upper stand

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plate girder to the bottom chord of the roof girder truss. These columns are 215mm in diameter and are considered pinned top and bottom.

These columns have been assessed by hand as steel columns in simple construction with the reactions from the girders at roof and upper stand level applied to the edges of the column to account for the eccentricity in the applied load.

The girder to column connections at upper stand and roof level are not considered to be able to resist any significant uplift loads e.g. wind.

5.3.9 Internal stairs & platforms

Internal stairs and platforms were modelled in 3D using Spacegas version 12.00. The model geometry and member section sizes were based on a limited intrusive investigation and compared with available architectural record drawings. The internal stairs and platforms were in fair to good condition. Some cracking of the concrete wall elements supporting the stairs was evident.

The stair from Lvl 1 – Lvl 2 is timber in construction and full supported by regularly spaced studs and framing.

The stair from Lvl 2 – Lvl 3 was modelled as a “3 Pin Arch” as the original supporting steelwork was cranked with an “idealised pin” located at the crank location. As an arch, the structure is strong and stiff and the capacity of the stair is therefore limited by the ability of the supporting elements to carry the large axial thrust loads developed in the stair framing members.

The stair and platform above Lvl 3 were modelled as flexural members supporting timber joists and framing.

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5.4 Summary of results

The following table has a summary of the capacity of the primary structural elements of the Grand National Stand to resist gravity and wind loads. A description of the limiting element capacity is provided where the component capacity is less than 100%NBS

Table 5-2 Summary of gravity analysis

Area	Item	Minimum %NBS	Comment
Primary Building Framing	Internal concrete / steel beams	70%	Internal beams capacity limited in both sagging and hogging moment regions. Ductile failure mechanism.
	Spandrel concrete / steel beams	>100%	Nil.
	Concrete columns	>100%	Not assessed quantitatively. Columns are “massive” and are adequate by inspection
	Slabs	>100%	Nil.
Roof	Typical Roof Truss	20%	Based on a top chord member under wind uplift. Capacity under gravity loads including snow > 100%NBS
	Roof Truss connections	35%	Based on a top chord splice connection resisting an in-plane moment and axial force due to wind uplift. Note connection capacity under gravity loads including snow > 100%NBS. Another splice has 86%NBS capacity under wind uplift. All other connections have >100%NBS capacity
	Girder truss	35%	Based on the lateral buckling of the truss bottom chord under 1.2G+1.5Q
	Roof Purlins	15%	The roof purlins span approximately 4m between the typical roof trusses. Based on wind uplift of the purlins located along the northern edge of the roof. All other purlins have adequate wind uplift capacity. All purlins have >100%NBS capacity for uniformly distributed gravity loads but only have 77%NBS capacity for concentrated imposed loads (as may occur during roof access for maintenance etc.)
Upper stand	Typical Truss	90%	Based on a single angle strut member in each truss under 1.2G+1.5Q loading. All other truss member types have a capacity >100%NBS
	Truss connection	90%	Based on the shear capacity of a single in the connection of a diagonal member under 1.2G+1.5Q loading. One of the remaining connections has a capacity of 97%NBS and all other connections have capacities >100%NBS
	Plate girder	>100%	Plate girder along grid A.

	Upper Stand Bleacher Joists	>100%	Timber members spanning approximately 4m between the typical upper stand trusses supporting the bleachers
	Bleacher stairs	75%	Based on the flexural capacity of a single equal angle steel beam supporting the timber stringers. The minimum timber member capacity is 98%NBS and this is based on the shear capacity of the 225x70 stringers assuming a half notch joint .
Lower Stand	Lower stand raking beams	80%	Inadequate for gravity loads. Strengthening suggested.
	Lower stand horizontal transfer girders (twin beams)	60%	Inadequate for gravity loads. Strengthening suggested.
	Lower stand common girders, ie, support for Lvl 1	70%	Inadequate for gravity loads. Strengthening suggested. Improve lateral restraint.
	Lower stand bleacher joists	95%	These are timber members spanning approximately 4m between the lower stand beams and support the bleachers The minimum capacity is based on the combined flexural and axial capacity of timber joist under 1.2G+1.5Q
Circular steel columns	Columns members supporting northern edge of upper stand and roof	>100%	Based on columns being steel and with the base of the lower columns effectively fixed at their bases
	Column to girder connections	55%	This is based on the capacity of the beam column connection to resist the uplift force due to the worst case ULS wind uplift case. This is based on a design wind speed > 175kph.
Internal Stairs & Platforms	Internal stairs Lvl 1 – Lvl 2	90%	Timber stair supported by regularly spaced studs and bearers which are in turn supported by concrete slab at Lvl 1. Conservative assumptions mean that calculated %NSB conservative and likely to be 100% or better.
	Internal stairs Lvl 2 – Lvl 3	70%	“3 Pin Arch” used to resolve structural system. Plausible load path for thrust generated by 2.5kPa live load. Limiting criterion being 520 thick concrete wall on Grid C spanning 7.9m vertically.
	Internal stairs above Lvl 3 + infill platform	>100%	Steelwork installed as a retrofit circa 1980.

6.0 Conclusions

6.1 Seismic assessment conclusions

The capacity of the Grand National Stand was checked against the requirements of AS/NZS 1170.5:2004 using 3D modal response spectrum and 3D non-linear pushover analysis.

6.1.1 3D modal response spectrum analysis

The capacity-demand ratios obtained from the 3D response spectrum analysis have not been used to assess the seismic capacity of the building. The main goals of the RSA are outlined in section 4.3.1.

The most beneficial component of the RSA was the modal analysis which identified the mode shapes and periods of the building. The periods in the longitudinal and transverse directions are 0.434s and 0.545s respectively. These periods are considered to be relatively low and are attributed to the fact that the structure is relatively stiff due to the large member sizes (e.g. columns and spandrel beams).

The elastic assessment of the circular columns supporting the upper stand and the roof indicate capacity issues with the lower columns (supporting upper stand). The RSA estimated the capacity of these columns to be at 54%NBS in the worst case scenario. This triggered further investigation of these columns in the non-linear pushover analysis.

The cursory capacity checks on selected members show flexural failure of beams which suggested inelastic behaviour would be expected in the subsequent nonlinear analyses. The capacity checks also indicated some shear issues which were later investigated in the pushover analyses.

The minimum seismic capacity based on the RSA is approximately 2 to 5%NBS based on the flexural failure of the beams as shown in the Figure 4-3.

As previously noted the results of %NBS from the RSA were not used to assess the seismic capacity of the building. They are considered to be conservative and form a lower-bound estimation when compared to the NLPO analysis which is a more accurate analysis.

6.1.2 3D non-linear pushover analysis

The 3D non-linear pushover analyses indicate that the seismic capacity of the building is governed by brittle shear failure. All the analyses suggest that there is a major issue at the interface between the main structure and the elevator core in the southern part of the building with the short and stiff connecting beams failing in shear at low drift levels. The incompatibility of lateral displacement between the core and the building impose large shear demands and would promote rapid degradation of the connection in the case of cyclic loading. This behaviour is also evidenced by the damage observed in this area (refer to DAR) with cracks observed to the connecting beams at various levels. As such the core was not part of the original structure but was added as a part of retrofit works carried out in 1980’s and it appears that the compatibility with the main building was not considered.

The seismic assessment considers the first shear failure to determine the minimum %NBS for the building. The shear failures occur at 8%, 9% 20% and 25%NBS for analyses in the south, north, west and east direction respectively. The overall seismic capacity of the building has been assessed to be at **8%NBS**, based on the minimum value from the four pushover analyses. The governing failure mechanism was observed at the interface between the elevator core and the main building with connecting beams failing in shear in all seismic analyses.

The seismic assessment of the circular steel columns indicates that at levels of drifts achieved in the NLPO their capacity is not exceeded. As none of the analyses reaches target displacement (100%NBS) the seismic capacity of these columns in terms of New Building Standards cannot be reported. It is envisaged that their capacity can be investigated along with the potential retrofit/strengthening scheme.

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6.2 Gravity and wind assessment conclusions

The ultimate limit state capacity of the Grand National Stadium primary structural members were checked against the code requirements in NZS 1170, parts 1, 2 and 3.

6.2.1 Primary building frame

Typical concrete framing members were selected as being representative of all primary structural elements in the building.

Intrusive investigations revealed that the primary structural system incorporated steel beams within concrete encasement. It generally appears that the primary framing for main building was designed for 2.0kPa – 3.0kPa. Failure modes are flexural, and hence ductile.

Spandrels, columns and slabs all possess sufficient capacity in flexure and shear.

Deflection calculations were not considered as part of this assessment.

6.2.2 Roof

Given the relatively lightweight construction of the roof the most significant loading applied is uplift due to wind. The roof structure appears to have been designed and constructed to resist gravity loads only.

The roof capacity under gravity loads is limited by the truss girder supporting the northern edge of the roof. The bottom chord of the girder truss has no lateral restraints between grids 2 and 20 producing an unrestrained length of 73m. The intermediate column supports at grids A/8 and A/14 are connected only to the bottom chord and the column connection is not considered to be able to provide any lateral restraint to the girder. The capacity of the girder truss could be significantly improved by the installation of fly bracing to the bottom chord.

The typical roof trusses have adequate capacity under gravity loading but do not meet the code requirements for wind uplift loading. The arrangement of the truss includes tension only vertical members which do not contribute to the truss structural system under uplift loading. This causes the effective length of the top chord of the truss to double under the most onerous loading conditions.

The timber purlins adjacent to the northern edge of the roof do not meet the code requirements for loading due to wind uplift. The majority of the purlins however are have adequate capacity wind uplift capacity and are only limited by their capacity to resist concentrated imposed loads. Access to the roof is restricted and it is thought that the risk of overloading the roof by concentrated loads can be managed by ensuring that any such loads are adequately distributed.

6.2.3 Upper Stand and Bleacher Stairs

The most significant loading on the upper stand and bleacher stairs is due to gravity.

The typical Upper Stand trusses were found to have a minimum capacity of 90%NBS under gravity loads with the minimum capacity governed by an equal angle member near grid C and a single rivet connection approximately mid span within the truss. The equal angle member is located within an area to the north of gird C that was modified circa 1981 to increase the footprint of level 3 providing bathroom and storage facilities. This modification has significantly increased the loading on this truss member element.

The timber joists supporting the upper stand bleachers were found to have adequate capacity.

The capacity of the Upper Stand Bleacher stairs was limited by a steel equal angle member spanning approximately 4 metres between trusses. The equal angle is provided with sufficient lateral restraint to develop the section capacity of the member but only achieves 75%NBS. The timber stringers supporting the stairs were assessed as having a minimum capacity of 98%NBS and are considered to be adequate for purpose.

6.2.4 Building Framing grids C to D, Levels 1 to 4

The typical concrete encased steel beams spanning along the numbered grids which support the floor slabs were found to have a capacity of only 70%NBS based on flexural strength. This relates to a characteristic imposed load capacity of 2kPa which is significantly less than the code demand of 5.0kPa.

The remaining floor beams, perimeter beams and the slabs were assessed as having adequate capacity.

The columns were not quantitatively assessed but by inspection of their sectional dimensions they will be very lightly stressed and are considered to have adequate capacity.

6.2.5 Lower Stand Framing

The capacity of the steel beams supporting the lower stand is 60%NBS based on their flexural strength. Cursory strength checks at 2.5kPa (50% LL) were also completed. The structure satisfies this criteria. AECOM recommends that strengthening measures are implemented to improve the capacity of the lower stand and achieve modern code live load capacity.

The minimum capacity of the timber joists supporting the lower stand bleachers was assessed as 98%NBS based on their shear strength. These joists are considered to be adequate to meet the code demand.

6.2.6 Circular Steel Columns

The circular steel columns that support the northern edge (grid A) of the upper stand and roof were assessed as having adequate capacity to resist gravity loads. However the column to girder connection appear to consist of a cast iron collar acting in bearing with a limited friction clamping connection utilizing bolts passing through the collar and screwed against the outside wall of the columns. It is possible that a more substantial connection has been constructed and is hidden within the collar however it is considered unlikely that this connection would provide any effective restraint under nett wind uplift cases. The stability of the roof under wind uplift has been assessed as 55%NBS and under the most onerous ULS wind uplift case the roof structure could potentially lift off the upper level columns. It should be noted that the design wind speed for this event is approximately 175kph and this would require the roof sheeting and purlins to remain intact.

6.2.7 Internal Stairs & Platforms

Lvl 1 – Lvl 2. Stairs are timber framed and stringers are supported by a regular arrangement of stud walls and bearers. A %NBS of 90% has been assessed and the stairs are considered adequate given the conservative nature of the analysis assumptions (materials and framing).

Lvl 2 – Lvl 3. There is no beam continuity (no flange plates) at crank locations; therefore the framing system has been assessed as a “3 Pin Arch”. The stair can accommodate 2.5kPa live load whereas modern codes requirements call for a live load of 5.0kPa. The thrust load developed in order to support the stair is delivered to the Lvl 2 and Lvl 4 diaphragms via flexural action of 520 thick concrete wall on Grid C spanning 7.9m. A plausible structural system for resisting the thrust loads uses the encased structural steel angles as reinforcement for the 520 thick wall which acts in flexure (note, by inspection, the axial loads are small). The stair achieves approximately 70%NBS. AECOM recommends remedial works to improve the capacity of the stair.

Lvl 3 + platforms. Steelwork and timber members are a recent addition (circa 1980’s). Framing appears to possess sufficient capacity to resist full live loads (i.e. 100%NBS).

7.0 Disclaimer

1) It should be noted that the remedial measures made in this report do not preclude the possibility of future differential settlement of the building following future significant earthquakes. This settlement will be cumulative and may result in further structural damage, settlement of ground slab and requirement for re-inspection. The requirement for ground improvement should be considered on a cost-benefit basis in accordance with the geotechnical report, taking consideration of cost, time and disruption and likelihood of future damage.

2) This report is for the sole use and benefit of our Client. No other party should rely on this report without the prior written consent of AECOM. AECOM undertakes no duty, nor accepts any responsibility, to any third party who may rely upon or use this report. The basis of AECOM's advice and our responsibility to our Client is set out above and in the terms of engagement with our Client.

Appendix A

Design Features Report

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Grand National Stand - Design Features Report

3D Response Spectrum, 3D Non-linear Pushover & Vertical analysis

D R A F T

Grand National Stand - Design Features Report

3D Response Spectrum, 3D Non-linear Pushover & Vertical analysis

Client: Canterbury Jockey Club

Co No.: N/A

Prepared by

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Reviewed by	Mark Ferfolja, Craig Oldfield

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Revision	Revision Date	Details	Authorised	
			Name/Position	Signature
	22-Jan -016	For Issue	Mark Ferfolja Associate Director	

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Table of Contents

1.0 General

1.1 Objective

This Design Features Report (DFR) incorporates AECOM's 3D Non-linear Pushover (NLPO), 3D Response Spectrum Analysis (RSA) & Vertical Analysis and is a detailed document defining the Grand National Stand's (GNS's) structural assessment criteria, key assumptions, inspection findings, methods of analysis, key decisions and outcomes.

It provides commentary on the following matters:

- lateral load resisting systems,
- soil properties,
- geometric assumptions,
- loading assumptions,
- structural modelling assumptions,
- methodology of analysis,
- material properties,
- design standards and industry guidelines used.

1.2 Scope

The scope of this report was broadly defined in AECOM's Project Change Record 10 (PCR10) dated 12 October 2015 and refined during a series of formal face-to-face, and site meetings conducted between AECOM and Thornton Tomasetti between 16/9/15 & 22/10/15.

In general terms, the scope of work included:

- Intrusive investigation of the beam / column joint(s) providing lateral resistance for the concrete frames in the North / South loading direction,
- Excavation of the footings on Grid A and Grid B to determine size and extent and confirm bearing strata and soil properties,
- Liaison with Thornton Tomasetti (TT) to agree the structural analysis approach / strategy as proposed by AECOM,
- Determination of building weights and likely live loads acting on the structure and completion of "load take downs" to estimate the overall building weight,
- Identification of significant critical structural weaknesses such as soft stories, strong beam / weak columns etc. which may limit the ductile response of the structure,
- Completion of a vertical analysis for the building considering the effects of gravity, wind and snow loads,
- Completion of an assessment of the seismic capacity of all the main structural framing elements excluding secondary structure(s) such as suspended ceilings, balustrades / railings, parapets, chimneys, lightweight cladding such as glazing etc. Refer to Section 8.0 for a list of other items excluded from the analysis (note that this list may not be exhaustive),
- Development of a full 3D model in ETABS analysis software. Elements such as the steel truss roof were modelled using "proxy" elements to simplify the analysis,
- Completion of a Non-Linear Push Over (NLPO) analysis for the entire 3D model loaded unilaterally in orthogonal directions to assess and verify the seismic response and demand of the structure. The pushover analysis provides realistic seismic response and highlights collapse mechanisms that require attention,

- Completion of a 3D Response Spectrum Analysis (RSA) to gain an appreciation of the global building behaviour including torsional response, evaluation of the building period of vibration and assessment of selected elements to determine lower-bound capacity in terms of the current building code,
- Completion of an assessment of the seismic capacities of specific critical member connections as deemed necessary or identified as under capacity during the analysis,
- Determination of seismic capacity of the building in terms of percentage of new building standard (%NBS, i.e. NZS1170.5:2004-Earthquake Actions),
- Liaison and active involvement of Thornton Tomasetti (TT) in the development of assumptions, analysis processes, and discussion of findings / results during the structural analysis process.

1.3 Previous reports

This report should be read in conjunction with the following:

- AECOM's Damage Assessment Report dated 14th July 2015,
- AECOM's (original) Design Feature Report (DFR) for the GNS dated 29th July 2015.

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2.0 Building Description

2.1 General Description

The Grand-National Stand is a concrete structure which has a number of framing systems including traditionally reinforced concrete elements, concrete encased steel beams, concrete encased “steel angle columns”, structural steel frames, structural steel trusses, plated steel girders and load bearing timber frames. The building was constructed circa 1920.

The Grand National Stand retains heritage status and is listed as Group 4 in the Christchurch City Council (CCC) South-West Christchurch Area Plan: Phase 1 Report – European Cultural Heritage [1] [2] [3]. It is also understood (at the time of writing) that the heritage classification of the building is under review, and has the potential to be changed.

The building is orientated with the long axis parallel to the “home straight” of the race track and 37° off east-west or approximately northwest-by-west (NWbW) to southeast-by-east (SEbE) in direction. For the purpose of reference, “Project North” has been defined as perpendicular to the “home straight”. This reference convention is shown in Figure 2-1.

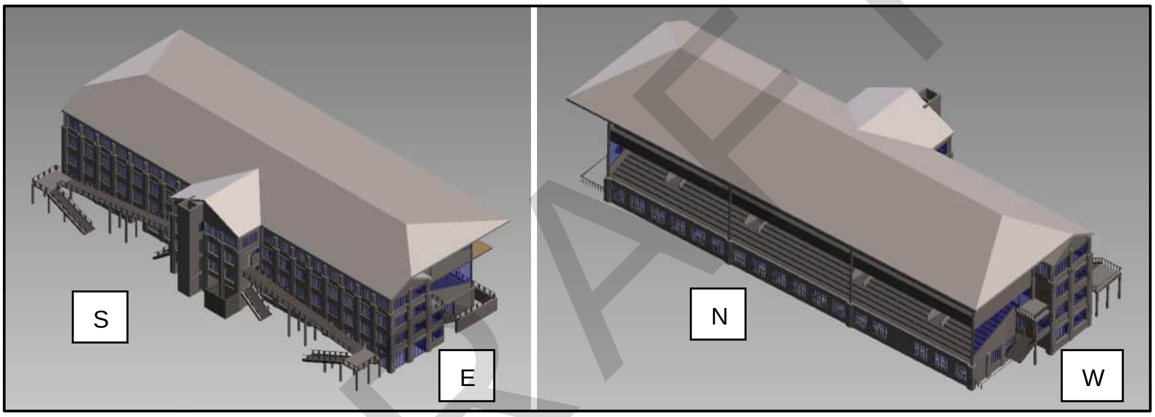

Figure 2-1 Elevation naming convention at Grand National Stand (GNS)

The structure consists of five above ground stories with two grandstand seating levels and has a footprint of approximately 82m parallel to the racetrack and 25m perpendicular to the racetrack. The main structure is generally rectangular in plan, measuring approximately 82m x 9.5m. There is an attached foyer and elevator core area measuring approximately 15.8m (east-west) x 6.5m (north-south) extending out on the southern elevation (see Figure 2-1). The elevator core is not an original feature.

There are two grandstand levels on the northern elevation, as shown in Figure 2-2. Both the (smaller) lower stand and (larger) upper stand are of structural steel and timber construction. The “bleachers” (stepped seating areas) are supported by timber joists, which are in turn supported by steel trusses and plated girders (for the upper stand) and structural steel frames (lower stand). Both grandstand areas are approximately 73m long but vary in width and gradient. The lower grandstand is narrower and flatter with a seating area of approximately 825m^2. The upper grandstand is steeper and wider with a seating area of approximately 1080m^2.

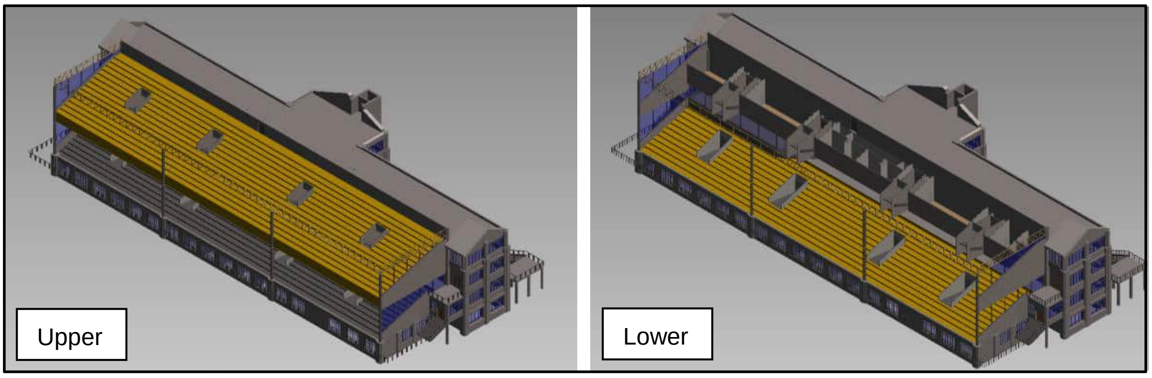

Figure 2-2 Cutaway showing the grandstand seating areas at GNS

The ground floor consists of a bar (known as 'The Parade Ring') at the eastern end of the structure and a storage and workshop area at the western end of the structure. The first, second, and third floors, consist of tote offices, bar areas, café facilities, kitchens, and general public assembly areas. The fourth floor is a maintenance level with no public access.

Access to these upper floors, (first, second, and third) is via several ramps and concrete steps or via an elevator; all located on the south elevation (see Figure 2-1). Access to the fourth floor is via a service door on the upper stand (see Figure 2-2) or via the elevator (see Figure 2-1). The lower stand can be accessed directly from the track on the northern side and from the first and second stories on the south side. Access to the upper stand is via four sets of stairs on the third floor only.

A brief summary of the building is provided in Table 1and Table 2.

Table 1 Building Summary

Grand National Stand	
Total Length	~ 82 m
Total Width	~ 25 m
Total Height	~ 18.6 m
Importance Level (IL)	3
Number of Stories	5 floor levels 2 grandstands
Total Plan Area (Approximate)	7700m^2

Table 2 Level-by-level Building Information

Level	Occupancy	Area	Storey Height
Ground	Workshop & Storage Public Access	1170 m^2 565m^2	0 m (reference level)
First	Public Access	1230 m^2	4 m
Lower Stand	Public Access	825 m^2	4 m – 7.7 m
Second	Public Access	1000 m^2	7.7 m
Third	Public Access	1065 m^2	11.5 m
Upper Stand	Public Access	1080 m^2	12.145 m – 16.375 m
Fourth	Maintenance Access Only	765 m^2	15.6 m
Roof	No Access	~ 2873 m^2	18.6 m

2.2 Structural layout and load paths

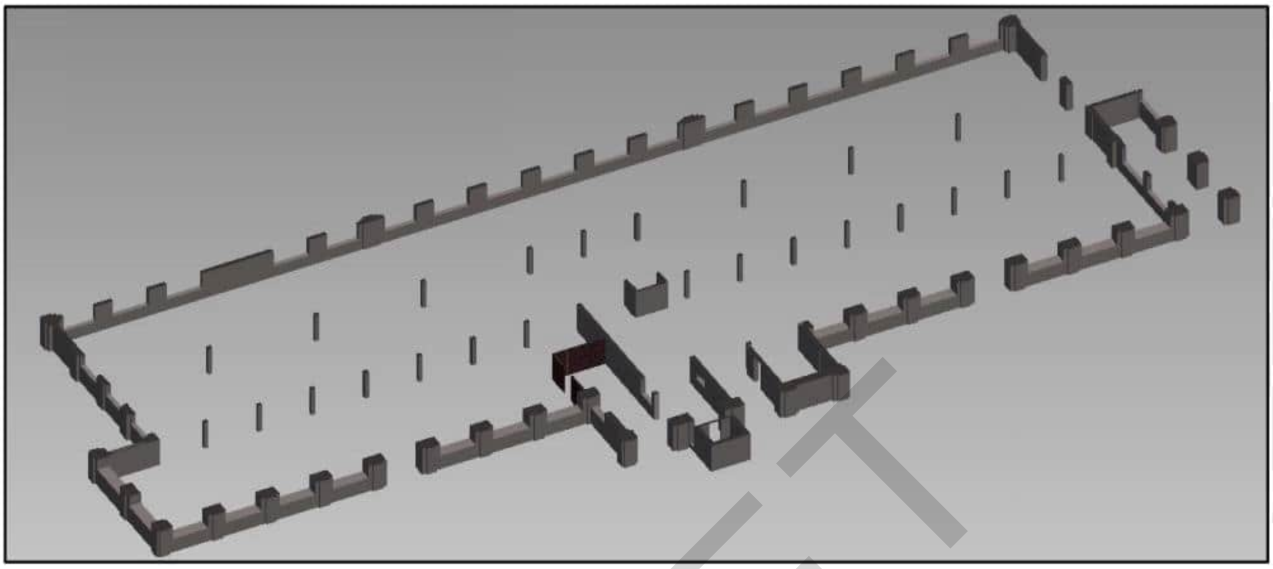

Figure 2-3 Cutaway showing walls and columns at the ground floor level at GNS

The ground floor plate is of slab on grade construction. The reinforced concrete columns that support the upper floors are supported by pad footings. The gravity loads from the upper levels are transferred to the ground through concrete columns. At the centre of the ground floor there is one ‘u-shaped’ shear wall, which transfers both gravity and lateral loads, as shown in Figure 2-3. There are also shear walls on grids 2 and 20, which run in the North-South direction. All other walls at ground level are partition walls and are not intended to be load bearing elements. The lateral load-transfer system in the north-south direction are moment frames consisting of concrete columns with embedded steel angles and concrete encased steel beams and shear walls alongside external elevations. In the east-west direction the lateral load resisting system comprises reinforced concrete moment frames and a shear wall.

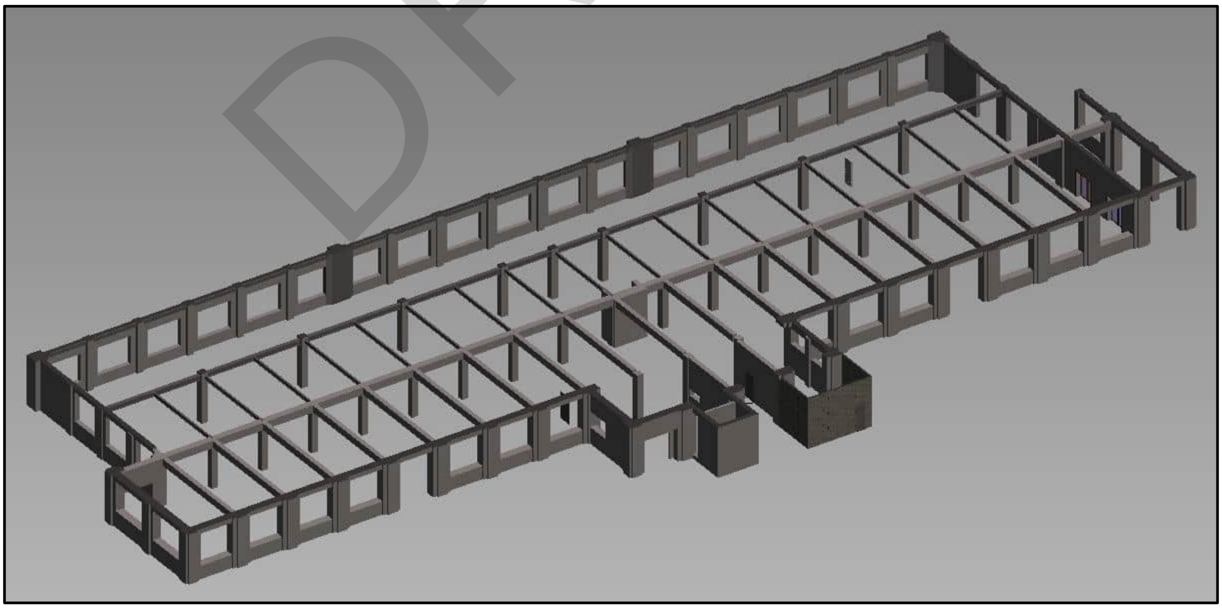

Figure 2-4 Cutaway showing beams, walls, and columns at the ground floor level at GNS

The first floor is a cast-in-situ reinforced concrete floor which sits on concrete encased steel beams. This floor plate spans in the east-west direction, between beams, as shown in Figure 2-4. The gravity loads from the first floor are transferred through this floor plate and beams, and eventually to the ground through concrete columns which contain embedded steel angles.

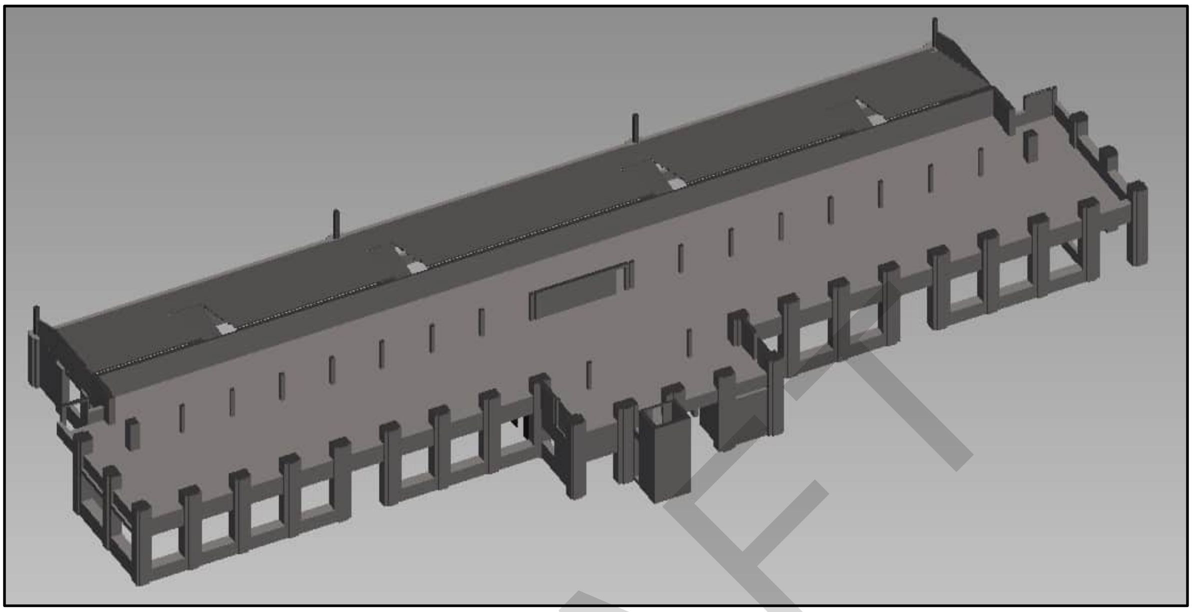

Figure 2-5 Cutaway showing walls and columns at the first floor level at GNS

The concrete columns with embedded steel angles that support the upper floors are present at first floor level. The ground floor ‘u-shaped’ shear wall length extends in the longitudinal direction and the return walls discontinue as shown in Figure 2-5. The gravity loads from the upper levels are transferred to the ground floor columns through both reinforced concrete columns with embedded steel angles and the central shear wall. The lateral load-transfer system at first floor level in the north-south direction comprises moment frames consisting of concrete columns with embedded steel angles and concrete encased steel beams and shear walls alongside the elevations. In the east-west direction, the lateral load resisting system comprises reinforced concrete moment frames and shear walls. All other internal walls at first floor level are lightweight partition walls and are not intended to be load bearing elements. There is direct access to the lower stand from first floor level via four stepped passageways.

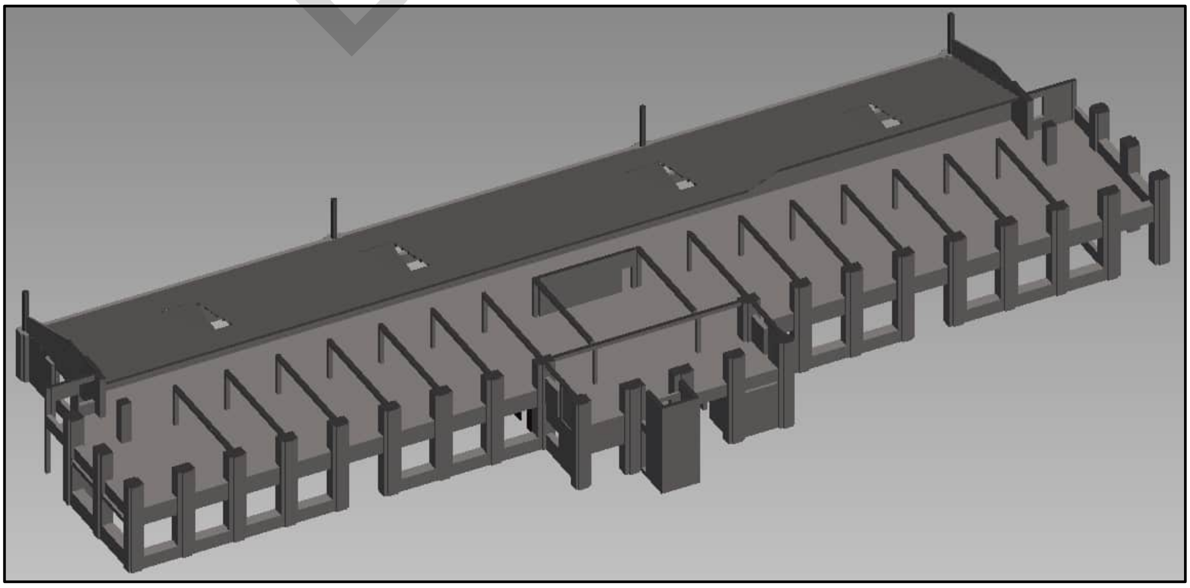

Figure 2-6 Cutaway showing beams, walls, and columns at the first floor level at GNS

D R A F T

The second floor is a cast-in-situ reinforced concrete floor which sits on concrete encased steel beams. This floor plate spans in the east-west direction, between beams, in a similar manner to the first floor and this is shown in Figure 2-6. The occupancy loads from the second floor are transferred through the second floor plate and beams and eventually to the ground through a combination of reinforced concrete columns with embedded steel angles and the central shear wall. The lower stand is supported directly by steel girders which bear on concrete columns. The (north elevation) upper stand supporting circular columns can be seen in Figure 2-6. These columns do not contribute to the lateral resistance system in the structure and transfer vertical gravity loading only.

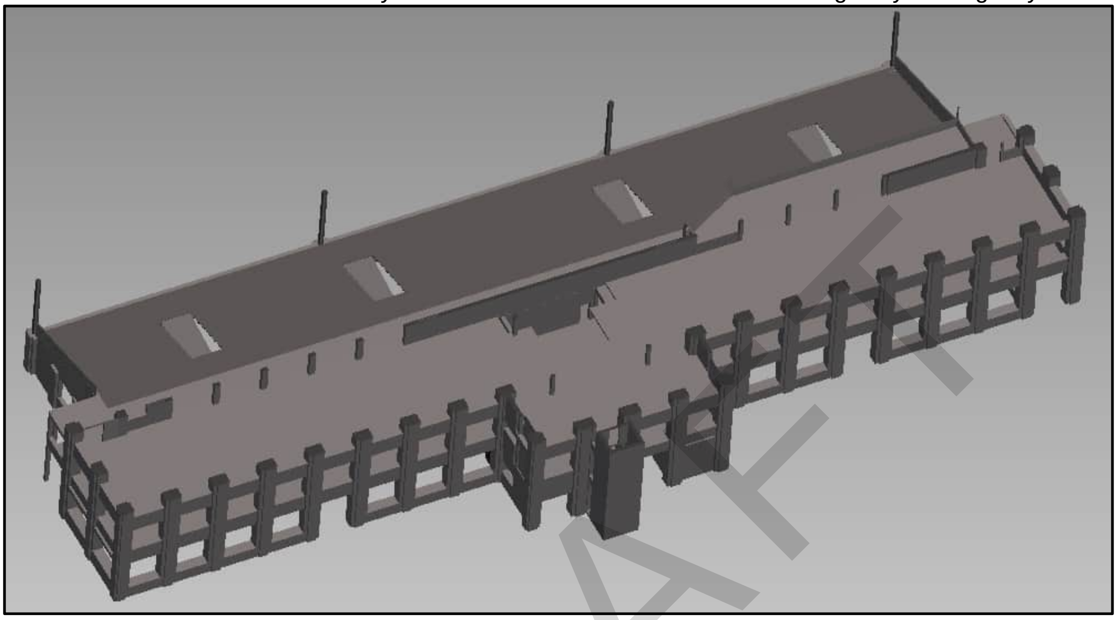

Figure 2-7 Cutaway showing walls and columns at the second floor level at GNS

The longitudinal shear wall on Grid C is larger at second floor level than at first floor level, as shown in Figure 2-7. This shear wall was modified in the early 1980's and is now different from the original 1920's design. The concrete columns with embedded steel angles that support the upper floors are present at second floor level. The gravity loads from the upper levels are transferred to the ground floor columns through both concrete columns with embedded steel angles and the central shear wall. The lateral load-transfer system at first floor level in the north-south direction comprises moment frames consisting of concrete columns with embedded steel angles and concrete encased steel beams. In the east-west direction, the lateral load resisting system is comprised of concrete moment frames and shear walls. All other internal walls at first floor level are lightweight partition walls and are not intended to be load bearing elements. There is direct "free-flow" access to the top of the lower stand from second floor level. The (north elevation) upper stand supporting circular columns can be further seen in Figure 2-7.

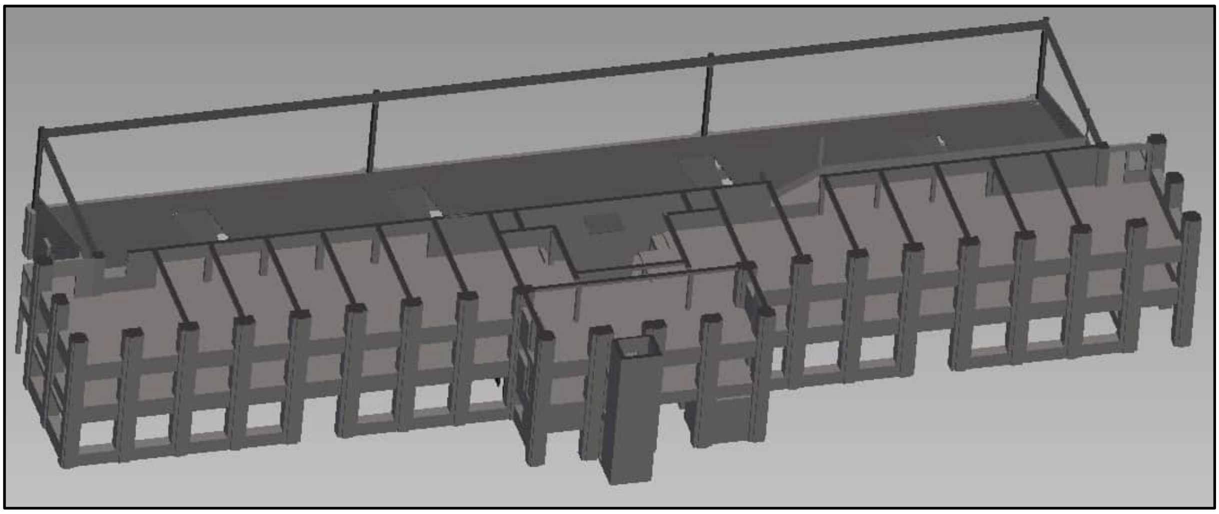

Figure 2-8 Cutaway showing beams, walls, and columns at the second floor level at GNS

D R A F T

The third floor is a cast-in-situ reinforced concrete floor which sits on concrete encased steel beams. This floor plate spans in the east-west direction, between beams, in a similar manner to the second floor and this is shown in Figure 2-8. The occupancy loads from the third floor are transferred through the third floor plate and beams and eventually to the ground through a combination of concrete columns with embedded steel angles and the central shear wall.

The upper stand timber decking and seating is supported on timber joists which span between the top chords of steel trusses located on the numbered grids. These steel trusses span between plated steel perimeter girder running along Grid A and the shear wall on Grid C and have been omitted for clarity. The steel perimeter girders are fabricated from riveted steel plates and are supported on circular steel columns as shown in Figure 2-8. A series of six diagonal tension braces in the horizontal plane provide lateral restraint to the perimeter girders in the east-west direction. The bracing is laid out in an XXX pattern with connections to the web of the plate girder adjacent to each of the circular steel columns. The bracing ties directly into the reinforced concrete frame and is omitted from Figure 2-8for clarity.

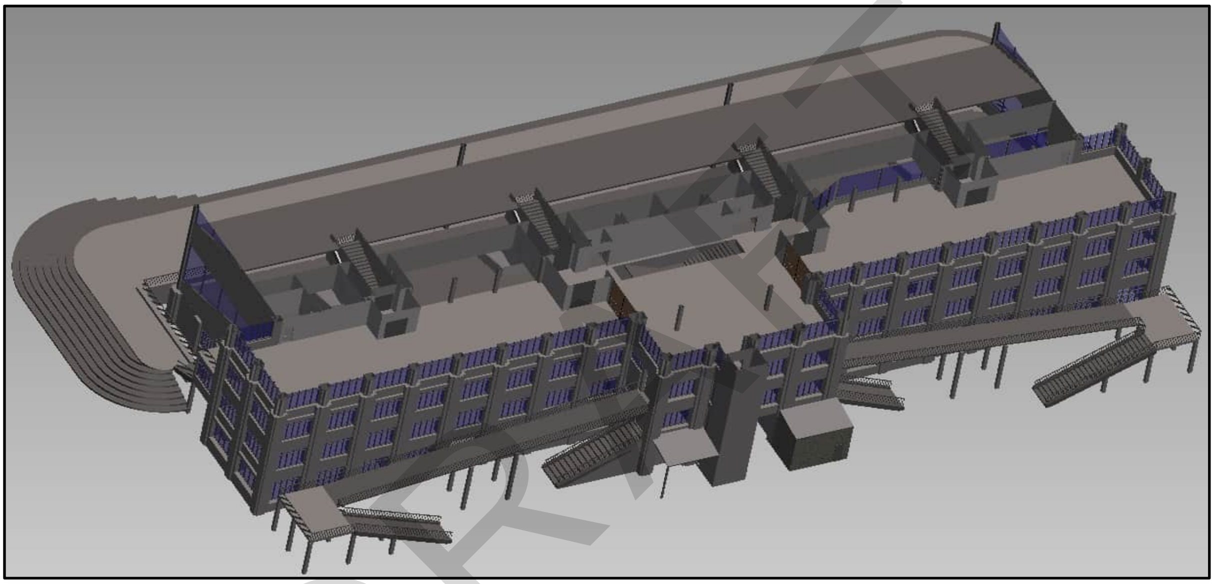

Figure 2-9 Cutaway showing walls and columns at the third floor level at GNS

The longitudinal shear wall on Grid C is larger at third floor level than at second floor level, as shown in Figure 2-9. This shear wall was modified in the early 1980’s and is now different from the original 1920’s design. In its original layout, the shear wall at level 3 ran the full length of the structure, with designed openings for ramps to access the upper stand. Extra openings were cut into this wall in the early 1980’s to allow access to new tote and kitchen areas. The concrete columns with embedded steel angles that support the upper floors are present at third floor level. The gravity loads from the upper levels are transferred to the ground floor columns through both concrete columns with embedded steel angles and the central shear wall. The lateral load-transfer system at third floor level in the north-south direction comprises moment frames consisting of concrete columns with embedded steel angles and concrete encased steel beams. In the east-west direction the lateral load resisting system is comprised of concrete moment frames and shear walls. All other internal walls at third floor level are lightweight partition walls and are not intended to be load resisting elements. There is direct access to the upper stand from third floor level via four stepped passageways, as shown in Figure 2-10. This is the only public access to the upper stand.

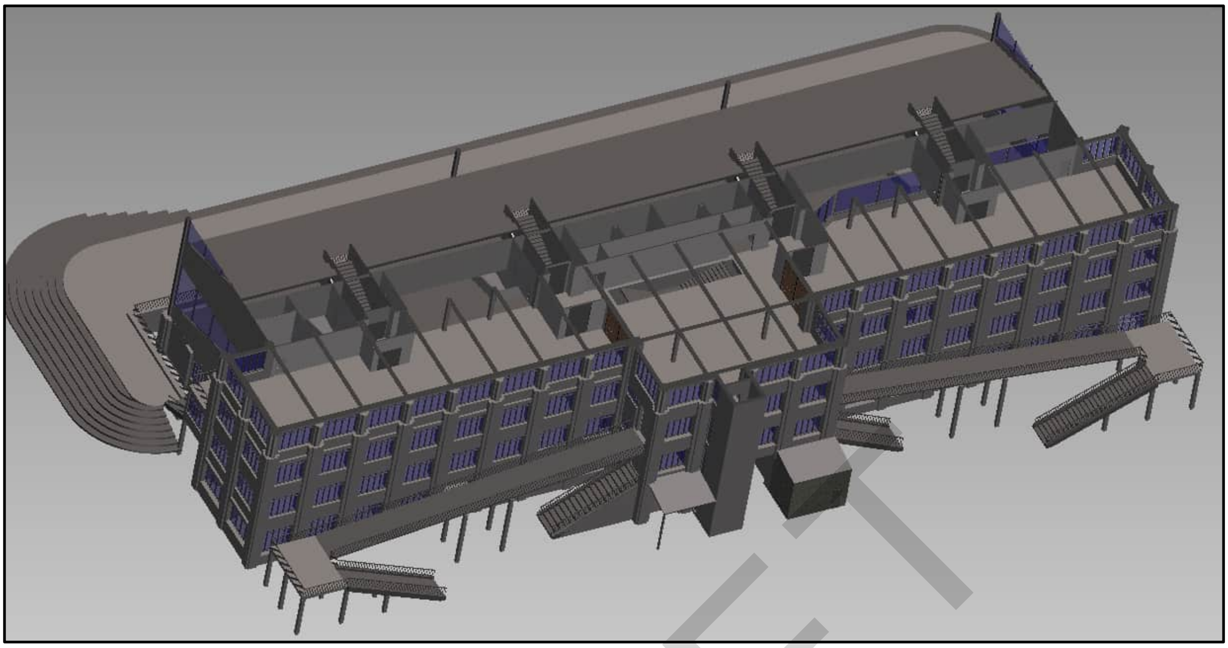

Figure 2-10 Cutaway showing beams, walls, and columns at the third floor level at GNS

The fourth floor is a cast-in-situ reinforced concrete floor which sits on concrete encased steel beams. This floor plate spans in the east-west direction, between beams, in a similar manner to the third floor and this is shown in Figure 2-10. The maintenance access occupancy and storage loads from the fourth floor are transferred through the fourth floor plate and beams and eventually to the ground through a combination of concrete columns with embedded steel angles and the central shear wall.

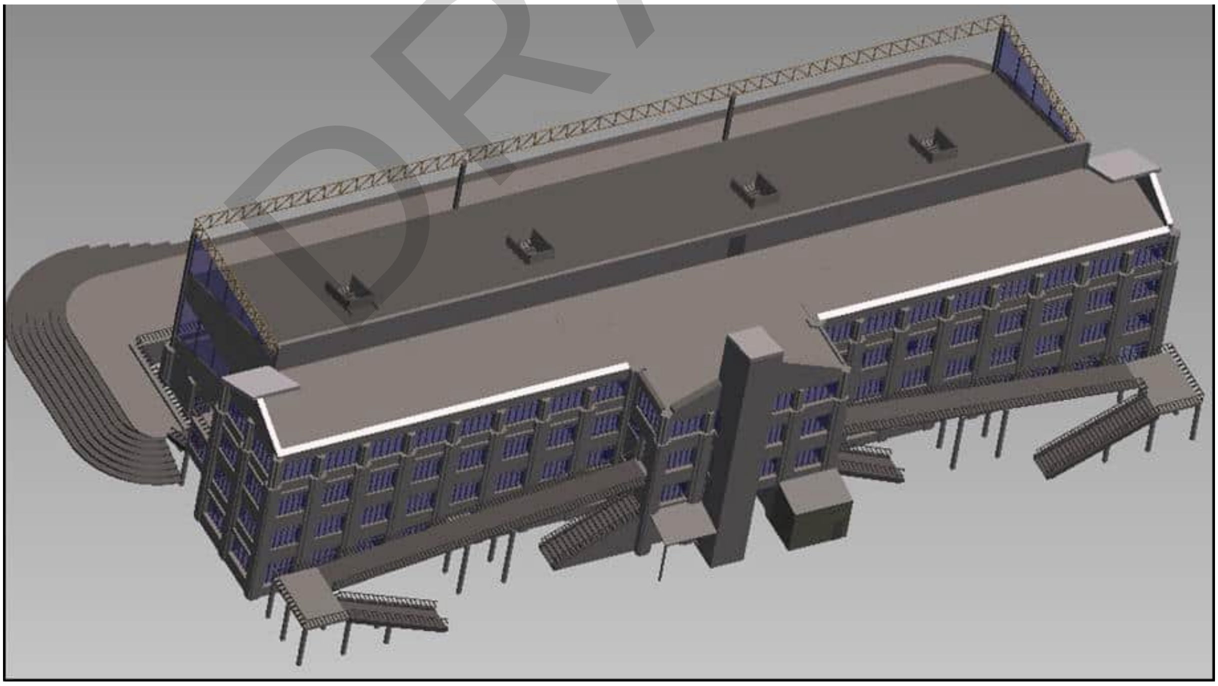

Figure 2-11 Cutaway showing walls and columns at the fourth floor level at GNS

Between Grid C and D the roof is supported on timber purlins spanning between steel rafter beams fabricated from back to back unequal steel angles. The steel rafter beams are supported by the walls on Grid C and D and by three intermediate steel columns fabricated from single equal angle sections. There is no bracing in this section of the roof.

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Between Grid A and C the roof is supported on timber purlins spanning between steel roof trusses located on each numbered grid. The steel roof trusses span between the shear wall on Grid C and the steel perimeter trusses on Grid A with a cantilevered section beyond Grid A. The steel perimeter trusses on Grid A are supported by circular steel columns as shown on Figure 2-11. A series of six diagonal tension braces provide lateral restraint in the east-west direction to the perimeter trusses in a horizontal plane level with the bottom chord of the roof trusses. The bracing is laid out in an XXX pattern and ties directly into the longitudinal reinforced concrete shear wall on Grid C as shown in Figure 2-13.

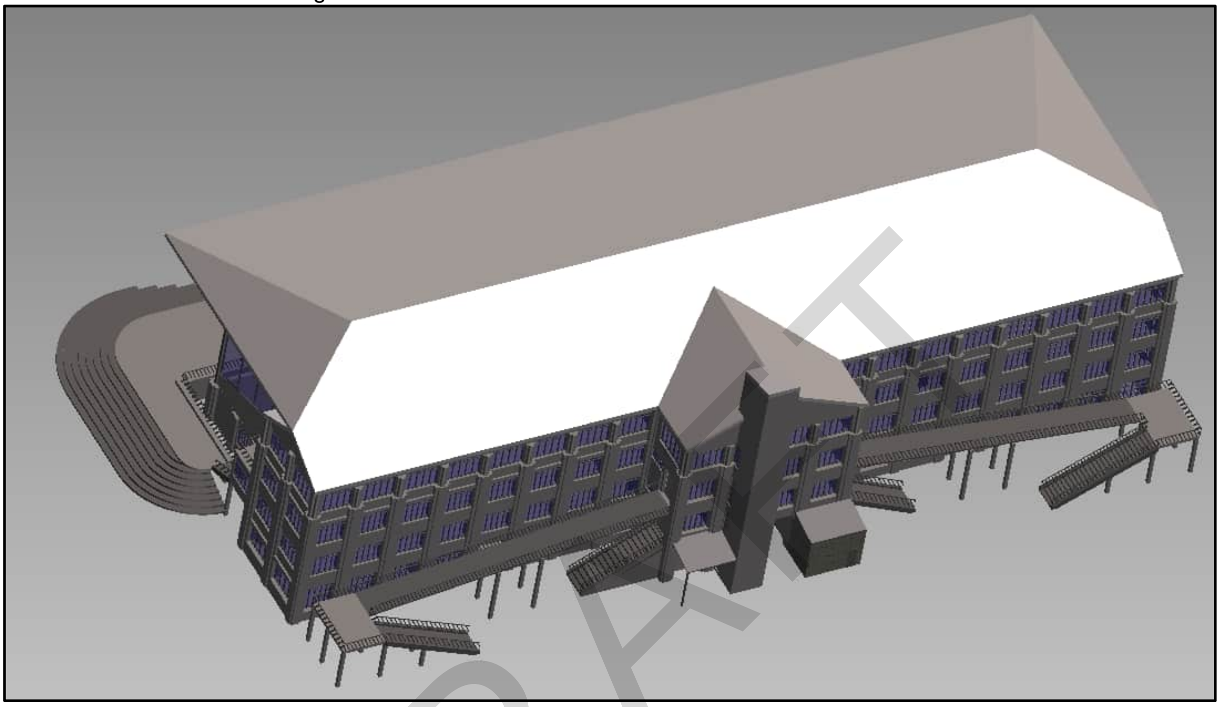

Figure 2-12 3D model showing roof level at GNS

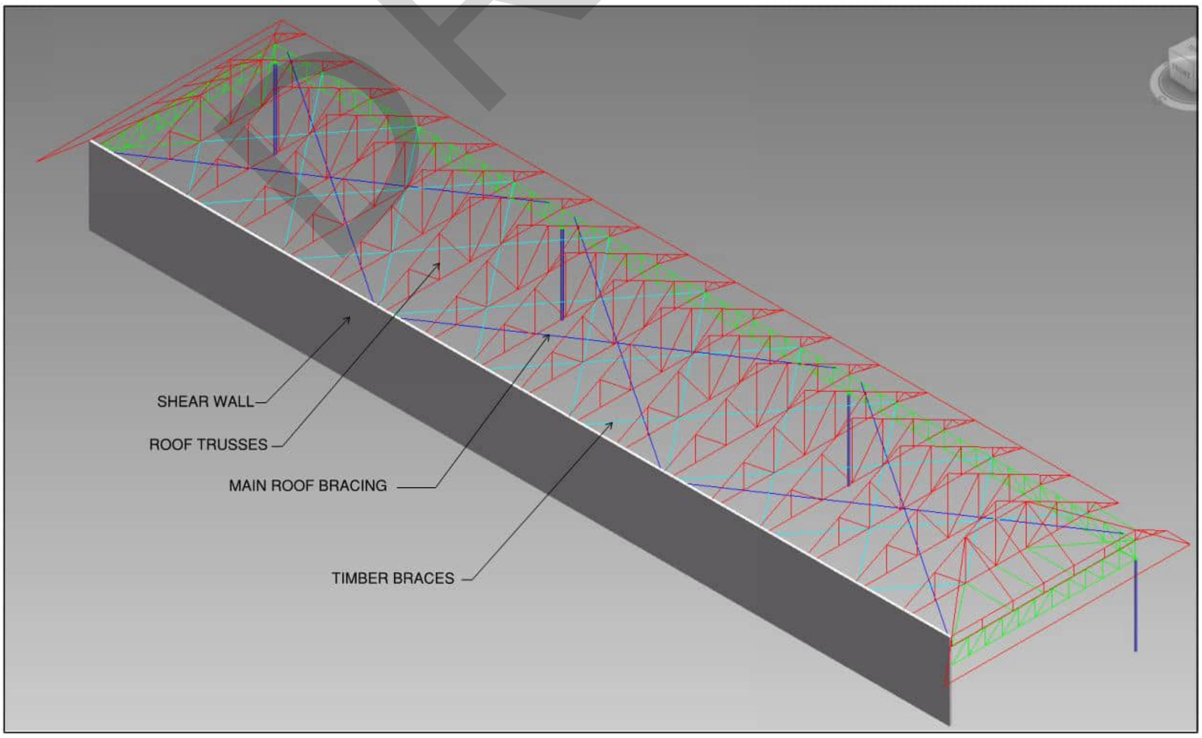

Figure 2-13 Upper stand roof layout

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3.0 Soil Conditions

3.1 Site Investigation

An Engineering Geologist from AECOM undertook a series of shallow intrusive geotechnical inspections on 25 June, 2 July, and 23 October 2015. The inspections were completed in the excavations at the base of column C7 and B4, and alongside the base of strip footing D7 and A4. The purpose of the geotechnical investigation was to confirm the dimensions of the footings, confirm the existing ground conditions at the locations of the footings and determine the bearing strength of the material supporting the foundations.

3.2 Observations

All excavated foundations to date were observed to be founded in natural materials. The footings at C7 and D7 are founded in natural medium dense silty fine sand, whilst the footings and A4 and B4 are founded in natural stiff silt.

The observed foundation dimensions are summarised in Table 1.

Table 3 Observed and estimate foundation dimensions

Foundation Location	Foundation Type	Depth below base of slab (m)	Width (m)
C7	Pad	2.0^{A}	0.76 by 0.77^{A}
D7	Strip	1.0^{B}	Unknown
A4	Strip	0.7^{B}	Unknown
B4	Pad	1.0^{B}	1.6^{A} by $1.6^{\mathbf{B}}$ (up to 2.0^{A} x 2.0^{B} due to rough cast width)

A- Assumed

B- Observed

Notes

- The maximum excavation depth at C7 was 1.8 m below the base of the slab. Therefore the total depth of the pad is assumed at 2.0 m below the base of the slab.
- A variably sized piece of site concrete was observed around the base of the D7 strip. This is not structurally tied to the strip and is assumed to not form part of the strip foundation system.
- The strip at A4 was rough cast against the edge of the excavation. Therefore the width is variable along the length of the strip.
- The top 0.2 m of the pad at B4 is 1.6 m by 1.6 m square and is 0.2 m thick. Below the 0.2 m thickness the foundation has been rough cast against the excavation walls resulting in a variable thickness observed as up to 0.4 m wider along the exposed edge.

At the locations of C7 and D7, where the foundations were constructed within boxing, backfill material was observed in the excavation walls around the footings. At the location of A4 and B4, where the foundations were cast against the excavation, fill is only observed directly beneath the ground slab.

Annotated site photography's of the observed footings are presented in Figure 3-1, Figure 3-2, Figure 3-3, and Figure 3-4.

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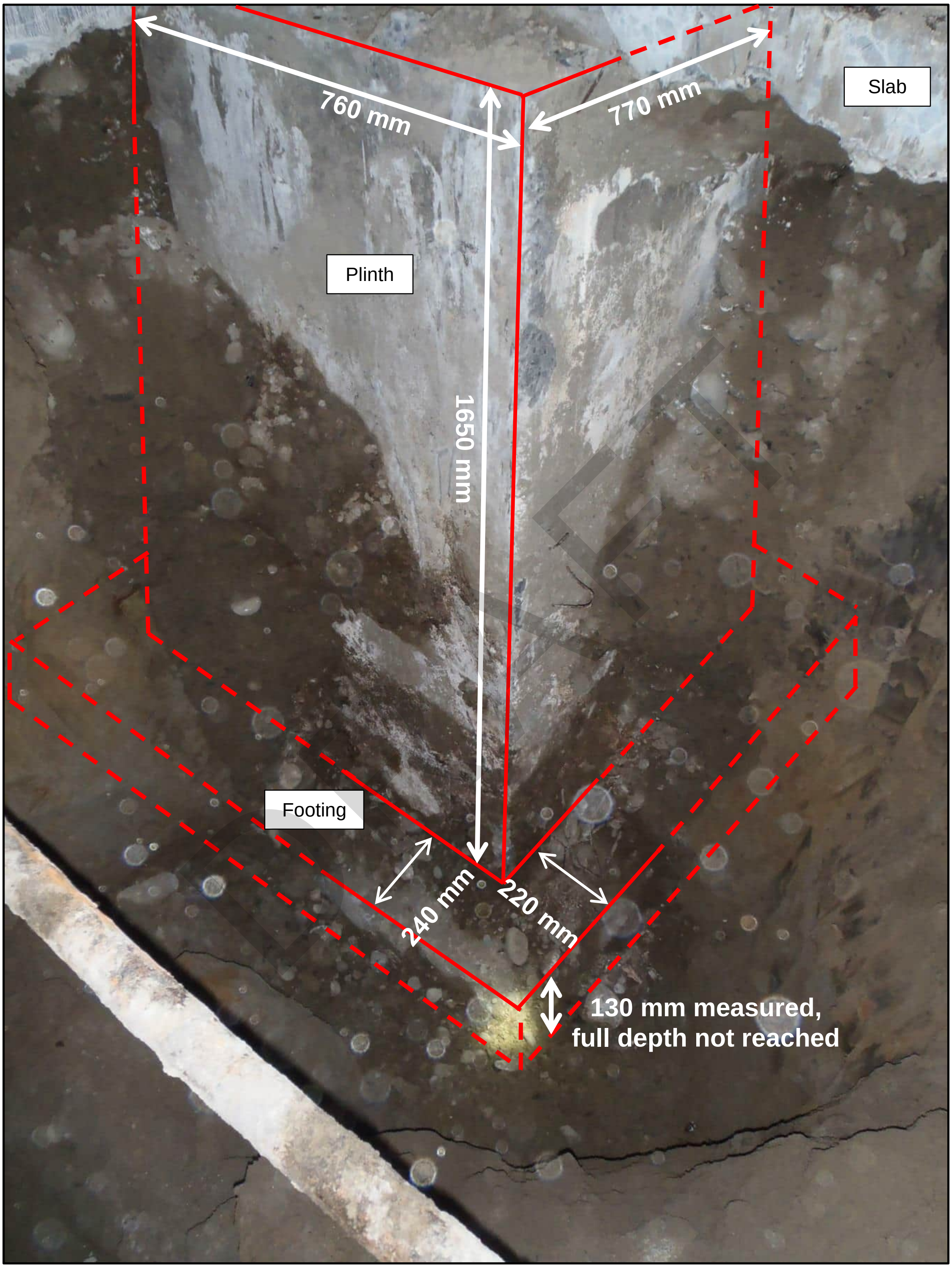

Figure 3-1 Column C7 footing annotation

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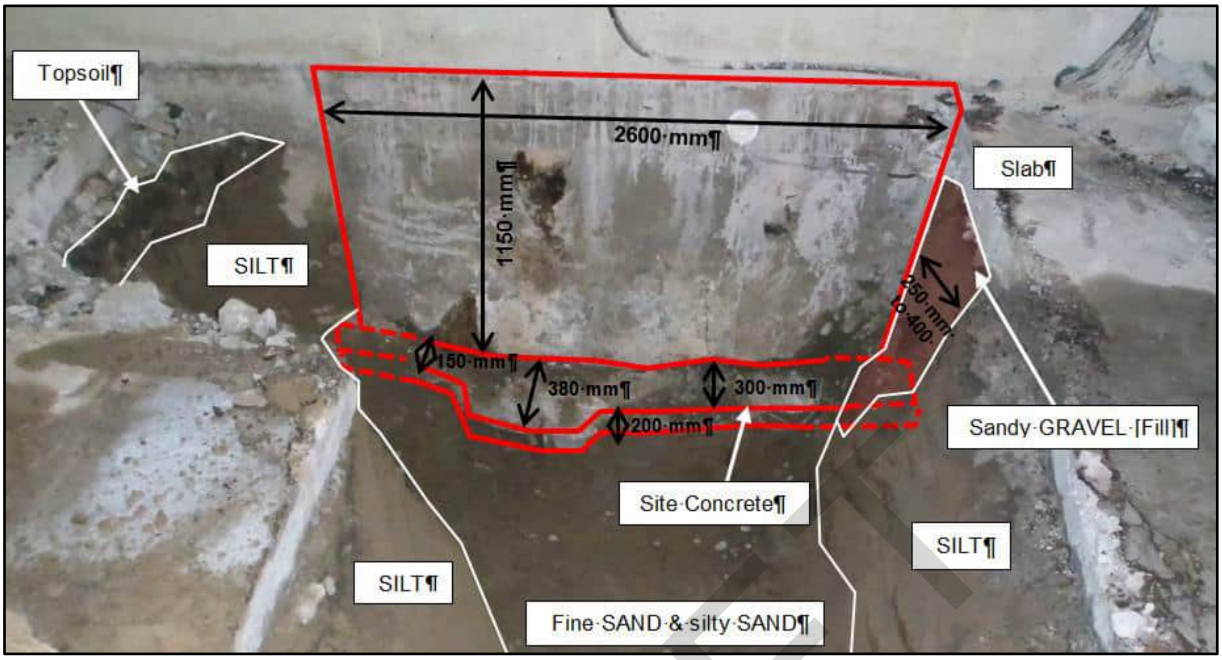

Figure 3-2 **Column D7 footing annotation**

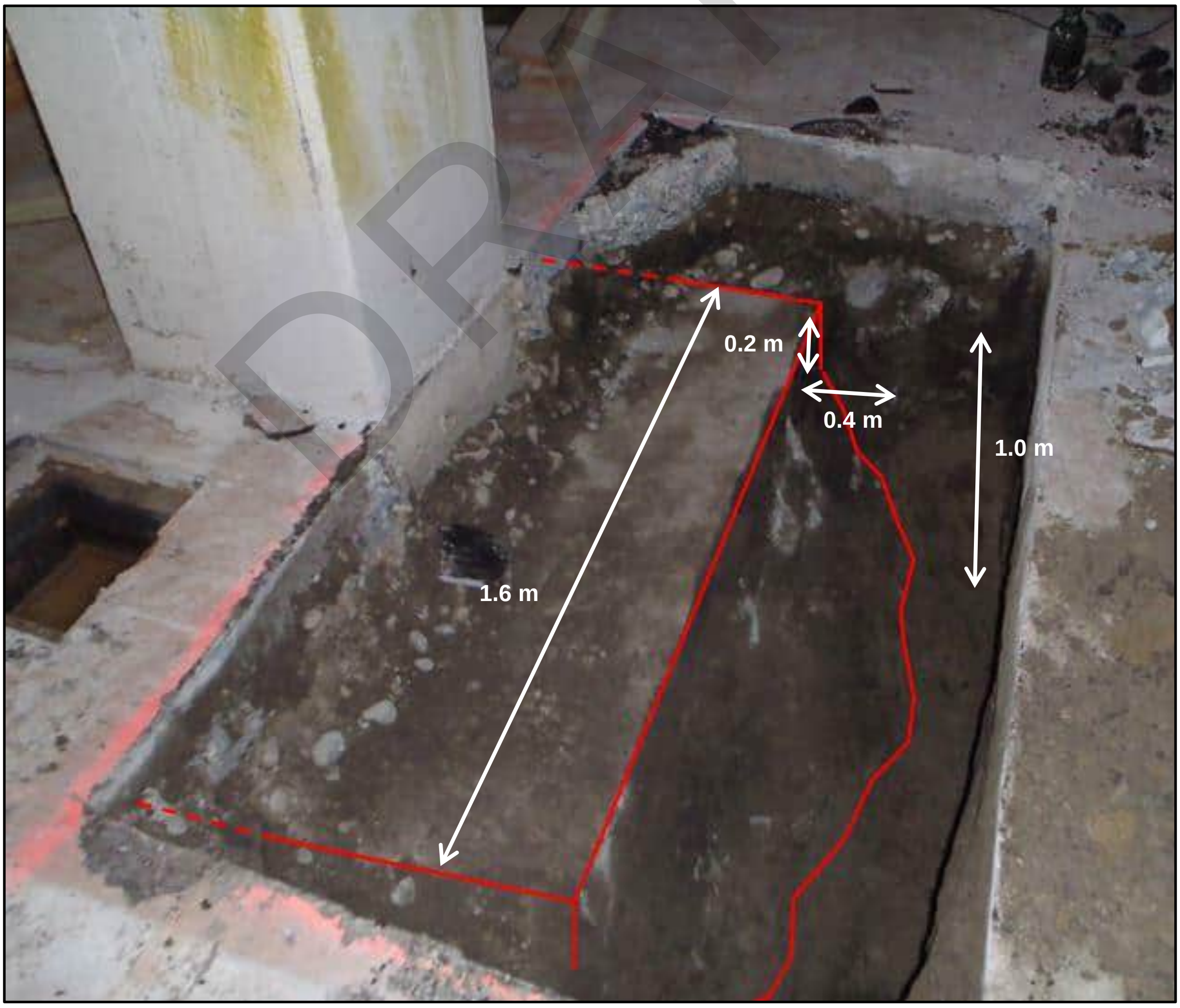

Figure 3-3 **Column B4 footing annotation**

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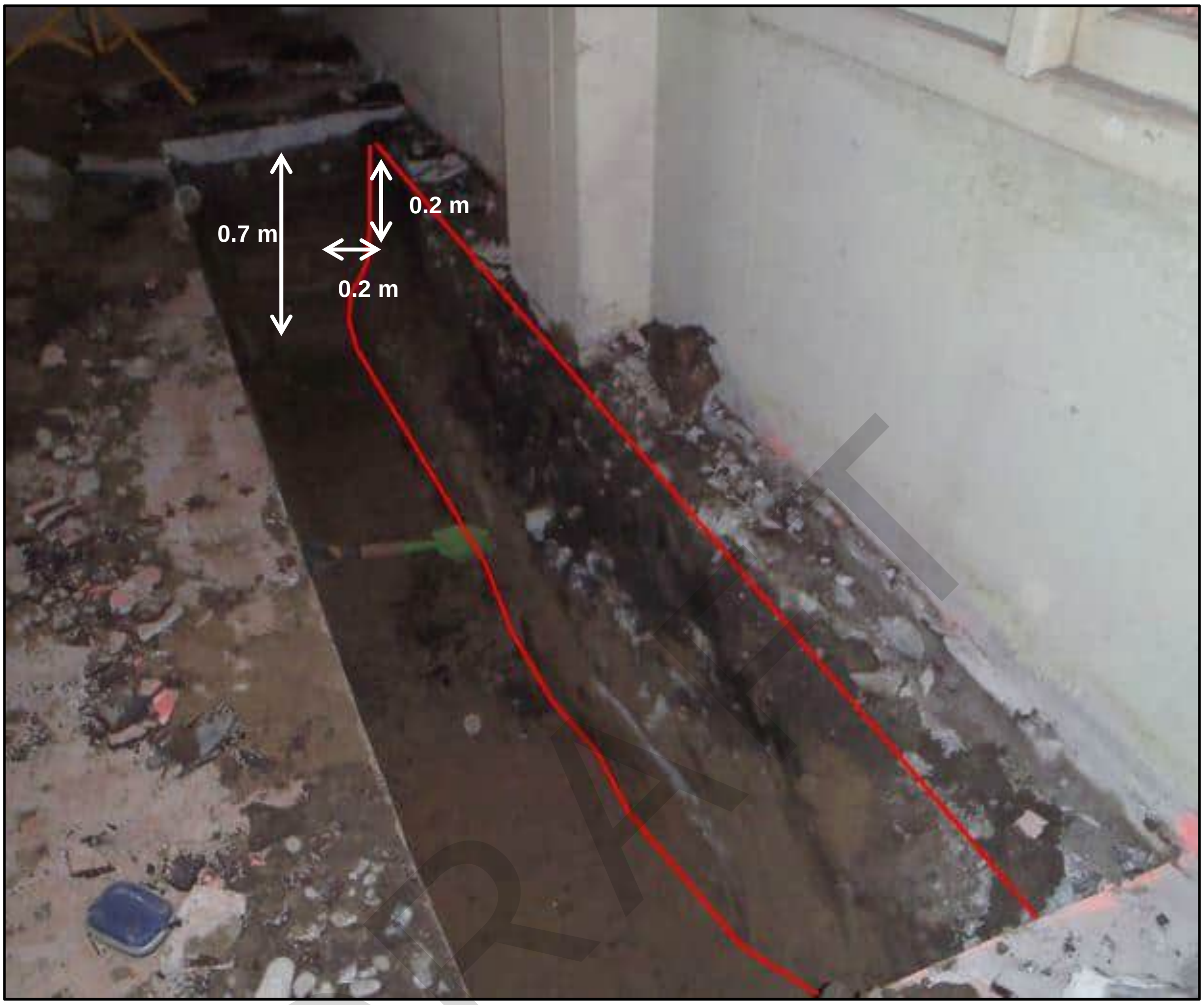

Figure 3-4 Column A4 footing annotation

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3.3 Shallow Investigation

The following investigations were undertaken in each excavation;

C7

- One Hand auger with two adjacent Dynamic Cone Penetrometer (DCP) tests and hand held shear vane tests in cohesive materials on 25 June 2015.
- One Hand auger with one Dynamic Cone Penetrometer (DCP) tests on 2 July 2015.

D7

- One DCP and hand held shear vane tests in cohesive materials on 25 June 2015
- One Hand auger with one Dynamic Cone Penetrometer (DCP) tests on 2 July 2015.

A4

- One Hand auger with an adjacent Dynamic Cone Penetrometer (DCP) test and hand held shear vane tests in cohesive materials on 23 October 2015.

B4

- One Hand auger with an adjacent Dynamic Cone Penetrometer (DCP) test and hand held shear vane tests in cohesive materials on 23 October 2015.

The shallow AECOM investigation confirms that the near surface material is broadly consistent with the ground model outlined in the AECOM geotechnical desk study. The inferred site geology is summarised in Table 4, with depths taken from below base of slab (bbs).

Table 4 Inferred site geology

Material Description	Depth from below base of slab (bbs) (m)	Thickness (m)
Loosely packed sandy GRAVEL [Non engineered fill][A]	0.0	0 – 1.8
Very stiff SILT [Topsoil]	0.0	0.2 – 0.4
Stiff to very stiff SILT and sandy SILT [Loess]	0.2 – 0.4	0.6 – 1.35
Interbedded fine SAND and silty SAND [Springston Formation]	0.8 – 1.6	2.0 – 2.6
Very dense GRAVEL [Springston Formation]	3.3 – >3.7	> 12.0

Notes: A - Encountered in HA/DCP01 only

DCP testing by AECOM in the non-engineered fill encountered in the excavation of column C7 indicates it is of low, inconsistent strength to a depth of 2.4m bbs.

3.4 Soil Springs

The vertical modulus of subgrade reaction used for modelling soil springs was calculated following a method recommended in Bowles [4]. Due to the percentage of settlement associated with elastic compression and liquefaction of fine sand layers within the gravels being unknown, the variability of soil conditions across the site, and the variability of foundation dimensions across the building, a range of subgrade reactions were calculated. The engineering properties shown in Table 5 were used to calculate the recommended vertical modulus of subgrade reaction.

The calculated vertical moduli of subgrade reaction were based on the observed pad foundation dimensions and ground conditions observed, presented in Section 3.2 and 3.3. Should foundation dimensions or ground conditions vary, the calculations should be amended. Should a single value be required for each material type, in-situ testing could be conducted to measure directly.

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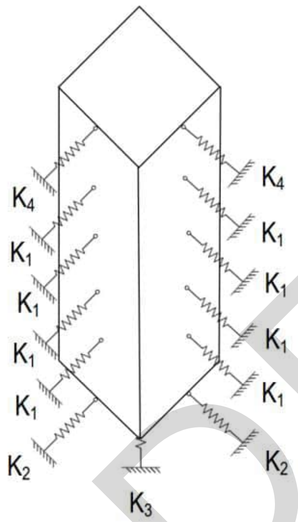
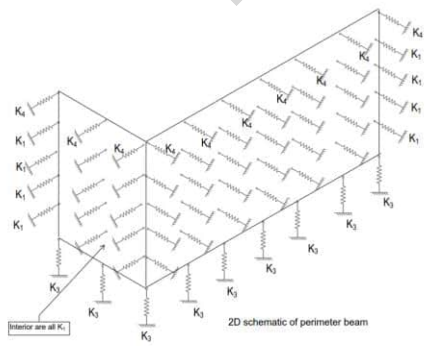
Table 5 **Engineering Properties used in Bowles Method**

	Engineering Properties						
	Static Stress Strain Modulus (MPa)	Bulk Unit Weight (kN/m^3)	Equivalent Undrained Shear Strength (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Poisson's Ratio	Recommended Vertical Modulus of Subgrade Reaction (kN/m2)
Fine Silty SAND overlying GRAVEL	25	19	140	0	32	0.3	20E3 – 60E3
Stiff SILT	15 - 60	18	128	7	30	0.1 – 0.3	20E3 – 45E3

Table 6: Soil springs for model

Type	Location	Spring	Value
FA	Internal Column Grid C	K_1	27 kN/mm at 400mm centres
		K_2	13.5 kN/mm
		K_3	86.4 kN/mm
		K_4	1000 kN/mm
FB	Perimeter Beam Foundation – South	K_1	60 kN/mm/m^2
		K_3	26.1 kN/mm at 400mm centre
		K_4	1000 kN/mm at 400mm centre
FC	Elevator Slab -Perimeter beam modelled same as FB	K_1	60 kN/mm/m^2
		K_2	12 kN/mm at 400mm centre
		K_5	60 kN/mm/m^2

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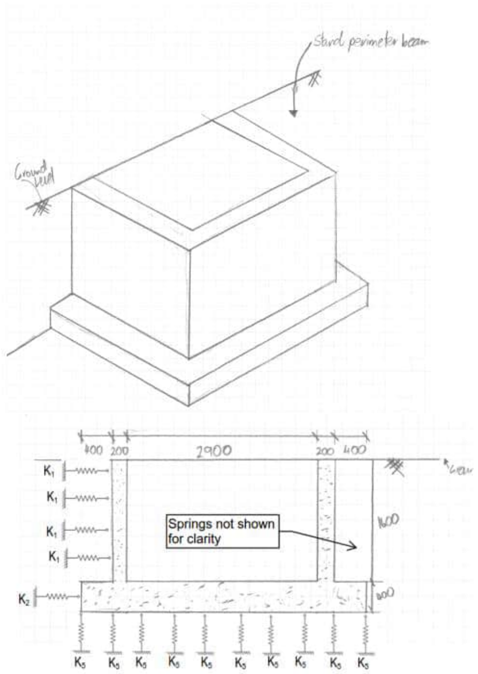
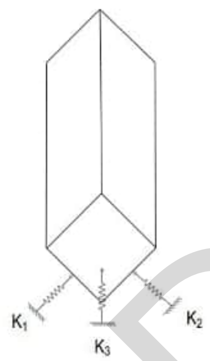
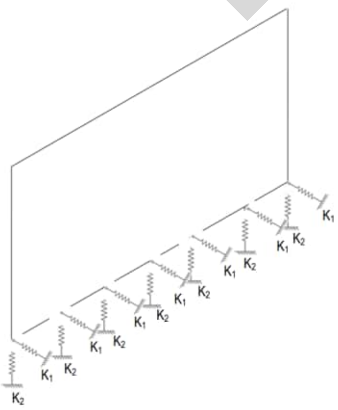
			
FD	Internal Column Grid B 	K_1	1000 kN/mm
		K_2	1000 kN/mm
		K_3	18 kN/mm
FE	Perimeter Beam Foundation – North 	K_1	1000 kN/mm
		K_2	21 kN/mm at 400mm centres

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4.0 Frame Geometry

The geometry used for the analytical model was determined by approximating each member as a line element to form a ‘stick model” of the building. The following modelling assumptions have been used:

- Beams and columns are represented by line elements on their centre lines,
- Walls are represented by shell elements,
- Potential plastic hinge locations on beams are located at each beam / column joint, at the column face, see Figure 4-1,
- Potential plastic hinge locations on columns are assumed to be at the base of columns, and at each beam column joint located at the top and bottom face of each beam (or slab face as applicable), see Figure 4-1,
- Where there are walls between columns, additional potential plastic hinges are located on the line (beam) element at the face of wall openings, see Figure 4-2.
- The foundation conditions have been approximated with multiple vertical and horizontal spring supports and in cases where uplift was expected non-linear gap (compression only) supports were used. The spring stiffness was based on geotechnical recommendations which have assumed that the foundations have been founded on natural medium dense fine silty sand. For further information refer to section 3.

Refer to Appendix B for a graphical representation of geometry used in the analytical model.

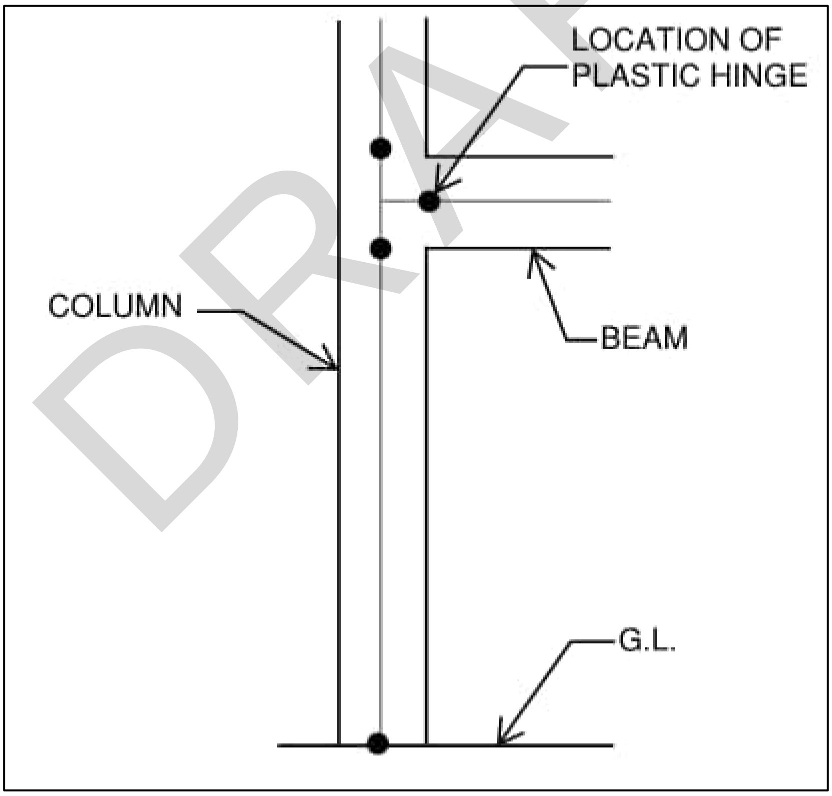

Figure 4-1 Representation of plastic hinge locations

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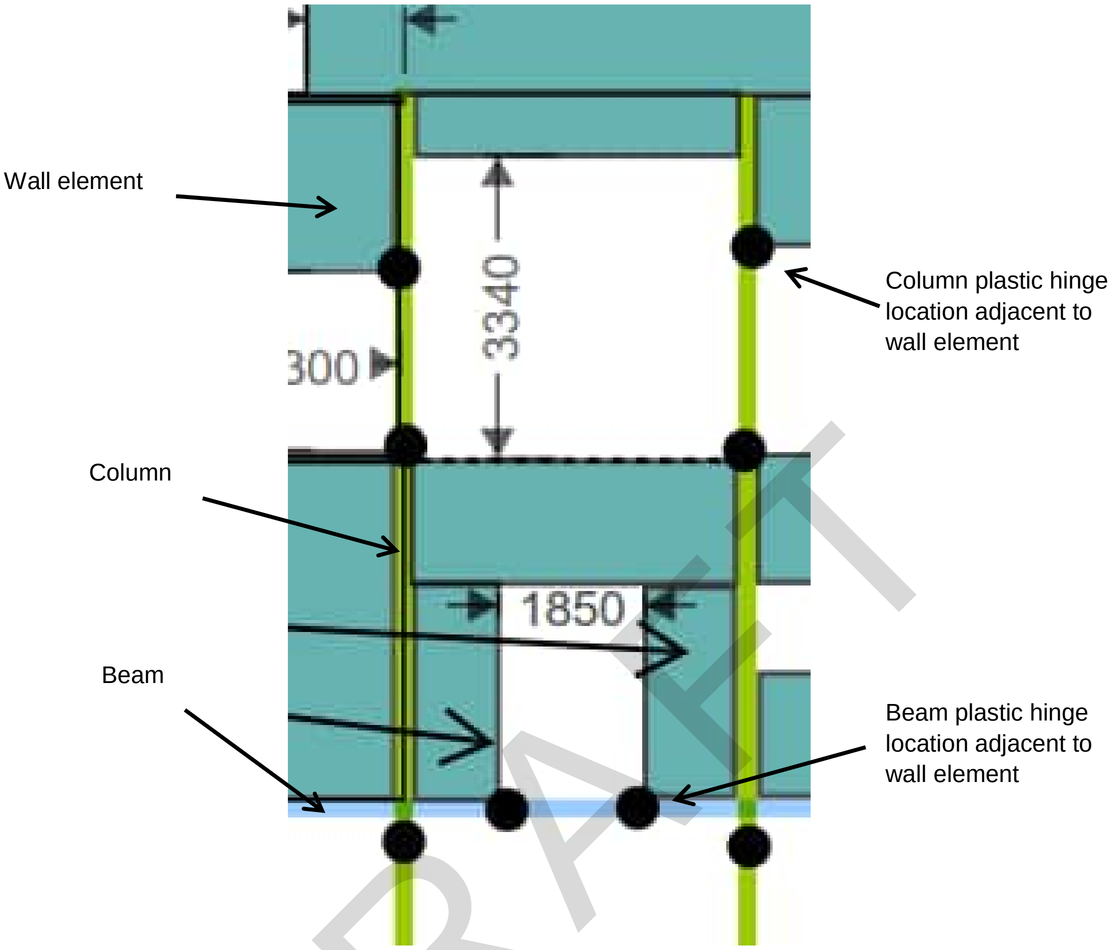

Figure 4-2 Extract from drawing B.21, showing plastic hinge interaction with wall elements

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5.0 Analysis methodology

Three types of analyses have been performed on the building:

- 3D modal response spectrum analysis (RSA),
- 3D non-linear pushover analysis (NLPO),
- Gravity, wind and snow analysis.

The objective of the first two analyses was to determine the likely seismic performance of the building. The gravity, wind and snow analysis was performed to evaluate the structure (generally) for vertical load effects.

The RSA has been performed to gain appreciation of the overall behaviour of the building and its torsional response, evaluate the period of vibration and assess the lower-bound capacity in terms of the current building code. Refer to 5.1 for a detailed procedure of this analysis.

Following on from the RSA, which is a linear elastic method, the building's seismic capacity has been assessed using a NLPO analysis. The NLPO is an analysis technique used to estimate the capacity of a structure beyond its elastic limit up to its ultimate strength in the post-elastic range. It is used to determine how progressive failure is likely to occur in buildings, and can identify the final failure mechanism. Refer to section 5.2 for a detailed description of this procedure.

The gravity, wind and snow analysis has been performed to evaluate the building for loads imposed during "routine" (vertical) loading events.

The building has been modelled in ETABS 2015 software with the sections of various members outlined in Appendix A, geometrical assumptions presented in Appendix B and loadings shown in Appendix C.

When considered appropriate, a separate / independent model of part(s) of the structure (e.g. roof truss) has been created to evaluate a specific element or sub-assembly of the building, which for practical reasons was not necessarily incorporated into the building's global model.

5.1 3D modal response spectrum analysis

5.1.1 General

The modal response spectrum method is a computer-based approach. As with the equivalent static method, an analytical model of the building is developed. The analysis software calculates the different modes of vibration of the structure, finding the period and deformed shape of each mode together with the effective mass of each mode. On the basis of the response spectrum, the lateral force coefficient for each mode is found and the associated structural actions are determined.

Once the structural actions in each mode have been assessed, the next task is to combine the actions. There are a variety of techniques available for deriving appropriate values for design purposes (e.g. SRSS, CQC).

5.1.2 Analysis procedure

Table 7 outlines proposed RSA analysis procedure for the assessment of the building. It also provides references and basic assumptions made in the analysis.

Table 7 RSA analysis procedure

Step	Description	Notes / References
1	**Develop an analytical model and investigate mode shapes**	
1.1	Create a computer model of the building	Analytical model as per Appendix B
1.2	Define and assign material and section properties as required	Section and material properties as per Appendix A
1.3	Assign loads and masses	Refer to Appendix C for loads.

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1.4	Assign restraint conditions	Spring supports to foundation plinths. In cases where uplift was predicted gap elements (compression only supports) are utilized.
1.5	Perform modal analysis	Sufficient modes shall be included to achieve at least 90% of the total mass participation NZS 1170.5:2004, cl. 6.3.3.2
2	**Combine spectral responses**	
2.1	Calculate maximum values of displacements and forces in each mode	Automatically calculated in computer model
2.2	Combine modal action effects to obtain maximum probable response	Use complete quadratic combination (CQC) for deriving appropriate values for assessment purposes. NZS 1170.5:2004, cl. 6.3.4.2 Automatically calculated in computer model
2.3	Account for orthogonal effects in spectral analysis	Use CQC3 method to account for orthogonal effects Automatically calculated in computer model
3	**Scale results**	
3.1	Scale actions and displacements	NZS 1170.5:2004, cl. 5.2.2.2
4	**Review results**	
4.1	Obtain dominant period for the building in two orthogonal directions	
4.2	Obtain displacement demand on the columns supporting upper stand and roof (grid A)	
4.3	Check selected critical structural elements of the structure	
5	**Determine probable member flexural and shear strengths**	
5.1	Probable flexural strength	NZS 3101: Part 1: 2006, cl. 7.4 for concrete NZS 3404: Part 1: 19997, cl. 5.2 for steel Strength reduction factor, ϕ = 1.0 as per NZSEE - Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (NZSEE Guidance) cl. 7.1.1c
5.2	Probable shear strength of beams	NZSEE Guidance, page 7-8, Eq. 7(5) for concrete NZS 3404: Part 1: 1997, cl. 5.11 for steel Strength reduction factor, ϕ = 0.85 as per NZSEE - Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (NZSEE Guidance) cl. 7.1.1c
5.3	Determine probable flexural strength of columns for various levels of axial loads	Response-2000 in combination with Excel spreadsheets utilized to plot moment-axial force interaction diagrams to derive columns capacities

5.4	Probable shear strength of columns	NZSEE Guidance, page 7-8, Eq. 7(6) NZS 3101: Part 1: 2006, cl 7.5 Strength reduction factor, ϕ = 0.85 as per NZSEE - Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (NZSEE Guidance) cl. 7.1.1c
6	**Assess %NBS of the structure**	

5.2 Nonlinear static (pushover) analysis

5.2.1 General

Pushover analysis is a static, nonlinear procedure in which the magnitude of the lateral loading is incrementally and proportionally increased in accordance with a predefined pattern (e.g. inverted triangle, mode-based). With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with effects of the cyclic behaviour and load reversals estimated using a modified monotonic force-deformation criteria and with damping approximations.

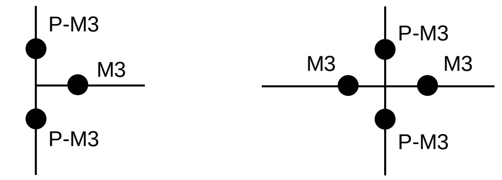
The non-linear static push-over analysis was adopted because it can identify the post-elastic failure mechanism and determine the associated strength and deformation capacity of the structure.

P-delta effects were considered in the non-linear push over analysis.

5.2.2 Analysis procedure

Table 8 outlines nonlinear procedure adopted for the assessment of the building. It also provides references and basic assumptions made in the analysis.

Table 8 Nonlinear static analysis procedure

Step	Description	Notes / References
1	**Determine probable member, flexural and shear strengths**	Refer to Table 7, steps 1.1 to 1.4
2	**Nonlinear static pushover analysis using ETABS 2015 software**	
2.1	Create an analytical computer model of the building, define and assign material and section properties as required, assign loads and masses	Refer to Table 7, steps 2.1 to 2.4
2.3	Define non-linear link properties and assign them to beams and column	Definition based on intrusive investigation to beam-column joints and T-member flexural capacity  Exterior beam-column joint Interior beam-column joint
2.5	Define load patterns for pushover analysis	Gravity loads (dead load + 0.3 live load) acting on a structure. Lateral load pattern in proportion to the first mode shape.

2.6	Define nonlinear static load case	Pre-load the structure with gravity using load control. Apply lateral pushover loads under displacement control. Select node at top of the structure to monitor displacement.
2.7	Run pushover analysis	
2.8	Review the pushover results (for two orthogonal directions)	Plot pushover curve. Plot model showing links at various stages. Show ADRS along with the performance point. Export all results to Excel for post-processing.
3	**Assess %NBS of the structure**	

5.3 Vertical Analysis

5.3.1 General

The vertical analysis considered the effects of dead load (ie structure mass), superimposed dead load (dead load in addition to structure mass), live load (derived from crowd effects, furniture etc), wind load and snow load.

Not all the structure was analysed during the vertical analysis. Instead, representative elements were isolated, modelled and analysed. It is assumed that structural behaviour will not significantly deviate in the remainder of the structure from that observed in the representative elements selected.

Isolation of structural elements into sub-assemblies is considered valid and appropriate for the investigation of vertical load effects.

5.3.2 Analysis procedure

Table 9 outlines the procedure adopted for the assessment of the building for vertical load effects. It also provides references and basic assumptions made in the analysis.

Table 9 Vertical Analysis

Step	Description	Notes / References
1	**Develop an Analytical Model**	
1.1	Create a computer model of structural sub assembly.	Generally 2D, but some 3D models created in both SpaceGass and ETABS.
1.2	Define and assign material and section properties as required.	Steel properties were taken from “Dorman Long” catalogue published circa 1920’s.
1.3	Assign loads.	-
1.4	Assign restraint conditions	Assign sub assembly supports. Note that supports are generally assumed to be unyielding with exceptions as noted.
1.5	Perform linear static and buckling analysis	2nd order analysis completed to identify potential member buckling issues.
2	**Post Process Analysis Outputs**	

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2.1	Determine maximum moments, shears, axial loads and deflections.	Use excel or software post processing capabilities to determined most heavily stressed elements or combinations of stressed elements.
3	**Calculate Design Capacity (D/C) Ratios**	
3.1	Verify that element has sufficient strength and stiffness to resist imposed loads.	Use spreadsheets or proprietary software to determine strength and stiffness characteristics of element or sub assembly and compare these results to imposed loads.

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6.0 Loading assumptions

6.1 General

The basic loading requirements are given in the table below, along with annual probability of exceedance (APE), which has been determined in accordance with clause 3.3 of AS/ NZS 1170.0.

Applicable design loadings are based on the following criteria:

Table 10 Basic Design Criteria

Description	Criteria
Design working life of building	50 years
Importance category	3
Annual probability of exceedance (ULS)	1/1000 (earthquake) 1/250 (snow) 1/1000 (wind)
Annual probability of exceedance (SLS)	1/25 (earthquake) 1/25 (snow) 1/25 (wind)

The design life of 50yrs was used to determine the appropriate loading for the building. AECOM makes no warrant of the actual residual life of the building.

See Appendix C for loads.

6.2 Dead (permanent) loads

Dead loads are deemed to be all permanently fixed structural materials, and include the self-weight of the structural roof system, walls and floors. Dead loads are calculated from unit material weights and structural component dimensions. Weights of material have been allowed for as follows:

- Reinforced concrete: 25 kN/m^3
- Structural steel: 77.5 kN/m^3
- Wrought iron: 76 kN/m^3

Table 11 presents typical sizes and associated dead loads for various structural elements within the building. Refer to Appendix A for details of columns and beams.

Table 11 Selected typical weights

Item	Location	Detail	Dead Load [kPa]	Dead load [kN/m]
Roof	-	- cladding - purlins - trusses / bracing - ceiling / mesh Total	0.2 0.2 0.3 0.15 0.85	- - - - -
Slab	Level 4	- 150mm concrete slab	3.75	-
	Levels 1 to 3	- 200mm (average) concrete slab	5.0	-

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Upper stand	-	- deck/seating - seating plywood/board - joists - trusses/bracing - ceiling - services Total	0.2 0.15 0.2 0.3 0.3 0.2 <u>1.35</u>	- - - - - - -
Lower stand	-	- deck/seating - joists - corrugated board - services Total	0.2 0.2 0.1 0.2 <u>0.7</u>	- - - - -
Shear wall	Level 4, gridline C, typical	200mm concrete wall	5.0	-
	Gridline C, typical	250mm concrete wall	6.3	-
	Level 3, gridline C, locally between gridlines 10 and 12	550mm concrete wall	14	-
Perimeter columns	Type A	approx. 625x650mm	-	16
	Type B	approx. 1100x1100mm	-	40
	Type C	approx. 460x620mm	-	13
	Type D	approx. 770x770mm	-	21
	Type E	approx. 940x940mm	-	30
	Type F	approx. 620x1370mm	-	24
	Type G1	approx. 720x1360mm	-	28
	Type G2	approx. 720x1360mm	-	28
	Type I	approx 1200x1010mm	-	40
	Type J1	approx. 500x660mm	-	10
	Type J2	approx. 500x660mm	-	10
	Type K	approx. 660x750mm	-	16
	Type L	approx. 200x500mm	-	3
	Type M	approx. 600x600mm	-	9
	Type N	approx. 920x940mm	-	22
	Type O	approx. 235mm diameter	-	4
	Type P	approx. 213mm diameter	-	3
	Type Q	approx. 250x250mm	-	3
	Type R	approx. 310x230mm	-	2
Internal columns	Type H1	approx. 520x520mm	-	7
	Type H2	approx. 520x520mm	-	7
	Type H3	approx. 520x520mm	-	7
Internal beams	Type 3	BSB28 (encased in concrete 540x350mm)	-	5

	Type 4	approx. 500x700mm	-	9
	Type 8	BSB21	-	0.7
	Type 9	BSB25	-	0.7
	Type 10	2x BSB23	-	1.4
	Type 11	360UB44.7	-	0.5
Perimeter beams	Type 1A	approx. 1450x610mm	-	22
	Type 1B	approx. 1300x610mm	-	20
	Type 2	approx. 530x610mm	-	8
	Type 5	approx. 1600x580mm	-	23
	Type 6	approx. 500x580mm	-	7
	Type 7	approx. 540x720mm	-	10
	Type 12	approx. 660x210mm	-	4
	Type 13	approx. 550x450mm	-	7

6.3 Imposed loads

Table 12 summarizes all vertical live loads.

Table 12 Imposed loads

Level/Area	Use	Live Load [kPa][1]
Level 0	Workshop / Public access	5.0
Level 1	Public access	5.0[2]
Level 2	Public access	5.0
Level 3	Public access	5.0
Level 4	Maintenance access only / water tanks	5.0[3]
Lower stand	Public access	5.0
Upper stand	Public access	5.0
Roof	Maintenance access only	0.25
Stairs	Public access	5.0

6.4 Wind loads

The wind structural actions were calculated using AS/NZS 1170.2: 2011. The factors used in the calculation of the design wind actions are described below in Table 13.

Table 13: Wind parameters

Item	Factor	Comment / Reference
Region	A7	AS/NZS 1170.2: 2011, figure 3.1(B)

[1] Elements of the structure were "designed" for an imposed load of approx. 2kPa – 3kPa (established via back calculation), therefore this load range was used in some instances as a credible, lower live load limit to investigate a number of live load performance characteristics.
[2] Area susceptible to overcrowding, refer to table 3.1 (type C5) of AS/NZS1170.1:2002
[3] Areas for equipment and plant, refer to table 3.1 (type E) of AS/NZS1170.1:2002

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Regional wind speed, V_R	46 m/s		AS/NZS 1170.2: 2011, table 3.1
Directional multiplier, M_d	N	0.9	AS/NZS 1170.2: 2011, table 3.2
	NE	0.9	
	E	0.8	
	SE	0.9	
	S	0.9	
	SW	0.9	
	W	1.0	
	NW	1.0	
	Any	1.0	
Terrain category	Category 2		AS/NZS 1170.2: 2011, cl. 4.2.1
Height, z	18.6m		
Terrain height multiplier, $M_{z,cat}$	1.07 (interpolated)		AS/NZS 1170.2: 2011, table 4.1(A)
Shielding multiplier, M_s	1.0		AS/NZS 1170.2: 2011, table 4.3
Topographic multiplier, M_t	1.0		AS/NZS 1170.2: 2011, cl. 4.4.1
Site wind speed, $V_{sit,\beta}$	N	44.3 m/s	AS/NZS 1170.2: 2011, cl. 2.2
	NE	44.3 m/s	
	E	39.4 m/s	
	SE	44.3 m/s	
	S	44.3 m/s	
	SW	44.3 m/s	
	W	49.2 m/s	
	NW	49.2 m/s	
	Any	49.2 m/s	
Design wind speed, $V_{des,\theta}$	49.2 m/s		AS/NZS 1170.2: 2011, cl. 2.3

6.5 Snow and ice loads

The snow structural actions were calculated using AS/NZS 1170.3:2003. The factors used in the calculation of the design snow actions are described below in Table 14.

Ice actions have not been considered as the site region is N4 subalpine.

Table 14: Snow parameters

Item	Factor	Comment / Reference
Region	N4 subalpine	AS/NZS 1170.3: 2003, figure 2.2

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Probability factor, k_p	1.35		AS/NZS 1170.3: 2003, table 5.1
Ground snow load, s_g	0.9 kPa		B1 Building Code (incl. amendment 9, September 2010)
Exposure reduction coefficient, C_E	0.6		AS/NZS 1170.3: 2003, cl. 4.2.2
	0.95		AS/NZS 1170.3: 2003, figure 6.3 (duo pitch)
Roof slope, α	30°		
Shape coefficient, μ_i	0.42		AS/NZS 1170.3: 2003, cl. 7.2 & 6.2 (for mono pitch)
	0.34		AS/NZS 1170.3: 2003, cl. 7.2 & 6.4 (for duo pitch)
Roof snow load, s	Mono	0.29 kPa	AS/NZS 1170.3: 2003, cl. 4.2.1
	Duo	0.23 kPa	
Roof edge load / overhang s_e	$6.0x10^{-3}$ kN/m (can be ignored)		AS/NZS 1170.3: 2003, cl. 4.2.3

6.6 Horizontal imposed actions

Horizontal imposed actions due to crowd movement as per clause 3.9 of AS/NZS 1170.1:2002 has not been considered in the analysis as only seismic performance was assessed.

6.7 Seismic loads

The earthquake structural design actions were calculated using AS/NZS 1170.5. The factors used in the calculation of the seismic design coefficient, C_d, are described below:

Table 15 Seismic parameters

Item	Factor		Comment / Reference
Soil Category	D		Deep or soft soil
Location	Christchurch		
Period, T	N-S	0.545	Calculated during the 3D modal response spectrum analysis, in the two orthogonal directions
	E-W	0.434	
Spectral shape factor, $C_h(T)$	N-S	3	NZS 1170.5: 2004, table 3.1
	E-W	3	
Hazard Factor, Z	0.3		B1 Building Code (incl. amendment 12, February 2014)
Annual probability of exceedance	ULS	1000 years	Importance level 3 structure, refer table 3.3 in AS/NZS 1170.0:2002
	SLS	25 years[1]	
Return period factor, R	ULS	1.3	Table 3.5 of AS/NZS 1170.5: 2004
	SLS	0.25[1]	
Near fault factor, N(T,D)	1		NZS 1170.5: 2004, table 3.7
Elastic Site Spectra, C(T)	N-S	1.17	NZS 1170.5: 2004, Eq. 3.1(1)

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	E-W	1.17	
Ductility, μ	1.0		Linear for the 3D modal response spectrum analysis
Structural performance factor, S_p	1.0		NZS 1170.5: 2004, cl. 4.4.2
Structural ductility factor, k_{μ}	N-S	1	NZS 1170.5: 2004, cl. 5.2.1.1
	E-W	1	
Horizontal design action coefficient, $C_d(T)$	N-S	1.17	NZS 1170.5: 2004, cl. 5.2.1.1
	E-W	1.17	

[1] Assessments of the structure under SLS conditions was not undertaken

7.0 Serviceability criteria

7.1 Seismic deflections

Maximum Allowable:	ULS:	2.5% inter-storey in accordance with 7.5.1 of AS/NZS 1170.5:2004
	SLS:	not applicable as only ultimate limit state considered

7.2 Gravity deflections

Deflections under gravity were not considered as the gravity assessment was limited to a strength analysis only.

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8.0 Limitations

The structural assessment of the GNS was divided into three main parts being:

- 3D Non Linear Push Over (NLPO),
- 3D Response Spectrum Analysis (RSA),
- Vertical analysis (including gravity, wind and snow effects).

8.1 Lateral Analysis

All parts of the structure were considered in the lateral analysis models, however some elements were simplified to limit the modelling time and computational effort.

8.1.1 Roof

The perimeter trusses, used to transfer loads from the common trusses to the circular steel perimeter columns located on the west, north and east of the roof were modelled as equivalent beam elements for the lateral analysis.

The common roof trusses (at approximately 4.0m cts) were represented by their bottom chords only for the lateral analysis and are only located at brace attachment locations.

The timber purlins attached to the roof sheet were not modelled.

The in-plane bracing (tension only) at the bottom chord location of the common roof trusses was modelled true for the lateral analysis.

All structure mass and live load was distributed to these elements as appropriate to simulate the insitu conditions.

8.1.2 Upper Stand

The plated steel transfer girder (located on the northern perimeter) used to transfer loads from the common trusses to the circular steel perimeter columns was modelled as a beam element for the lateral analysis.

The common trusses supporting the upper stand were modelled true for the lateral analysis.

The timber joists were not modelled.

The in-plane bracing (tension only) at the bottom chord location of the common roof trusses was modelled true for the lateral analysis.

8.1.3 Lower Stand

The lower stand steel framing was modelled true for the lateral analysis.

The timber joists were not modelled.

8.1.4 External Stairs and Ramps

The external stairs and ramps were not modelled in the lateral analysis as it was considered that they would have little impact on the overall response of the building. However their contributing seismic mass was considered and has been added to the structure. This concession was agreed with Thornton Tomasetti, the peer reviewing engineer.

8.2 Vertical Analysis

The vertical analysis was generally conducted by modelling isolated sub-assemblies of the frames and trusses present in the building.

8.2.1 Roof Trusses

The perimeter trusses, used to transfer loads from the common trusses to the circular steel perimeter columns located on the west, north and east of the roof were modelled as sub-assemblies for the vertical analysis.

The common roof trusses (at approximately 4.0m cts) were modelled as a sub assembly for the vertical analysis.

The timber purlins were modelled as simple beam elements for the vertical analysis.

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All structure mass and live load was distributed to these elements as appropriate to simulate the insitu conditions.

8.2.2 Upper Stand

The plated steel transfer girder used to transfer loads from the common trusses to the circular steel perimeter columns located on the northern side was modelled as a simple beam element.

The common trusses supporting the upper stand were modelled as a sub assembly.

The timber joists were modelled as simple beam elements.

8.2.3 Lower Stand

The lower stand steel framing was modelled as a sub assembly.

The timber joists were modelled as beam elements.

8.2.4 External Stairs and Ramps

The external stairs and ramps were considered in a separate item of work commissioned by Canterbury Jockey Club (and therefore did not form part of this scope of works).

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9.0 Material properties

9.1 Concrete Sample Testing

9.1.1 Test results

Analysis of 25 concrete test results show a statistically significant difference in the compressive strength of samples taken from horizontal elements (floors) and vertical elements (walls). In the absence of NZ-specific guidance on the assessment of in-situ compressive strength of concrete in existing structures, international best practice has been adopted.

The advice provided in BS EN 13791:2007 [5] has been adopted. Test results yield the following distinct concrete grades:

Table 16 Concrete grades

Material designation	Compressive strength fc' (MPa)	Mean specific weight γ_{conc} (kN/m^3)
C25	25	23.3
C15	15.3	23.1

9.1.2 Concrete properties for analysis

The following concrete characteristics will be used for all analysis, whether carried out by hand or using software, and for all design checks:

Table 17 Concrete properties

Characteristic	C15	C25	Formula (if applicable)	Commentary and Reference
Specific weight (γ_{conc})	23.1 kN/m^3	23.3 kN/m^3	Derived from testing	Mean value of samples adopted
Compressive strength (f_c')	15.3 MPa	25 MPa	Derived from testing	Value determined in accordance with BS EN 13791:2007 [5]
Modulus of elasticity (E_c)	19900 MPa	23700 MPa	$E_c = 3320\sqrt{f_c'} + 6900$	cl. 5.2.3 NZS3101:2006 [6]
Modulus of rupture (f_r)	2.35 MPa	3.03 MPa	$f_r = 0.6\,\lambda\sqrt{f_c'}$	cl. 5.2.4 NZS3101:2006 [6]
Direct tensile strength (f_{cr})	1.41 MPa	1.82 MPa	$f_{cr} = 0.36\sqrt{f_c'}$	cl. 5.2.6 NZS3101:2006 [6]
Poisson's Ratio (v)	0.2	0.2	Codified value	cl. 5.2.7 NZS3101:2006 [6]
Coefficient of thermal expansion (α)	0.000012 /K (12x10^6 /°C)	0.000012 /K (12x10^6 /°C)	Codified value	cl. 5.2.9 NZS3101:2006 [6]
Shear Modulus (Gc)	7950 MPa	9470 MPa	$G_c = 0.4\,E_c$	cl. C7.6.1.3 NZS3101:2006 [6]

As per the terms of AECOM's engagement, and as directed by the insurer's engineer (Thornton Tomasetti) AECOM have adopted a concrete strength of 25MPa in the analysis.

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9.2 Steel Reinforcement Sample Testing

Test results show that there is a statistically significant difference in the steel properties of the 'large' diameter and 'small' diameter bars found during the intrusive works onsite. 13 large diameter samples were tested and 3 small diameter samples were tested. For the purpose of analysis, two distinct materials have been defined, as follows:

Table 18 Reinforcement steel grades

Callout	Bar type	Nominal Size	Yield strength (fy)	Tensile strength (fu)	Stress ratio (Rm / Re)	% elongation	Design size (SAP)	Design area (SAP)
R307B	Round	7 mm	307 MPa	340 MPa	1.11	17%	6.8 mm	36.3 mm^2
R296C	Round	19 mm	296 MPa	451 MPa	1.51	20%	19 mm	284 mm^2

The reinforcement callout is a three part coding system, (*XYYYZ*) based on EN10080 and NZS3101. This system allows the reinforcing material to be described in terms of type, yield stress, and ductility.

- *X:* bar type – Round (R) or deformed (D)
- *YYY:* bar yield grade - yield strength (fy) of material expressed in MPa
- *Z:* bar ductility grade – example below shown is for 350MPa steel. Grade A, B, or C based on ductility measurements with thresholds defined in accordance with NZS3101:2006.

Note that the standard New Zealand ductility grading L, N, and E have intentionally not been used as although the tested steel may exhibit similar elongation properties to these categories, insufficient testing has been carried out to suggest that the tested steel can be accurately classified in accordance with NZS3101:2006.

Table 19 sets out the stress ratios and elongation limits used to define each reinforcement steel class:

Table 19 Steel reinforcement grades

Grade YYYZ	Yield stress fy (MPa)	Stress ratio Rm / Re	Total elongation %	Comment
350A	350[4]	>= 1.03	>= 1.5%	Low ductility – analogous to NZS3101 grade 'L'
350B	350	>= 1.08	>= 5.0%	Normal ductility – analogous to NZS3101 grade 'N'
350C	350	>= 1.15	>= 15%	High ductility – analogous to NZS3101 Seismic grade 'E'

Where a sample exhibits properties which place the sample in a transitional zone between grades, i.e. the stress ratio corresponds to ductility grade B and total elongation corresponds to ductility grade C, then the lower bound conservative ductility grading has been chosen.

For the analysis the reinforcement grade R296C, as per Table 18, was used.

9.3 Steel Reinforcement Scanning

There are no original construction drawings available for the Grand National Stand therefore reinforcement used in the concrete elements could not be readily evaluated for the analysis. To build a representative analytical model of the structure, selected elements were scanned by a specialist subcontractor. A range of beams, columns, walls and slabs were investigated to determine reinforcement patterns, reinforcement sizes and cover depth.

Two types of scanners were used, the PS200 which as a guidance can scan up to 100mm into the scanned element and gives the size of the reinforcement and the PS1000 which as a guidance can scan up to 300mm

[4] yield grade 350 is used as an illustrative example only

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deep and provides a detailed picture of the location of reinforcement. The PS200 scanner was used to scan in all locations except for the level 4 shear wall. The PS1000 was used more sparingly, mainly as a verification of the PS200 results or if the PS200 did not detect any reinforcement.

It was envisaged that reinforcement scanning would provide a comprehensive understanding of the reinforcement present in the structure, but the results were largely inconclusive. In some locations, no reinforcement was detected and in others the PS200 and PS1000 investigations did not provide definitive results. The reasons for the inconclusive results were likely to be:

- depth of the concrete cover,
- steel sections such as beams and angles imbedded within the concrete elements (and thereby invalidating the scan results),
- presence of random reinforcement arrangements throughout the structure.

To properly verify the amount and extent of reinforcement, further intrusive works involving removal of concrete cover and exposure of steel reinforcing bar was undertaken. This highlighted the inclusion of structural steel beams and structural steel angles encased within concrete elements and the variability of steel reinforcement positioning.

9.4 Structural steel

AECOM were furnished with a digital copy of the steel properties tables [7] used in the design of the original structure, circa 1922. AECOM have relied upon this set of tables for all material information relating to the structural steel used in the construction of this building. The following table summarizes the information acquired:

Table 20 Structural steel characteristics

Characteristic	British units	SI
Specific weight (γ_{steel})	490 lbs/ cubic foot	76.9 kN/m^3
tensile strength (f_u)	28 ton / square inch	433 MPa
Elongation at failure	20%	0.2
Max permissible stress (f_b)	7.5 ton / square inch	116 MPa
Estimated Yield Stress (f_y)	13.1 ton / square inch	203 MPa
Young's Modulus (E)	12000 ton / square inch	185 GPa (assumed) [8]

The load capacities quoted in the Dorman Long design manual are based on the assumption of full lateral torsional buckling restraint and therefore it was not necessary to consider pre-yield buckling behaviour or any strength reduction over the numbers quoted. The 1906 structural steel design was based on permissible stress and was not based on limit state theory. Therefore, to define an appropriate structural steel for use in the seismic assessment, a yield stress needed to be estimated from permissible stresses given in the tables. Based on the existing literature [9] a ratio between the yield and permissible stresses for structural steel from 1920's is approximately 1.75. Therefore the yield strength for structural steel was estimated as 203MPa.

Upon discussion with the insurer's engineer, it was understood that they favoured an increase in yield strength. As per the terms of AECOM's engagement, and as directed by the insurer's engineer (Thornton Tomasetti) AECOM have adopted an increased structural steel yield strength. This structural steel has been given a designation of S230 and its properties are shown below in Table 21. 1980's alterations to the structure introduced more modern structural steel and this has been labelled as S250 and its properties are also shown in Table 21 below.

Table 21 Structural steel properties used

Characteristic	S230	S250
Specific weight (γ_{steel})	77 kN/m^3	77 kN/m^3
Poisons Ratio	0.3	0.3
Tensile strength (f_u)	287.5 MPa	410 MPa

Estimated Yield Stress (f_y)	230 MPa	250 MPa
Young's Modulus (E)	200 GPa	200 GPa

9.5 Circular Steel Columns (supporting roof and upper stand)

Uncertainty existed regarding the steel alloy used in the Columns on Grid A which support the forward edge of the upper stand and the GNS roof.

5mm holes were easily drilled into the columns to a depth of 70mm at third points around the circumference of the member. Although not drilled to the column centre, the depth of the drill holes suggested that the columns were likely to be solid. The columns were also tested with a magnet and found to be ferrous.

A sample of the column was removed and chemical composition and mechanical testing was undertaken by a local laboratory. Importantly, it was determined that the material was not a "cast iron".

Generally, the sample was found to conform with a material compliant with a UNS Number of G10210 and SAE-AISI Number of 1021. Our materials research indicated that steel classified as AISI 1021 generally had the following mechanical properties:

- Yield Stress, fy = 395 MPa,
- Ultimate Tensile Stress, fu = 470 MPa,
- Youngs modulus, E = 190-210GPa,
- Shear modulus, G = 80 GPa,
- Rockwell B hardness = 68.

The material sampling was limited to one location on one of the eight columns and the only mechanical property testing undertaken was a Rockwell B hardness test giving a hardness of 63. Due to the limited testing undertaken and the poor correlation between the Rockwell B test value and the characteristic AISI 1021 value the following mechanical properties were adopted:

Table 22 Circular Steel Column Properties

Characteristic	S230
Ultimate strength (f_u)	287.5 MPa
Shear Modulus (G)	76.9 GPa
Yield Stress (f_y)	230 MPa
Young's Modulus (E)	200 GPa

9.6 Timber members

In the absence of specific testing, all timber members were assumed to have equivalent properties of Grade SG8

Characteristic	S230
Density	5 kN/m^3
Bending Stress (f_b)	14.0 MPa
Compressive Stress – parallel to grain (f_c)	18.0 MPa
Compressive Stress – perpendicular (f_p)	8.9MPa
Tensile Stress (f_t)	6.0MPa
Shear Stress (f_s)	3.8MPa
Young's Modulus (E)	8.0 GPa
Shear Modulus (G) based on E/15	0.53 GPa

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10.0 References

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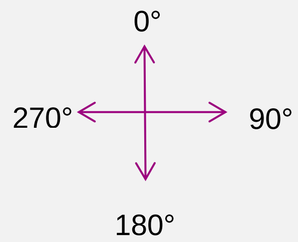
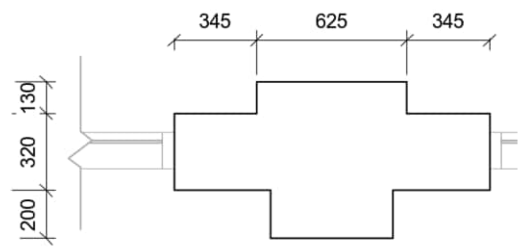
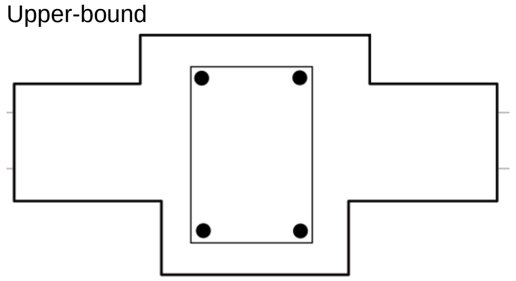
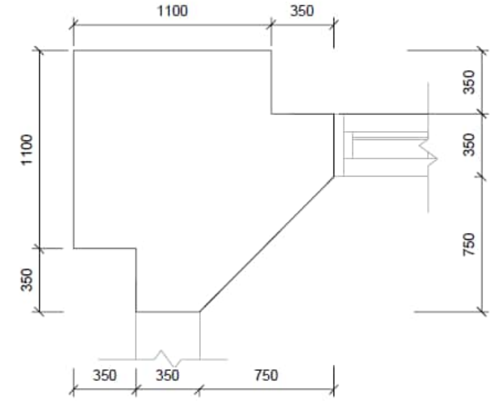
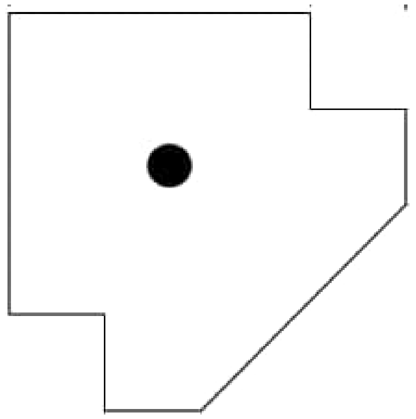
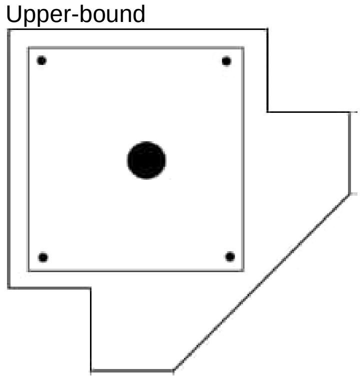
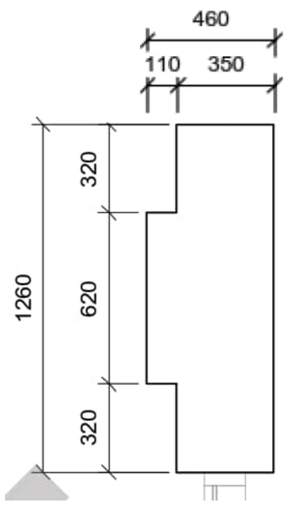
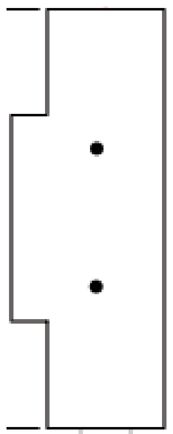
[8] Standards New Zealand, *NZS 3404:1997 (& 2009, Partial Update); Steel Structures,* Wellington: Standards New Zealand, 2009.

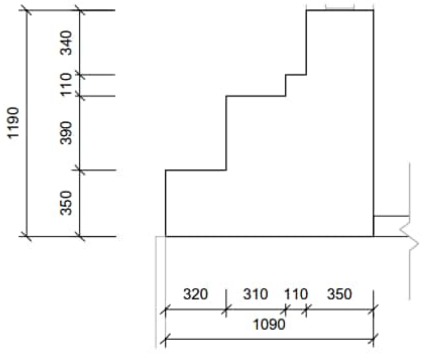
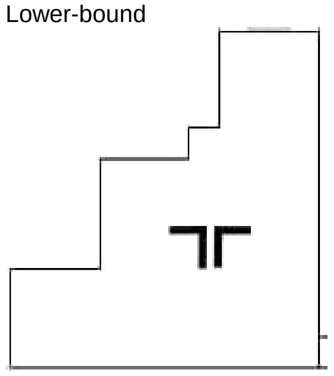
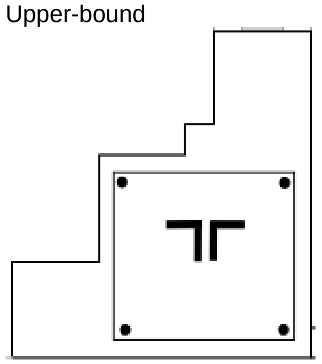
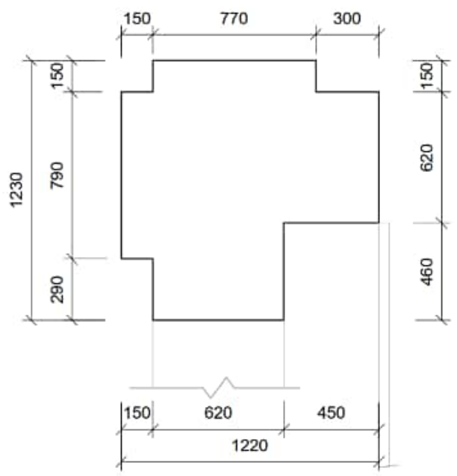
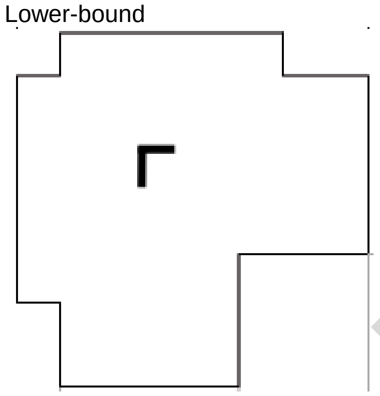
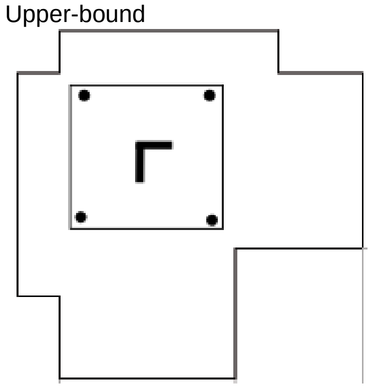
[9] A. N. Beal, “A history of the safety factors,” The Structural Engineer 89(20) 18 October 2011, 2011.

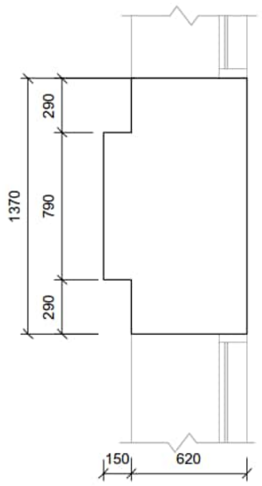
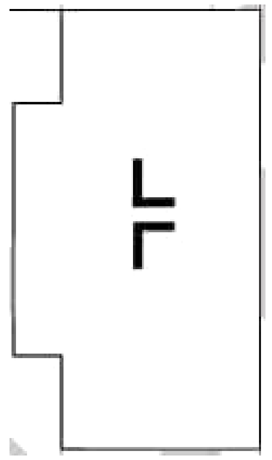
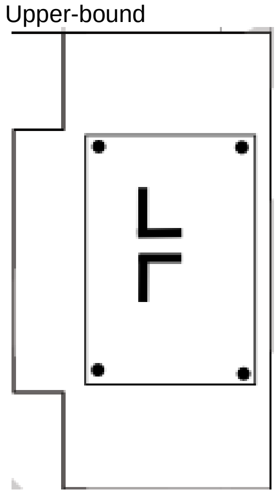
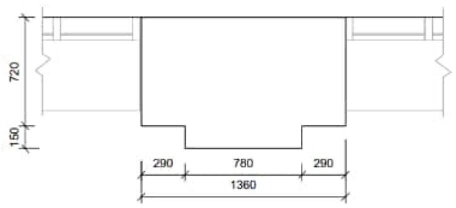
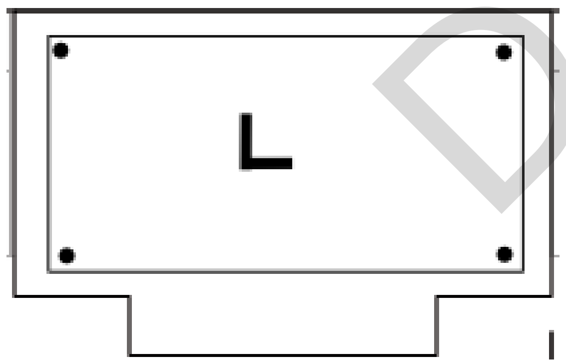
Appendix A

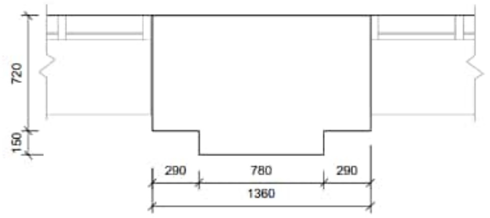
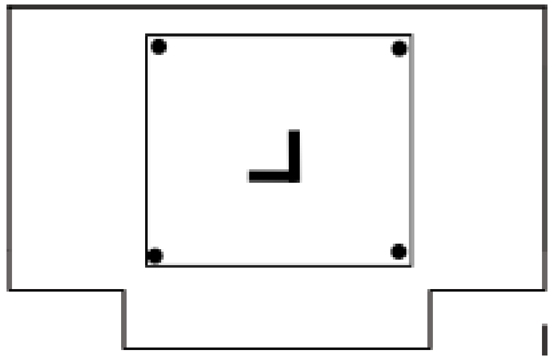
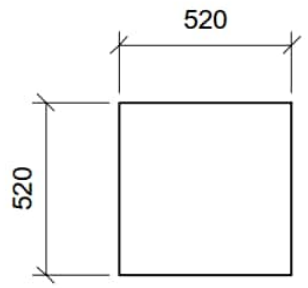
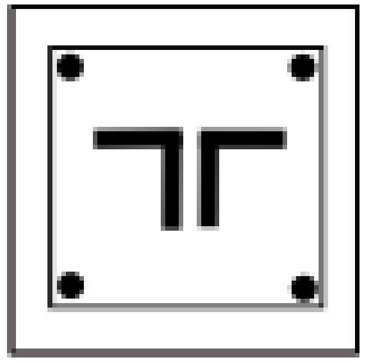
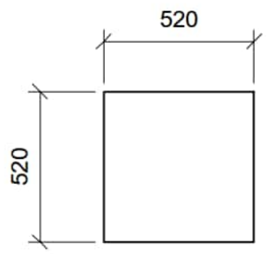
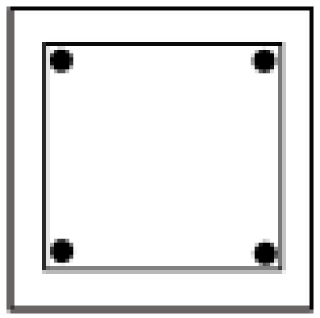
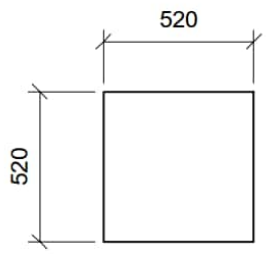
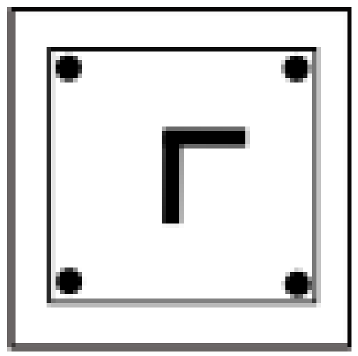
Sections Summary

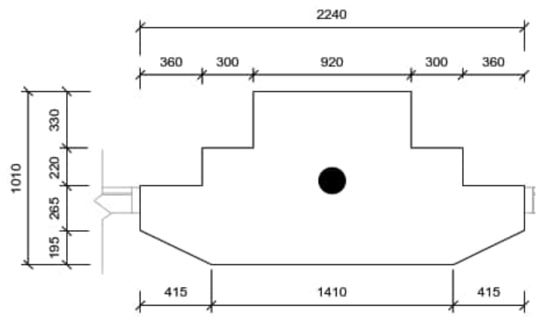
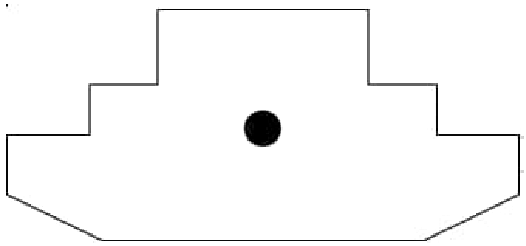
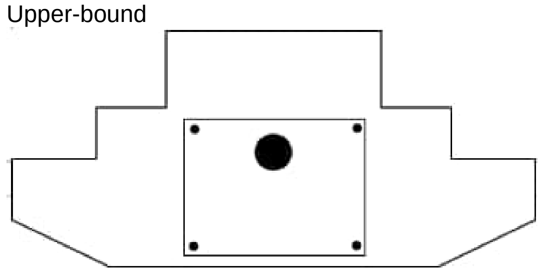
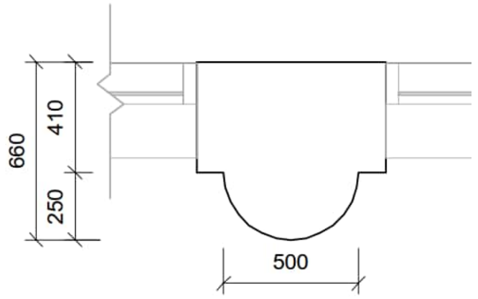
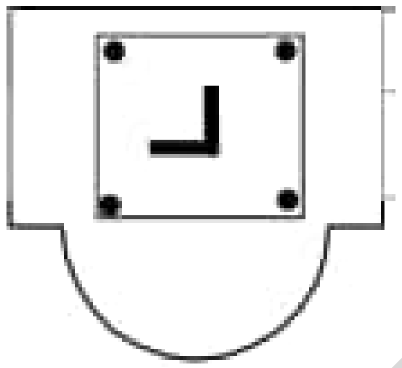
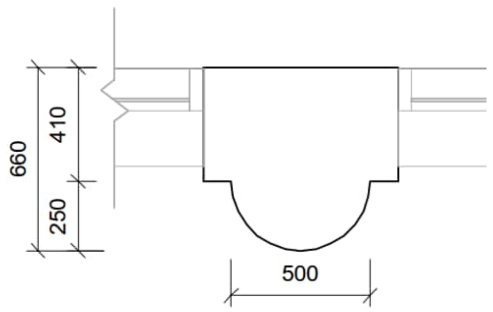
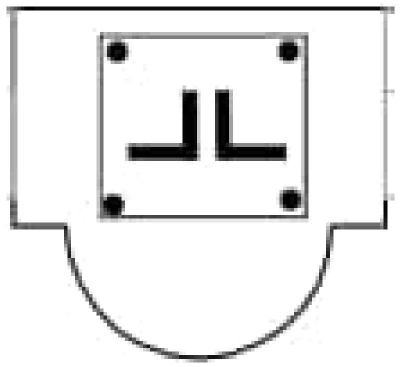
Section type	Section diagram	Section modelled	Material properties	Section properties	Stiffness modifiers	Section capacities estimate				
						Bending, M_n (at axial load N)			Shear, V_n	Axial, N_n
Notes	Section diagram shows the actual size of the element as measured on site. In cases where direct measurements could not be undertaken a section has been inferred from indirect measurements, historical drawings and photos.	Section modelled shows the reinforcement of the element used in the analysis software. A lower and upper bound reinforcement layout has been considered if appropriate.	Refer to DFR material section for detailed material properties. C25 = 25MPa R296C = 296MPa S230 = 230MPa S250 = 250MPa Reinforcement layout based on intrusive investigation. Cover is 100mm U.N.O	I_{gross} is the moment of inertia of the gross (uncracked) section. I_{mod} is the moment of inertia of the modelled section. x-x is the moment about the horizontal axis and y-y is the moment about the vertical axis.	The stiffness modifier is the quotient of I_{sec} and I_{mod}. (I_{sec} / I_{mod}) For concrete this is multiplied by 0.5 to account for the cracked stiffness of the section. This modifier is used in the model to account for the actual area of the section used in the analysis.	The moment in kNm for columns is shown for two axial forces 0 kN and 1300 kN and in four directions. 			For reinforced concrete sections with no encased structural steel. Two values of shear strength calculated in accordance with NZS3101 and NZSEE Guidance are shown. For sections with encased structural steel the shear is developed from the steel section.	Maximum axial compression force for columns with no moment applied.
Type A column		Upper-bound  Inferred from site observations	Concrete C25 Rebar R296C 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	Cover = 100mm $I_{gross\ x\text{-}x}$ =1.62 x10^{10} mm^4 $I_{gross\ y\text{-}y}$ =6.7 x10^{10} mm^4	x-x =0.5 y-y =0.5		0 kN	1300 kN	158 kN	9776 kN
						0°	88	386		
						90°	193	831		
						180°	88	386		
						270	193	831		

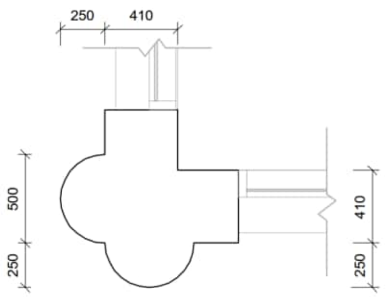
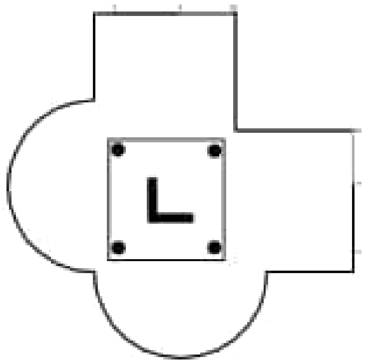
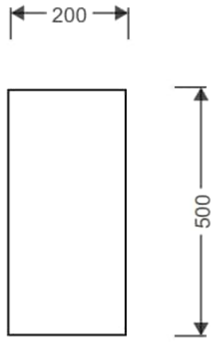
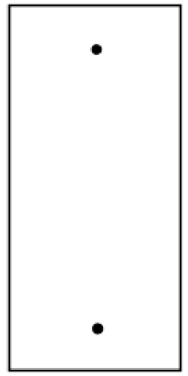
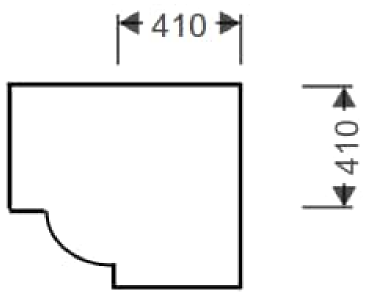
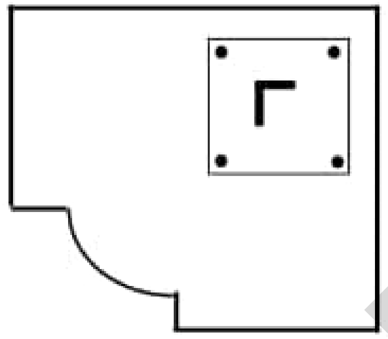
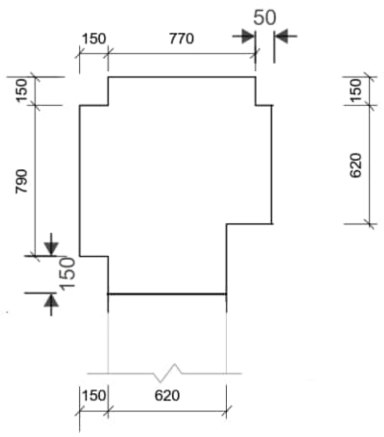
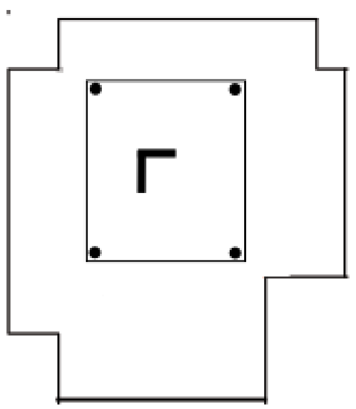
Type B Column		Lower-bound  Inferred from site observations	Concrete C25 Steel S230 ϕ235mm steel column in the centre	$I_{gross\ x\text{-}x} = 2.16 \times 10^{11}\ mm^4$ $I_{gross} = 2.16 \times 10^{11}\ mm^4$	x-x =0.1 y-y = 0.1		0 kN	1300 kN	4920 kN (steel section only)	35724 kN
						0°	3299	3586		
						90°	3299	3586		
						180°	5132	5184		
						270°	5132	5184		
		Upper-bound  Inferred from site observations	Concrete C25 Steel S230 Rebar R296C ϕ235mm steel column in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover = 100mm $I_{gross\ x\text{-}x} = 2.16 \times 10^{11}\ mm^4$ $I_{gross\ y\text{-}y} = 2.16 \times 10^{11}\ mm^4$	x-x =0.5 y-y =0.5		0 kN	1300 kN	4920 kN (steel section only)	35987 kN
						0°	3417	3707		
						90°	3417	3707		
						180°	5372	5591		
						270°	5372	5591		
Type C coluimn		 Inferred from site observations	Concrete C25 Rebar R296C 2 no. ϕ19mm bars in the centre	cover = 100mm $I_{gross\ x\text{-}x} = 6.05 \times 10^{10}\ mm^4$ $I_{gross\ y\text{-}y} = 7.7 \times 10^{9}\ mm^4$	x-x =0.1 y-y =0.1		0 kN	1300 kN	213 kN (NZS3101) 220 kN (NZSEE)	9330 kN
						0°	97	741		
						90°	64	305		
						180°	97	741		
						270	28	258		

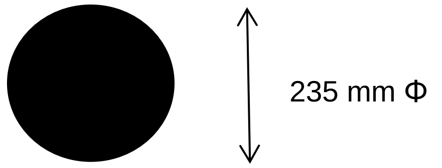
Type D Column		Lower-bound  Inferred from site observations	Concrete C25 Steel S230 2x BSEA14. 150mm leg length located in the centre	$I_{gross\ x\text{-}x} = 8.55\text{x}10^{10}\ mm^4$ $I_{gross\ y\text{-}y} = 7.05\text{x}10^{10}\ mm^4$	x-x = 0.1 y-y = 0.1		0 kN	1300 kN	800 kN (steel section only)	16529 kN
						0°	1128	1683		
						90°	961	1485		
						180°	540	1020		
						270°	545	968		
		Upper-bound  Inferred from site observations	Concrete C25 Rebar R296C Steel S230 2x BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover = 100mm $I_{gross\ x\text{-}x} = 8.55\text{x}10^{10}\ mm^4$ $I_{gross\ y\text{-}y} = 7.05\text{x}10^{10}\ mm^4$	x-x =0.5 y-y =0.5		0 kN	1300 kN	800 kN (steel section only)	16794 kN
						0°	1333	1836		
						90°	1129	1556		
						180°	634	1108		
						270°	653	1084		
Type E Column		Lower-bound  Inferred from site observations	Concrete C25 Steel S230 BSEA14. 150mm leg length located in the centre	$I_{gross\ x\text{-}x} = 1.22\text{x}10^{11}\ mm^4$ $I_{gross\ y\text{-}y} = 1.2\text{x}10^{11}\ mm^4$	x-x = 0.1 y-y = 0.1		0 kN	1300 kN	400 kN (steel section only)	21936 kN
						0°	328	943		
						90°	375	997		
						180°	588	1334		
						270°	543	1286		
		Upper-bound  Inferred from site observations	Concrete C25 Rebar R296C Steel S230 BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover = 100mm $I_{gross\ x\text{-}x} = 1.22\text{x}10^{11}\ mm^4$ $I_{gross\ y\text{-}y} = 1.2\text{x}10^{11}\ mm^4$	x-x =0.5 y-y =0.5		0 kN	1300 kN	400 kN (steel section only)	22201 kN
						0°	466	1060		
						90°	508	1101		
						180°	806	1507		
						270°	754	1446		

Type F Column		Lower-bound  Inferred from site observations	Concrete C25 Steel S230 2x BSEA7 located in the centre	$I_{gross\ x\text{-}x} = 1.39\text{x}10^{11}\ mm^4$ $I_{gross\ y\text{-}y} = 4.29\text{x}10^{10}\ mm^4$	x-x = 0.1 y-y = 0.1		0 kN	1300 kN	800 kN (steel section only)	17721 kN
						0°	269	1025		
						90°	175	613		
						180°	269	1025		
						270°	103	524		
		Upper-bound  Inferred from site observations	Concrete C25 Rebar R296C Steel S230 2x BSEA7 located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 1.39\text{x}10^{11}\ mm^4$ $I_{gross\ y\text{-}y} = 4.29\text{x}10^{10}\ mm^4$	x-x =0.5 y-y =0.5		0 kN	1300 kN	800 kN (steel section only)	17986 kN
						0°	423	1157		
						90°	285	680		
						180°	423	1157		
						270°	183	567		
Type G1 Column			Concrete C25 Rebar R296C BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 6.21\text{x}10^{10}\ mm^4$ $I_{gross\ y\text{-}y} = 1.57\text{x}10^{11}\ mm^4$	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	400 kN (steel section only)	20640 kN
						0°	358	769		
						90°	637	1281		
						180°	486	934		
						270	716	1391		

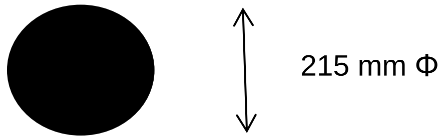
Type G2 Column		 Inferred from site observations	Concrete C25 Rebar R296C Steel S230 BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 6.21\text{x}10^{10}\ mm^4$ $I_{gross\ y\text{-}y} = 1.57\text{x}10^{11}\ mm^4$	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	400 kN (steel section only)	20640 kN
						0°	358	769		
						90°	663	1379		
						180°	486	934		
						270°	729	1448		
Type H1 Column		 Measured on site	Concrete C25 Rebar R296C Steel S230 2x BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 300mm crs.	520x520 mm square cover 50mm $I_{gross\ x\text{-}x} = 6.32\text{x}10^9\ mm^4$ $I_{gross\ y\text{-}y} = 6.49\text{x}10^9\ mm^4$ (Difference is because ETABS treats steel angle as equivalent concrete section)	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	800 kN (steel section only)	6300 kN
						0°	353	450		
						90°	377	500		
						180°	392	497		
						270°	377	500		
Type H2 Column		 Inferred from site observations	Concrete C25 Rebar R296C 4 no. ϕ19mm bars in the corners ϕ12mm links at 300mm crs.	520x520 mm square cover 50mm $I_{gross\ x\text{-}x} = 6.09\text{x}10^9\ mm^4$ $I_{gross\ y\text{-}y} = 6.09\text{x}10^9\ mm^4$	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	214 kN (NZS3101) 236 kN (NZSEE)	5149 kN
						0°	74	271		
						90°	74	271		
						180°	74	271		
						270°	74	271		
Type H3 Column		 Inferred from site observations	Concrete C25 Rebar R296C Steel S230 BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 300mm crs.	520x520 mm square cover 50mm $I_{gross\ x\text{-}x} = 6.21\text{x}10^9\ mm^4$ $I_{gross\ y\text{-}y} = 6.24\text{x}10^9\ mm^4$ (Difference is because ETABS treats steel angle as equivalent concrete section)	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	400 kN (steel section only)	5724 kN
						0°	238	411		
						90°	280	473		
						180°	256	442		
						270°	208	402		

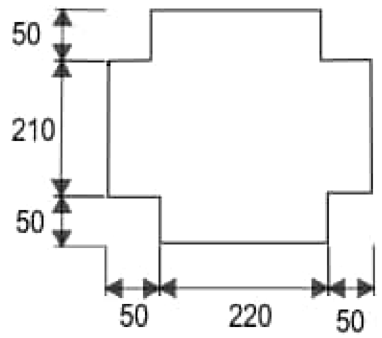
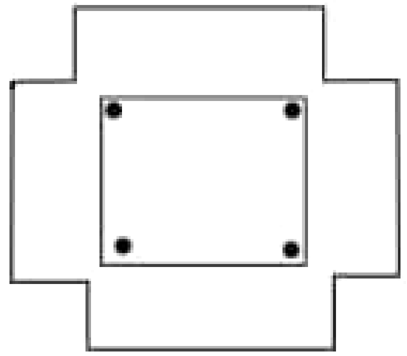
Type I column		Lower-bound  Inferred from site observations	Concrete C25 Steel S230 ϕ235mm steel column in the centre	$I_{gross\ x\text{-}x} = 1.17\text{x}10^{11}\ mm^4$ $I_{gross\ y\text{-}y} = 5.12\text{x}10^{11}\ mm^4$	x-x = 0.1 y-y = 0.1		0 kN	1300 kN	4920 kN (steel section only)	36802 kN
						0°	4175	4366		
						90°	2114	2012		
						180°	1855	2099		
						270°	2114	2012		
		Upper-bound  Inferred from site observations	Concrete C25 Steel S230 Rebar R296C ϕ235mm steel column in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover = 100mm $I_{gross\ x\text{-}x} = 1.17\text{x}10^{11}\ mm^4$ $I_{gross\ y\text{-}y} = 5.12\text{x}10^{11}\ mm^4$	x-x =0.5 y-y =0.5		0 kN	1300 kN	4920 kN (steel section only)	37067 kN
						0°	4016	4180		
						90°	6769	7163		
						180°	2117	2363		
						270°	6769	7163		
Type J1 Column	 Note: Obtained from sources other than site measurement	 Inferred from site observations	Concrete C25 Rebar R296C Steel S230 BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 1.08\text{x}10^{10}\ mm^4$ $I_{gross\ y\text{-}y} = 9.05\text{x}10^{9}\ mm^4$	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	400 kN (steel section only)	7045 kN
						0°	193	420		
						90°	246	456		
						180°	386	587		
						270°	294	478		
Type J2 Column	 Note: Obtained from sources other than site measurement	 Inferred from site observations	Concrete C25 Rebar R296C Steel S230 2x BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 1.08\text{x}10^{10}\ mm^4$ $I_{gross\ y\text{-}y} = 9.05\text{x}10^{9}\ mm^4$	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	800 kN (steel section only)	7621 kN
						0°	280	459		
						90°	417	584		
						180°	581	696		
						270°	417	584		

Type K Column	 Note: Obtained from sources other than site measurement	 Inferred from site observations	Concrete C25 Rebar R296C Steel S230 BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 3.18x10^{10}$ mm^4 $I_{gross\ y\text{-}y} = 3.18x10^{10}$ mm^4	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	200 kN (steel section only)	11844 kN
						0°	516	896		
						90°	384	737		
						180°	402	755		
						270°	526	913		
Type L Column	 Note: Obtained from sources other than site measurement	 Inferred from site observations	Concrete C25 Rebar R296C 2 no. ϕ19mm	500x200 mm rectangle cover 100mm $I_{gross\ x\text{-}x} = 2.08x10^{9}$ mm^4 $I_{gross\ y\text{-}y} = 3.33\ x10^{8} mm^4$	x-x = 0.1 y-y = 0.1		0 kN	1300 kN	54 kN (NZS3101) 52 kN (NZSEE)	1939 kN
						0°	32	151		
						90°	16	60		
						180°	32	151		
						270°	16	60		
Type M Column	 Note: Obtained from sources other than site measurement	 Inferred from site observations	Concrete C25 Rebar R296C Steel S230 BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 1.89x10^{10}$ mm^4 $I_{gross\ y\text{-}y} = 1.89x10^{10}$ mm^4	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	400 kN (steel section only)	9201 kN
						0°	166	444		
						90°	480	771		
						180°	516	815		
						270°	191	486		
Type N Column		 Inferred from site observations	Concrete C25 Rebar R296C Steel S230 BSEA14. 150mm leg length located in the centre 4 no. ϕ19mm bars in the corners ϕ12mm links at 350mm crs.	cover 100mm $I_{gross\ x\text{-}x} = 8.29x10^{10}$ mm^4 $I_{gross\ y\text{-}y} = 6.4x10^{10}$ mm^4	x-x = 0.5 y-y = 0.5		0 kN	1300 kN	400 kN (steel section only)	17833 kN
						0°	467	1017		
						90°	485	1012		
						180°	651	1209		
						270°	471	956		

<table>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Type O Column</td><td>Stand columns lower</td><td>
Measured on site</td><td>Steel S230

Solid circular member</td><td>235mm ϕ circular column

$I_{gross} = 1.5 \times 10^{8}\ mm^{4}$</td><td>-</td><td>My=293 kNm</td><td>4920 kN</td><td>-</td></tr>
<tr><td>Type P Column</td><td>Stand Columns upper</td><td>
Measured on site</td><td>Steel S230
Solid circular member</td><td>215mm ϕ circular column

$I_{gross} = 1.05 \times 10^{8}\ mm^{4}$</td><td>-</td><td>My=224 kNm</td><td>4117 kN</td><td>-</td></tr>
<tr><td>Type Q Column</td><td></td><td>
Inferred from site observations</td><td>Concrete C25
Rebar R296C

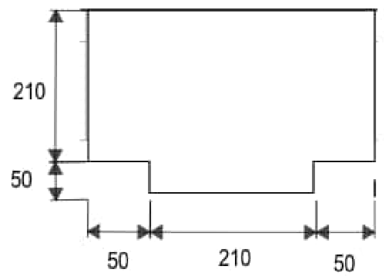
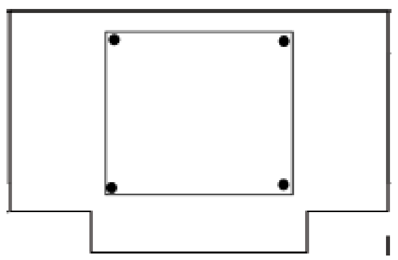
4 no. ϕ19mm
bars in the corners

ϕ12mm links
at 350mm crs.</td><td>cover 50mm

$I_{gross\ x\text{-}x} = 6.23 \times 10^{8}\ mm^{4}$

$I_{gross\ y\text{-}y} = 6.62 \times 10^{8}\ mm^{4}$</td><td>x-x = 0.5

y-y = 0.5</td><td><table><tr><td></td><td>0 kN</td><td>1300 kN</td></tr><tr><td>0°</td><td>40</td><td>84</td></tr><tr><td>90°</td><td>40</td><td>87</td></tr><tr><td>180°</td><td>40</td><td>84</td></tr><tr><td>270°</td><td>40</td><td>87</td></tr></table></td><td>138 kN
(NZS3101)

69 kN
(NZSEE)</td><td>1876 kN</td></tr>
<tr><td>Type R Column</td><td></td><td>
Inferred from site observations</td><td>Concrete C25
Rebar R296C

4 no. ϕ19mm
bars in the corners

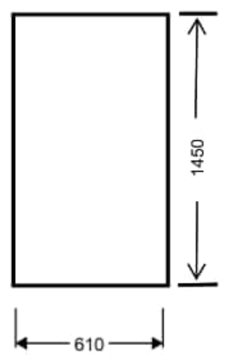
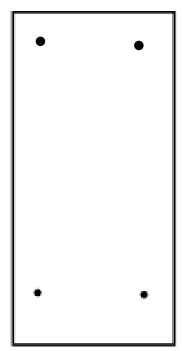
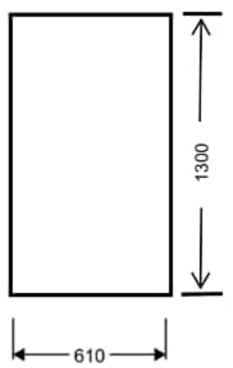
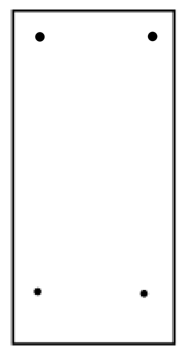
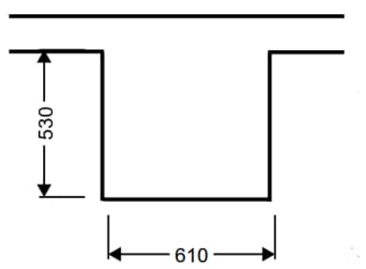
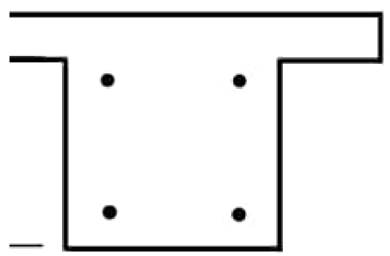
ϕ12mm links
at 350mm crs.</td><td>cover 50mm

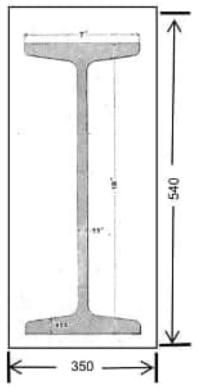
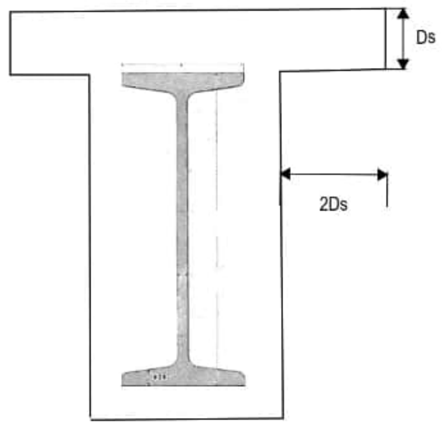
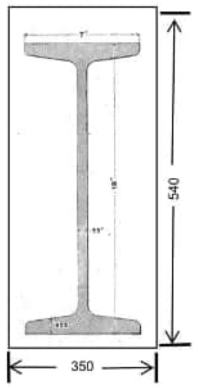
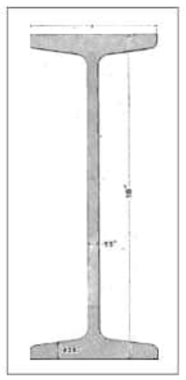
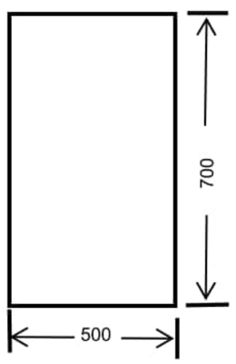
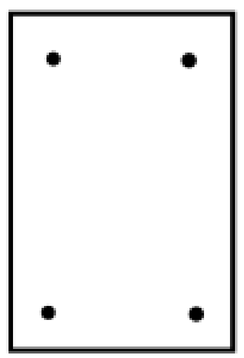
$I_{gross\ x\text{-}x} = 3.94 \times 10^{8} mm^{4}$

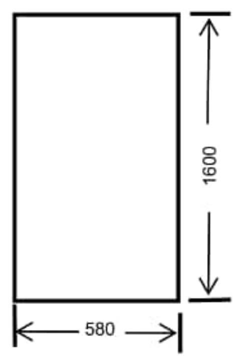
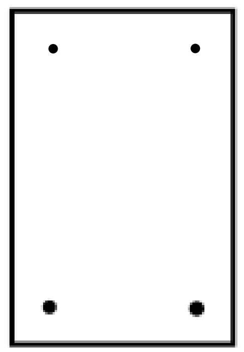
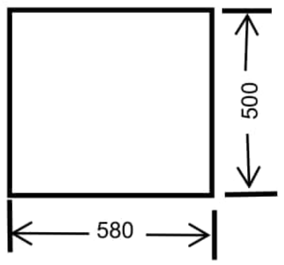
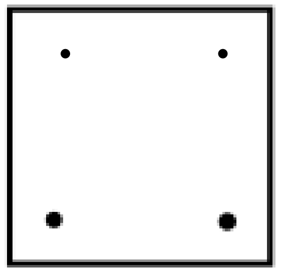
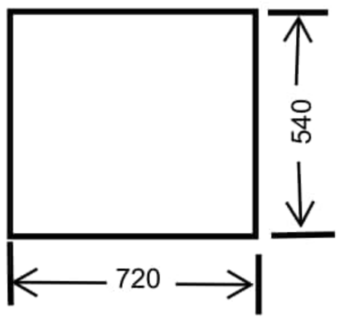
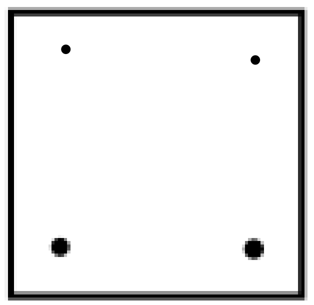
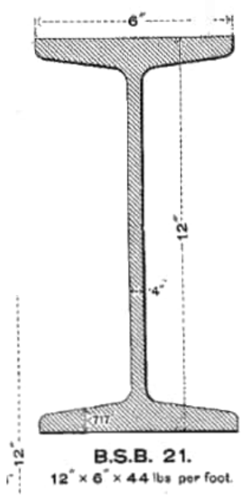
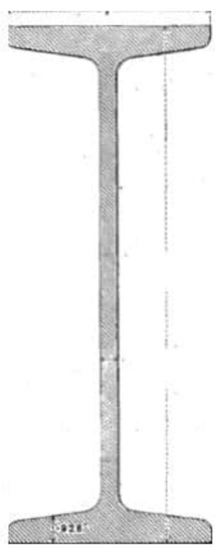
$I_{gross\ y\text{-}y} = 5.6 \times 10^{8} mm^{4}$</td><td>x-x = 0.5

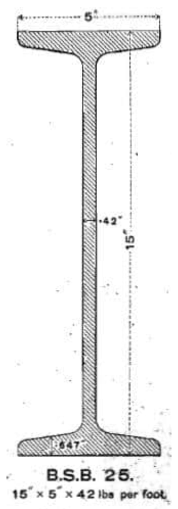
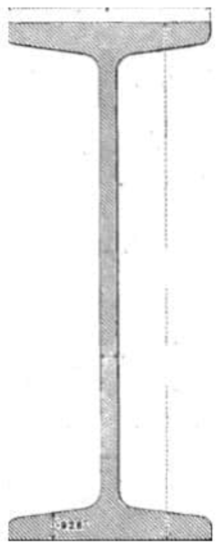
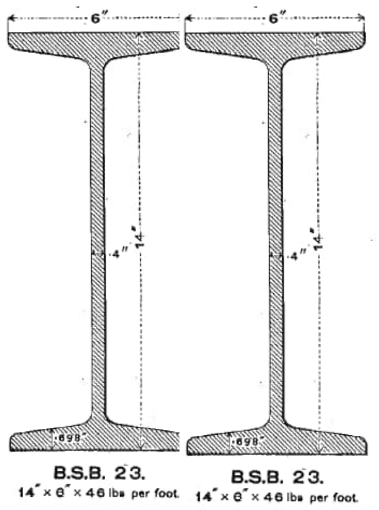
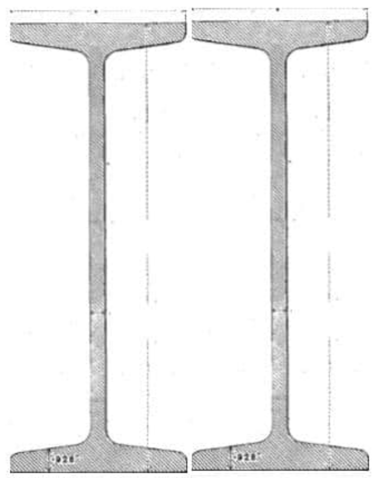
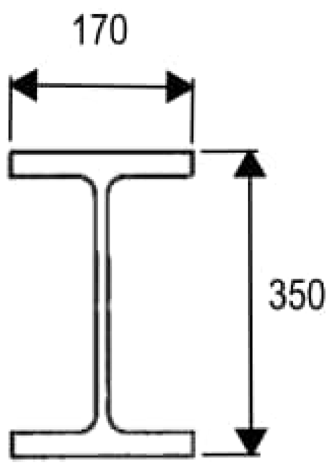
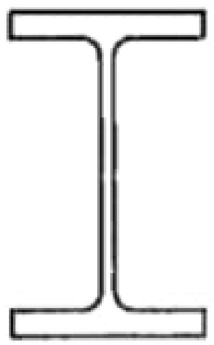
y-y = 0.5</td><td><table><tr><td></td><td>0 kN</td><td>1300 kN</td></tr><tr><td>0°</td><td>26</td><td>60</td></tr><tr><td>90°</td><td>38</td><td>70</td></tr><tr><td>180°</td><td>41</td><td>54</td></tr><tr><td>270°</td><td>38</td><td>70</td></tr></table></td><td>141 kN
(NZS3101)

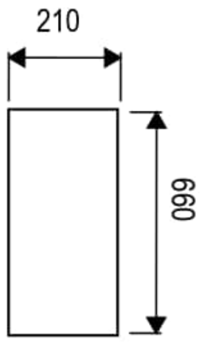
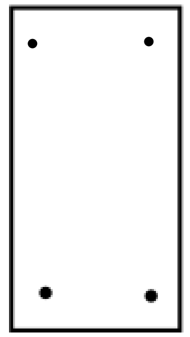
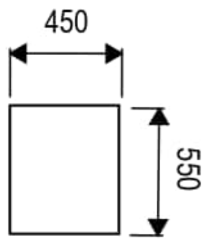
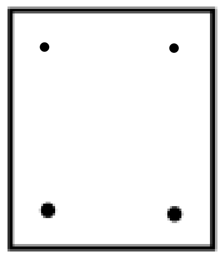
70 kN
(NZSEE)</td><td>16630 kN</td></tr>
</table>

Section type	Section diagram	Section modelled	Material properties	Section properties	Stiffness modifiers	Section capacities estimate	
						Bending, M_n	Shear, V_n
Notes	Section diagram shows the actual size of the element as measured on site. In cases where direct measurements could not be undertaken a section has been inferred from indirect measurements, historical drawings and photos.	Section modelled shows the reinforcement of the element used in the analysis software. A lower and upper bound reinforcement layout has been considered where reinforcement is unknown.	Refer to DFR material section for detailed material properties. C25 = 25.5MPa R296C = 296MPa S233 = 233MPa S250 = 250MPa	I_{gross} is the moment of inertia of the gross section. I_{mod} is the moment of inertia of the modelled section. x-x is the moment about the horizontal axis and y-y is the moment about the vertical axis.	The stiffness modifier is the quotient of I_{gross} and I_{mod} (I_{gross} / I_{mod}) For concrete this is multiplied by 0.5 to account for the cracked stiffness of the section.	The bending moment capacity for beams.	For concrete beams two values of shear strength calculated in accordance with NZS3101 and NZSEE Guidance are shown. For steel beams one value of shear strength calculated in accordance with NZS3404 is shown.
Type 1A Beam		 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars bottom Nominal 2 no. ϕ6mm bars top	1450x610 mm rectangle cover 100mm $I_{gross} = 1.55 \times 10^{11}$ mm^4	0.5	162KNm (NZS3101)	926kN (NZSEE) 512kN (NZS3101)
Type 1B Beam	 Note: Obtained from sources other than site measurement	 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars bottom Nominal 2 no. ϕ6mm bars top	1300x610 mm rectangle cover 100mm $I_{gross} = 1.12 \times 10^{11}$ mm^4	0.5	144KNm (NZS3101)	824kN (NZSEE) 460kN (NZS3101)
Type 2 Beam		 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars top 2 no. ϕ19mm bars bottom	530x610 mm rectangle cover 100mm $I_{gross} = 7.57 \times 10^{9}$ mm^4	0.5	103KNm (NZS3101)	295kN (NZSEE) 188kN (NZS3101)

Type 3A Beam			Concrete C25 Steel S230 BSB28 Contribution from the slab Type 3b is within 0.25L of end of beam (where L is length)	Steel section BSB28 encased in 540x350 mm rectangle $I_{gross} = 1.99 \times 10^{10}$ mm^4 (150mm slab) $I_{gross} = 2.66 \times 10^{10}$ mm^4 (200mm slab) 1920's structural steel beam BSB28 as inferred from intrusive investigation. Beam modelled as steel beam in ETABS 2015 with stiffness modifier applied to account for concrete surround.	0.5 (200mm slab) 0.5 (150mm slab)	424kNm	893kN (NZS3404) Steel beam only
Type 3B Beam			Concrete C25 Steel S230 BSB28 Type 3a is the middle 0.5L of beam (where L is length)	Steel section BSB28 encased in 540x350 mm rectangle $I_{gross} = 8.2 \times 10^{9}$ mm^4 1920's structural steel beam BSB28 as inferred from intrusive investigation. Beam modelled as steel beam in ETABS 2015 with stiffness modifier applied to account for concrete surround.	0.5	424kNm	893kN (NZS3404) Steel beam only
Type 4 Beam		 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars top 2 no. ϕ19mm bars bottom	700x500 mm rectangle cover 100mm $I_{gross} = 1.43 \times 10^{10}$ mm^4	0.5	144KNm (NZS3101)	355kN (NZSEE) 229kN (NZS3101)

Type 5 Beam	 Note: Obtained from sources other than site measurement	 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars bottom Nominal 2 no. ϕ6mm bars top	1600x580 mm rectangle cover 100mm $I_{gross} = 1.98 \times 10^{11}$ mm^4	0.5	180KNm (NZS3101)	991kN (NZSEE) 551kN (NZS3101)
Type 6 Beam	 Note: Obtained from sources other than site measurement	 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars bottom Nominal 2 no. ϕ6mm bars top	500x580 mm rectangle cover 100mm $I_{gross} = 6.04 \times 10^{9}$ mm^4	0.5	48.1KNm (NZS3101)	264kN (NZSEE) 174kN (NZS3101)
Type 7 Beam	 Note: Obtained from sources other than site measurement	 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars bottom Nominal 2 no. ϕ6mm bars top	540x720 mm rectangle cover 100mm $I_{gross} = 9.45 \times 10^{9}$ mm^4	0.5	52.9KNm (NZS3101)	344kN (NZSEE) 206kN (NZS3101)
Type 8 Beam			Steel S230 BSB21	Steel section BSB21 $I_{gross} = 1.31 \times 10^{8}$ mm^4 1920's structural steel beam BSB21 as inferred from intrusive investigation. Beam modelled as steel beam in ETABS 2015.	-	175 kNm	430kN (NZS3404)

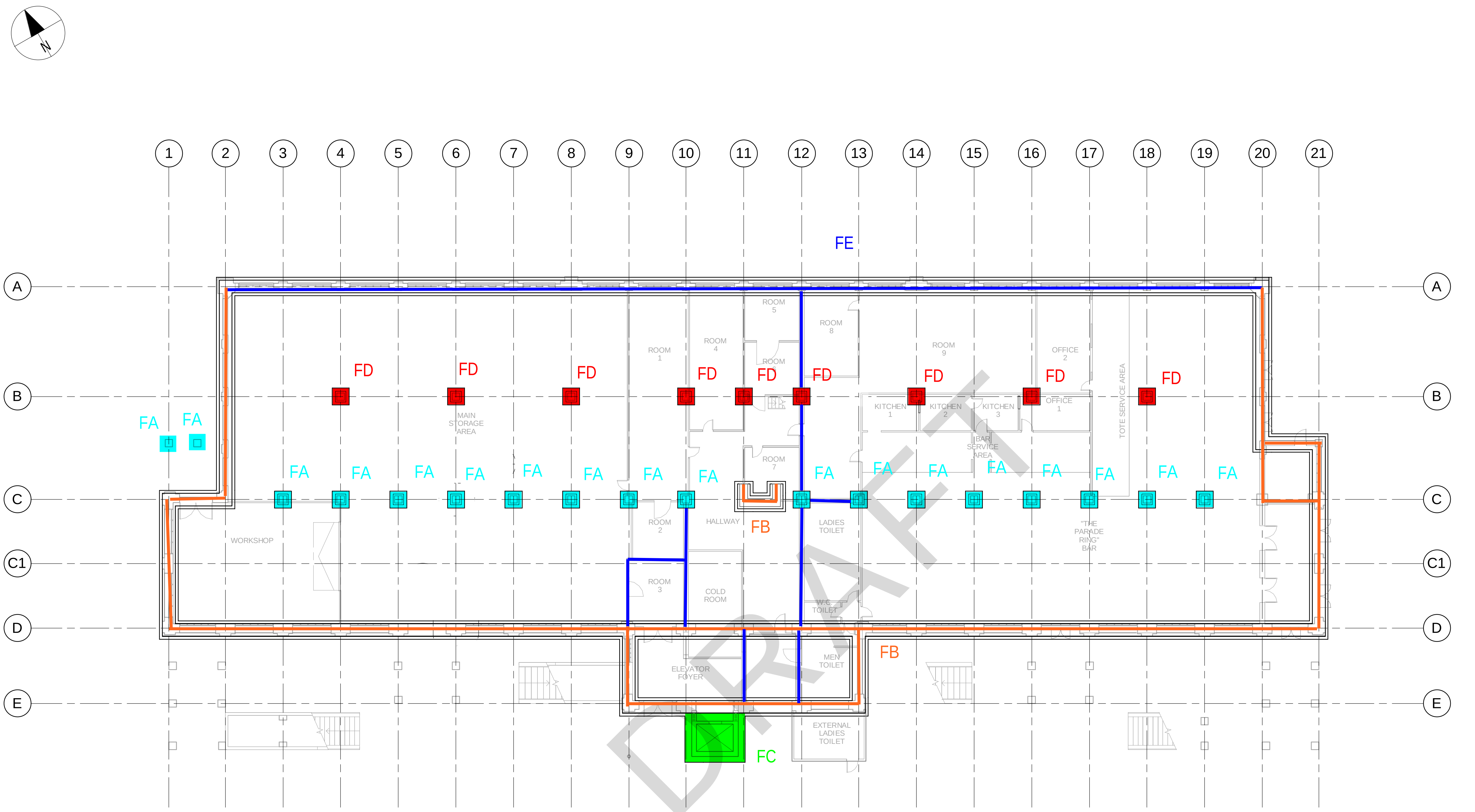
Type 9 Beam			Steel S230 BSB 25	Steel section BSB25 $I_{gross} = 1.78 \times 10^8$ mm^4 1920's structural steel beam BSB25 as inferred from intrusive investigation. Beam modelled as steel beam in ETABS 2015.	-	190 kNm	570kN (NZS3404)
Type 10 Beam			Steel S230 2x BSB 23	Steel section 2x BSB23 $I_{gross} = 3.66 \times 10^8$ mm^4 1920's structural steel beam 2x BSB23 as inferred from intrusive investigation. Beam modelled as steel beam in ETABS 2015.	-	418 kNm	1004kN (NZS3404) 2x shear of one steel beam
Type 11 Beam			Steel S250 360UB44.7	Steel section 360UB44.7 $I_{gross} = 1.21 \times 10^8 mm^4$ Structural steel beam 360UB44.7 as inferred from intrusive investigation. Beam modelled as steel beam in ETABS 2015.	-	172kNm	364kN (NZS3404)

Type 12 Beam		 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars bottom Nominal 2 no. ϕ6mm bars top	660x210 mm rectangle cover 100mm I_{gross} =3.78 x10^9 mm^4	0.5	67.4KNm (NZS3101)	192kN (NZSEE) 168kN (NZS3101)
Type 13 Beam		 Assumed reinforcement	Concrete C25 Rebar R296C 2 no. ϕ19mm bars bottom Nominal 2 no. ϕ6mm bars top	550x450 mm rectangle cover 100mm I_{gross} =6.24 x10^9 mm^4	0.5	54.1KNm (NZS3101)	247kN (NZSEE) 174kN (NZS3101)

Appendix B

Lateral Analysis Model Geometry

ISO A1 594mm x 841mm



FOUNDATION PLAN

AECOM

PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

CONSULTANT

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Christchurch, NZ 8024
+64 3 966 6000 tel +64 3 966 6001 fax
www.aecom.com

Key

REFER TO DFR SECTION 3 FOR DETAILS

PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

60439900

SHEET TITLE

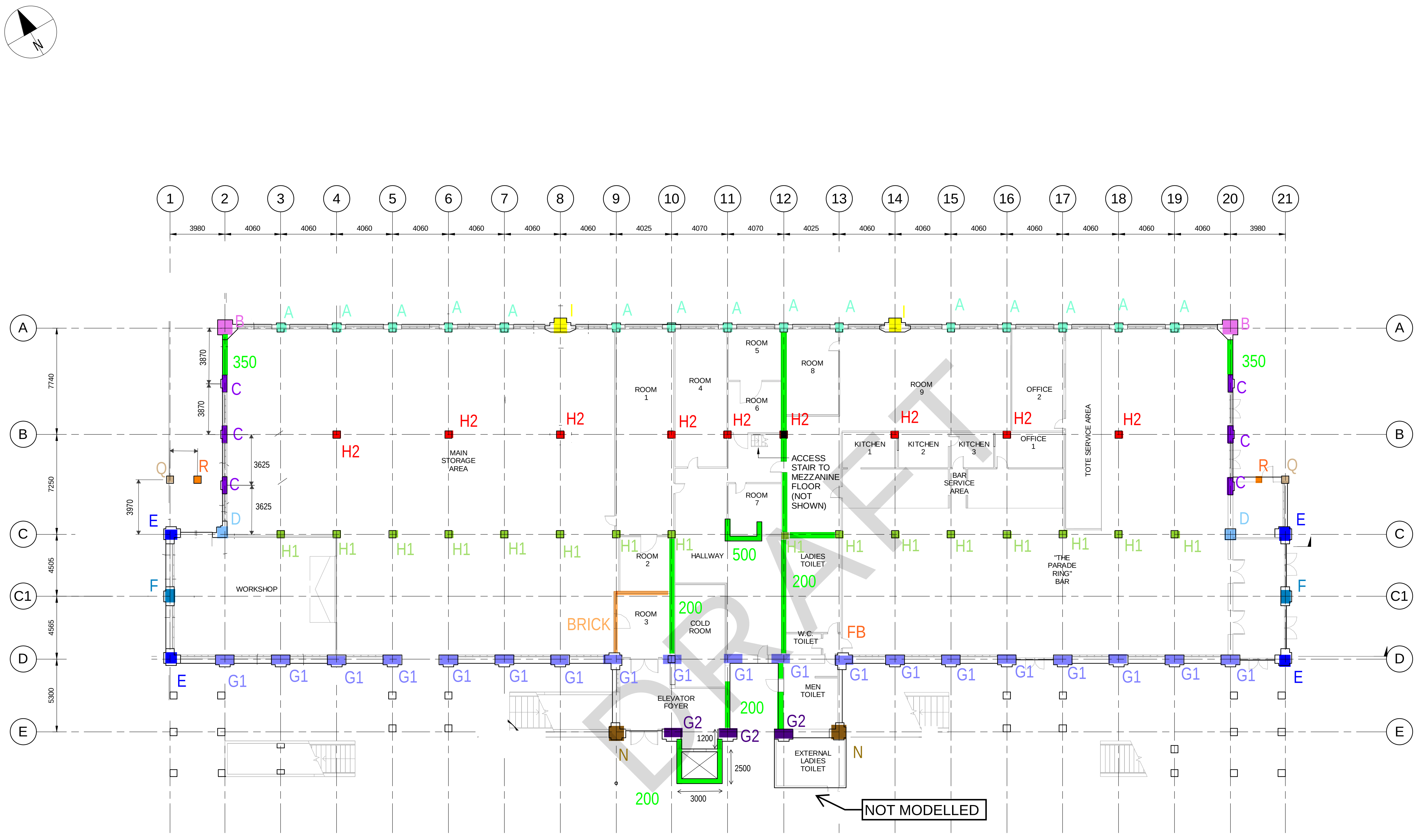
INFERRED FOUNDATION PLAN AND SECTIONS

SHEET NUMBER

B.1

Last Saved: 3/07/2015 4:40:55 p.m.
Filename: C:\Users\PettiC\Desktop\60332326_MOD_ST_2014_PettiC(Recovery).rvt

ISO A1 594mm x 841mm



GROUND FLOOR COLUMNS

AECOM

PROJECT
STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT
CANTERBURY JOCKEY CLUB

Prepared for:

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AECOM New Zealand Limited
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www.aecom.com

PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

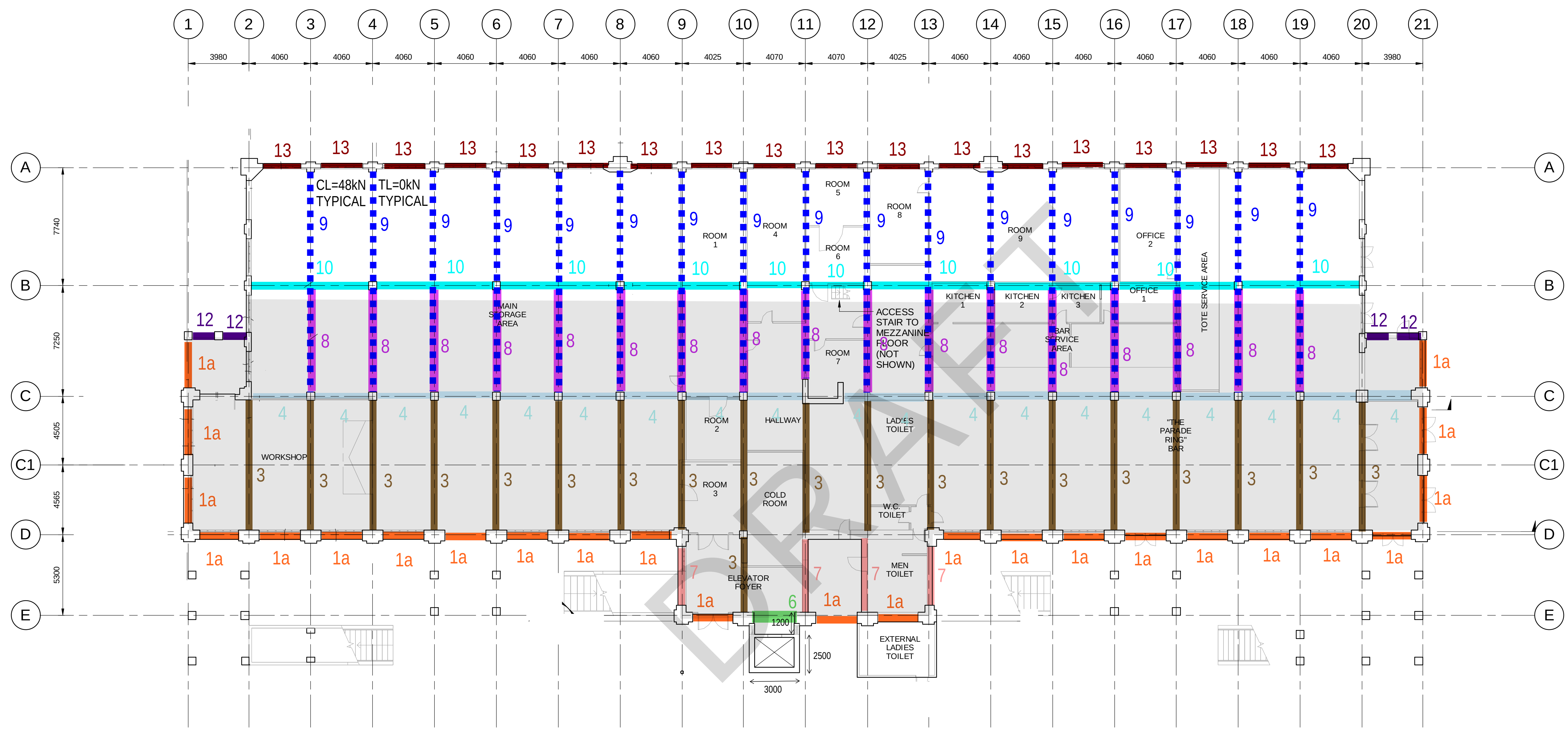
KEY PLAN

PROJECT NUMBER
60439900

SHEET TITLE
ANALYTICALMODEL

SHEET NUMBER
B.2

Last Saved: 8/06/2015 4:47:27 p.m.
Filename: C:\Users\PettiC\Desktop\60332326_MOD_ST_2014_PettiC(Recovery).rvt



BEAMS SUPPORTING LEVEL 1 FLOOR

HORIZONTAL
INCLINED
EXTENT OF SLAB

CL = Compression Limit
TL = Tension Limit

AECOM

PROJECT
STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT
CANTERBURY JOCKEY CLUB

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PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER
60439900

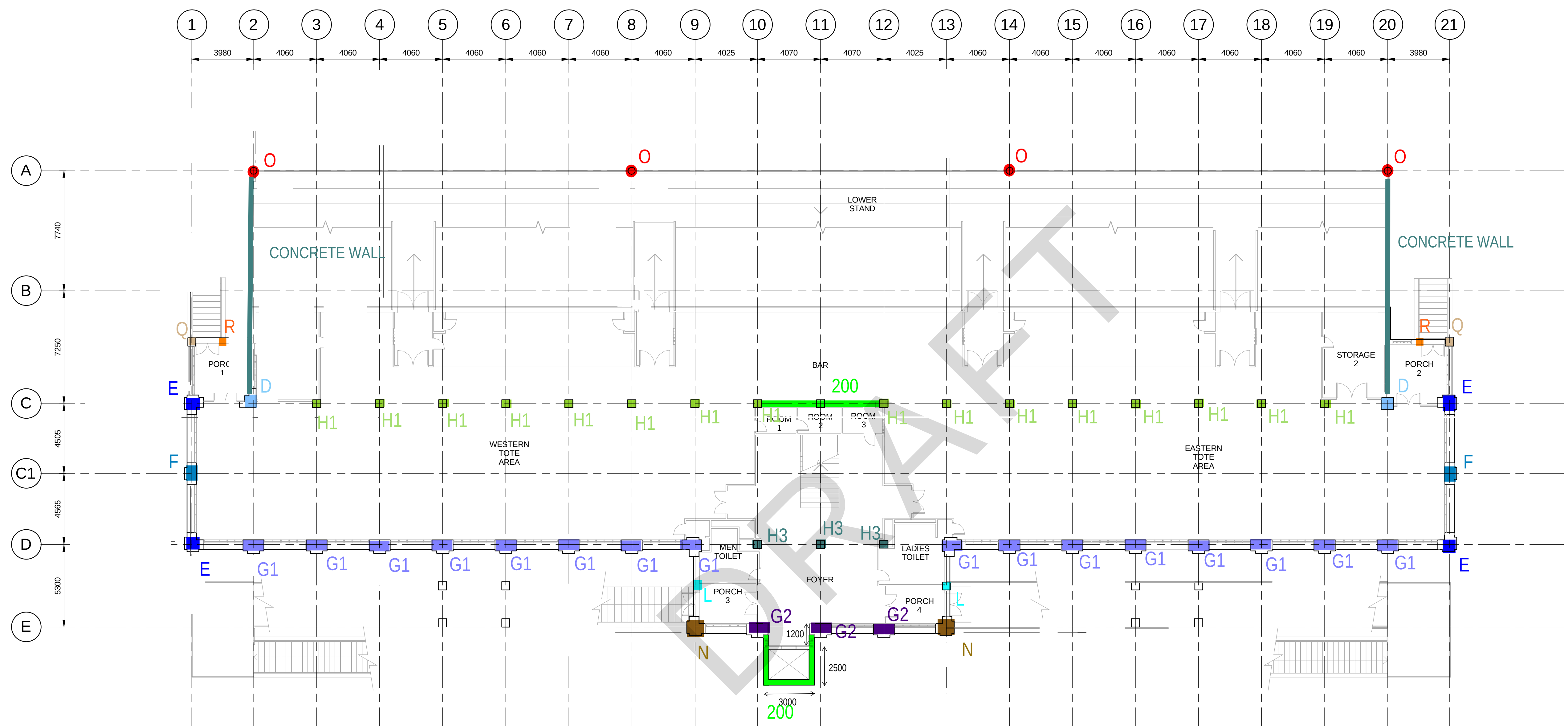
SHEET TITLE
ANALYTICALMODEL

SHEET NUMBER
B.3

ISO A1 594mm x 841mm

Last Saved: 8/06/2015 4:47:27 p.m.
Filename: C:\Users\PettiC\Desktop\60332226_MOD_ST_2014_PettiC(Recovery).rvt

ISO A1 594mm x 841mm



LEVEL 1 COLUMNS

AECOM

PROJECT

STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

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NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

60439900

SHEET TITLE

ANALYTICALMODEL

SHEET NUMBER

B.4

Last Saved: 8/06/2015 4:47:29 p.m.
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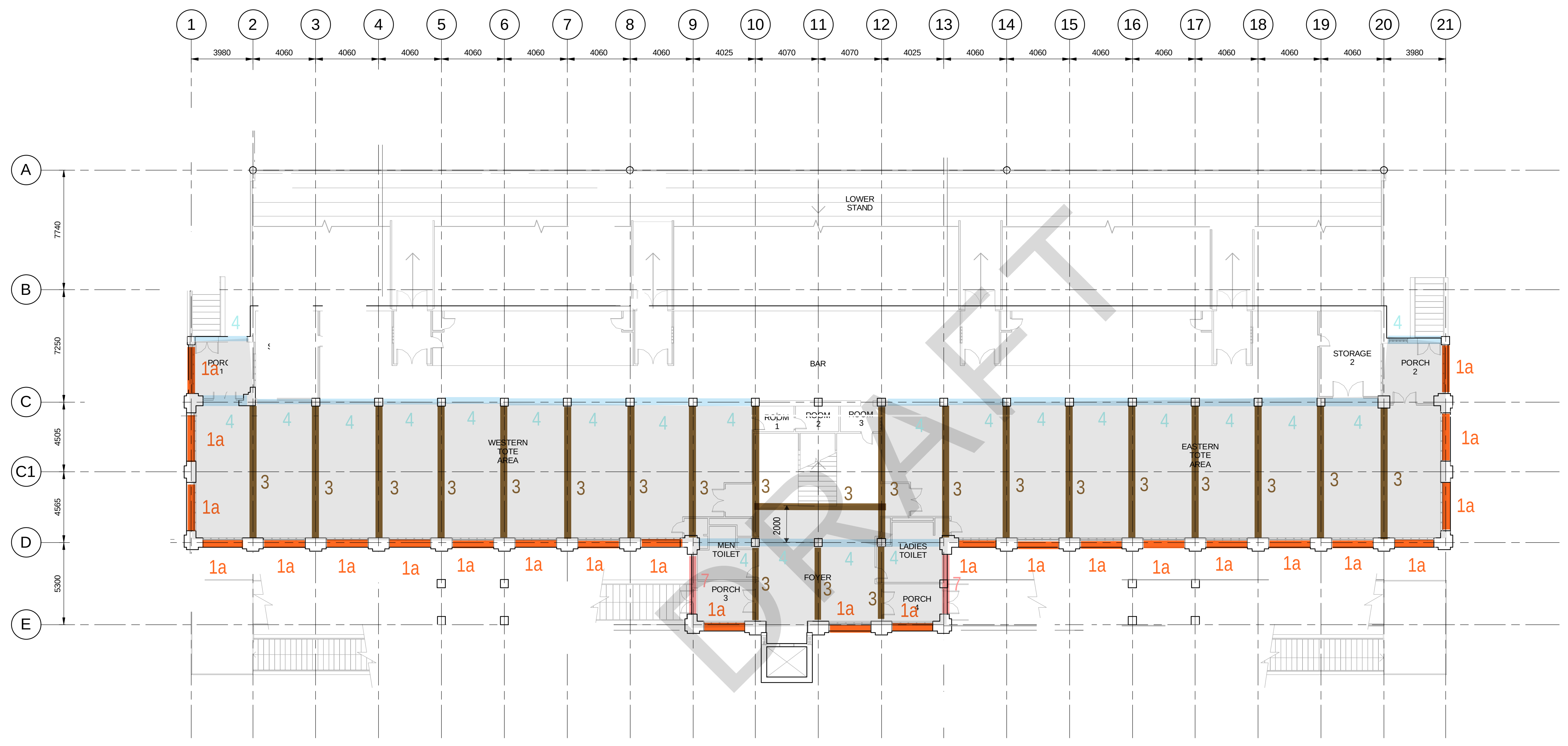

BEAMS SUPPORTING LEVEL 2 FLOOR

AECOM

CONSULTANT

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PROJECT

STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

Prepared for:

PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

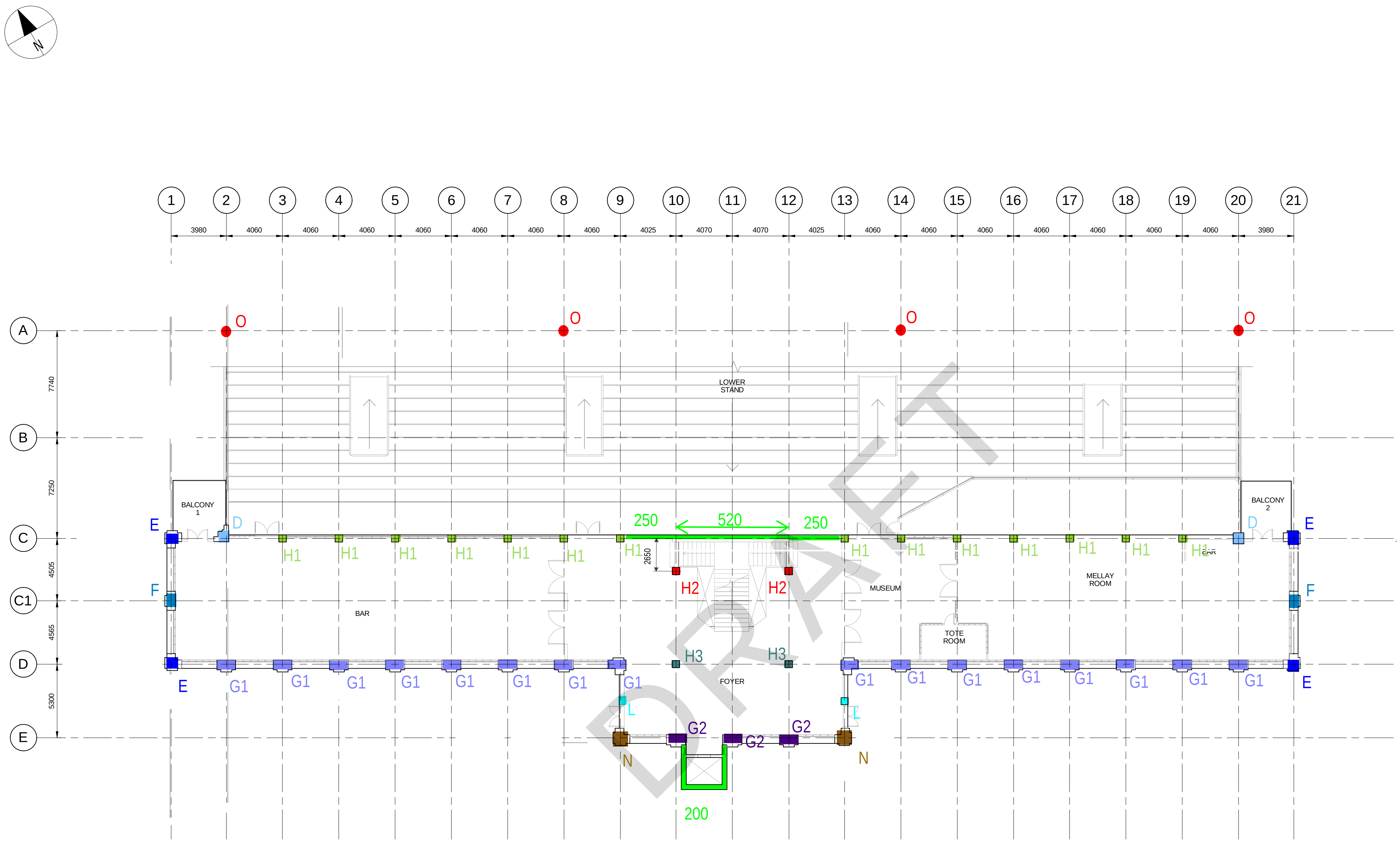
60439900

SHEET TITLE

ANALYTICALMODEL

SHEET NUMBER

B.5



LEVEL 2 COLUMNS

ISO A1 594mm x 841mm

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PROJECT

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CANTERBURY JOCKEY CLUB, CHRISTCHURCH

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NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

60439900

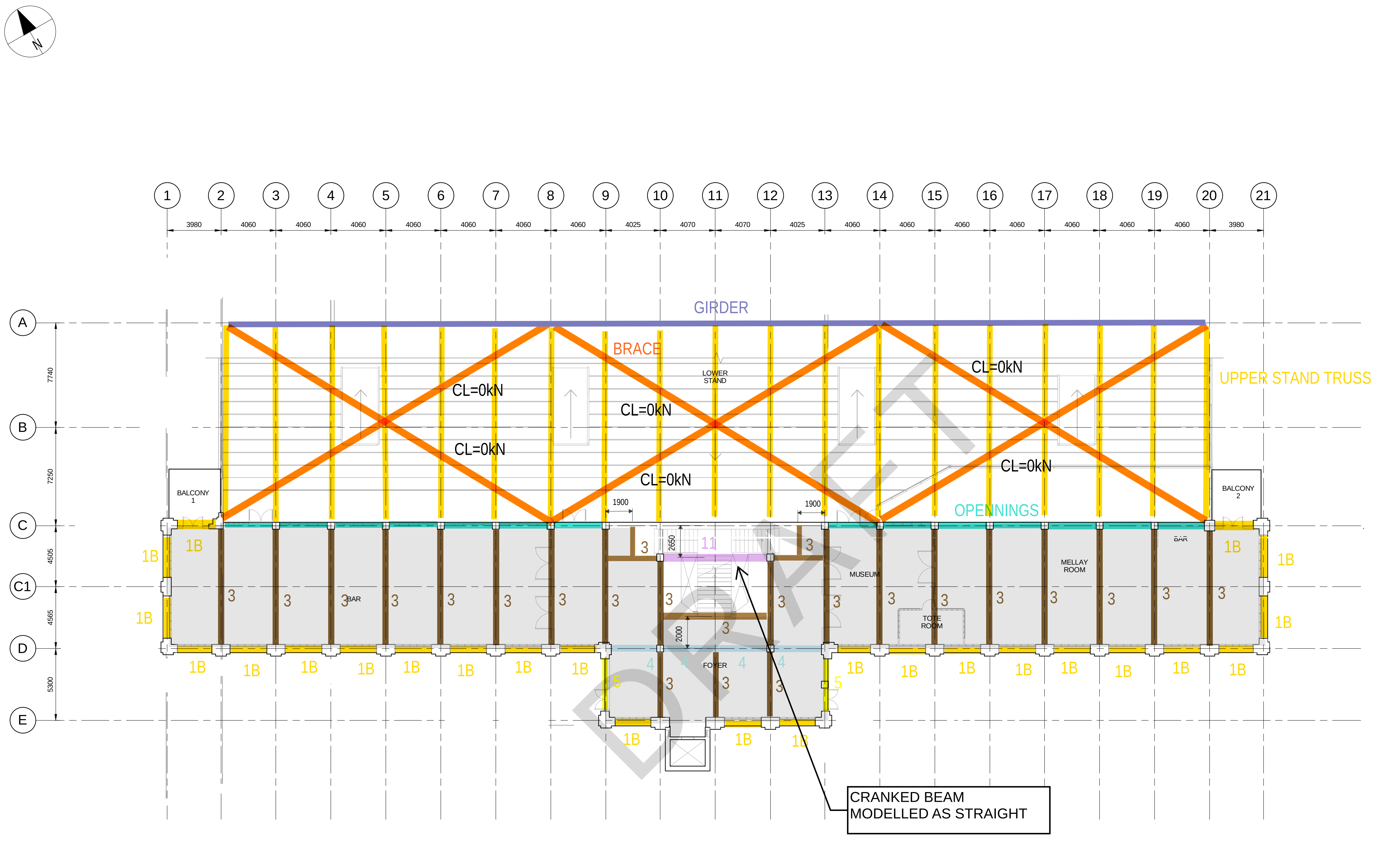
SHEET TITLE

ANALYTICALMODEL

SHEET NUMBER

B.6

Last Saved: 8/06/2015 4:47:30 p.m.
Filename: C:\Users\PettiC\Desktop\60332326_MOD_ST_2014_PettiC(Recovery).rvt



BEAMS SUPPORTING LEVEL 3 FLOOR

AECOM

PROJECT

STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

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CANTERBURY JOCKEY CLUB

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NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

60439900

SHEET TITLE

ANALYTICALMODEL

SHEET NUMBER

B.7

Last Saved: 8/06/2015 4:47:30 p.m.
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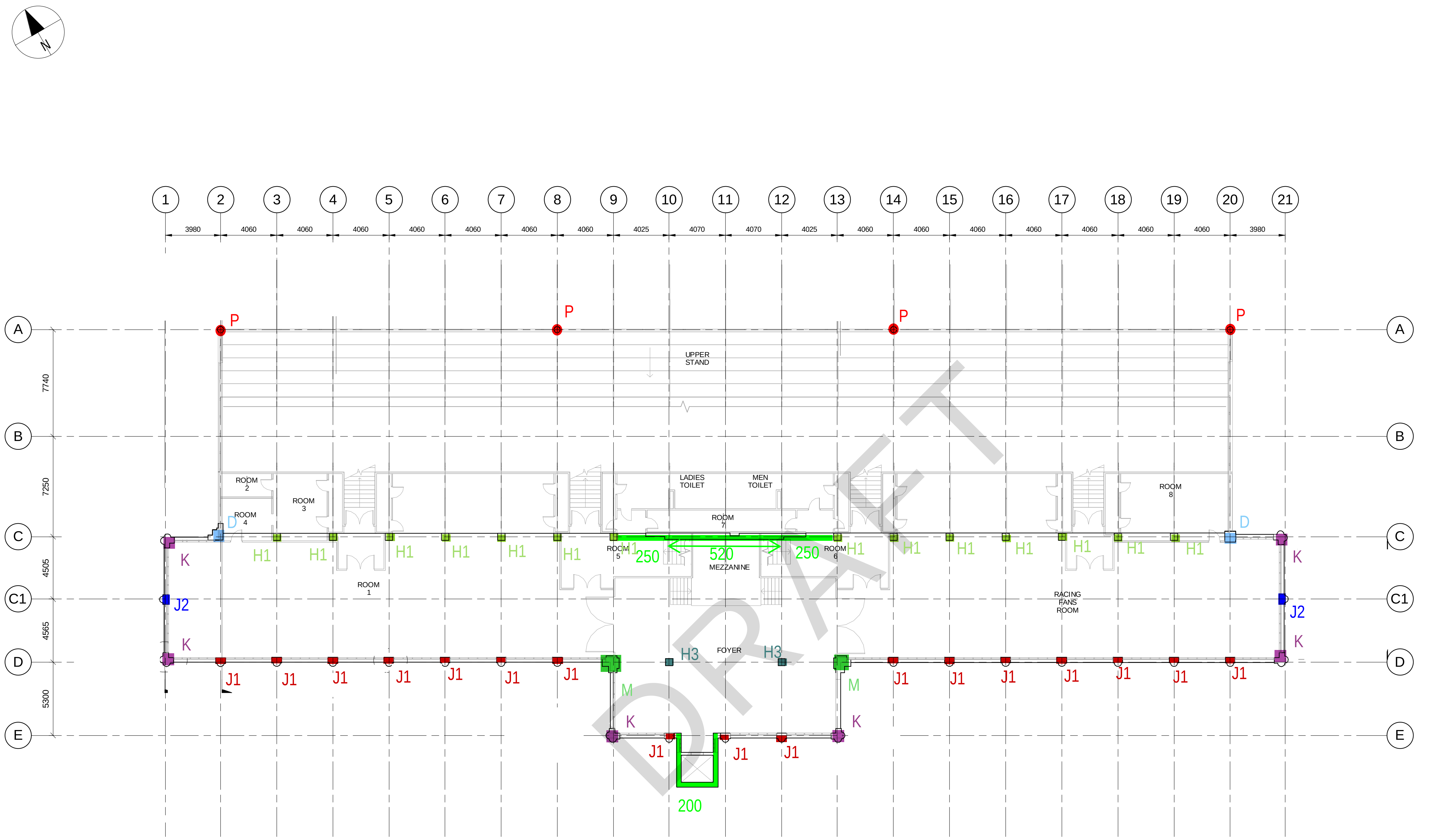

LEVEL 3 COLUMNS

ISO A1 594mm x 841mm

Last Saved: 8/06/2015 4:47:32 p.m.
Filename: C:\Users\PettiC\Desktop\60332326_MOD_ST_2014_PettiC(Recovery).rvt

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PROJECT

STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

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PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

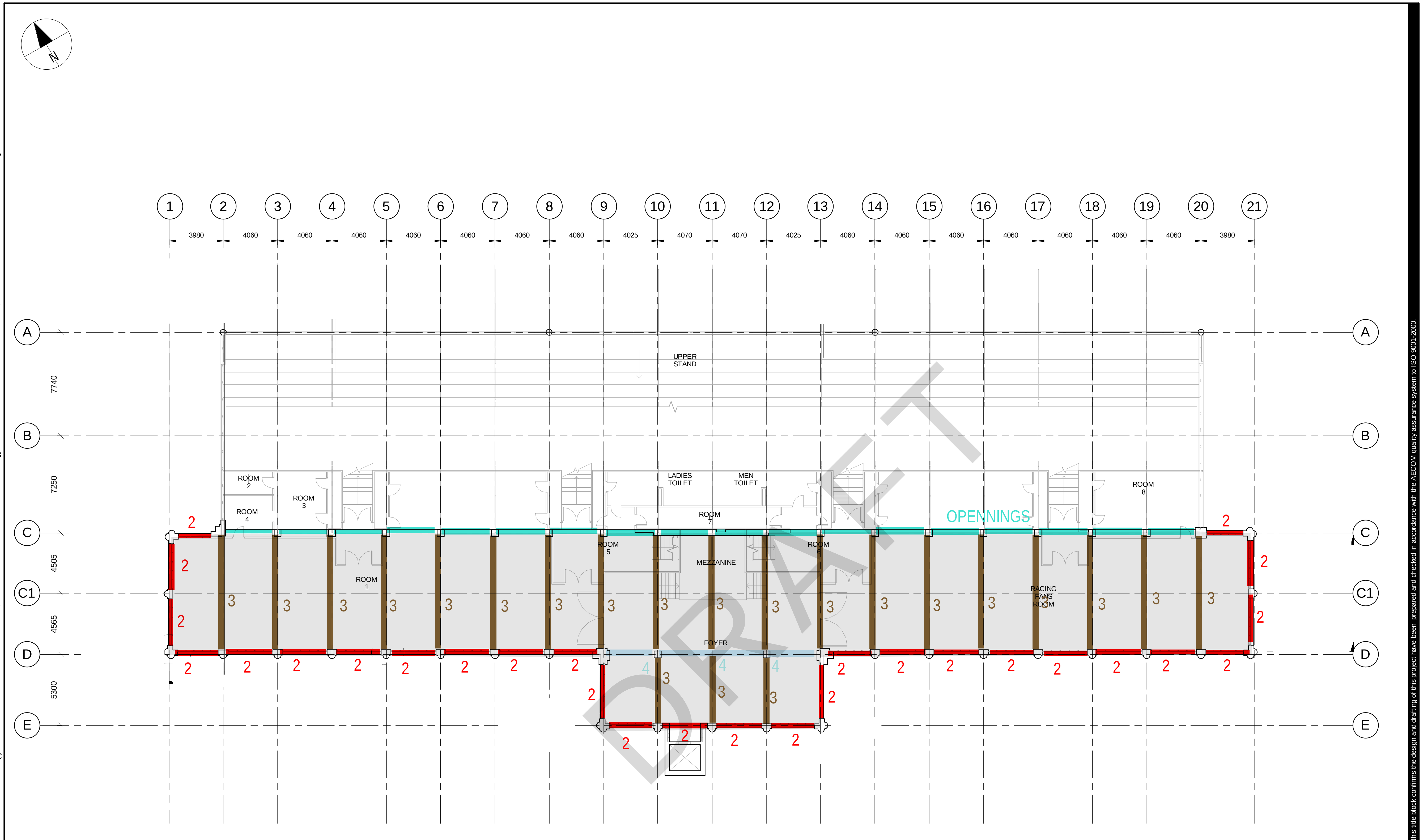
60439900

SHEET TITLE

ANALYTICALMODEL

SHEET NUMBER

B.8



BEAMS SUPPORTING LEVEL 4 FLOOR

AECOM

PROJECT

STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

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EXTENT OF SLAB

PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

60439900

SHEET TITLE

ANALYTICALMODEL

SHEET NUMBER

B.9

ISO A1 594mm x 841mm
Last Saved: 8/06/2015 4:47:32 p.m.
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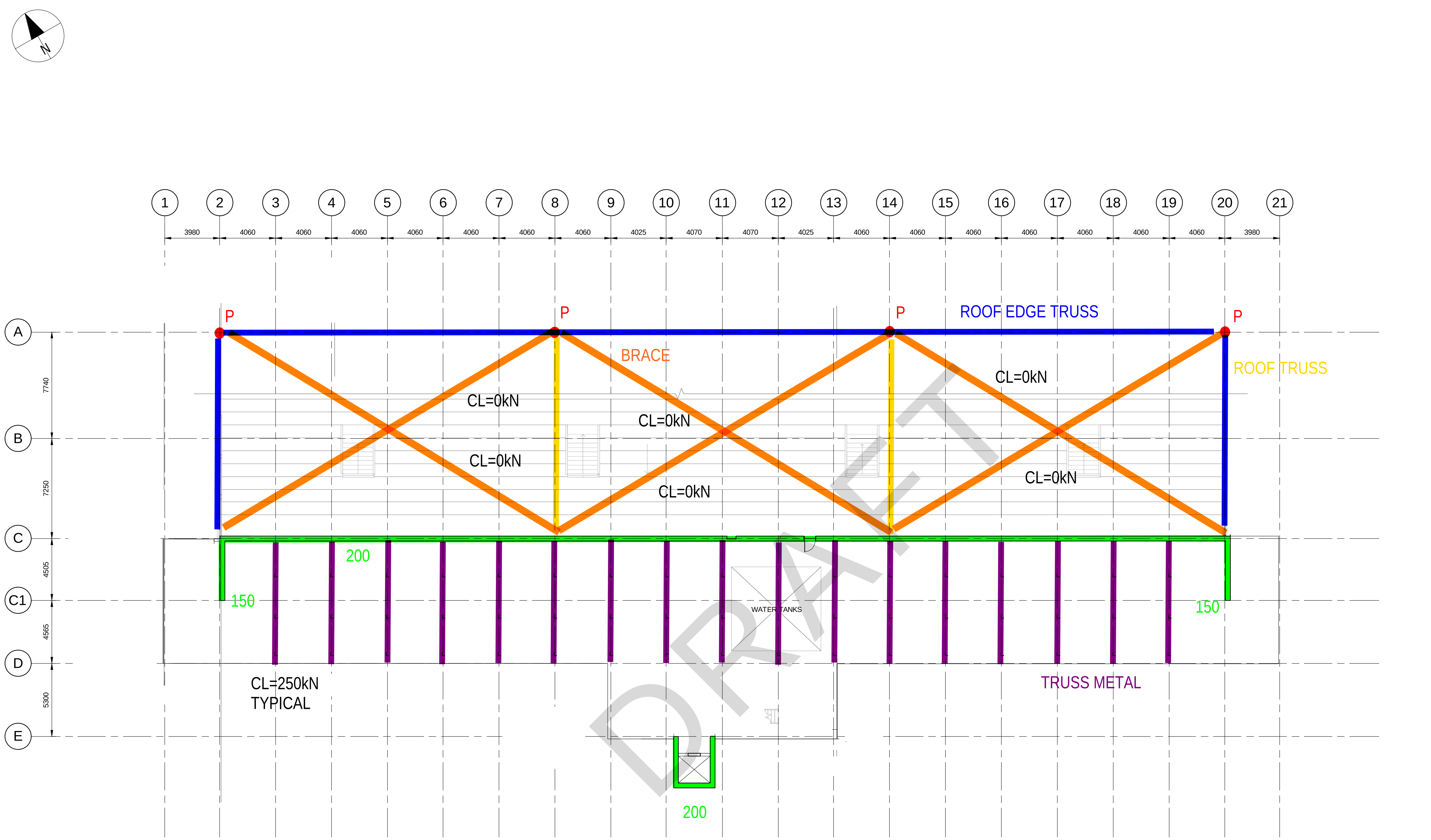

LEVEL 4 COLUMNS

CL = Compression Limit

AECOM

PROJECT

STRUCTURAL ASSESSMENT

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

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ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

60439900

SHEET TITLE

ANALYTICALMODEL

SHEET NUMBER

B.10

ISO A1 594mm x 841mm
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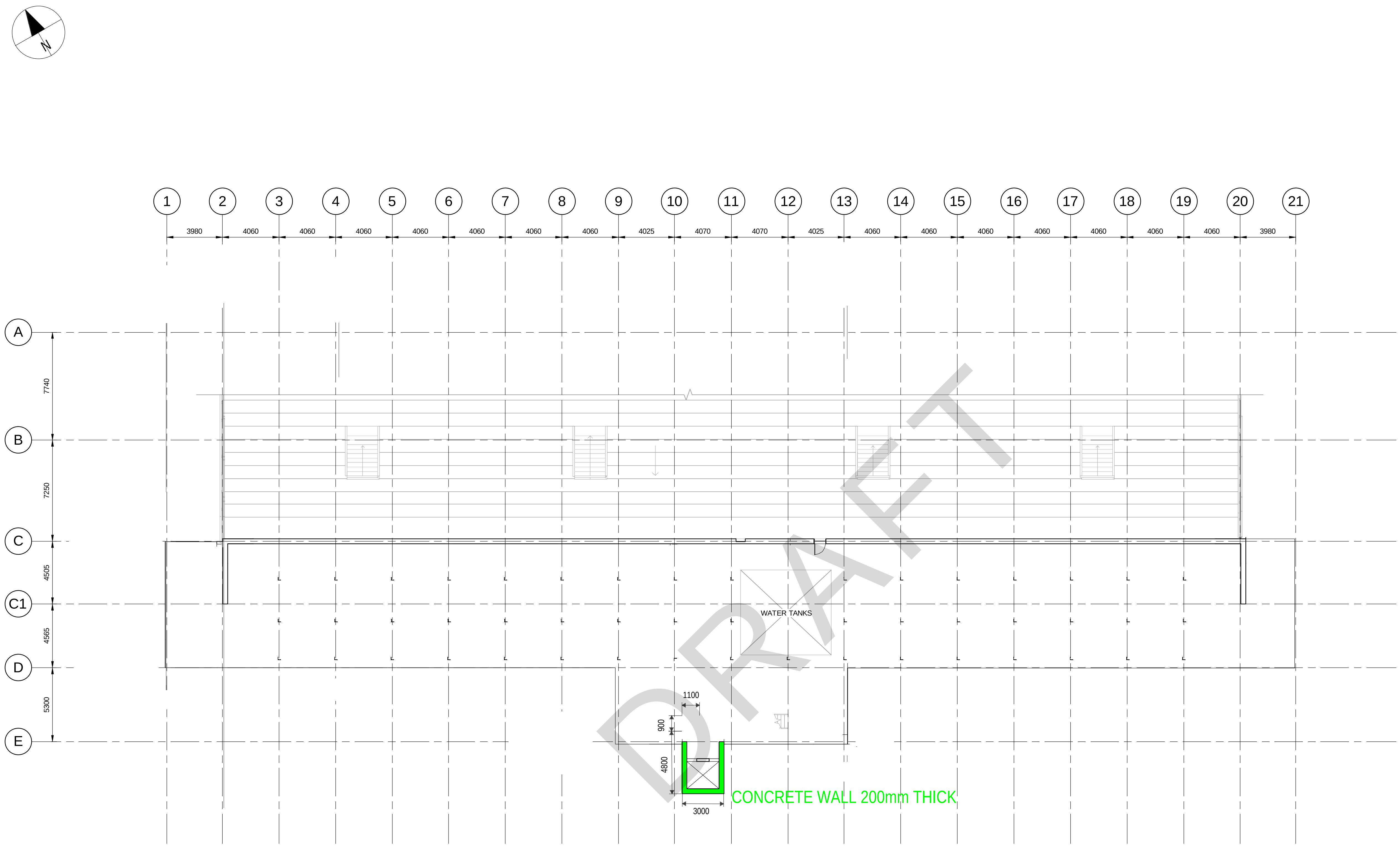

LEVEL 5 COLUMNS

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PROJECT

STRUCTURAL ASSESSMENT

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CLIENT

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PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A		

KEY PLAN

PROJECT NUMBER

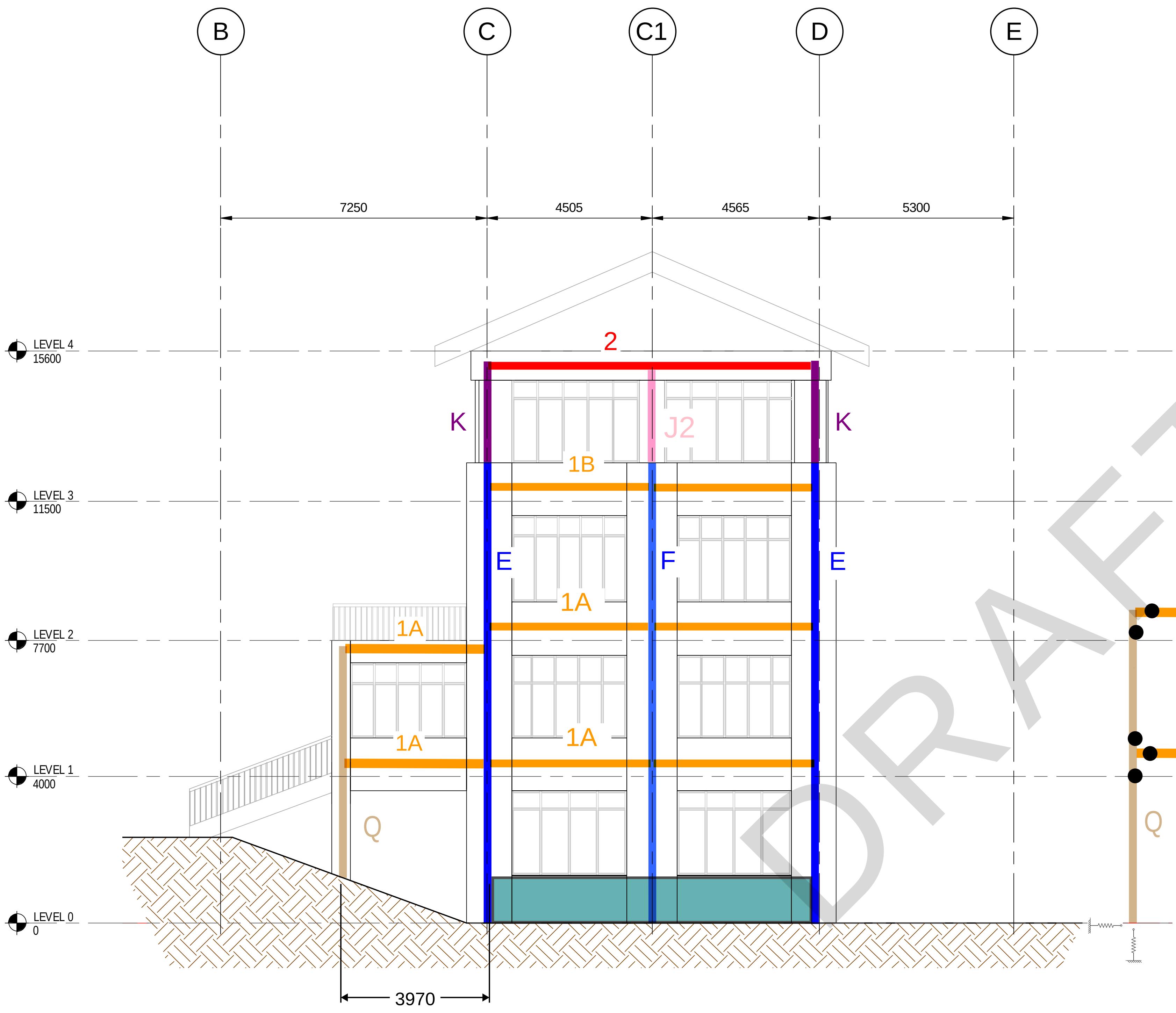
60439900

SHEET TITLE

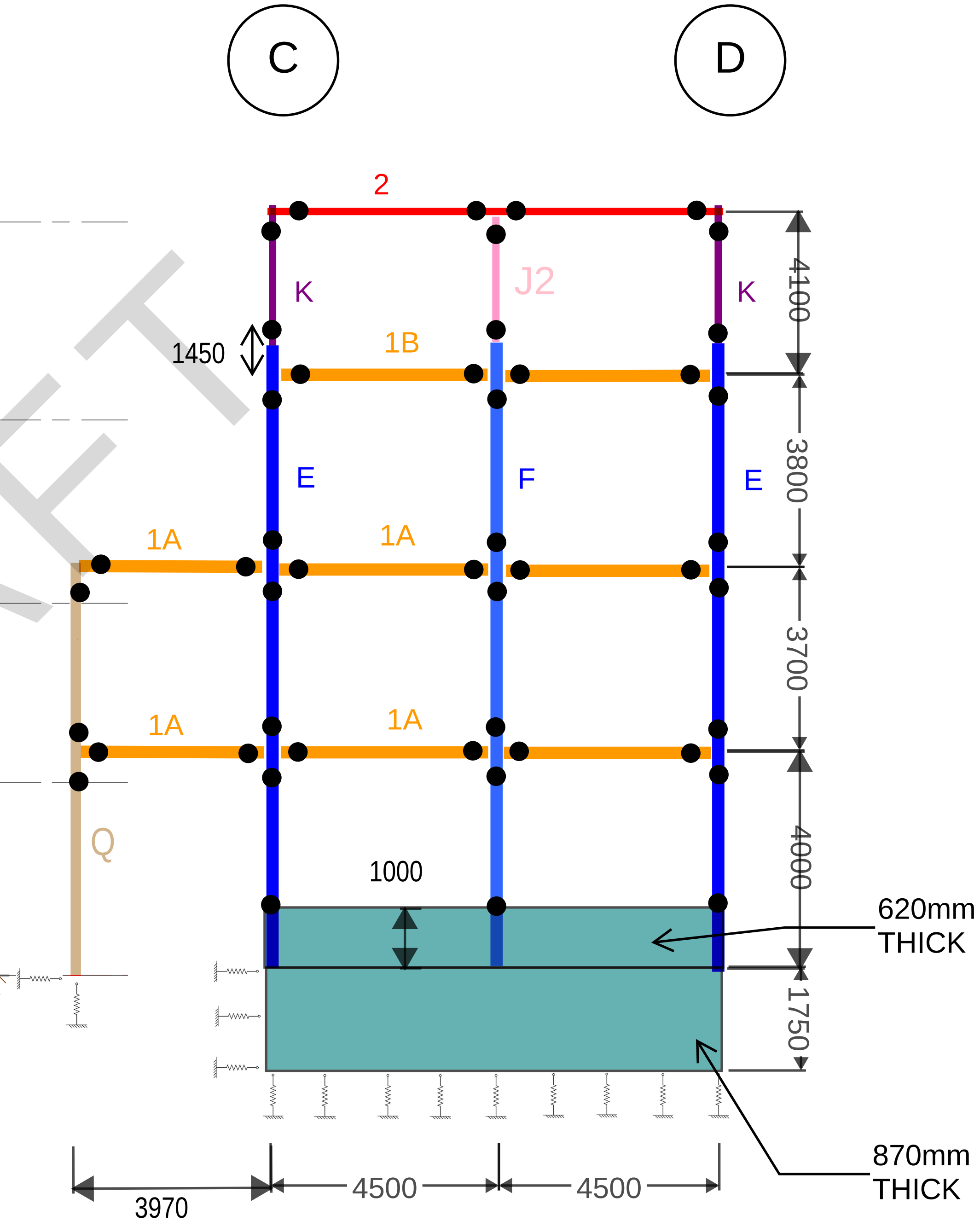
ANALYTICALMODEL

SHEET NUMBER

B.11



ELEVATION



ANALYTICAL MODEL

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PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

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Key

● POTENTIAL PLASTIC HINGE LOCATION

PROJECT MANAGEMENT INITIALS

DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

60439900

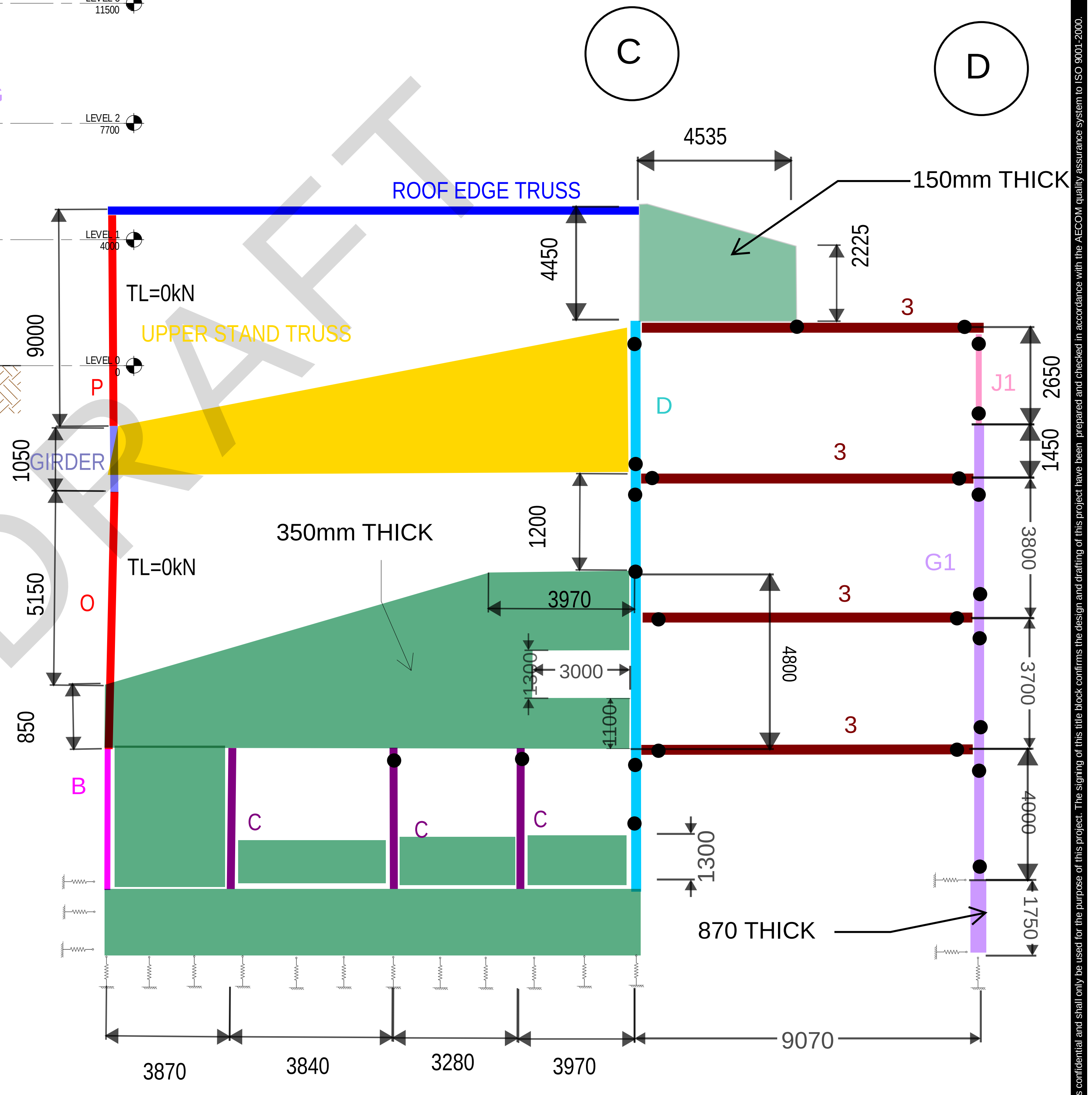
SHEET TITLE

FRAME IN GRID 1 ANALYTICAL MODEL

SHEET NUMBER

B.12

ELEVATION



ANALYTICAL MODEL

AECOM

PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

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Key

● POTENTIAL PLASTIC HINGE LOCATION

TL = Tension Limit
CL = Compression Limit

PROJECT MANAGEMENT INITIALS

DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

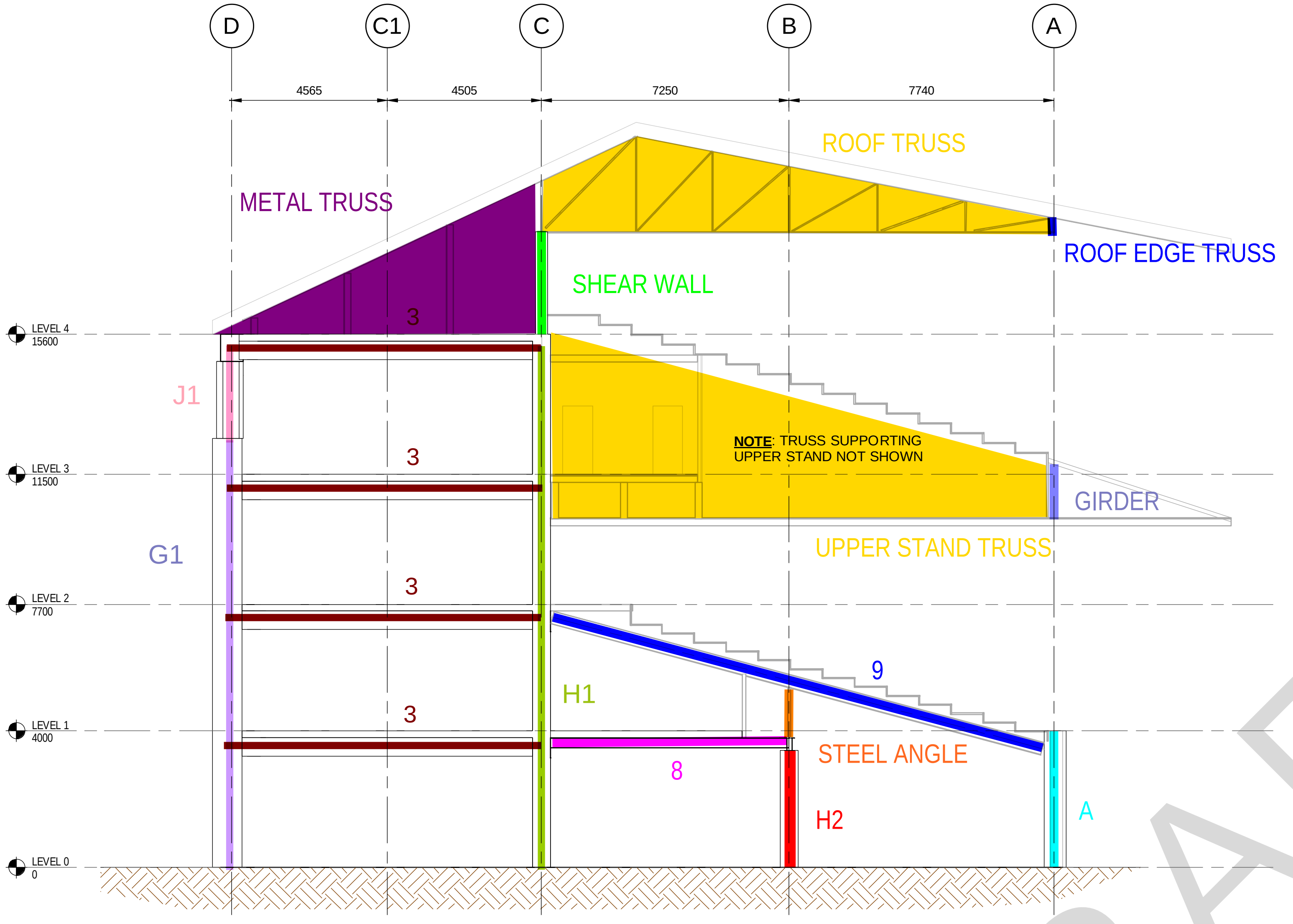
60439900

SHEET TITLE

FRAME IN GRID 2 ANALYTICAL MODEL

SHEET NUMBER

B.13



ELEVATION

AECOM

PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

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Key

● POTENTIAL PLASTIC HINGE LOCATION

PROJECT MANAGEMENT INITIALS

DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

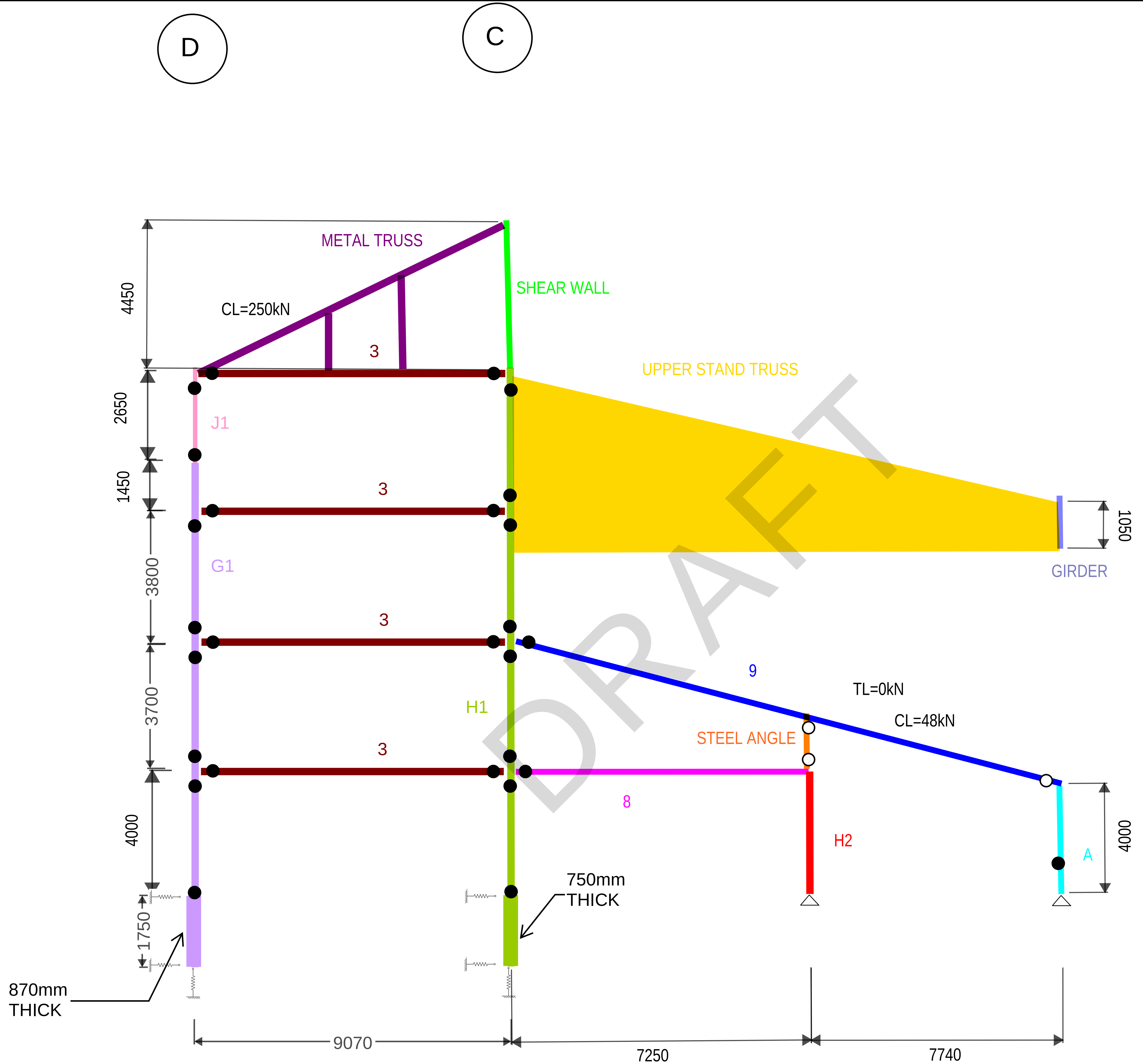
60439900

SHEET TITLE

FRAME IN GRID 4

SHEET NUMBER

B.14



ANALYTICAL MODEL

AECOM

PROJECT
STRUCTURAL ASSESSMENT GRAND NATIONAL STAND
CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT
CANTERBURY JOCKEY CLUB

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Key
● POTENTIAL PLASTIC HINGE LOCATION

CL = Compression Limit
TL = Tension Limit

PROJECT MANAGEMENT INITIALS

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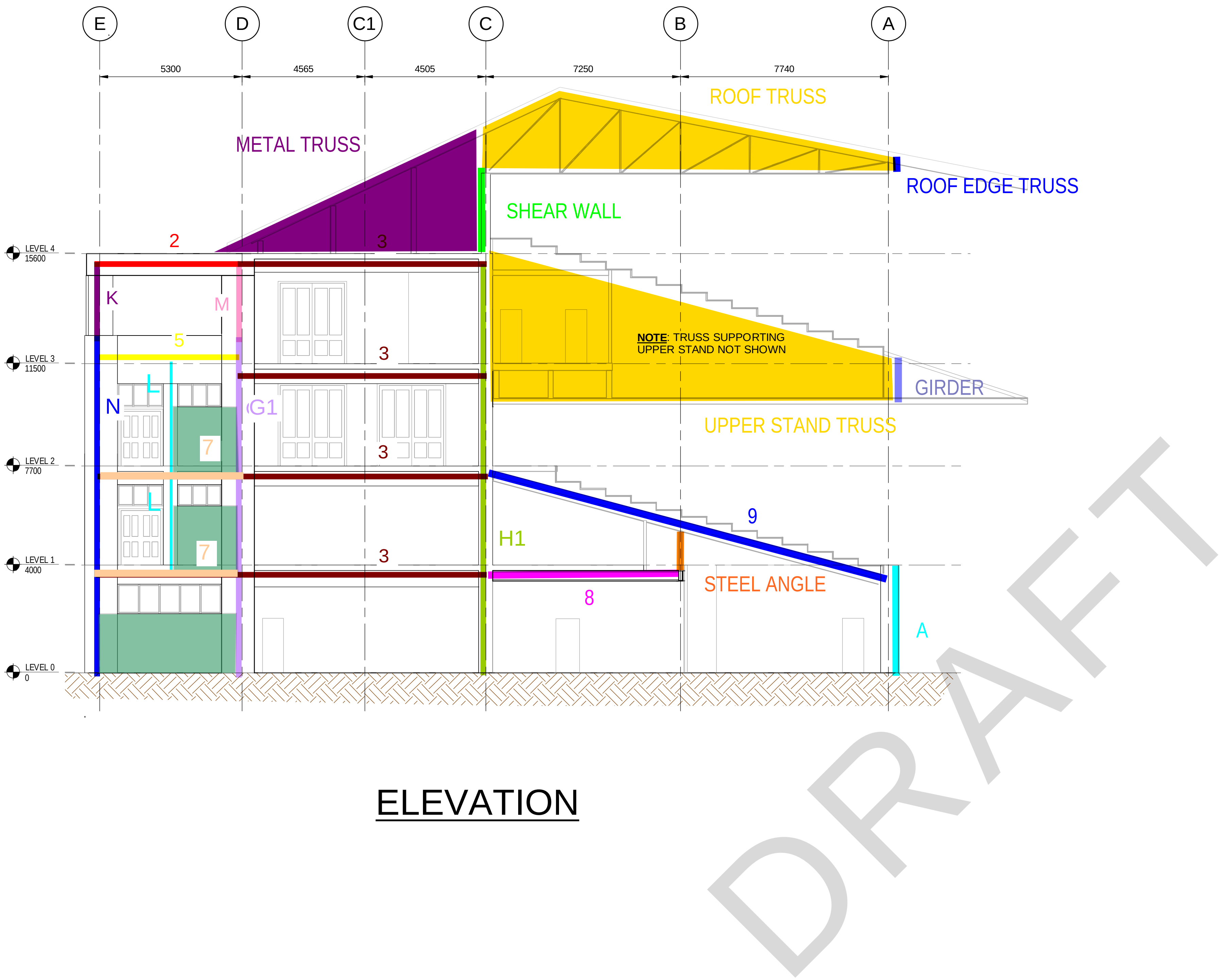
I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER
60439900

SHEET TITLE
FRAME IN GRID 4 ANALYTICAL MODEL

SHEET NUMBER
B.15



ELEVATION

AECOM

PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

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Key

● POTENTIAL PLASTIC HINGE LOCATION

PROJECT MANAGEMENT INITIALS

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KEY PLAN

PROJECT NUMBER

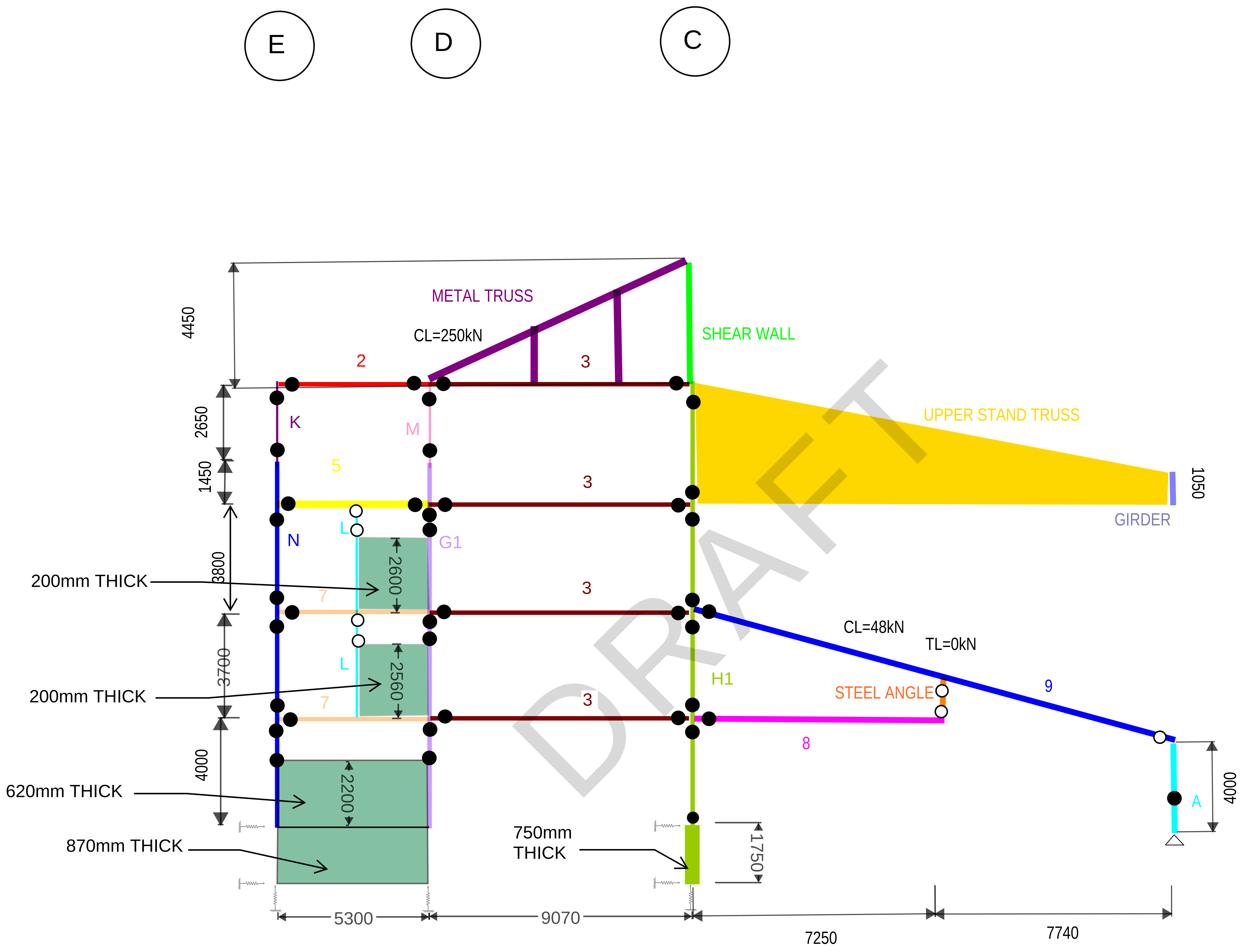
60439900

SHEET TITLE

FRAME IN GRID 13

SHEET NUMBER

B.16



ANALYTICAL MODEL

AECOM

PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

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Key

● POTENTIAL PLASTIC HINGE LOCATION

CL = Compression Limit

TL = Tension Limit

PROJECT MANAGEMENT INITIALS

DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

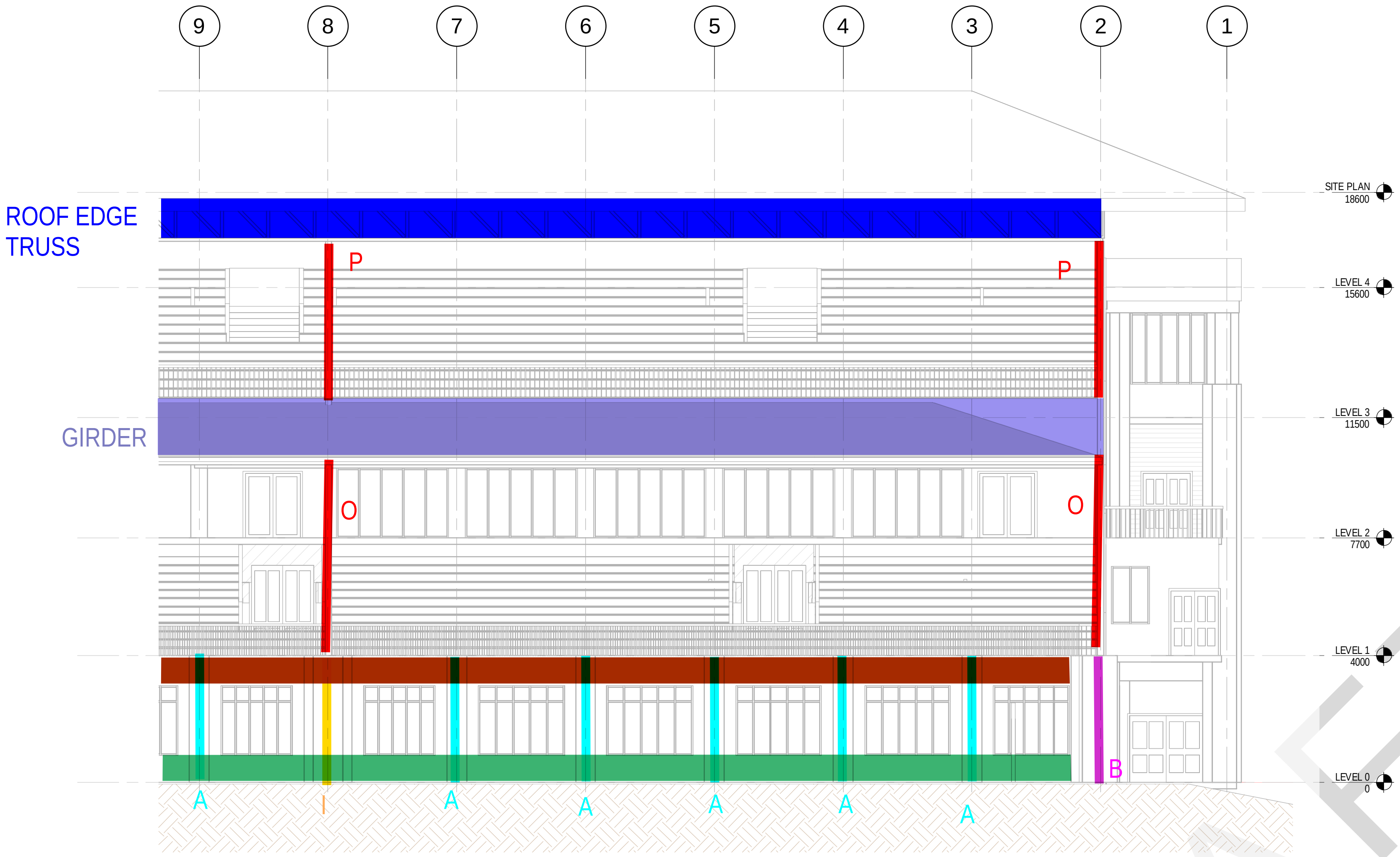
60439900

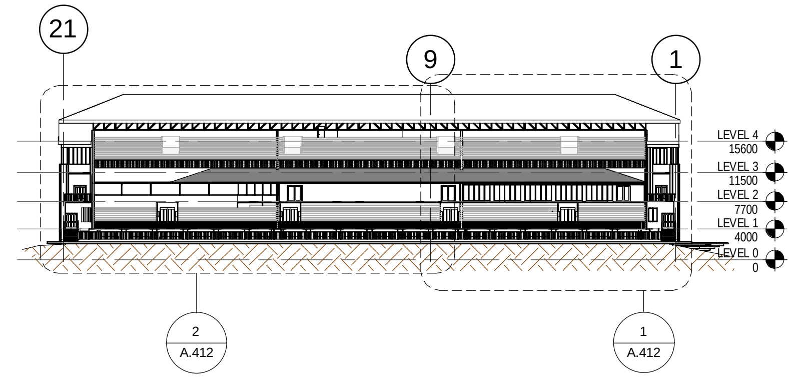
SHEET TITLE

FRAME IN GRID 13 ANALYTICAL MODEL

SHEET NUMBER

B.17

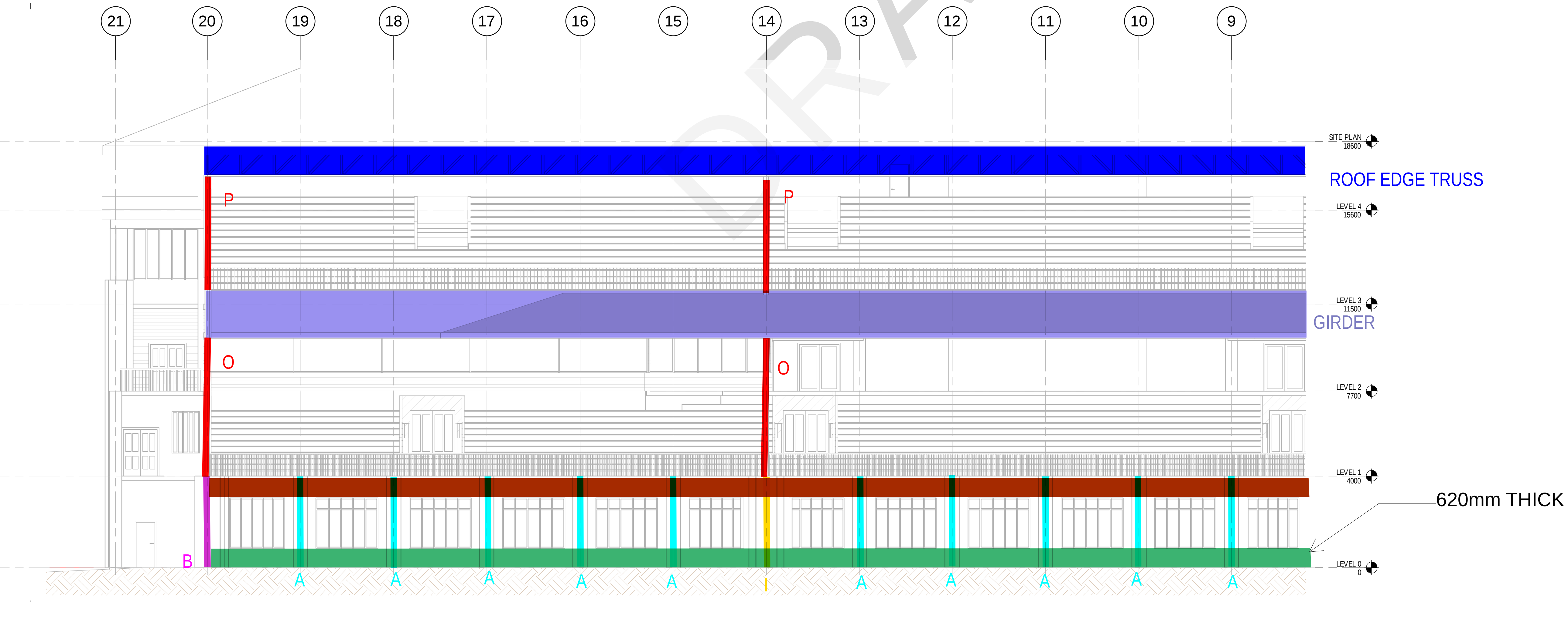




DAMAGE ON ELEVATIONS - ELEVATION NORTH

Scale: 1 : 500

ELEVATION - GRID A



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PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

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CONSULTANT

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ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

60439900

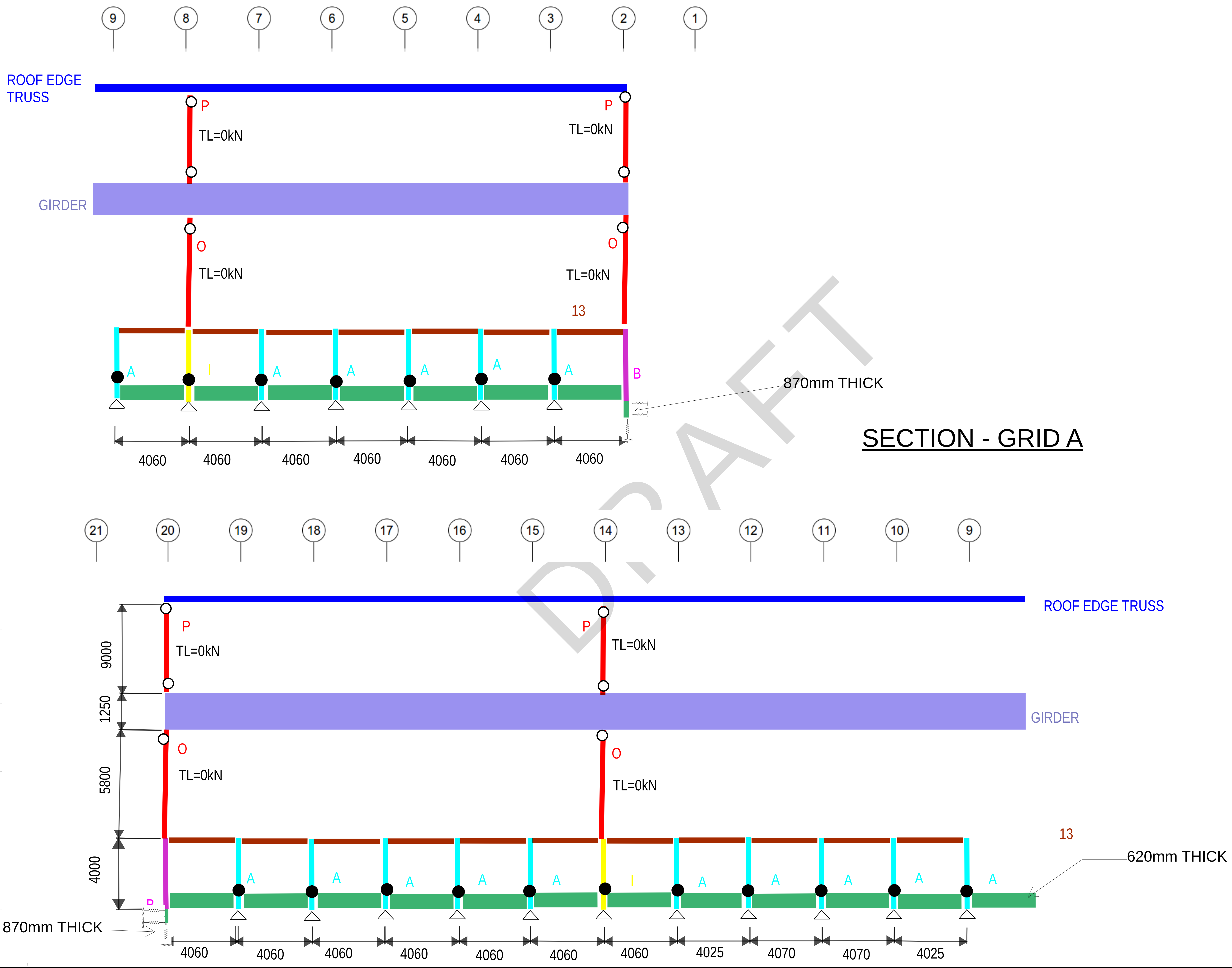
SHEET TITLE

FRAMEINGRIDA

SHEET NUMBER

B.18

SECTION - GRID A



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STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

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CLIENT

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TL = Tension Limit

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NR	MF	CO
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KEY PLAN

PROJECT NUMBER

60439900

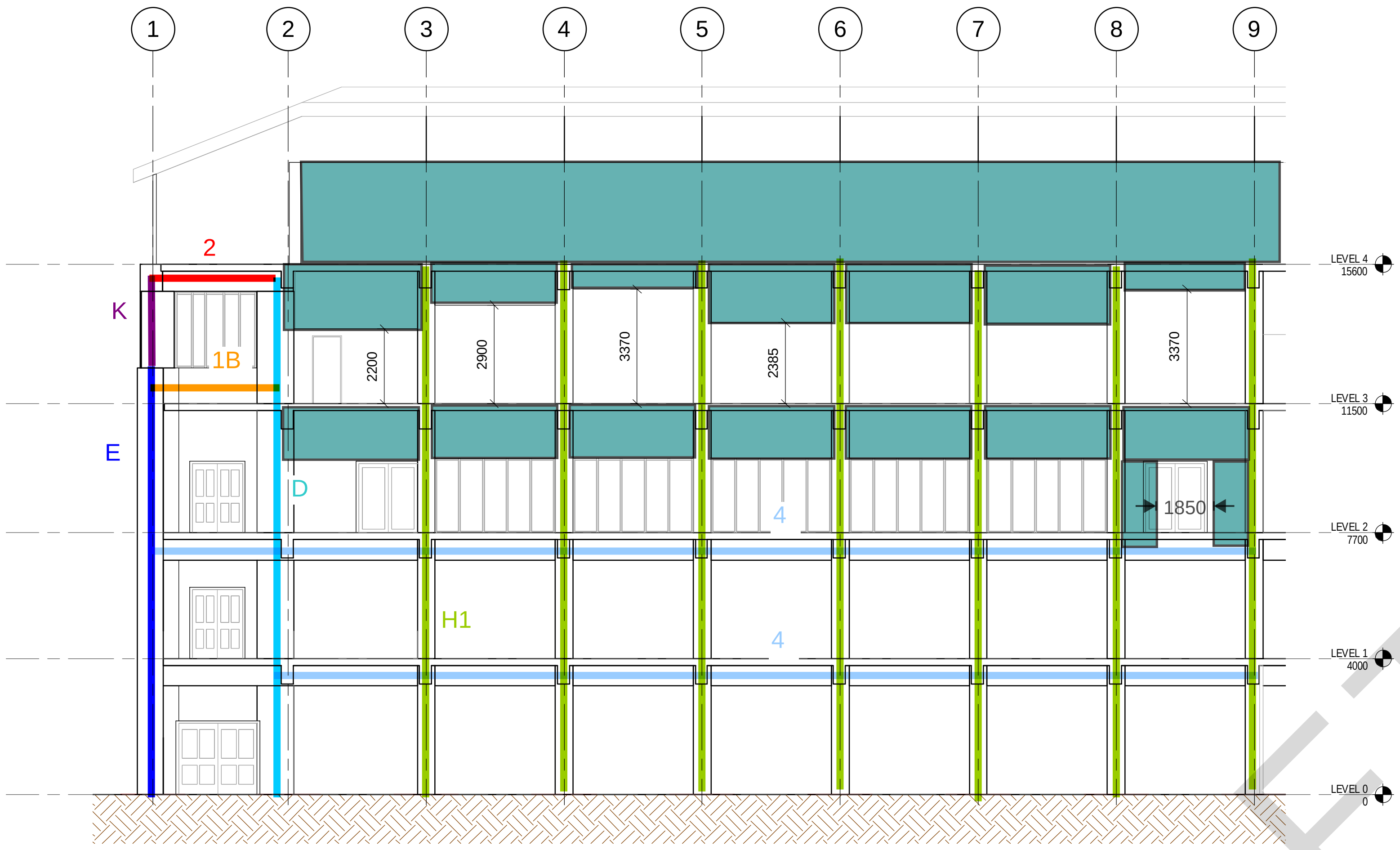
SHEET TITLE

FRAMEINGRIDA-ANALYTICAL MODEL

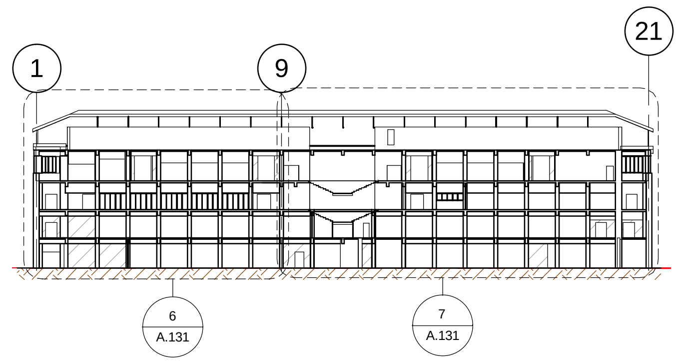
SHEET NUMBER

B.19

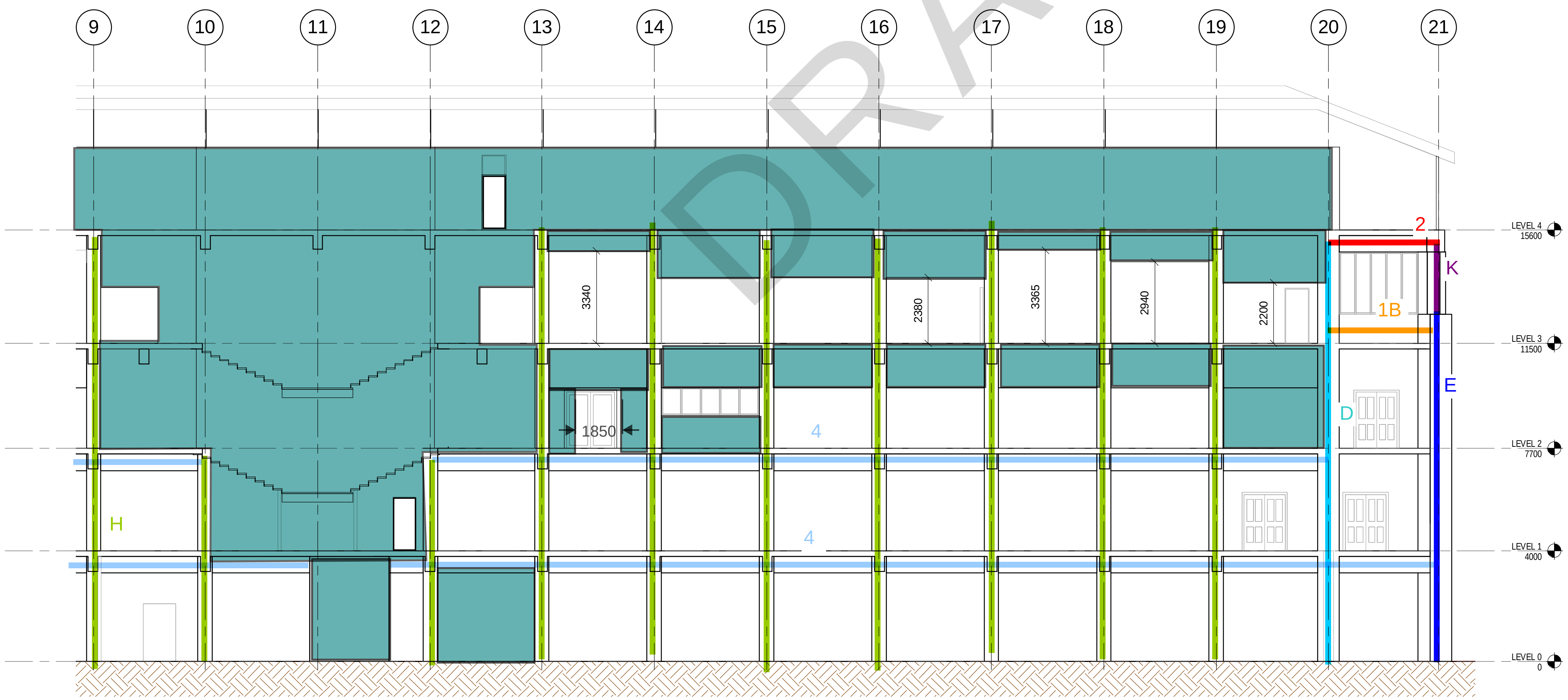
ISO A1 594mm x 841mm



6 | SECTION ON GRID C - GRID 1 TO GRID 9
A.131 | REF: A.131 Scale: 1 : 100



5 | SECTION ON GRID C
A.131 | REF: A.100 Scale: 1 : 500



7 | SECTION ON GRID C - GRID 9 TO 21
A.131 | REF: A.131 Scale: 1 : 100

DRAFT

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STRUCTURAL ASSESSMENT GRAND NATIONAL STAND
CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

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Key

PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

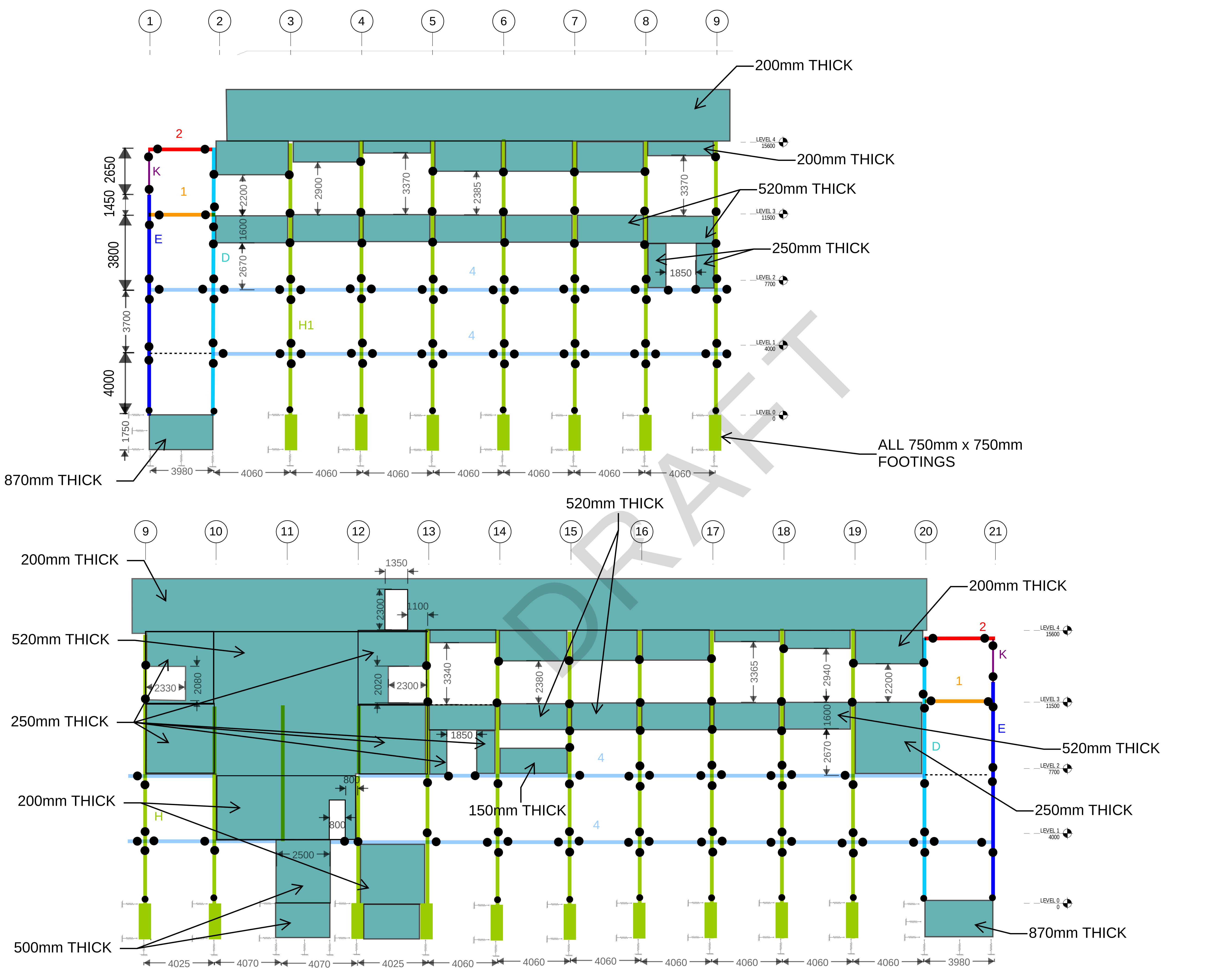
I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER
60439900

SHEET TITLE
FRAME IN GRID C

SHEET NUMBER
B.20



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PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

CONSULTANT

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Key

● POTENTIAL PLASTIC HINGE LOCATION

PROJECT MANAGEMENT INITIALS

DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

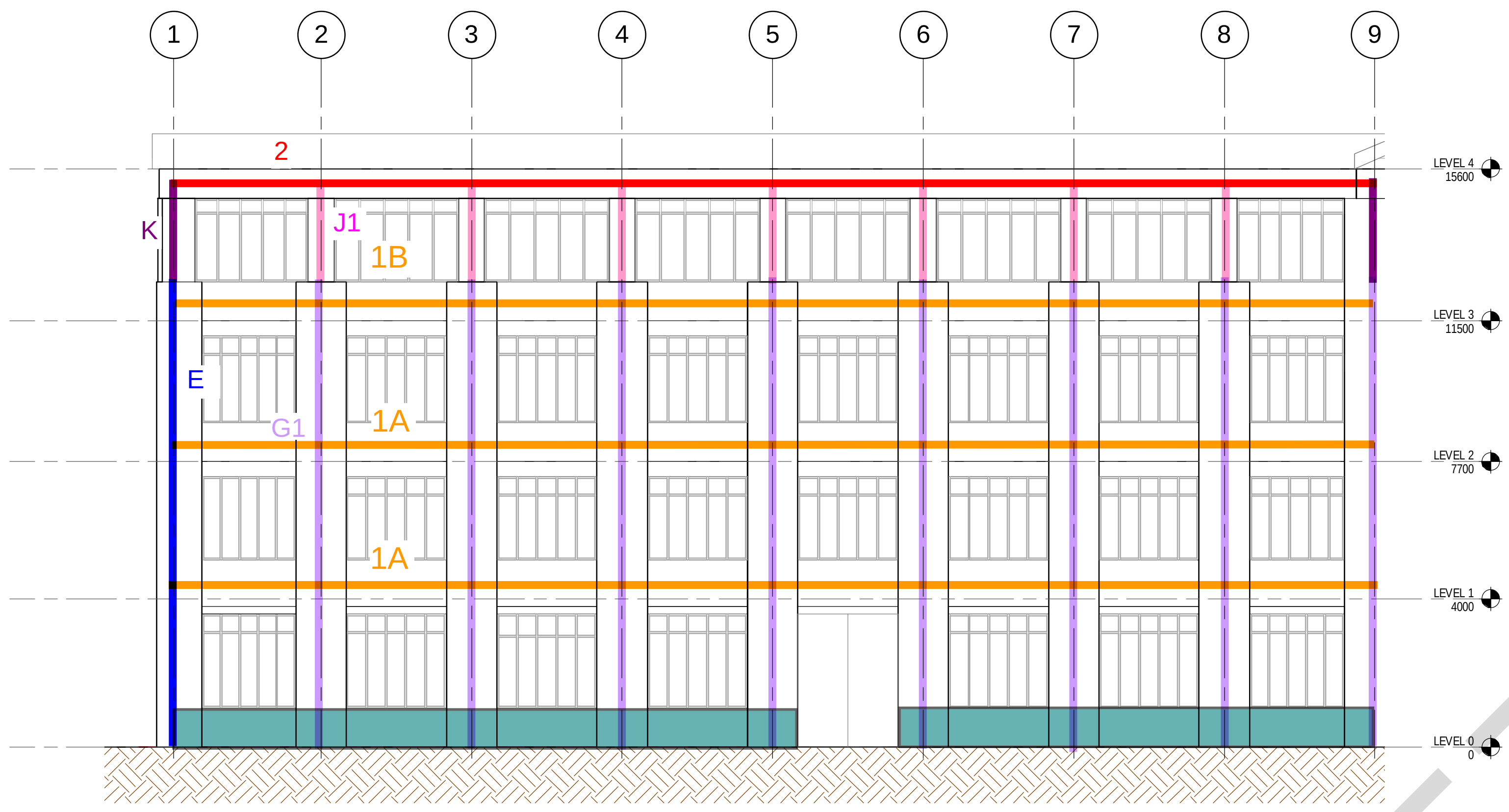
60439900

SHEET TITLE

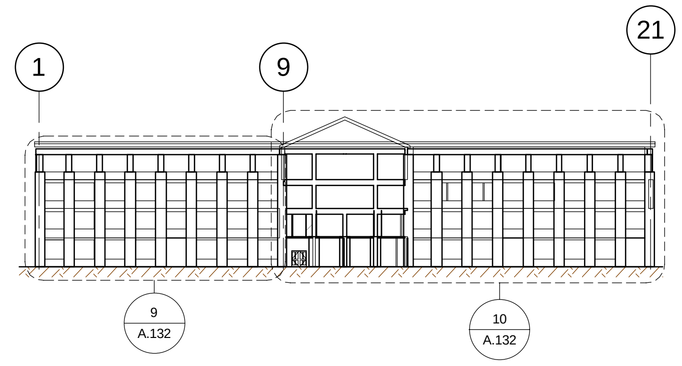
FRAME IN GRID C ANALYTICAL MODEL

SHEET NUMBER

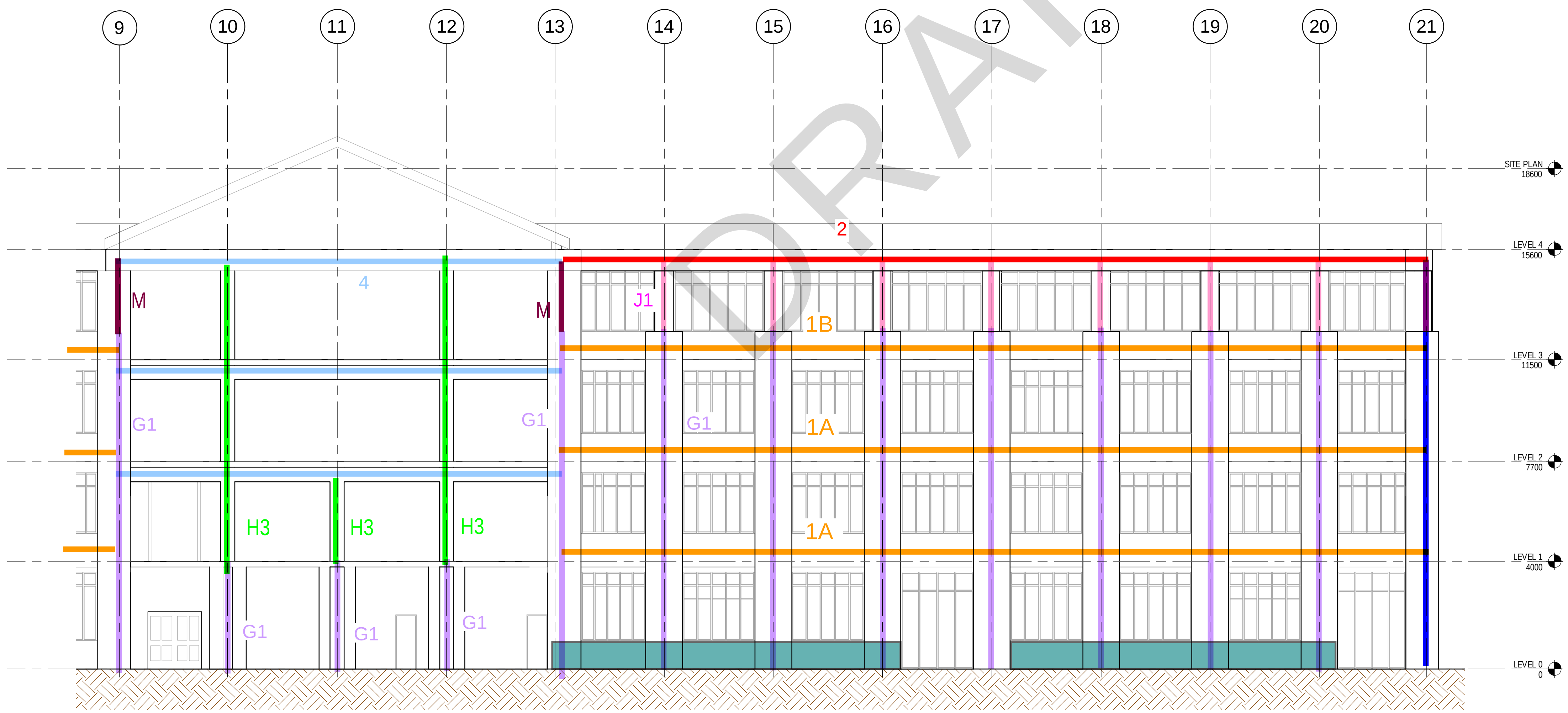
B.21



9 | SECTION ON GRID D - GRID 1 TO GRID 9
A.132 | REF: A.132 Scale: 1 : 100



8 | SECTION ON GRID D
A.132 | REF: A.100 Scale: 1 : 500



10 | SECTION ON GRID D - GRID 9 TO GRID 21
A.132 | REF: A.132 Scale: 1 : 100

DRAFT

ISO A1 594mm x 841mm

AECOM

PROJECT

STRUCTURAL ASSESSMENT GRAND NATIONAL STAND

CANTERBURY JOCKEY CLUB, CHRISTCHURCH

CLIENT

CANTERBURY JOCKEY CLUB

CONSULTANT

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Key

PROJECT MANAGEMENT INITIALS

NR	MF	CO
DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

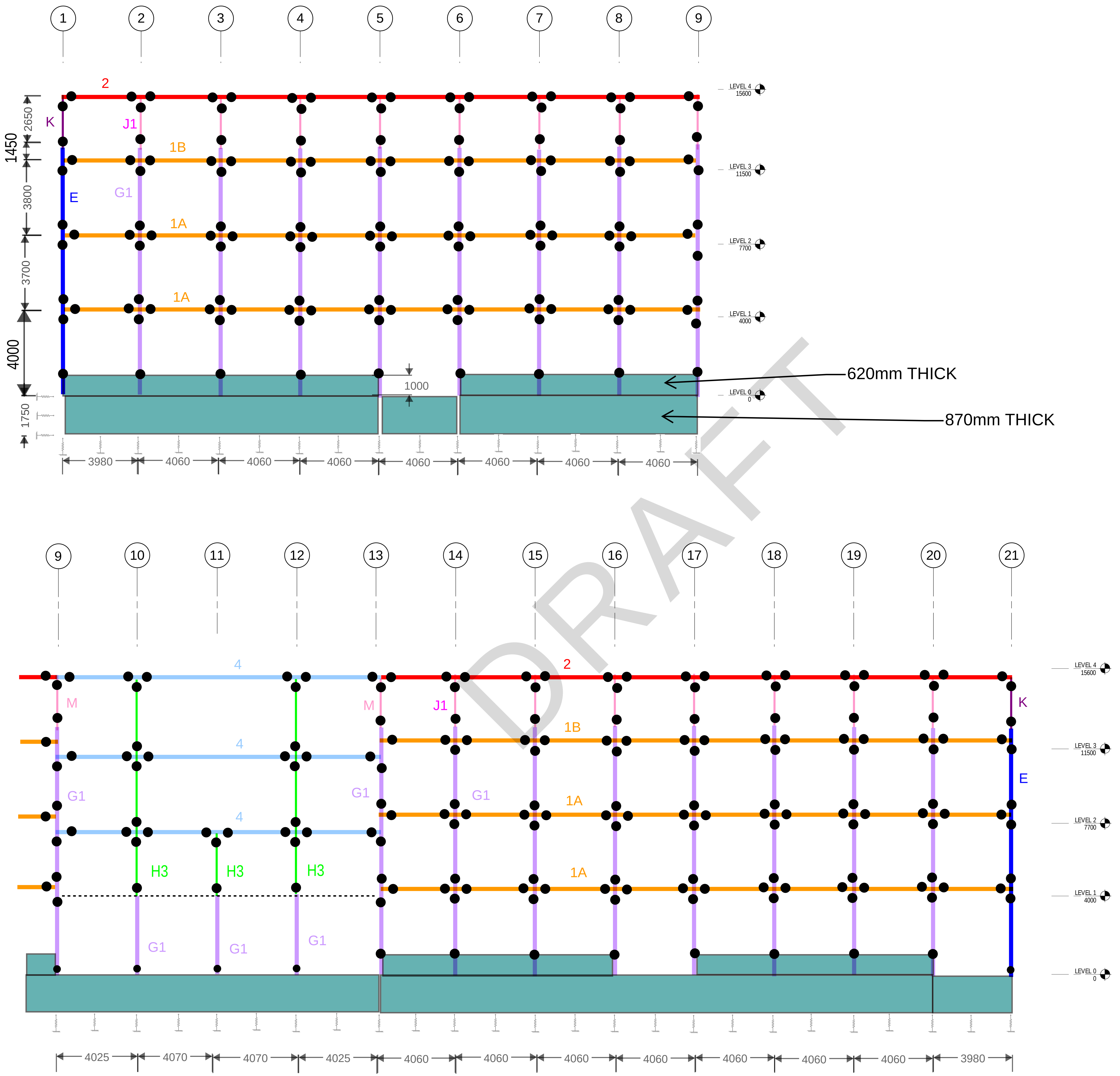
60439900

SHEET TITLE

FRAME IN GRID D

SHEET NUMBER

B.22



AECOM

PROJECT

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CANTERBURY JOCKEY CLUB, CHRISTCHURCH

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Key

● POTENTIAL PLASTIC HINGE LOCATION

PROJECT MANAGEMENT INITIALS

DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

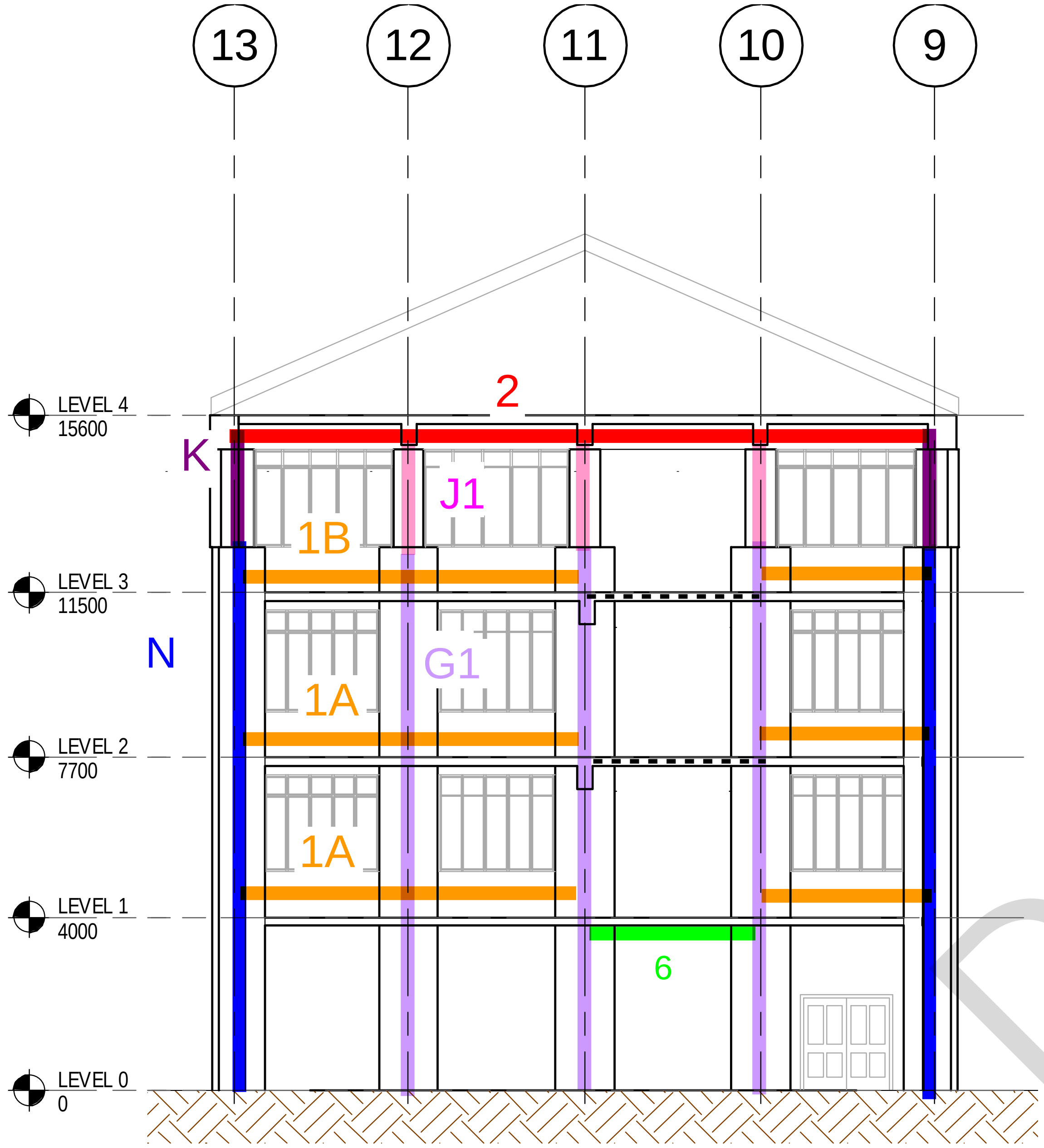
60439900

SHEET TITLE

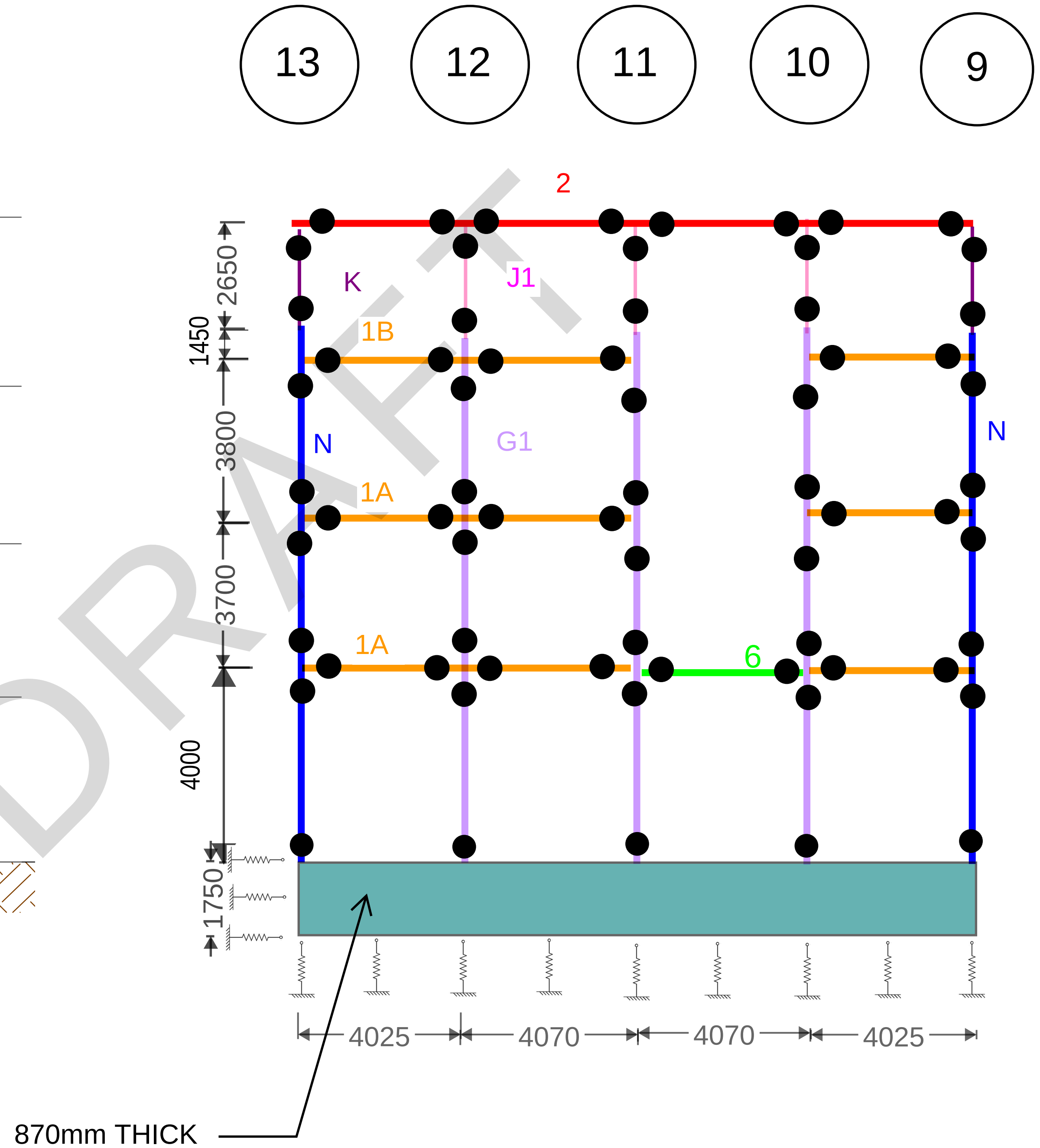
FRAME IN GRID D ANALYTICAL MODEL

SHEET NUMBER

B.23



ELEVATION



ANALYTICAL MODEL

AECOM

PROJECT

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CLIENT

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Key

- ● POTENTIAL PLASTIC HINGE LOCATION

PROJECT MANAGEMENT INITIALS

DESIGNER	CHECKED	APPROVED

ISSUE/REVISION

I/R	DATE	DESCRIPTION

KEY PLAN

PROJECT NUMBER

60439900

SHEET TITLE

FRAME IN GRID E ANALYTICAL MODEL

SHEET NUMBER

B.24

Appendix C

Additional Loads and Seismic Mass

ID	Area	Load Applied	Dead		Live	
1	Roof	Point loads applied to grid 3 – 17 truss support ends	Grid A Grid C Grid D	16.5 kN 26.5 kN 10 kN	Grid A Grid C Grid D	7.5 kN 12.1kN 4.6 kN
2		Point loads applied to grid 2 & 20 truss support ends	Grid C Grid D	10 kN 10 kN	Grid C Grid D	4.6 kN 4.6 kN
3		Line load applied to grid 1 & 21 between C & D	2.2 kN/m		1.0 kN/m	
4		Line load applied to grid A truss	3.4 kN/m		1.0 kN/m	
5		Line load applied to grid 2 & 20 between A to C	5.6 kN/m		2.0 kN/m	
6		Line load applied to grid D & E	2.2 kN/m		1.0 kN/m	
7		Line load applied to grid C between grid 1 & 2 and 20 & 21	2.2 kN/m		1.0 kN/m	
8		Line load applied to grid 9 & 13 between grid D & E	2.2 kN/m		1.0 kN/m	
9	Roof overhang lower	Line load applied to grid A from grid 2 to 17	3.4 kN/m		1.0 kN/m	
10	Stand trusses upper	Load applied to diaphragm	0.85 kPa		5.0 kPa	
11	Lower stand beam	Load applied to diaphragm	0.55 kPa		5.0 kPa	
12	Elevator core	Line load at level 5 floor level	47.7 kN/m		10.2 kN/m	
13	Ramps south elevation	Point load	182 kN		84 kN	
14		Seismic weight	182 kN		84 kN	
15	Stairs south elevation	Point load	80 kN		49 kN	
16	Stairs east/west elevation	Point load	33 kN		20 kN	

DRAFT

Appendix B

Intrusive Work Reports

Craig, hi

Please find below the brief site memo covering today's site visit:

Project Name	CJC, Intrusive Works - Beam Column Joints	**Project No.**	60439900
Venue	Grand National Stand	**Time**	1:30-2:30pm
Participants	Nik Richter (AECOM), Ian Reynolds (Dominion, 021718729)	**Date**	12/10/2015
Item No.	**Notes**		**Selected photos**
1.	Beam-column joints exposed in three locations: - Level 2, gridline C8 - Level 2, gridline C6 - Level 0, gridline D8 Beam-column joint has not yet been exposed at Level 0, gridline D5		
2	At Level 2, gridline C8 location the following has been observed: - The middle portion of the steel beam was exposed in the joint by core drilling. Web of the steel beam was exposed. - No reinforcement in the column was encountered in the course of drilling - The support length (steel beam embedment) was indirectly measured to be approximately 200mm - A steel section was identified beyond the steel beam support. It is presumed that the section may be a rectangular (or square) steel hollow section extending the full height of the column. The actual section size or its connection to the steel beam is unknown.		
3	At Level 2, gridline C6 location the following has been observed: - The support length (steel beam embedment) was indirectly measured to be approximately 170mm - The remaining observations same as for item 1		
4	At Level 0, gridline D8 location the following has been observed: - The support length (steel beam embedment) was directly measured to be approximately 300mm - The remaining observations same as for item 1		

		

Further actions	Description	Date
1	Further intrusive investigation recommended to confirm the section size of the steel section beyond the beam support and its connection to the steel beam in the beam-column joint. AECOM recommends the following actions: - Clean up the interface between the steel beam and the presumed RHS/SHS steel section beyond (needle gun or similar) to remove excess concrete and determine the connection between the steel beam and the presumed RSH/SHS - Drill into the steel section to check the wall thickness of the steel section beyond the steel beam and determine its actual size - Remove concrete from the top of the flange of the beam to determine the connection between the beam and the presumed RHS/SHS beyond AECOM engineer to attend site (13/10, 9:00am) to explain and discuss the practical methodology of this work with contractor.	13/10/2015

Regards

Nik Richter
Senior Structural Engineer
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Memorandum of Inspection

Attention	Craig Stracey	File No.	
Company	Dominion Constructors Ltd	Date	14-Oct-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Page	8
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Nik Richter		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Beam-column joint intrusive works	Inspection Date	13-Oct-2015
Attendees	Nik Richter, Mike Lowe (AECOM) Craig Stracey, Ian Reynolds (Dominion)		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Craig Oldfield	AECOM	
Matthew Crake	AECOM	
Mike Lowe	AECOM	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	
Kit Lawrence	AECOM	

Attachments ☒ Yes ☐ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

Site inspection introduction:

At the request of Dominion Constructors Ltd, AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Tuesday 13th October 2015. The inspection covered the following items:

- Level 0 intrusive works to beam column joints
- Level 2 intrusive works to beam column joints
- Level 4 intrusive works

AECOM

Observations and recommendations:

1. Level 0 (ground floor) beam column joint on gridline D8 observations:

   - The steel bearing length on concrete (steel beam embedment) was directly measured to be approximately 300mm.
   - The middle portion of the steel beam was exposed in the joint by core drilling. Web of the steel beam was exposed.
   - No reinforcement in the column was encountered in the course of drilling.
   - A steel section was identified beyond the steel beam support. After investigation it was determined that the section beyond the beam is likely to be a double angle (2 no. 6"x6"x ¾" equal angles, similar to the one observed at level 4 supporting roof trusses, refer to Photo 3) or potentially a cruciform section (4 no. 6"x6"). Refer to Figure 1.

   Recommendations:

   - Remove/scabble concrete from the face of the steel section beyond the beam (see Photo 1) and expose the face of this steel section to determine the connection detail.
   - In addition we recommend to core drill (approx. 100mm diameter) into one of the columns on gridline C and on gridline D (refer to Figure 2 and Figure 3) in two orthogonal directions at approximately mid-height. This is required to confirm if the steel sections continue full height and determine if this arrangement is a double angle or cruciform (or other).
   - Ensure that scabbled concrete is thoroughly cleaned and that dust and debri is removed using compressed air or similar.

2. Level 2 beam column joint observations:

   - On grid C8 the steel beam bearing length on concrete (steel beam embedment) was indirectly measured to be approximately 200mm
   - On grid C6 the steel beam bearing length on concrete (steel beam embedment) was indirectly measured to be approximately 170mm
   - The remaining observations same as for item 1.

   Recommendations:

   - Core drill at the beam / col joint on gridline C6 to expose the connection between the assumed angle section and steel beam (refer to Photo 2). Remove excess concrete and expose the face of the steel section.
   - Ensure that scabbled concrete is thoroughly cleaned and that dust and debri is removed using compressed air or similar.

3. Level 4 observations:

   - AECOM observed double angle sections supporting the roof trusses at level and it is inferred that these sections continue all the way down to the foundations and are encased in concrete.
   - It is possible that these section are cruciform sections as depicted in Figure 1

Recommendations:

- Break out a section (approximately 50mm deep) of concrete adjacent to the steel section to confirm if a second set of angles is present beyond the visible steel section (refer to Photo 3).
- Ensure that scabbled concrete is thoroughly cleaned and that dust and debri is removed using compressed air or similar.

Further Actions / Inspections:

- AECOM engineer to attend site during removal of concrete and core drilling.
- Dominion to advise time of commencement of the above works.

Kind Regards,

Richter

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Senior Structural Engineer
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AECOM

Photos:

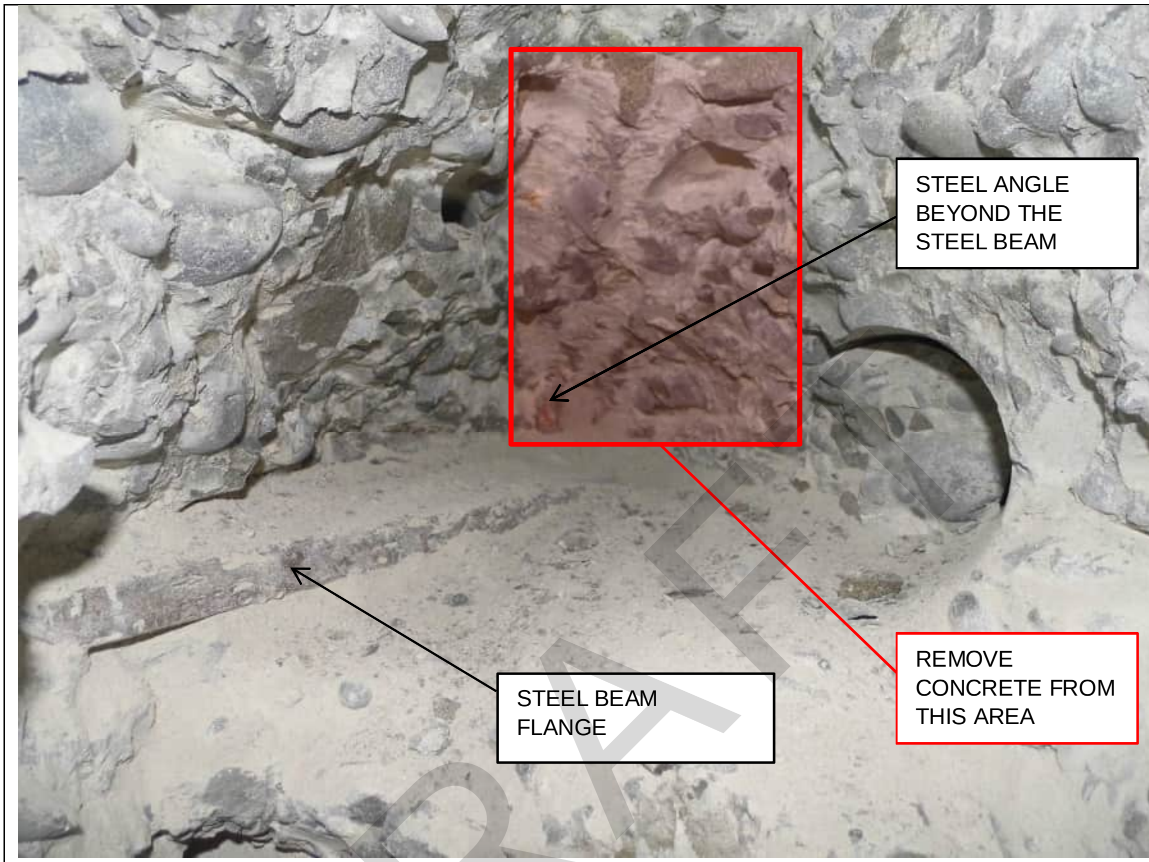

Photo 1 Beam column joint at Level 0, gridline D8

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Photo 2 Beam-column joint at level 2, gridline C6

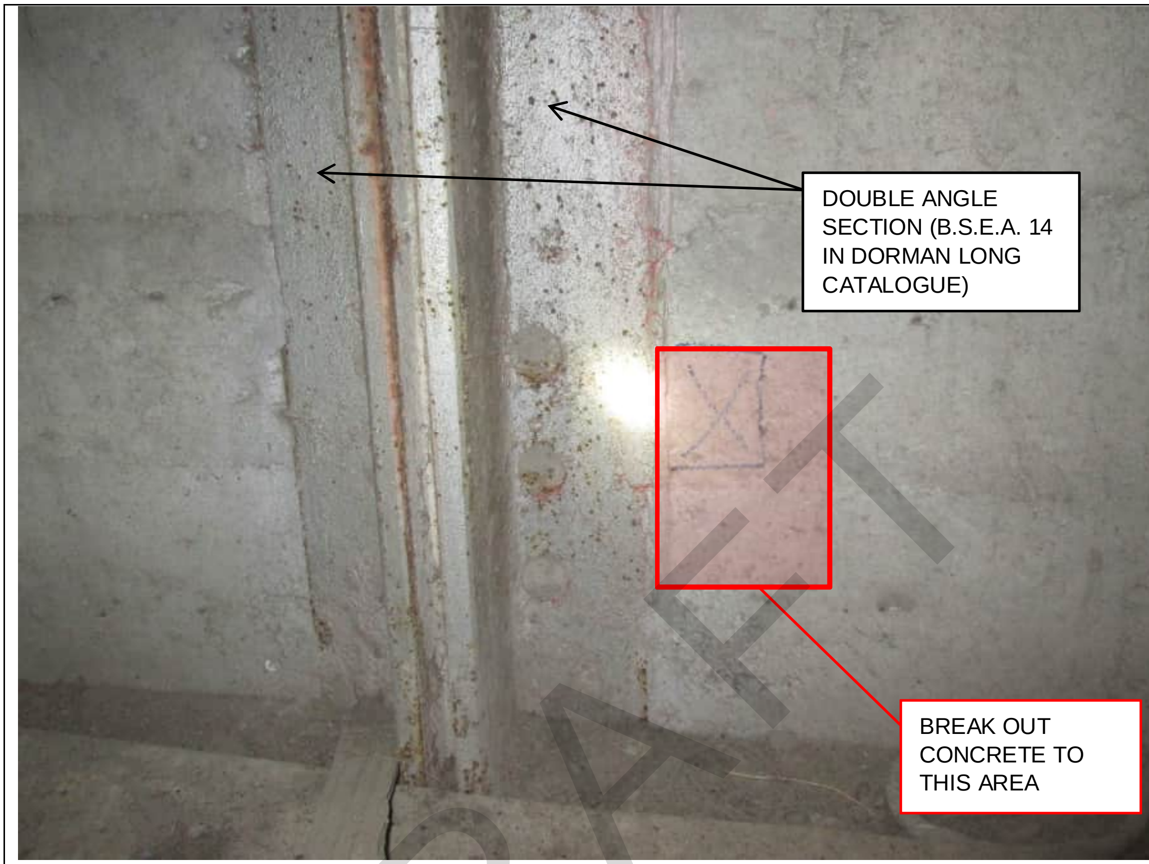

Photo 3 **Level 4 intrusive works on gridline C16**

Figures:

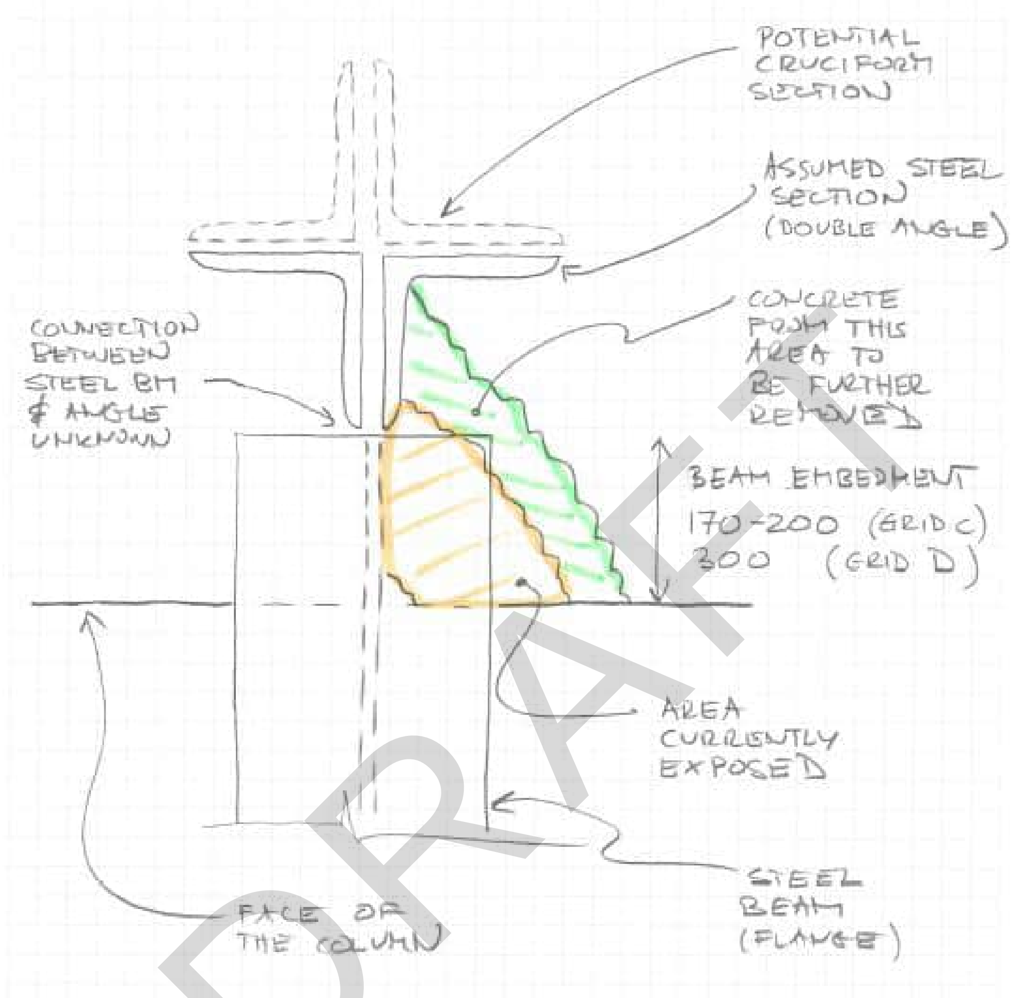

Figure 1 Beam-column joint at level 0, gridline D8 – further concrete removal recommendation

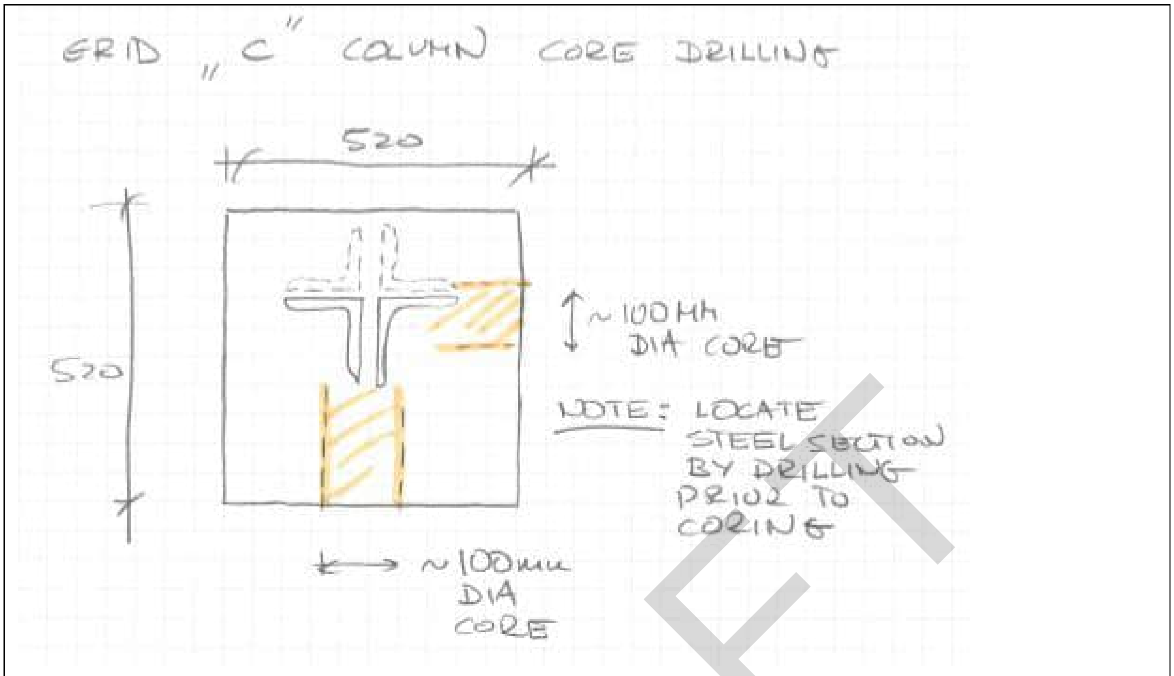

Figure 2 Core drilling to one of the column on gridline C5 to C8, level 0

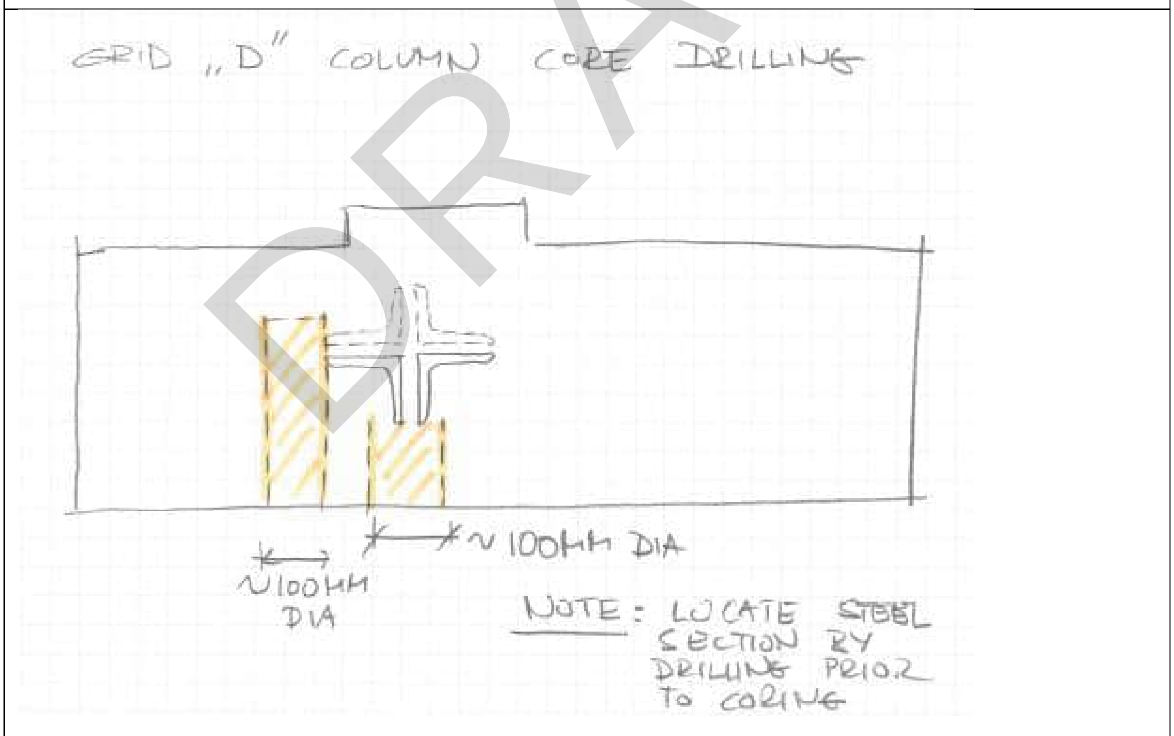

Figure 3 Core drilling to one of the columns on gridline D5 to D8, level 0

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-03
Company	Dominion Constructors Ltd	Date	15-Oct-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Page	3
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Matthew Crake		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	14-Oct-2015
Attendees	Matthew Crake (AECOM) Ian Reynolds (Dominion)		

‘Cc’ Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Andrew McMenamin	AECOM	
Kit Lawrence	AECOM	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☐ Yes ☒ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

Site inspection introduction:

AECOM attended an inspection at Grand National Stand at Riccarton Racecourse on Wednesday 14th October 2015. The inspection covered the following items:

- Level 0 intrusive works to beam column joints
- Level 4 intrusive works

Observations and recommendations:

1. Level 0 (ground floor) beam column joint on gridline D8 observations:
   - Dominion has continued to break out the concrete to expose the vertical steel element. It has been exposed approximately 30mm into the column and approximately 80mm vertically.
   - The flange of the steel beam terminates before the vertical steel element. The web continues past adjacent to the vertical steel element.
   - There appears to be a rivet extending through the vertical steel element and into the web of the beam

   Recommendations:

   - Continue to break out the concrete to expose more of the vertical element into the column. The objective is to determine what the element is (angle or flat) and its dimensions.
   - Break out more vertically to determine the connection between the web and vertical element.
2. Level 4 observations:
   - A hole has been broken out behind the vertical element. The hole extends approximately 100mm and is on an angle.
   - There is no cruciform shape vertical element (i.e. the steel vertical section is a double angle only).
   - It is unknown what the rivets are connecting to on the other side. It is now speculated that it may be the top connection of the upper stand truss.

   Recommendations:

   - At this stage terminate breaking out any more concrete. Further works may be required at later stage.

Further Actions / Inspections:

- Continue with the intrusive works on level 0 and level 2 as recommended above
- AECOM engineer will be on site Friday 16-Oct-2015, unless requested earlier, to inspect the intrusive works and oversee core drilling as per memo, dated 14-Oct-2015.

Kind Regards,

Prepared by:

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Reviewed by:

Richter

Nik Richter
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Photos:

Photo 1 **Level 4 intrusive works on gridline C16**

Photo 2 **Beam column joint at Level 0, gridline D8**

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-04
Company	Dominion Constructors Ltd	Date	19-Oct-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Pages	9
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Matthew Crake		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	16-Oct-2015
Attendees	Matthew Crake (AECOM) Ian Reynolds (Dominion) Vertec Concrete Cutting		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Andrew McMenamin	AECOM	
Kit Lawrence	AECOM	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☐ Yes ☒ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

Site inspection introduction:

AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Friday 16th October 2015. The inspection covered the following items:

- Level 0 (ground floor) intrusive works to beam column joints and columns
- Level 2 intrusive works to beam column joint
- Intrusive works to columns supporting upper stand and roof on grid A

Observations and recommendations:

1. Level 0 beam column joint gridline D8 observations:
   - Dominion has continued to break out the concrete to expose the vertical steel element, see Figure 1 for graphical representation,
   - It is believed that the steel vertical element is a single equal angle,
   - The connection between the web of the steel beam and equal angle is a riveted connection with the rivets offset into the column.

   Recommendations:
   - At this stage no further intrusive works are required.

2. Level 0 column gridline D7 observations:
   - Vertec have core drilled the column in two locations,
   - The core drilling has exposed a single equal angle in the centre of the column,
   - Dyna-drilling into the column has revealed a second piece of steel, see Figure 2 for a graphical representation.

   Recommendations:
   - Drill 1m above and below to confirm the extent of the second piece of steel,
   - Locally break out the concrete to expose the steel element as discussed on site and shown on Photo 3.

3. Level 0 column gridline C7 observations:
   - Vertec have core drilled the column in two locations, both are centrally located one from the south elevation and one from the east elevation,
   - This has exposed double equal angles, see Figure 3 for graphical representation.

   Recommendations:
   - At this stage no further intrusive works are required.

4. Level 2 beam column joint gridline C6 observations:
   - Vertec have core drilled into the beam column joint. Dominion have then locally broken out the remaining concrete to expose the connection between the web of the beam and double angles,
   - A packer plate is located between the web of the steel beam and the double equal angles,
   - Rivets connect the web to the angles and are offset into the column,
   - Dominion have drilled 400mm into the beam adjacent to the connection running along grid C. No steel was encountered, suggesting that there are no steel beams running along grid C.

   Recommendations:

- At this stage no further intrusive works are required.

5. Steel columns observations:
    - Dominion have drilled into the steel columns on the north elevation of the grand stand,
    - A 5mm hole was drilled into the lower and upper columns. Matthew Crake of AECOM was present during the drilling process. 70mm deep holes were drilled into both columns and steel fillings were observed to be existing the holes continuously during the drilling process,
    - A hand held battery drill was used and the hole was drilled with relative ease,

    Recommendations:

    - At this stage no further intrusive works are required.

Further Actions / Inspections:

- Continue with the intrusive works on the level 0 column gridline D7 as recommended above,
- AECOM to inspect the above works once works are completed

Kind Regards,

Prepared by:

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Reviewed by:

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Photos:

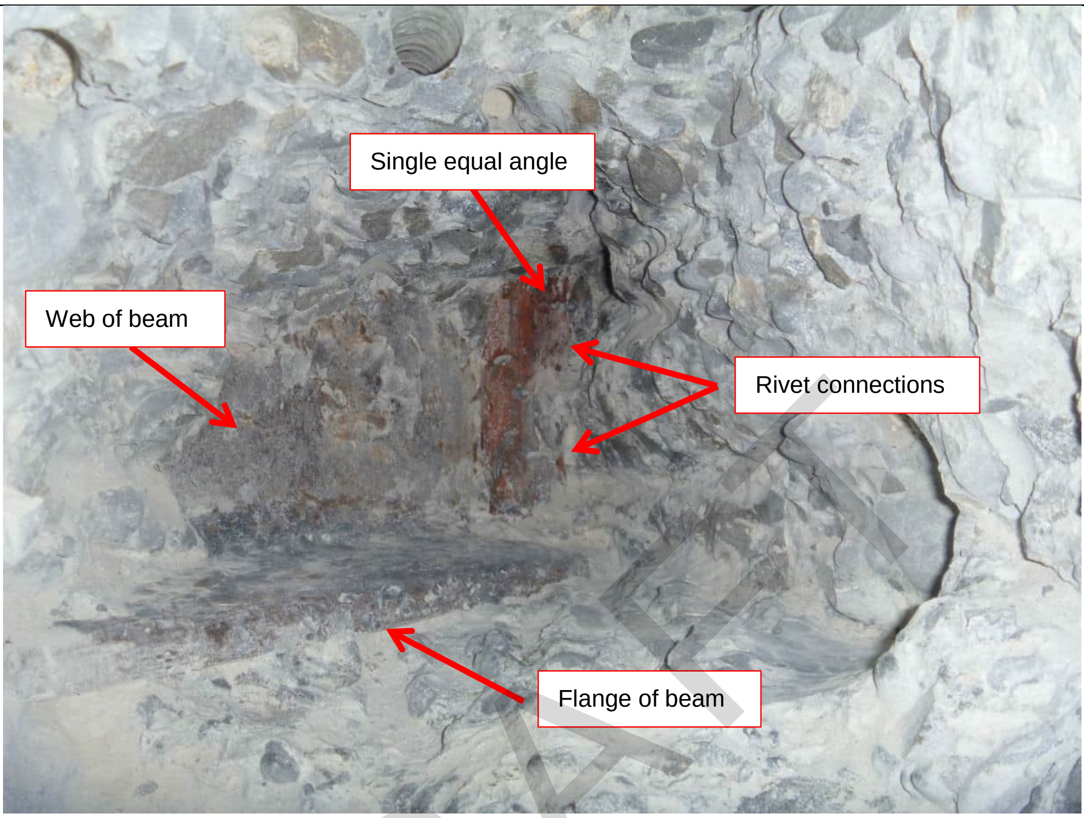

Photo 1 **Level 0 beam column joint gridline D8**

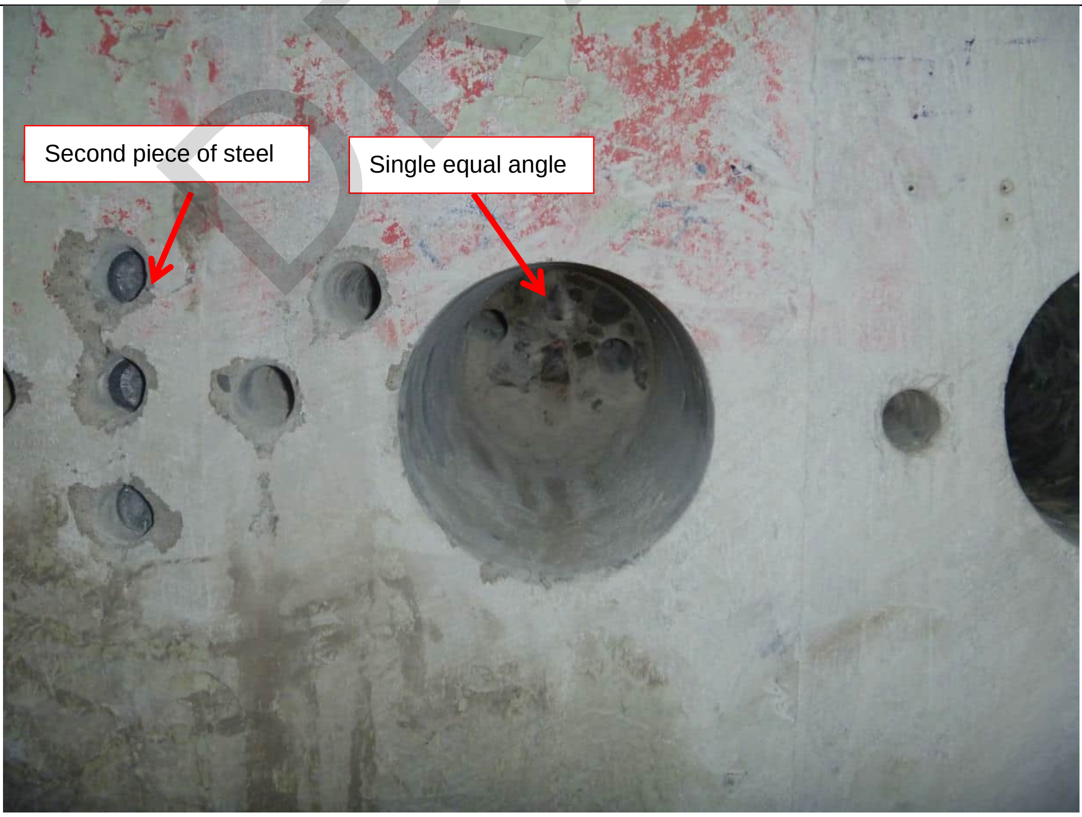

Photo 2 **Level 0 column gridline D7**

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Photo 3 Level 0 column gridline D7

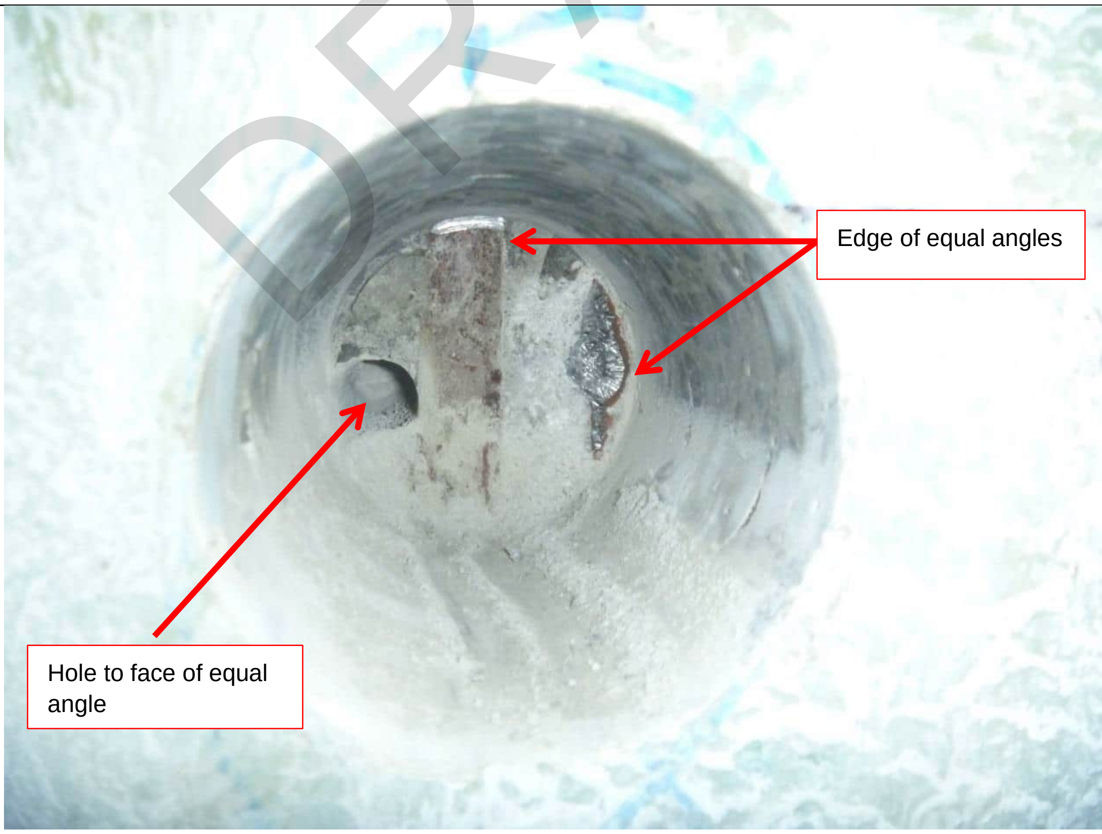

Photo 4 Level 0 column gridline C7 south elevation

Photo 5 **Level 0 column gridline C7 east elevation**

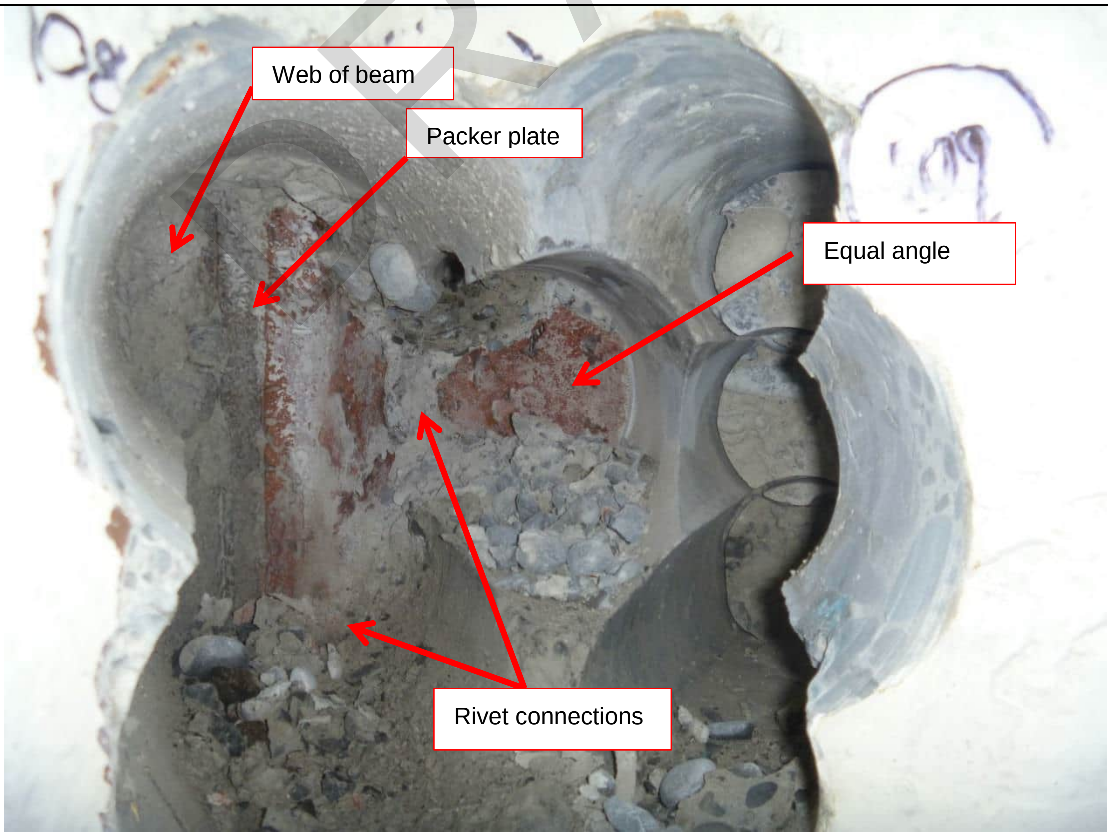

Photo 6 **Level 2 beam column joint gridline C6**

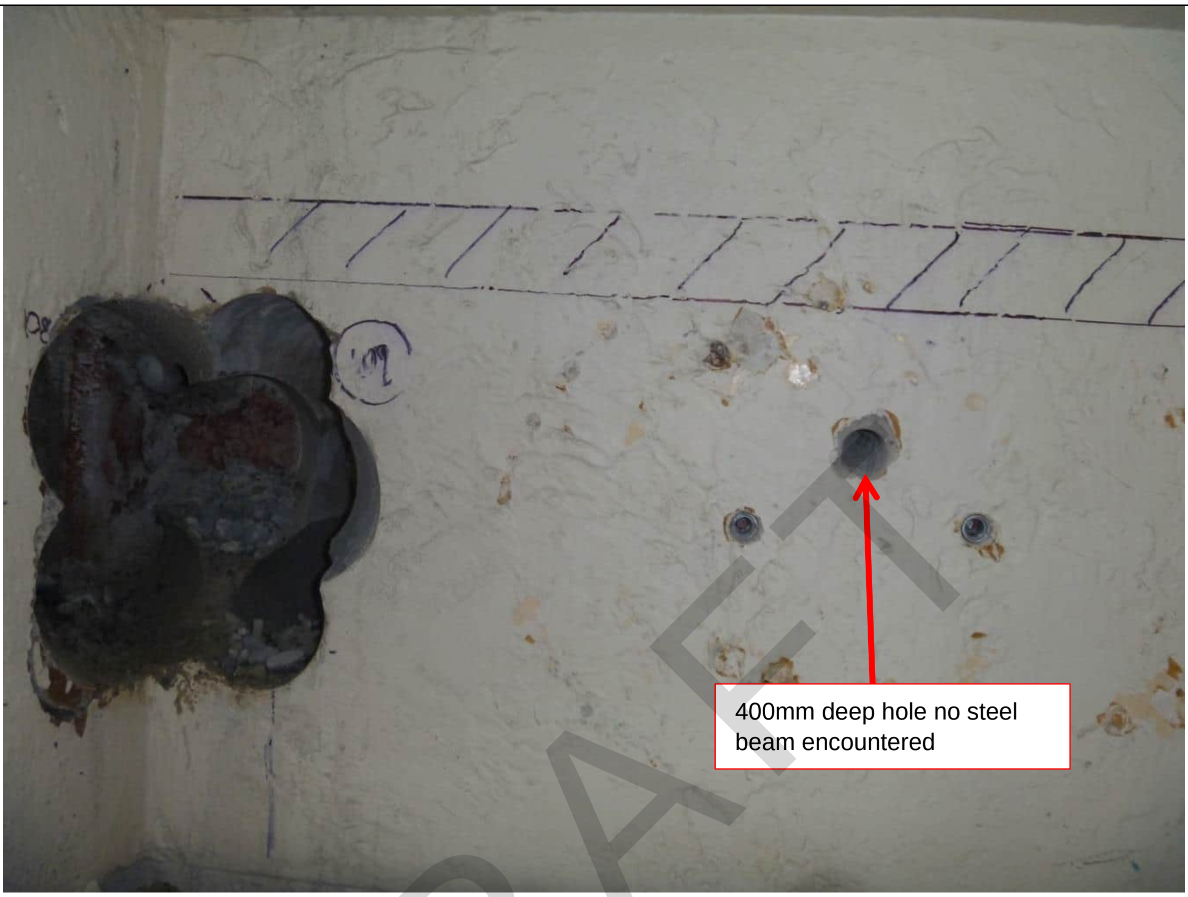

Photo 7 **Level 2 beam column joint gridline C6**

Figures:

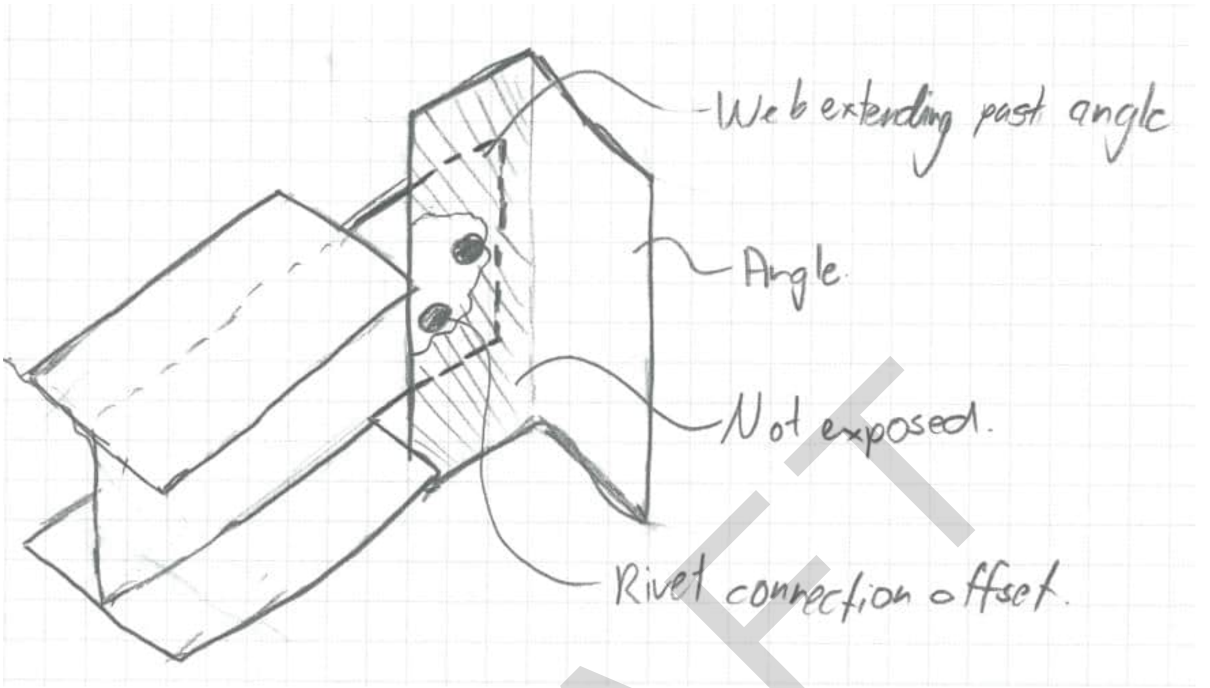

Figure 1 Level 0 gridline D8 beam column joint, showing rivet offset

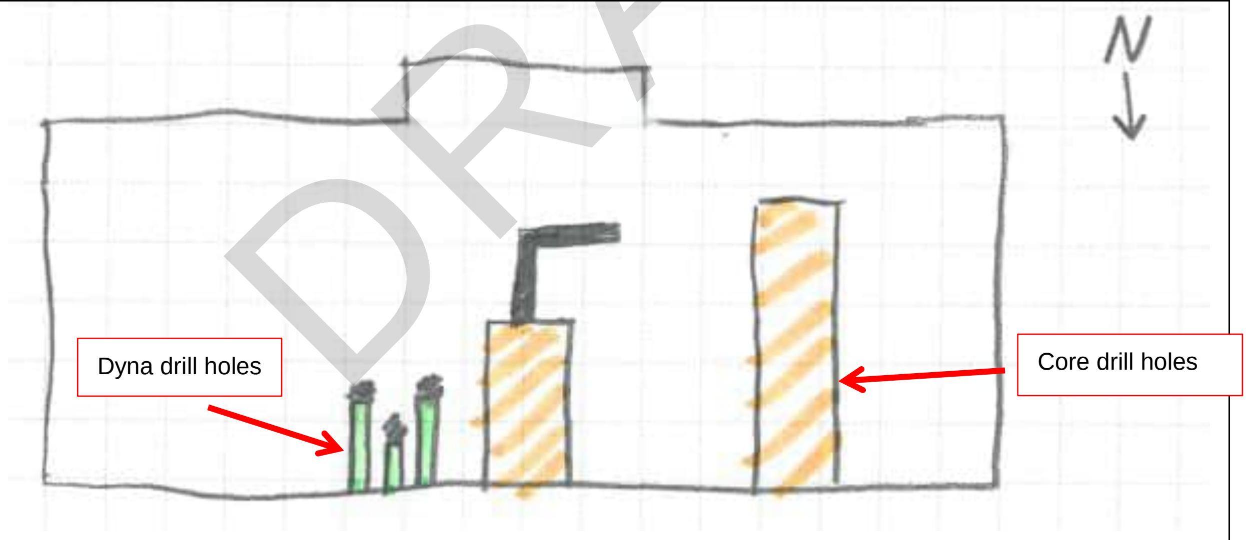

Figure 2 Level 0 gridline D7 column intrusive works

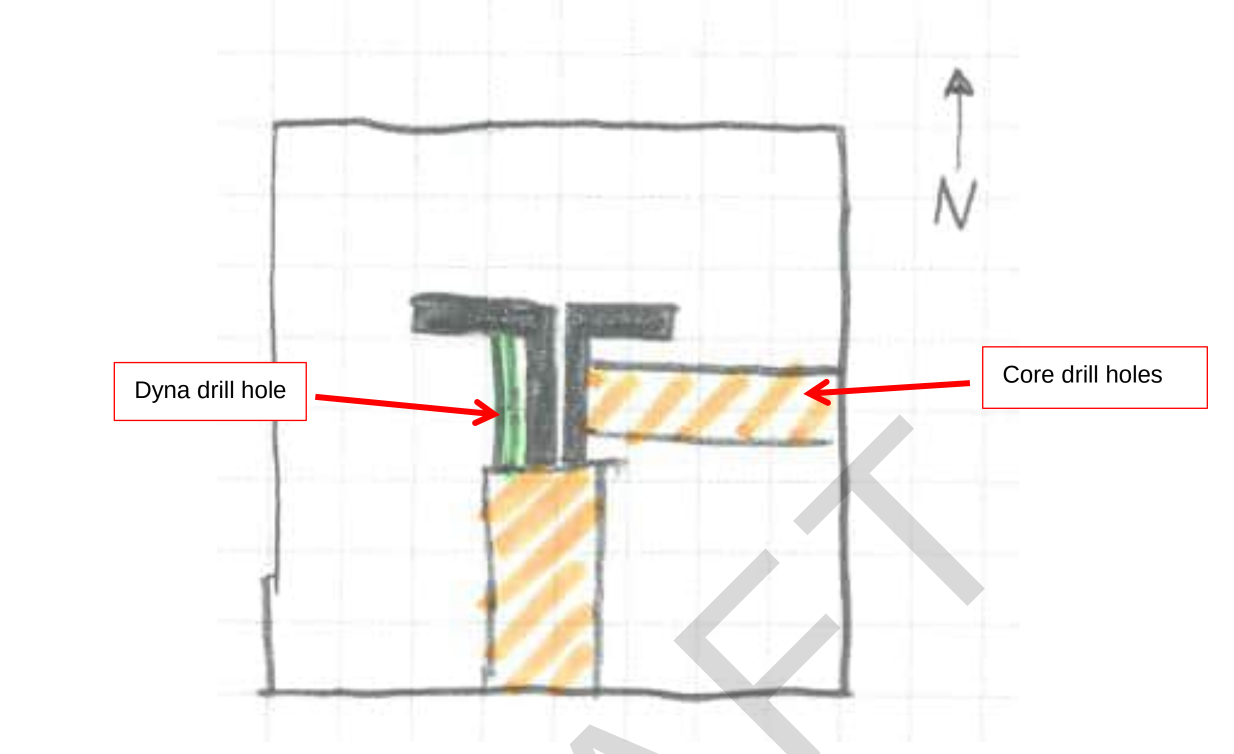

Figure 3 **Level 0 gridline C7 column intrusive works**

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-05
Company	Dominion Constructors Ltd	Date	22-Oct-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Page	10
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Matthew Crake		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	21-Oct-2015
Attendees	Kit Lawrence (AECOM) Mark Ferfolja (AECOM) Matthew Crake (AECOM) Ian Reynolds (Dominion) Craig Stracey (Dominion) Brent Nicholas (Dominion)		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Andrew McMenamin	AECOM	
Kit Lawrence	AECOM	
Ian Reynolds	Dominion	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☐ Yes ☒ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

AECOM

Site inspection introduction:

AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Wednesday 21st October 2015. The inspection covered the following items:

- Access to roof trusses,
- Cherry picker access to north truss of roof,
- Investigation of internal stairs,
- Ground floor gridline D5 column intrusive works,
- Ground floor gridline D7 column intrusive works,
- Ground floor grid B connection of double I beam to concrete columns,
- Measure up of various elements.

Observations and recommendations:

1. Access to roof trusses observations:

    - It was discussed on site gaining access to the roof structure to measure the trusses and braces,
    - It was decided that scaffolding is to be erected on the upper stand to provide access through the bottom of the roof structure.

    Recommendations:

    - The area where the scaffolding is to be erected is marked up on site with crayon and is shown in Photo 1, 2 and 3,
    - Remove the netting in the area to allow unimpeded access.

2. Cherry picker access to north truss of roof observations:

    - It was discussed on site gaining access to the front truss of the roof structure,
    - A cherry picker is going to be used to access the truss.

    Recommendations:

    - Organise a cherry picker that has the capacity to reach the truss.

3. Investigation of internal stairs observations:

    - The internal stairs in the centre of the structure were investigated for their structural form,
    - The level 1 stairs are constructed in timber and are supported on timber framing,
    - The level 2 stairs are constructed out of concrete and are supported by concrete beams,
    - The level 3 stairs are constructed out of concrete and are supported by steel beams,
    - The timber and steel beams were measured on site.

    Recommendations:

    - The concrete beams supporting level 2 are to be investigated with intrusive works. The intrusive works are to occur in two locations as discussed on site with Ian and shown in Photo 4 and 5.

4. Ground floor gridline D5 column intrusive works observations:

    - The column was investigated for possible longitudinal reinforcing in the corners,

- The exposed stirrup extends the full extent of the column, as shown in Photo 6. We are interested if there is longitudinal reinforcement present at the corner of this stirrup.

Recommendations:

- An area has been marked up on site with red crayon that is to be broken out to a depth of 150mm, this is shown in Photo 6.

5. Ground floor gridline D7 column intrusive works observations:

- The breaking out of concrete as requested in previous memo dated 19 October 2015 has been completed,
- This shows a 36mm round steel bar running vertically in the column,
- The further steel encountered by dyna-drilling, as shown in Photo 7, has not been exposed.

Recommendations:

- No further intrusive works are required at this stage.

6. Ground floor grid B connection of double I beam to concrete columns observations:

- The two I beams are only connected together at their top flange by cleats that attach the vertical steel angles,
- There is no positive connection between the I beams and the concrete columns. They are simply bearing on top of the columns.

Recommendations:

- No further actions are required at this stage.

7. Measure up of various elements observations:

- The double steel angles embedded in the shear wall on level 4 are 150x150 equal angles with a thickness of 20mm,
- The steel angle extending up from the slab on level 4 grid B is a 150x150 equal angle with a thickness of 20mm,
- The lower stand ramps are supported at their base on the double steel I beams and at the top on an I beam with approximate dimensions of, depth 210mm and flange width of 140mm,
- The upper stand stairs have five 225x70mm timber stringers which are connected to timber supports at each end by an assumed nailed connection. At two points along the stringers length they are supported by 150x150mm timber beams connected to a 150x150mm equal angle bolted to the upper stand truss, as shown in Photo 9,
- The dimensions of the columns supporting the east and west elevation stairs and porch area on the ground level and level 1 are shown in Figure 1,
- The beam supporting the top of the stairs extends 460mm below the slab and is 210mm in width,
- The beams on the level 1 porch are type 4 beams and the spandrels are to be modelled as type 1a beams,
- On the ground level the vertical steel angles supporting the inclined steel I beams on grid B are 105x105mm equal angles of 15mm thickness,
- The lower stand bleachers are supported on approximately 300x100mm timbers at 900mm centres, spanning between gridlines,
- The steel columns on grid A have heights of 6.05m on the lower stand and 6.09m on the upper stand. On the lower stand the columns appear to be embedded into the concrete column beneath.

Further Actions / Inspections:

- Installation of scaffolding to access roof structure,
- Organisation of a cherry picker to access front truss of roof structure,
- Intrusive works to concrete beams supporting stairs,
- Intrusive works to concrete column on the ground level gridline D5.

Kind Regards,

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Photos:

Photo 1 **Location of scaffolding**

Photo 2 **Location of scaffolding**

Photo 3 Location of scaffolding

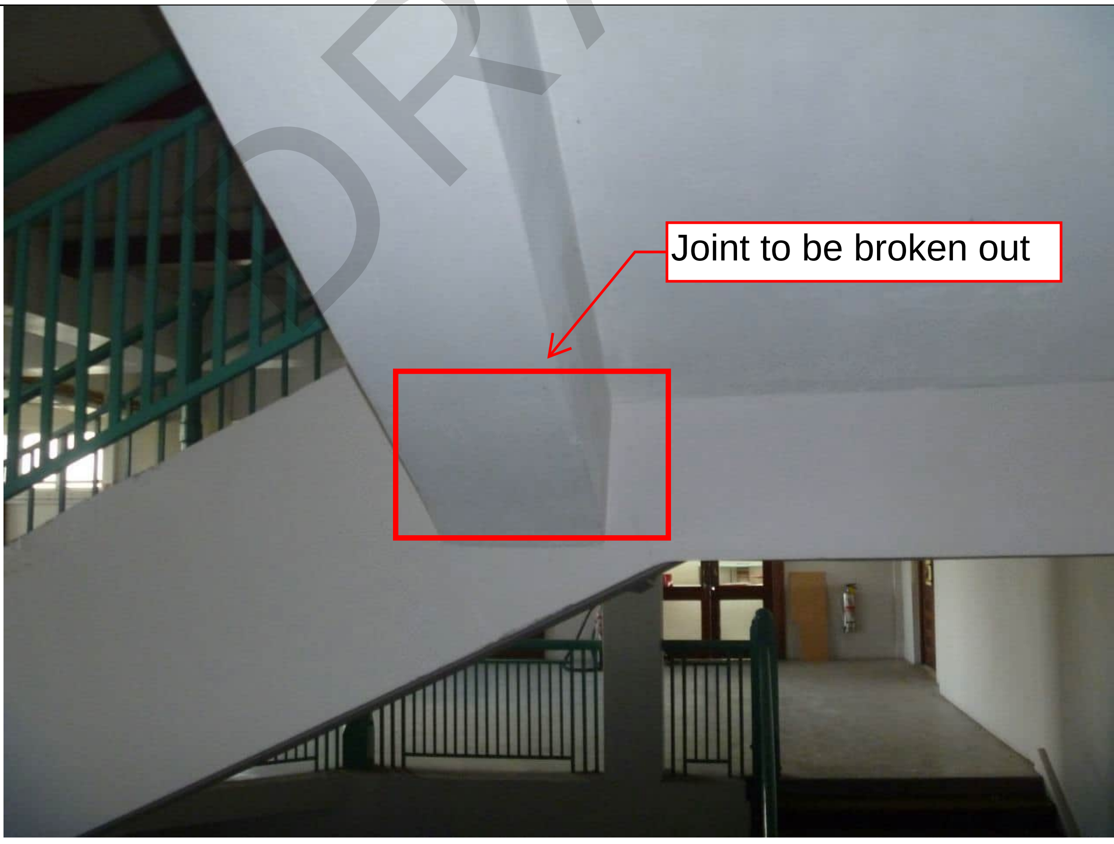

Photo 4 Location of intrusive works to the stairs

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Photo 5 **Location of second intrusive works to the stairs**

Photo 6 **Location of intrusive works to ground floor column on grid line D5**

Photo 7 **Location of intrusive works ground floor gridline D7**

Photo 8 **Connection between double I beam and concrete column**

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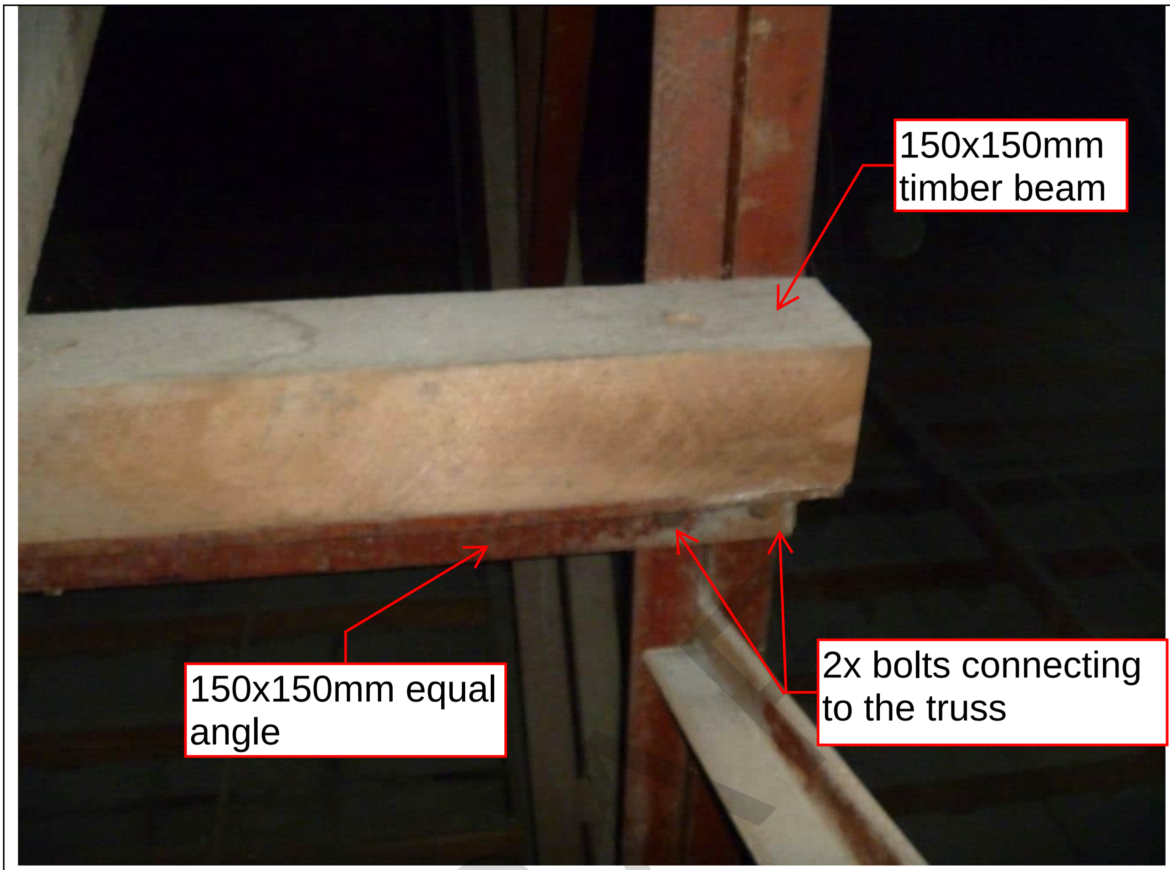

Photo 9 **Upper stand stair support**

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Figures:

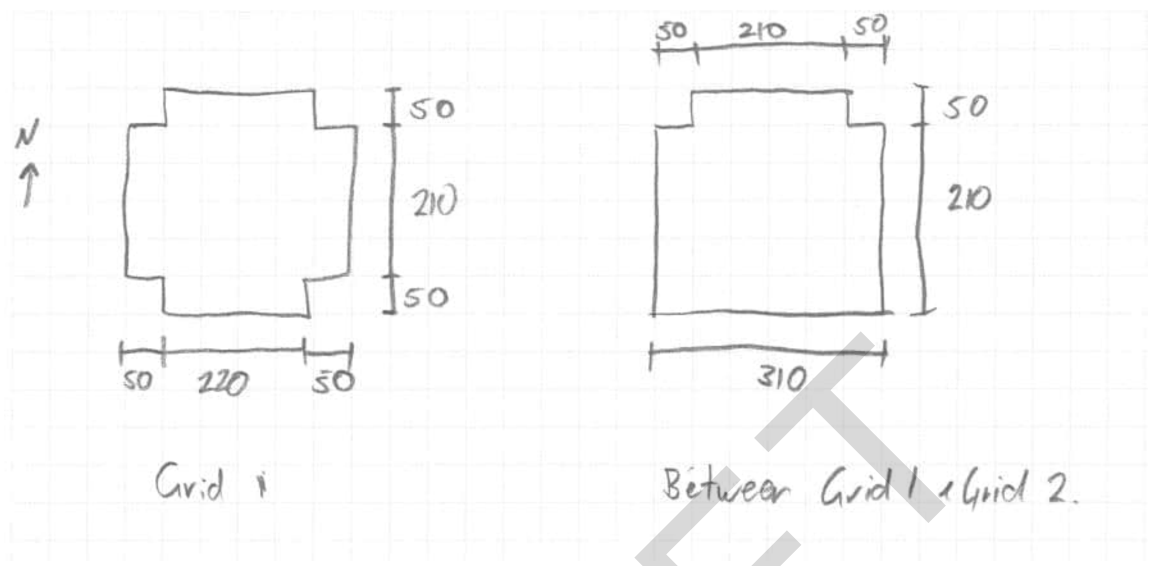

Figure 1 East and west elevation stair column supports

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-05
Company	Dominion Constructors Ltd	Date	19-Oct-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Pages	9
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Kit Lawrence		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Grid A and B Foundation Inspection	Inspection Date	23-Oct-2015
Attendees	Kit Lawrence (AECOM) Ian Reynolds (Dominion)		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Andrew McMenamin	AECOM	
Matthew Crake	AECOM	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☐ Yes ☒ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

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Site inspection introduction:

AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Friday 23 October 2015. The inspection covered the following items:

- Level 0 (ground floor) intrusive works to expose foundations on grid A-4 and B-4
- Inspection of the double steel angel on level 4, grid C1-21

Observations and recommendations:

1. Level 0 intrusive works to expose foundations on grid A-4:
   - Dominion has broken out the concrete slab adjacent to the strip foundation on the internal side of the ground floor, exposing a length of the strip foundation down to the base, see Figure 1 for graphical representation.
   - The foundation consists of a concrete strip founded at 0.7 m below the base of the slab in natural silt material. The foundation has been boxed for the top 0.2 m with the remaining 0.5 m being poured against the excavation face.
   - A single hand auger with an adjacent Dynamic Cone Penetrometer (DCP) test and shear vane tests in cohesive materials was undertaken in the excavation. The log of this investigation is attached to this memo.

   Recommendations:

   - At this stage no further intrusive works are required.
2. Level 0 intrusive works to expose foundations on grid B-4:
   - Dominion has broken out the concrete slab adjacent to the pad foundation on one side and excavated alongside the pad to expose the approximate base and two edges see Figure 2 for graphical representation.
   - The foundation consists of a concrete pad foundation 1.6 m by 1.6 m wide founded at 1.0 m below the base of the slab. The foundation has been boxed for the top 0.2 m with the remaining 0.8 m being poured against the excavation face forming a curve that extends 0.4 m outside the boxed edge. This pad is founded in natural silt material.
   - A single hand auger with an adjacent Dynamic Cone Penetrometer (DCP) test and shear vane tests in cohesive materials was undertaken in the excavation. The log of this investigation is attached to this memo.

   Recommendations:

   - At this stage no further intrusive works are required.
3. Level 4 double steel angle on column gridline C1-21 was measure as 60 mm by 60 mm with a thickness of 12 mm.

DRAFT

Kind Regards,

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Photos:

Photo 1 **Level 0 beam column joint gridline D8**

Photo 2 **Level 0 column gridline D7**

Figures:

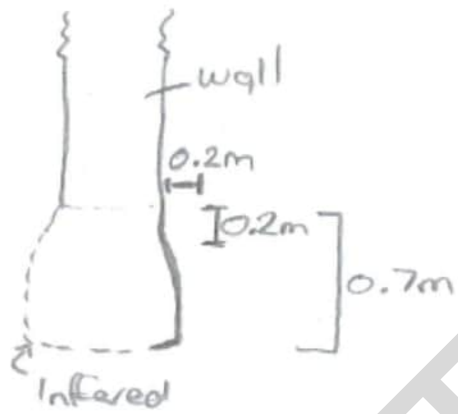

Figure 1 **Level 0 gridline A4 foundation diagram**

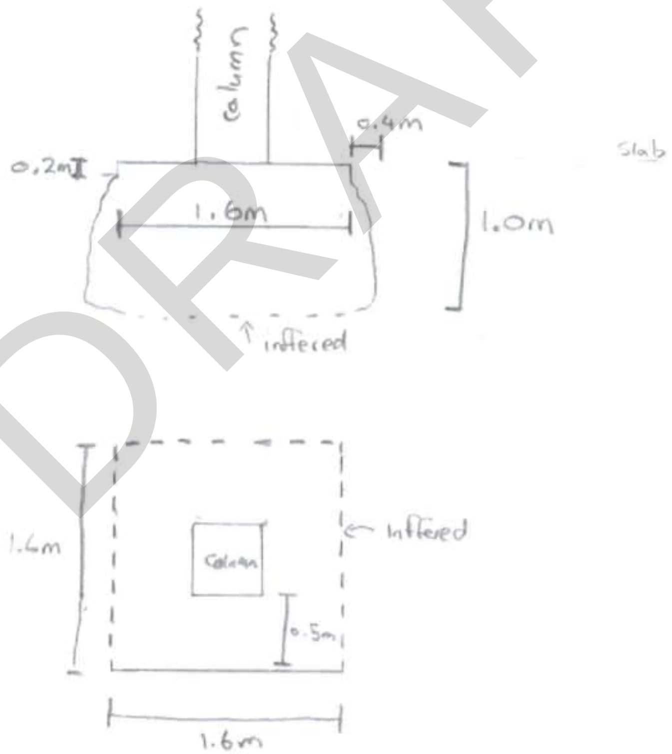

Figure 2 **Level 0 gridline B4 foundation diagram**

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-05
Company	Dominion Constructors Ltd	Date	30-Oct-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Page	5
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Kit Lawrence		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	29-Oct-2015
Attendees	Kit Lawrence (AECOM) Ian Reynolds (Dominion)		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
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Nik Richter	AECOM	
Andrew McMenamin	AECOM	
Kit Lawrence	AECOM	
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Alberto Cuevas	Thornton Tomasetti	

Attachments ☐ Yes ☒ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

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Site inspection introduction:

AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Thursday 29 October 2015. The inspection covered the following items:

- Inspection of intrusive investigation on level 0 gridline D-5
- Measure up of various elements on level 0.

Observations and recommendations:

1. Ground floor gridline D-5 intrusive works observations adjacent to column:
   - The concrete wall between the column and the doorway was investigated for possible vertical reinforcing which the longitudinal bars links to.
   - A single, vertical 19 mm diameter bar of reinforcement was found 40 mm from the doorway. The horizontal bar looped around this vertical bar.

   Recommendations:

   - No further actions are required at this stage.
2. Measure up of various elements observations:
   - Steel plate splice connection on the BSB25 beams (Type 9) supporting bleachers where inspected. The plates are 610 mm long, 160 mm wide and 20 mm thick. Plates are riveted to the beams with 12 steel rivets with 35 mm diameter heads an unfilled drill hole adjacent to the rivets indicate the rivets are 19 mm in diameter.
   - Timber beams supporting the bleachers are 300 mm by 80 mm with up to 80 mm notches.
   - Concrete slab tray deck is supported on a 160 mm tall concrete strip cast between two steel plates which are welded to the top flange of the BSB21 (type 8) beams between gridline B and C.

Further Actions / Inspections:

- No further actions are required at this stage.

Kind Regards,

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Photos:

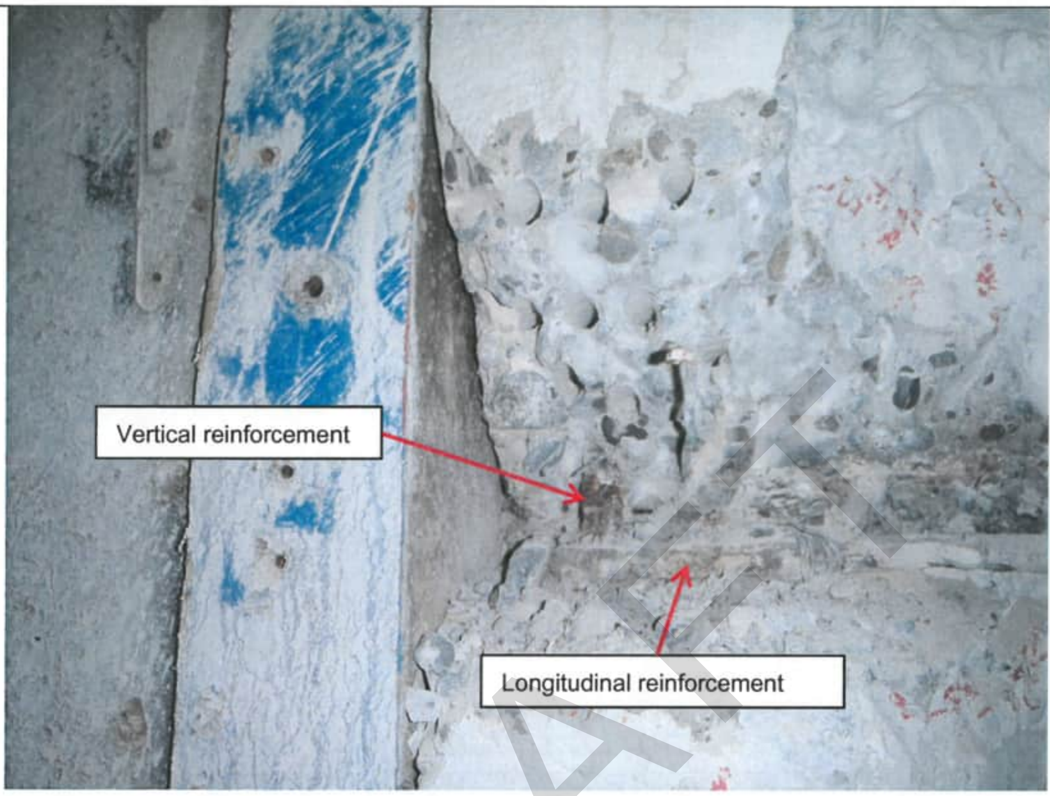

Photo 1 **Vertical reinforcement adjacent to door with longitudinal bar coming in**

Photo 2 **Steel splice plate on beam supporting bleachers**

Photo 3 Steel splice plate on beam supporting bleachers, showing rivets

Photo 4 Tray slab supported on beams between gridline B and C

Photo 5 Concrete strip with steel plates supporting tray slab above beam.

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-06
Company	Dominion Constructors Ltd	Date	30-Nov-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Page	8
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Matthew Crake		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	26 and 27-Nov-2015
Attendees	Ian Reynolds (Dominion Matthew Crake (AECOM) Kit Lawrence (AECOM)		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Andrew McMenamin	AECOM	
Steve Penny	AECOM	
Kit Lawrence	AECOM	
Ian Reynolds	Dominion	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☒ Yes ☐ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

Site inspection introduction:

At the request of Dominion Constructors Ltd, AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Thursday 26 and Friday 27 November 2015. The inspections covered the following items:

- Internal stairs intrusive works,
- Level 2 spandrel beam intrusive works,
- Lower stand plate girder top and bottom plate and splice connection,
- Ground floor grid 12 internal wall running from grid A to B.

Observations and recommendations:

1. Internal stairs intrusive investigation observations:
   - Two cores have been drilled into the beam supporting the level 3 floor and six holes drilled approximately 220 mm at varying heights up the height of the beam, see photo 1.
   - The six holes have been drilled approximately 20m m past the centre line of the beam, further intrusive works would be required to confidently determine the structural form of the beam.
   - The lower core as shown in photo 1 had a stirrup running vertically through the centre of the core. The cover to the stirrup is approximately 10 mm and is shown in photo 2. The bottom of the core, as shown in photo 3, appears to have cut through the bend in the stirrup and there doesn't appear to be a longitudinal bar seated in the bend.

   Recommendations:
   - No further action is required at this location at this stage. This may be reviewed depending on further findings at a later date.
   - Intrusive works are to continue at the lower end connection as shown in memo dated 22 Oct 2015, photo 4. Approximately six holes are to be drilled at different faces as discussed on site.

2. Level 2 spandrel beam intrusive works observations:
   - A vertical strip to the left of the column has been broken out to a depth of 110 mm, as shown in photo 4. No stirrups of longitudinal bars were observed in the strip.
   - An inverted T shape at the beam column connection has been broken out to a depth of 110 – 130 mm , as shown in photo 5. No stirrups or longitudinal bars were observed.

   Recommendations:
   - No further intrusive works are required at this location.

3. Lower stand plate girder top and bottom plate and splice connection observations:
   - The top and bottom plate were observed at the column connections, as shown in photo 6.
   - The top and bottom plate was terminated adjacent to the column and was absent for 2.5 m either side of the column, as shown in photo 7.
   - The girder is spliced together at approximately 12 m centres by a 1.25 m high, 0.3 m wide, 20 mm thick plate of steel riveted to the two pieces of girder. 30 Rivets are used in the splice, as shown in Photo 8.

   Recommendations:
   - A hole is to be drilled through the web of the plate girder to determine its thickness.

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4. Ground floor grid 12 internal wall running from grid A to B:
    - The reinforced concrete wall was not visible from inside the 'mattress room' due to the ceiling and walls being lined. The top of the wall was partially visible from approximately grid C looking north.
    - The wall has a height approximately the same as the top of the double I beams running along grid B. It remains at this height until reaching the angled I beam where it follows a similar height to the I beam.
    - The angled I beam and wall are on different vertical planes with the wall situated to the west of the beam.

    Recommendations:
    - No further intrusive works are required at this location.

Further Actions / Inspections:

- Intrusive investigation of the lower end connection of the internal stairs.
- Drill a hole through the web of the lower stand plate girder.

Kind Regards,

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Photos:

Photo 1	Internal stair core and hole locations

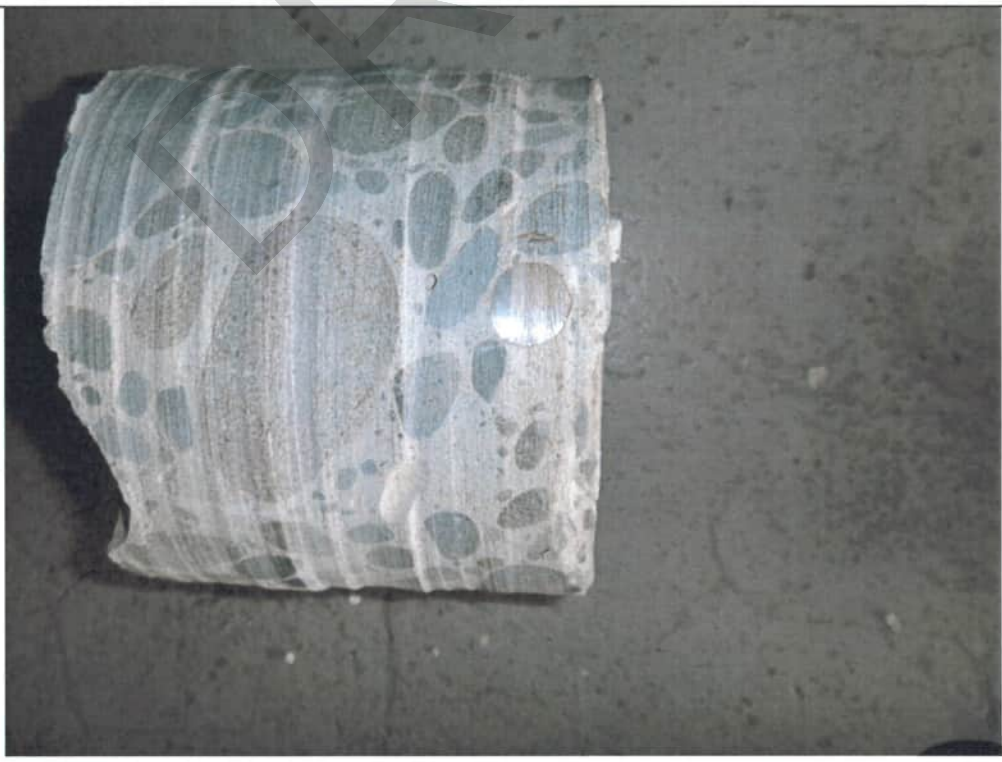

Photo 2	Stirrup in core approximately 10mm cover

Photo 3 Stirrup in core base

Photo 4 Broken out strip on spandrel beam, approximately 110mm deep

Photo 5 Broken out inverted T on beam column connection approximately 110 – 130mm deep

Photo 6 Top plate at column connection, terminating adjacent to column

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Photo 7	**Top plate beginning again approximately 2.5m from column**

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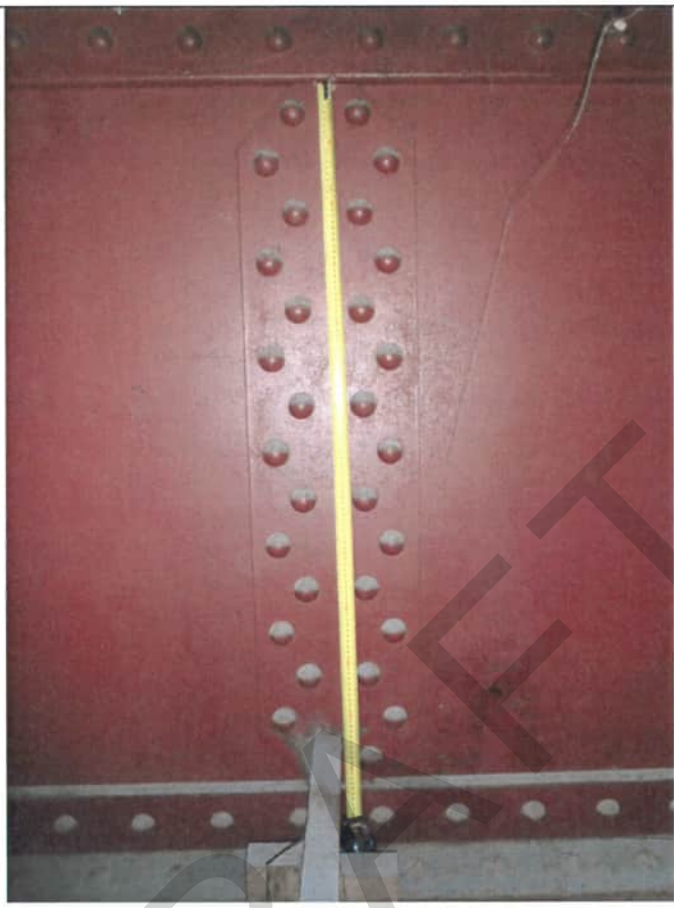

Photo 8	Plate girder splice connection

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-07
Company	Dominion Constructors Ltd	Date	2-Dec-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Page	11
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From	Matthew Crake		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	02-Dec-2015
Attendees	Craig Stracey (Dominion) Ian Reynolds (Dominion) Kyle (Thornton Tomasetti) Stevenson and Turner Representative Carl Burnett (City Care) Matthew Crake (AECOM)		

'Cc' Distribution Details

Attention	Organisation	Fax No.
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Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Andrew McMenamin	AECOM	
Steve Penny	AECOM	
Kit Lawrence	AECOM	
Ian Reynolds	Dominion	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☒ Yes ☐ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

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Site inspection introduction:

At the request of Dominion Constructors Ltd, AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Wednesday 2nd December 2015. The inspection covered the following items:

- Items accessible from scaffold on top stand bleachers:
  - Girder truss Grid 2,
  - Roof trusses,
  - Brace and brace connection.
- Girder truss Grid A from knuckle boom,
- Plate girder web thickness,
- Timber purlins supporting lower stand,
- Intrusive investigations to internal stairs.

Observations and recommendations:

1. Girder truss on Grid 2:

- See Figure 1 for dimensions,
- Rivets on angles connection top plate to partial web plate are at 100mm centres.

Recommendations:

- No further actions are required.

2. Roof trusses:

- Measurements were taken of the element sizes and locations of the following trusses:
  - Grid 3 truss (from Grid C to approximately Grid B),
  - Raked truss (from Grid 3 to Grid C),
  - Side truss (from Grid 3 to Grid 2).
- The layout and location of element types was recorded for Grid 4 truss.

Recommendations:

- No further actions are required.

3. Brace and brace connection:

- The brace was measured on Grid 3 to be 44mm in diameter,
- The connection of the brace to the top plate of the girder truss located at the intersection of Grid C and Grid 3 is through and I section with riveted angles to stiffen the I section, see Photo 1 and 2

Recommendations:

- No further actions are required.

4. Girder truss on Grid A from knuckle boom:

   - The knuckle boom was set up on the grass to the north of the stand and extended up to the Grid A girder truss approximately at Grid 5,
   - The knuckle boom was not able to completely reach the truss and measurements were taken from approximately 1m away,
   - See Figure 2 for dimensions.

   Recommendations:

   - No further actions are required.

5. Plate girder web thickness:

   - The plate girder located on Gird A in the roof of the lower stand had a hole drilled through the web adjacent to Grid 8,
   - The thickness of the plate girder web was found to be 27mm, see Photo 3.

   Recommendations:

   - No further actions are required.

6. Timber purlins supporting lower stand bleachers:

   - The timber purlins supporting the lower stand bleachers were accessed from a mobile work platform approximately at the intersection of Grid B and Grid 6,
   - Two types of purlins were identified, single and double members. The single members are 310x75mm with an approximately but varying notch of 80mm and the double members are 250x110mm with an approximately but varying notch of 20mm. See Photo 4 and 5.

   Recommendations:

   - No further actions are required.

7. Intrusive investigation to internal stairs:

   - Six 8mm holes have been drilled into the lower end connection, hitting steel at all locations, see Photo 6,
   - The cover to the assumed steel beam has been marked on the side of the beam with pencil and is approximately 80mm, see Photo 7,
   - Two 8mm hole were drilled into the upper connection and again no steel was found, see Photo 8.

   Recommendations:

   - See attached mark up for locations of further intrusive works, these include the following items:
     - Drill top and bottom of the stringer beam to determine if a steel beam is present and the depth of the beam, see Photo 9 for location,
     - Break out joint to determine how crank has been constructed and how other steel beam is connected, see photo 10 for location,
     - Drill bottom of middle stringer to see if steel beam is present, see photo 11 for location,
     - Drill bottom of top landing beam to see if steel beam is present, see photo 12 for location.

Further Actions / Inspections:

- Further intrusive investigations of the internal stairs, see attached mark up for locations.

Kind Regards,

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Reviewed by:

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Photos:

Photo 1 Roof brace connection

Photo 2 Roof brace connection

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Photo 3 **Hole in web of plate girder**

Photo 4 **Single member timber purlin**

Photo 5 Double member timber purlin

Photo 6 Six 8mm diameter holes in lower beam connection of internal stairs

Photo 7 Pencil line showing cover to assumed steel beam

Photo 8 Two more 8mm diameter holes drilled into top connection of internal stairs

Photo 9	**Location of intrusive works, drill top and bottom of stringer**

Photo 10	**Location of intrusive works break out joint**

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Photo 11 Location of intrusive works drill bottom of stringer

Photo 12 Location of intrusive works, drill bottom of landing

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Figures:

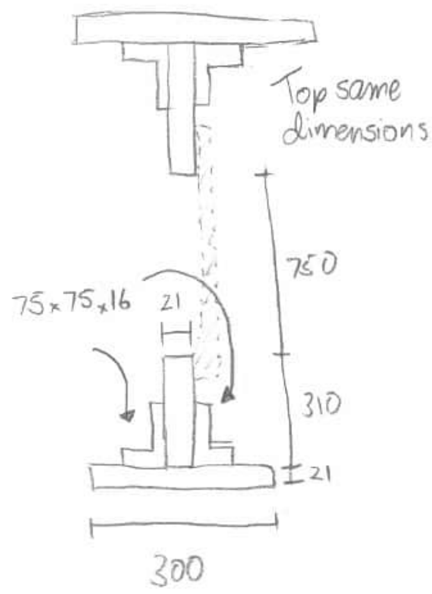

Figure 1 Dimensions of girder truss Grid 2

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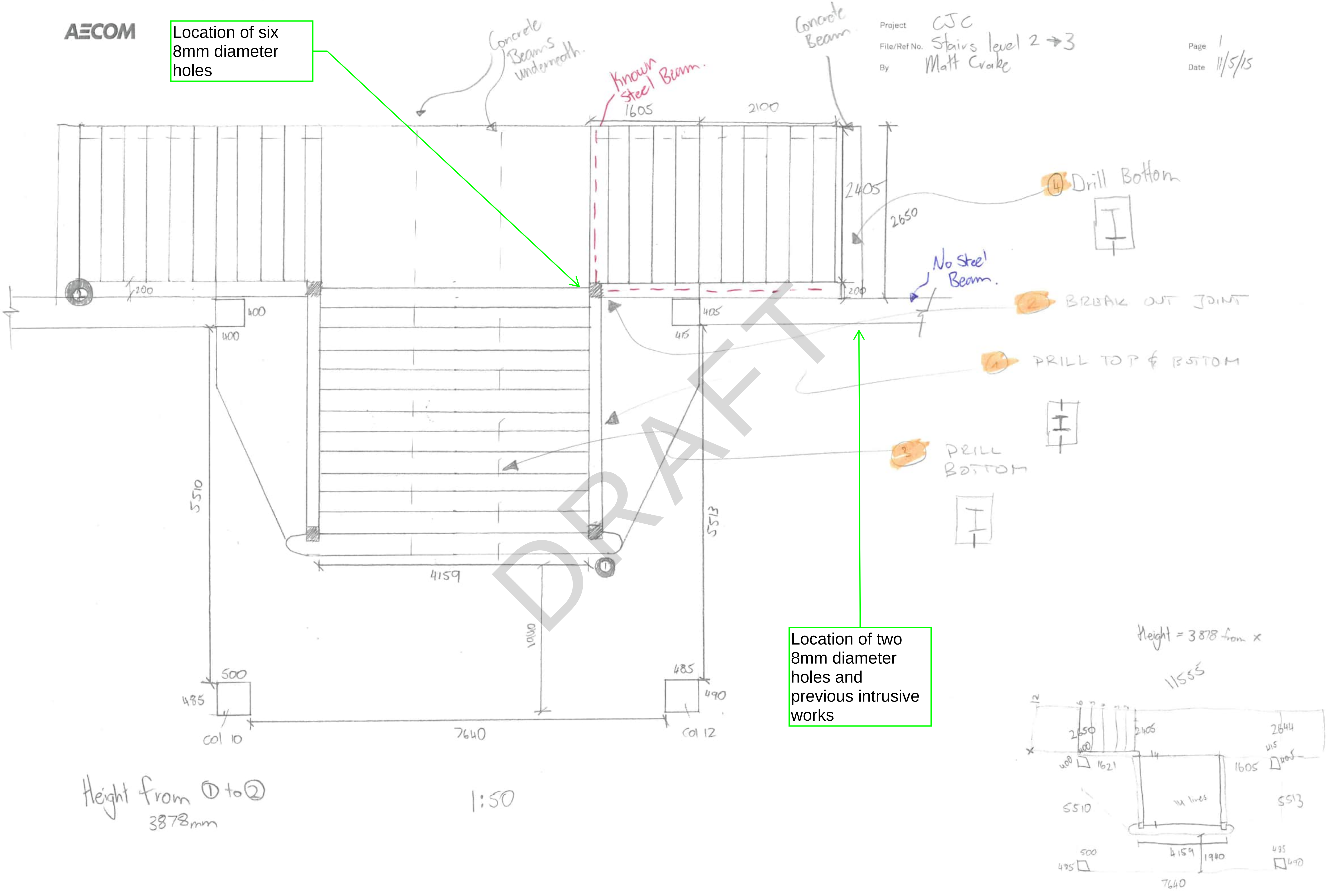
Project: CJC
File/Ref No.: Stairs level 2 → 3
By: Matt Crake
Page: 1
Date: 11/5/15



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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-08
Company	Dominion Constructors Ltd	Date	4-Dec-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	TotalPage	5
ProjectName	CanterburyJockeyClub,GrandNationalStand	ProjectNo.	60439900
From	Matthew Crake		
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	03-Dec-2015
Attendees	Matthew Crake (AECOM) Ian Reynolds (Dominion) Representatives from Concut Representatives from Dominion		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Kit Lawrence	AECOM	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☒ Yes ☐ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

AECOM

Site inspection introduction:

At the request of Dominion Constructors Ltd, AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Thursday 3rd October 2015. The inspection covered the following items:

- Intrusive works to internal stairs beam connection

Observations and recommendations:

1. Intrusive works to internal stairs beam connection:
   - The beam connection located on the ceiling of the level 2 to 3 stairs has been broken out exposing the connection of the cranked beams and adjacent stringer beam, see photo 1.
   - The cranked beams are butted against each other and connected by a web plate riveted to each beam, see photo 2.
   - The adjacent stringer beam has had its bottom flange and part of its web notched to sit on top of the cranked beam, see photo 3.
   - The cranked beams and adjacent stringer beam are assumed to be BSB13 sections based on approximate measurements taken on site consisting of height 200mm, single leg of flange 50mm less web plate and flange thickness of 10mm.

   Recommendations:

   - Carry out works as per previous memo dated 2nd December.

Further Actions / Inspections:

- Carry out works as per previous memo dated 2nd December.

Kind Regards,

Prepared by:

Matthew Crake
Graduate Structural Engineer
e: matthew.crake@aecom.com
d: +64 3 966 6027

Reviewed by:

Richter

Nik Richter
Senior Structural Engineer
e: nik.richter@aecom.com
d: +64 3 966 6016

Photos:

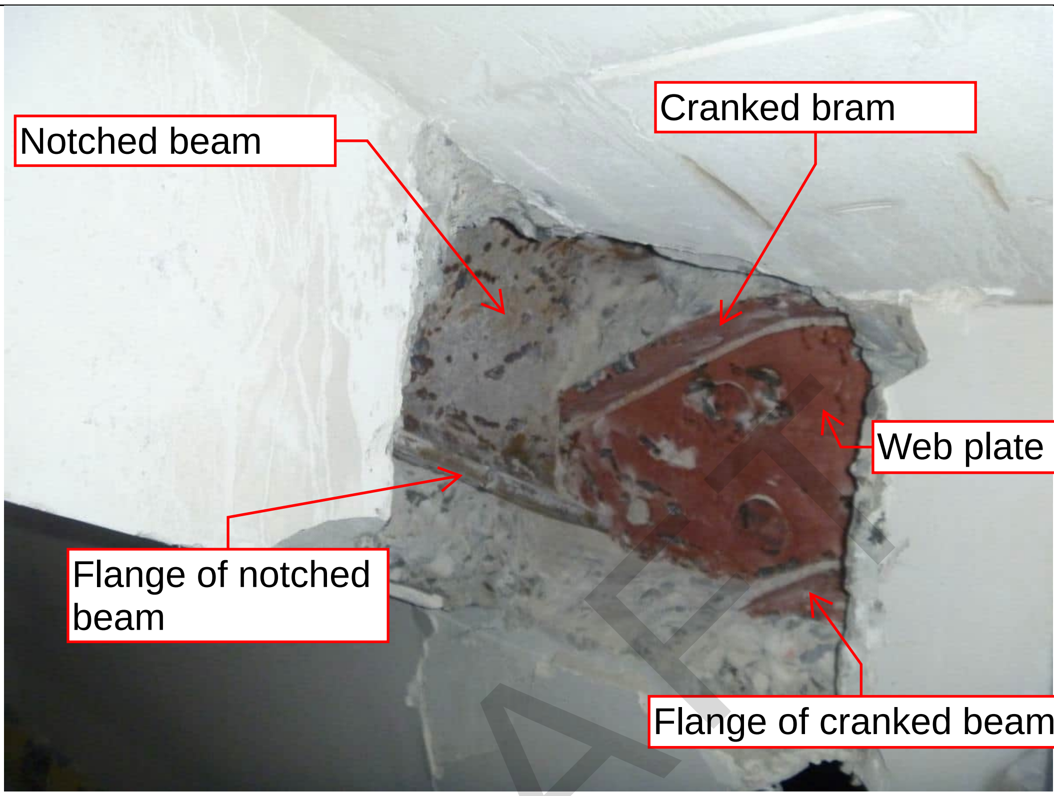


Photo 1 **Exposed connection**

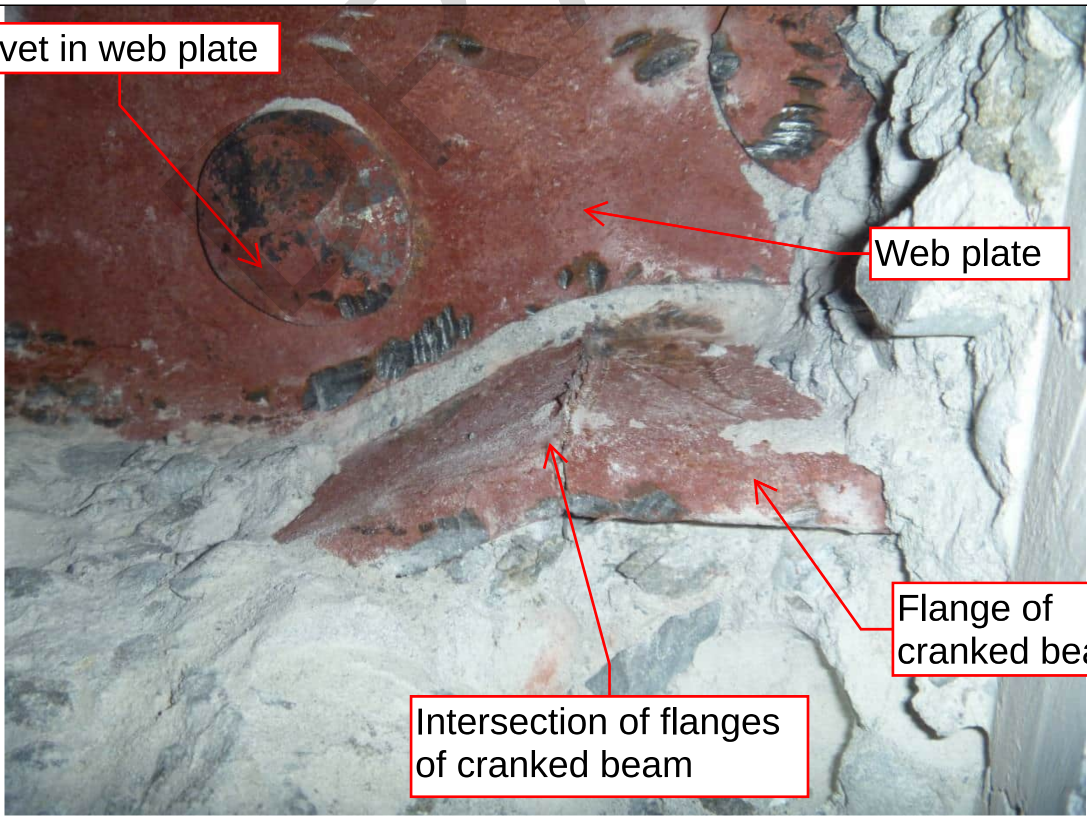


Photo 2 **Cranked beam connection showing riveted web plate**

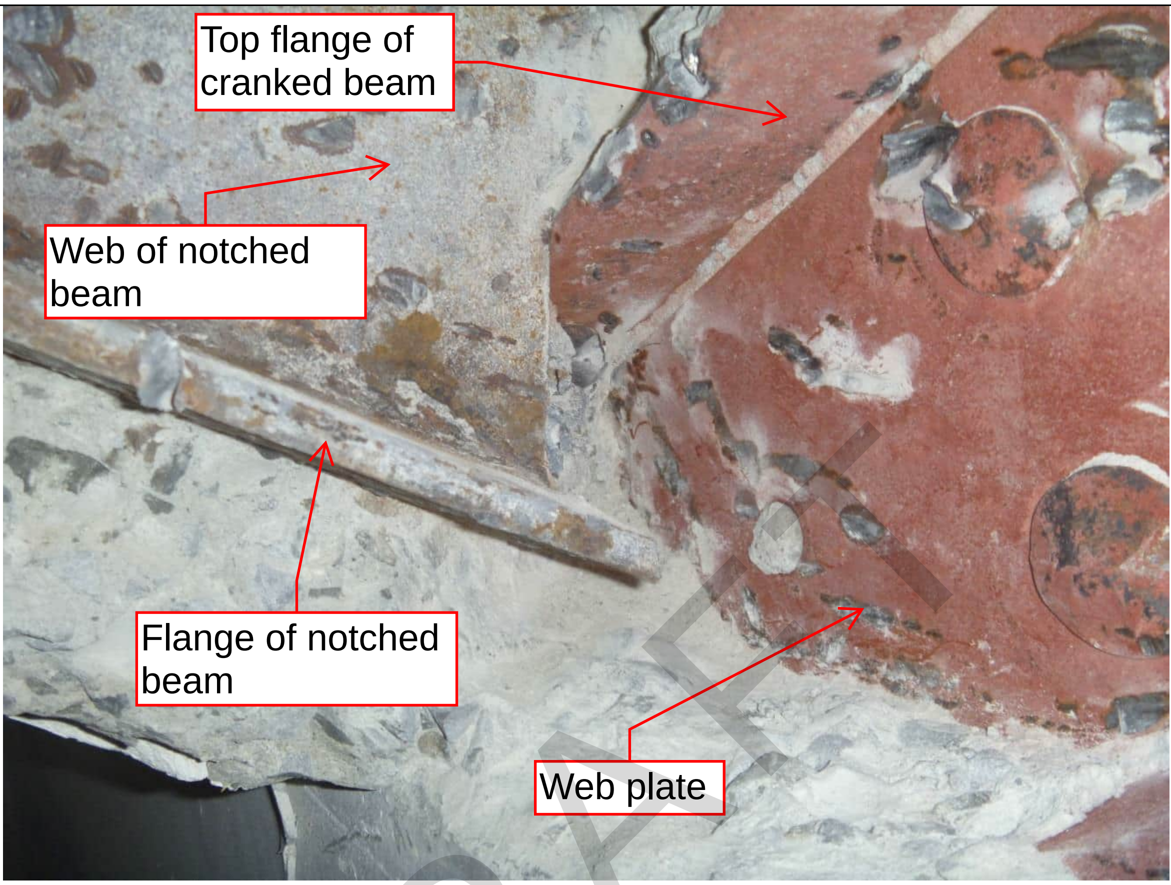
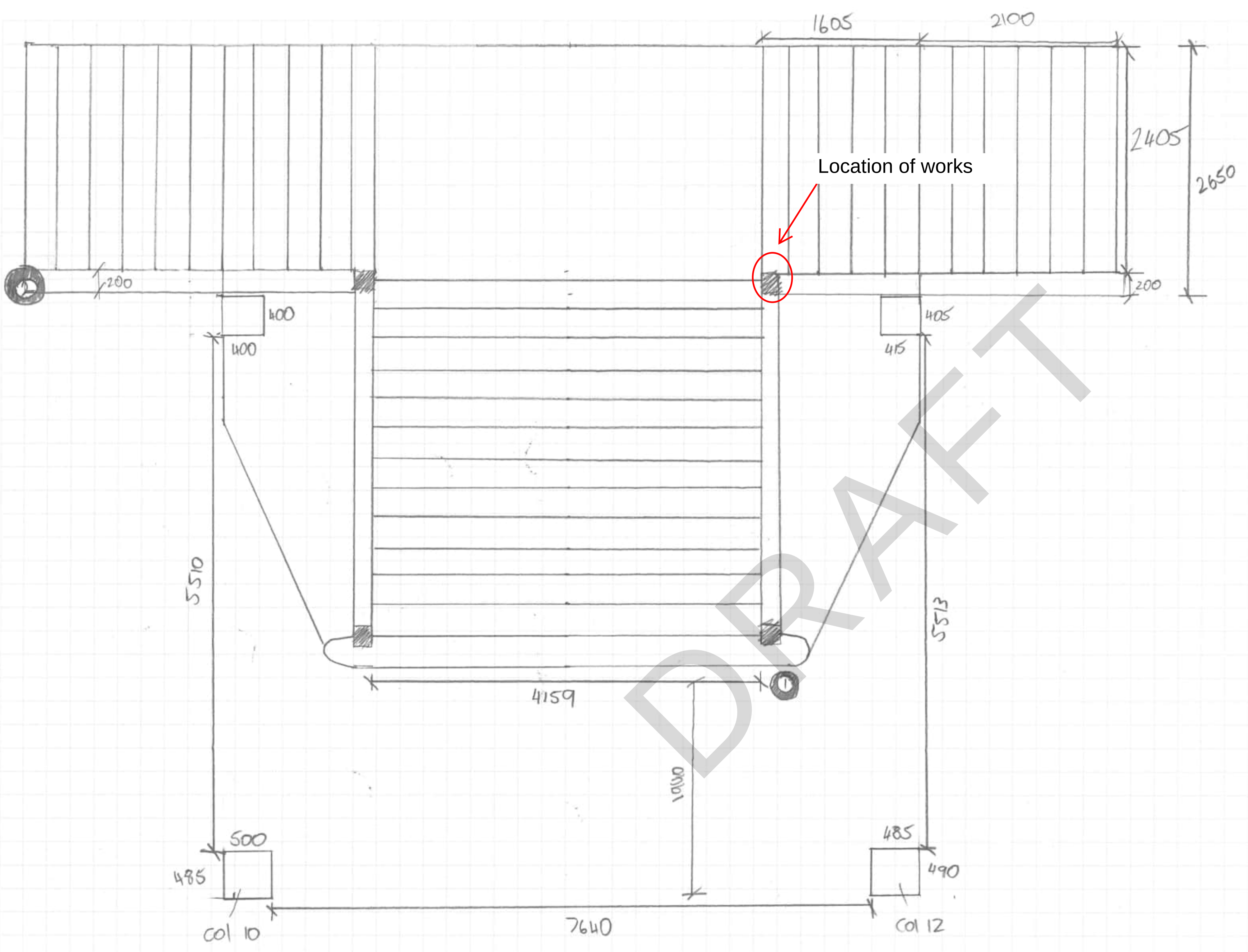


Photo 3 **Adjacent stringer beam showing notch through bottom flange and part of web**

Project	CJC
File/Ref No.	Stairs level 2
By	Matt Crake
Page	1
Date	11/5/15

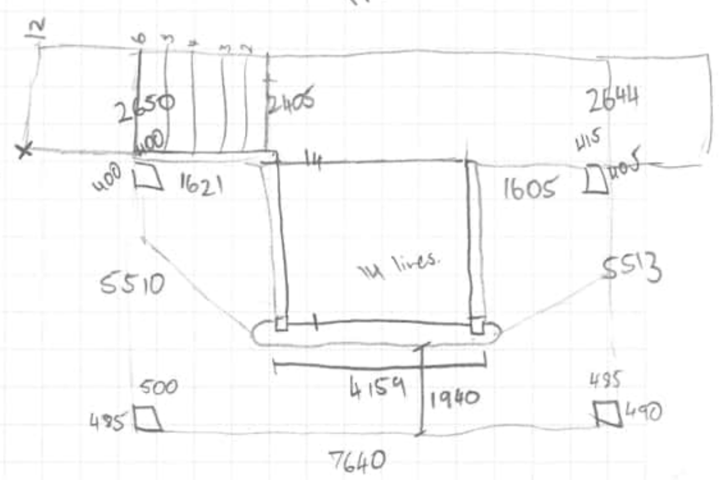


Height from ① to ② 3878mm

1:50

Height = 3878 from X

11555



AECOM

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Memorandum of Inspection

Attention	Craig Stracey	File No.	1.06-08
Company	Dominion Constructors Ltd	Date	14-Dec-2015
Address	292 Cashel Street, Christchurch PO Box 8824, Riccarton, Christchurch 8440, New Zealand	Total Page	7
Project Name	Canterbury Jockey Club, Grand National Stand	Project No.	60439900
From			
Service	Construction monitoring		
Fax No./Email	Craig.Stracey@constructors.co.nz		

We report on an inspection as follows:

Inspection Type	Intrusive Investigation	Inspection Date	11-Dec-2015
Attendees	Kit Lawrence (AECOM) Ian Reynolds (Dominion)		

'Cc' Distribution Details

Attention	Organisation	Fax No.
Nic Todd	Davis Langdon	
Mark Ferfolja	AECOM	
Mike Lowe	AECOM	
Craig Oldfield	AECOM	
Nik Richter	AECOM	
Matthew Crake	AECOM	
David Webster	Thornton Tomasetti	
Alberto Cuevas	Thornton Tomasetti	

Attachments ☒ Yes ☐ No **Mode of Delivery** ☐ Fax ☒ Email ☐ Hand ☐ Mail

AECOM

Site inspection introduction:

AECOM attended inspection at Grand National Stand at Riccarton Racecourse on Friday 11 December 2015. The inspection covered the following items:

- Intrusive works to internal stairs beam connection
- Intrusive works at beam column joint on gridline A5 on the ground floor
- Intrusive investigations on gridline C between gridline 2 and 3 on the ground floor
- Intrusive works at gridline D2 on level 2

Observations and recommendations:

1. Intrusive works to internal stairs beam connection:
   - Several of the beams located on the ceiling of the level 2 to 3 stairs have been drilled to determine the presence of steel, see photo 1, 2 and 3.
   - These investigations encountered steel at all locations.

   Recommendations:
   - No further investigations required
2. Intrusive works at beam column joint on gridline A5 on the ground floor:
   - The joint was cored out with three 85 mm diameter cores and then broken out to expose the beam, see photo 4.
   - The beam is embedded into the column 200 mm. No connection was observed from the web of the steel beam into the column, see photo 5.
   - No steel was encountered in the column with the exception of one horizontal 19 mm reinforcing bar, see photo 6.

   Recommendations:
   - Thornton Tomasetti to observe investigation and comment on need for any further investigations.
3. Intrusive investigations on gridline C between gridline 2 and 3 on the ground floor:
   - The existing slot in the horizontal beam was extended by approximate 150 mm. The extended slot was 50 to 60 mm deep, see photo 7.

   Recommendations:
   - Deepen extended slot to at least the same depth as the already exposed reinforcement bar.
4. Intrusive works at gridline D2 on level 2:
   - An additional vertical slot was cut into the column, see photo 8.
   - No steel was encountered

- Recommendations:
   - No further investigations required

AECOM

Further Actions / Inspections:

- Thornton Tomasetti to observe investigation at gridline A5 on ground floor and comment on need for any further investigations.
- Deepen the extended slot to at least the same depth as the already exposed reinforcement bar on gridline C between gridline 2 and 3 on the ground floor.

Kind Regards,

Prepared by:

Kit Lawrence
Engineering Geologist
e: kit.lawrence@aecom.com
d: +64 3 966 6059

Reviewed by:

Mark Ferfolja
Associate Director - Structures
e: mark.ferfolja@aecom.com
d: +64 3 966 6015

Photos:

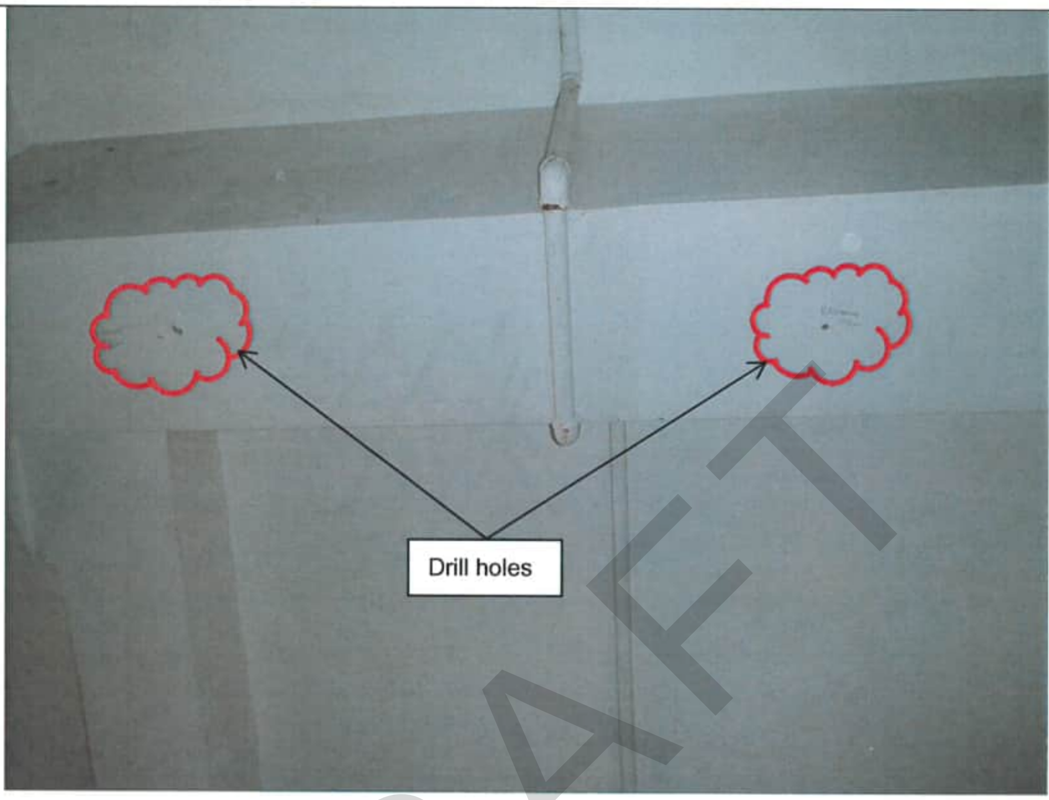


Photo 1	**Drill holes in ceiling beams on level 2-3 stairs.**

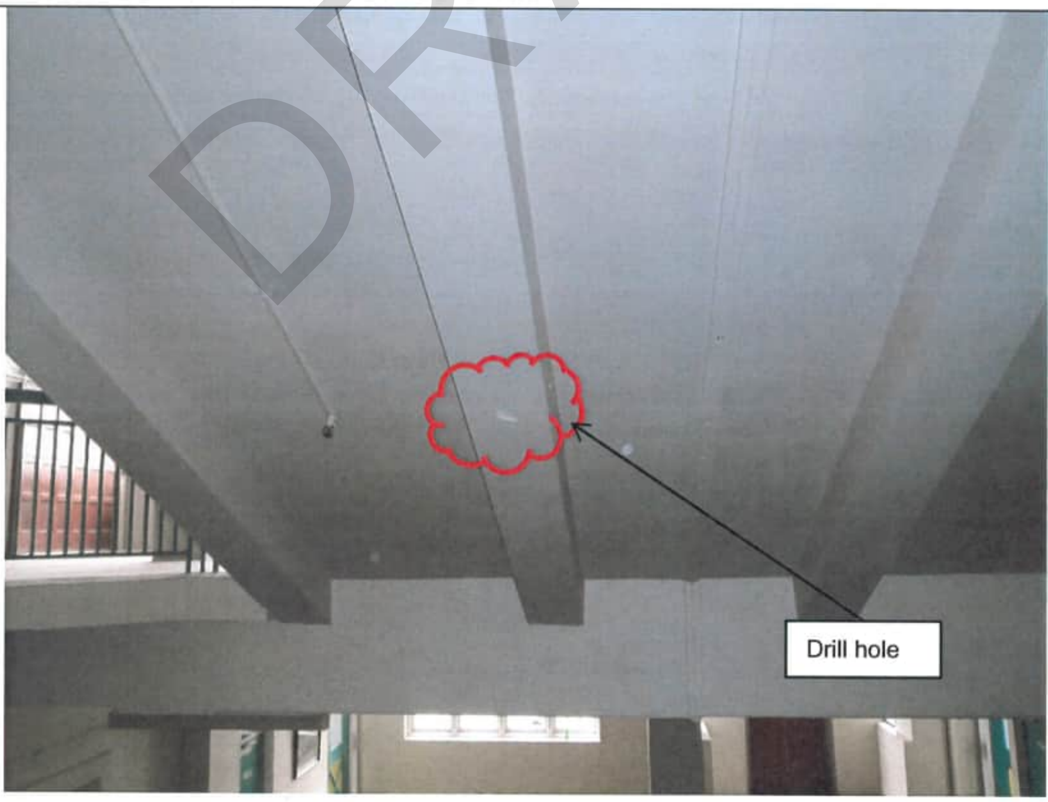


Photo 2	**Drill holes in ceiling beams on level 2-3 stairs.**

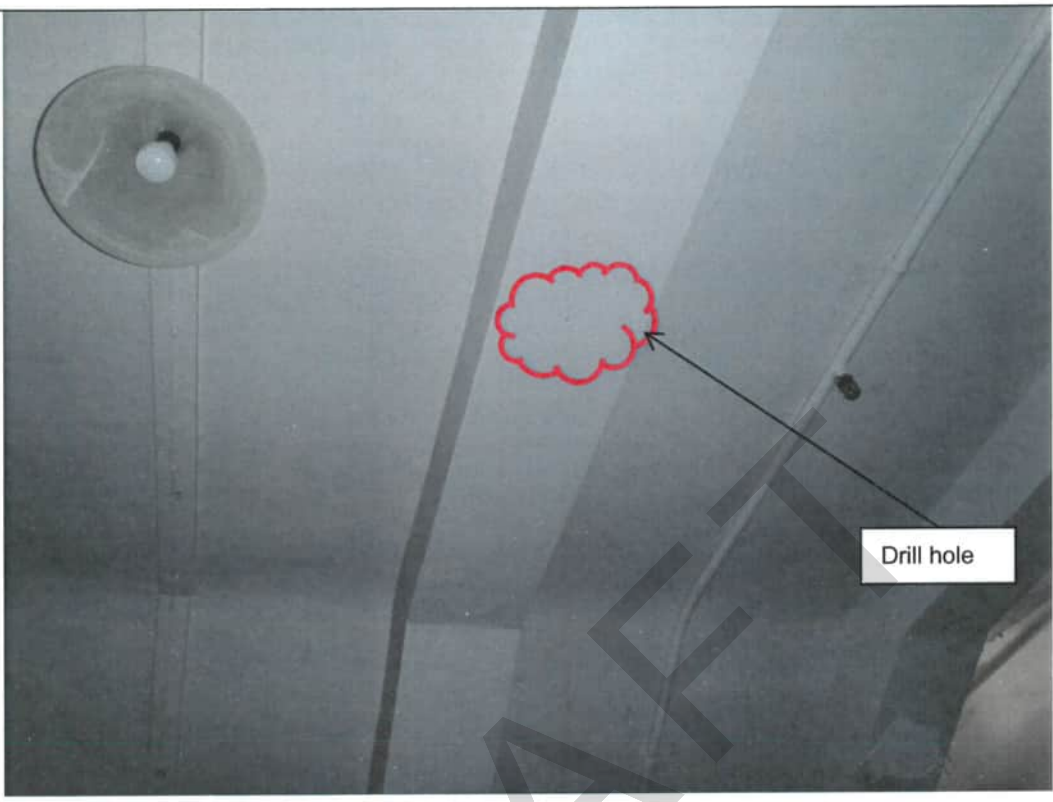


Photo 3 Drill holes in ceiling beams on level 2-3 stairs.

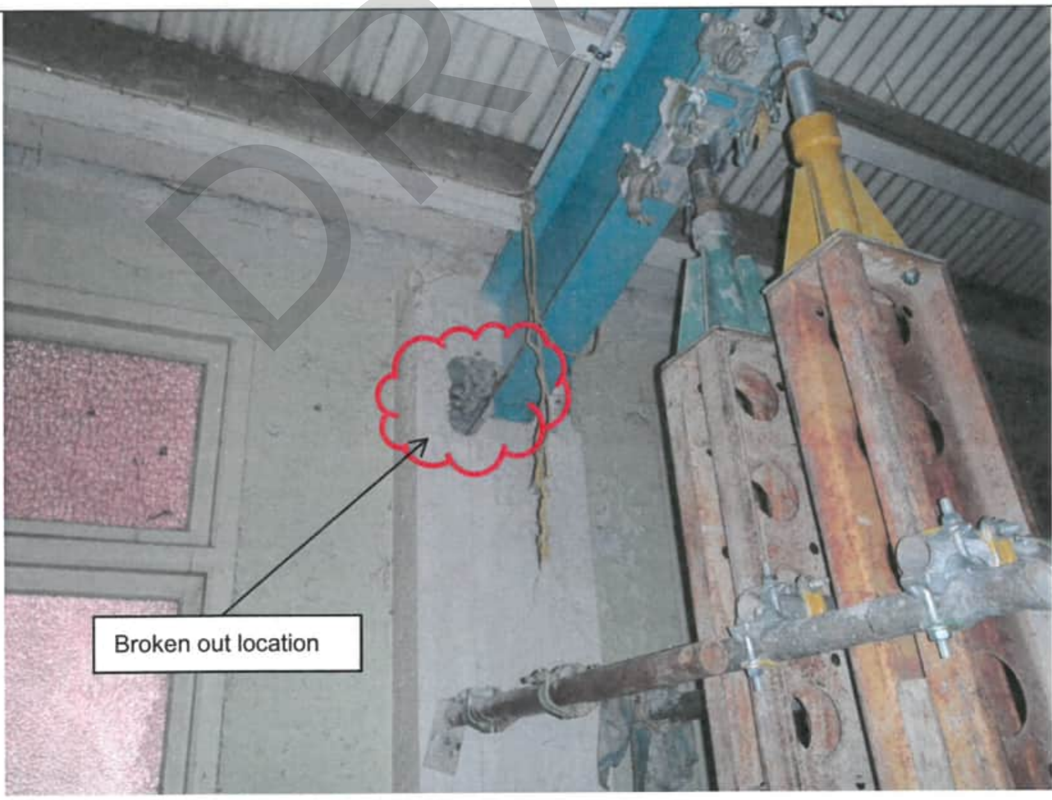


Photo 4 Broken out beam-column location on gridline A5.

AECOM

Photo 5 End of steel beam in column, no connection or column steel.

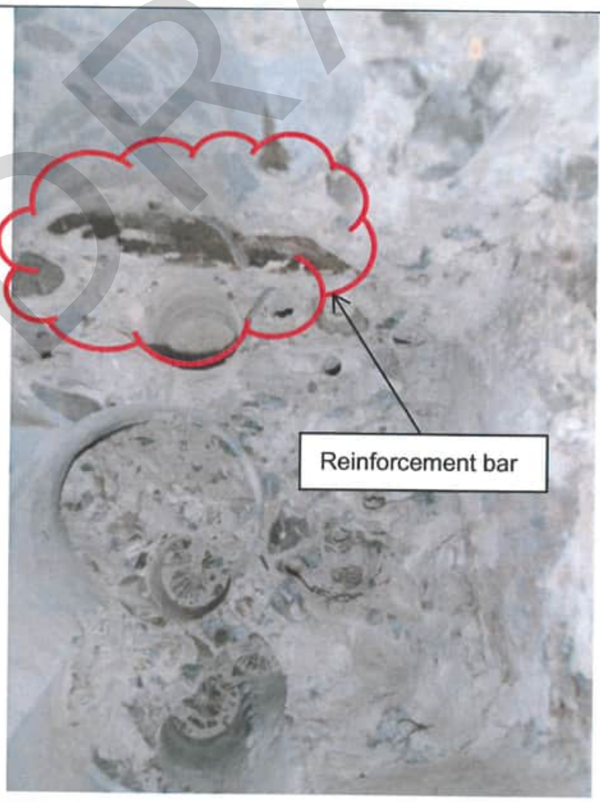


Photo 6 Horizontal reinforcing bar in broken out area.

AECOM

Photo 7 Existing slot between Gridline 2 and 3 on gridline C.

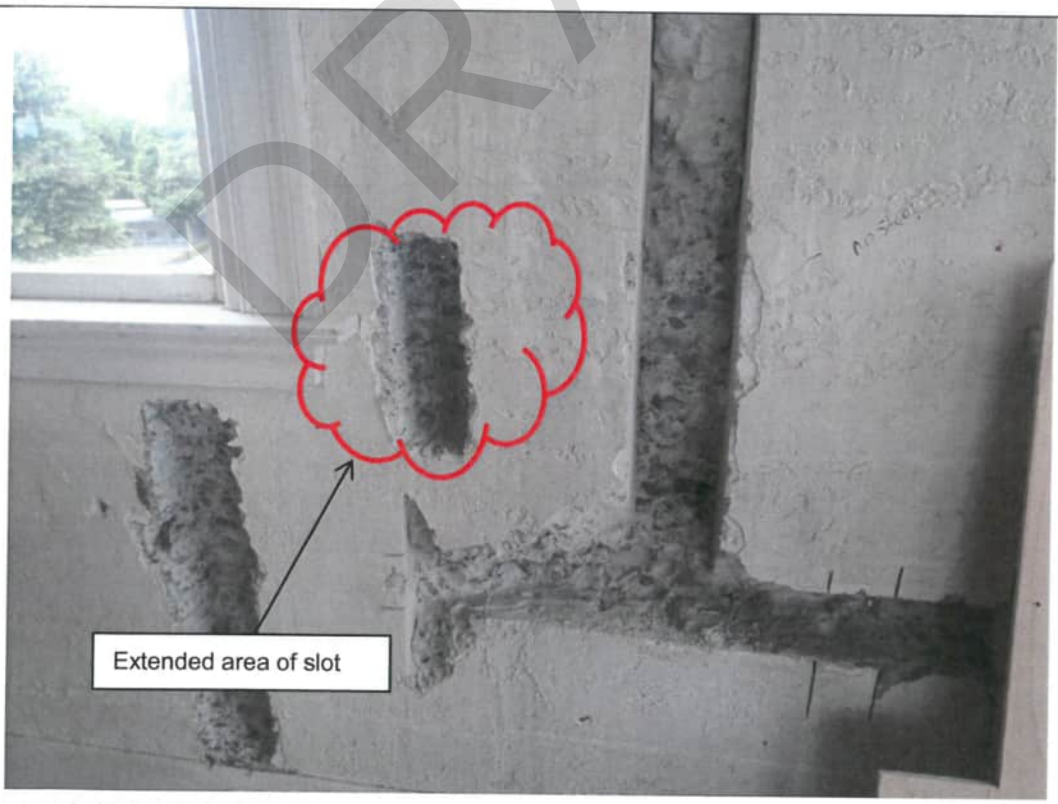


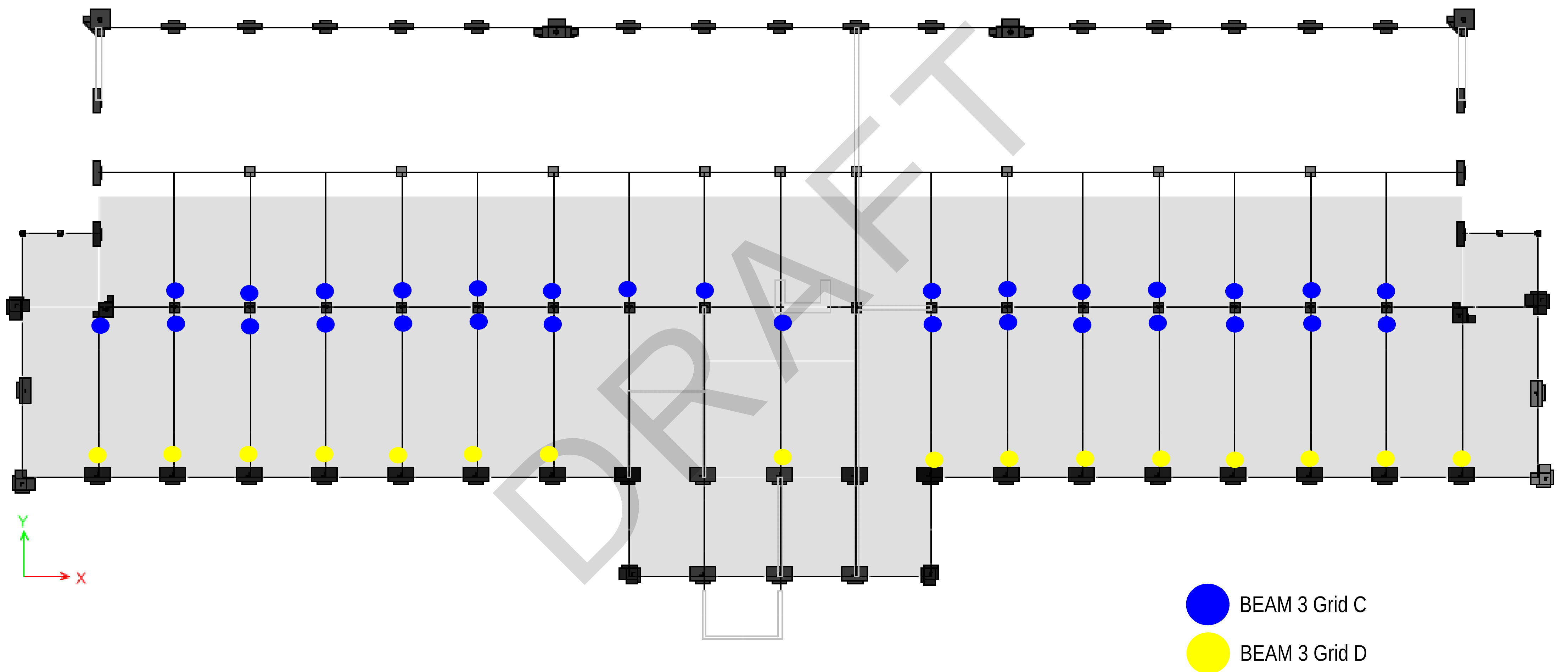
Photo 8 Vertical slot at gridline D2.

DRAFT

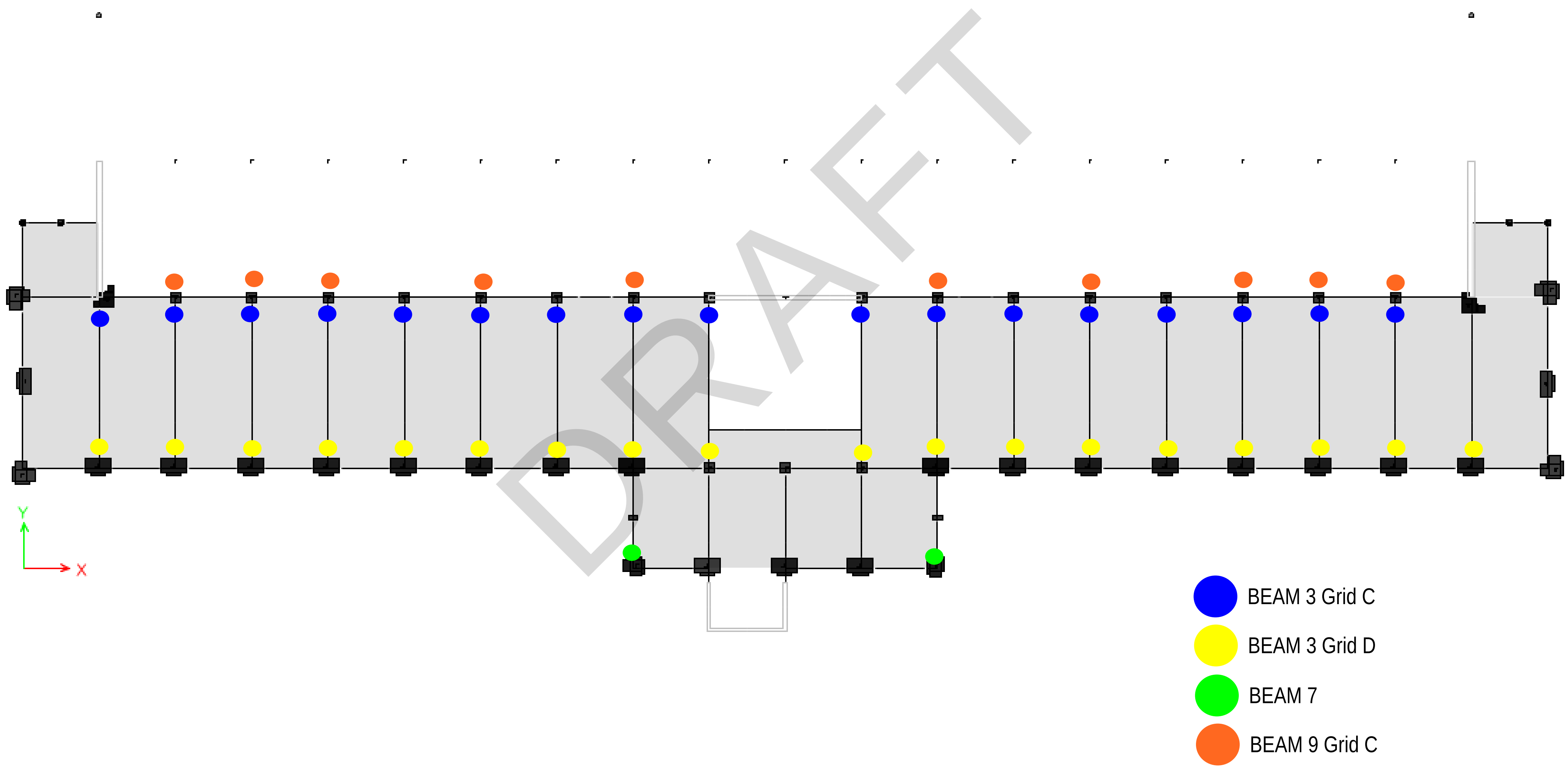
Appendix C

3D Non-linear Pushover Link Locations

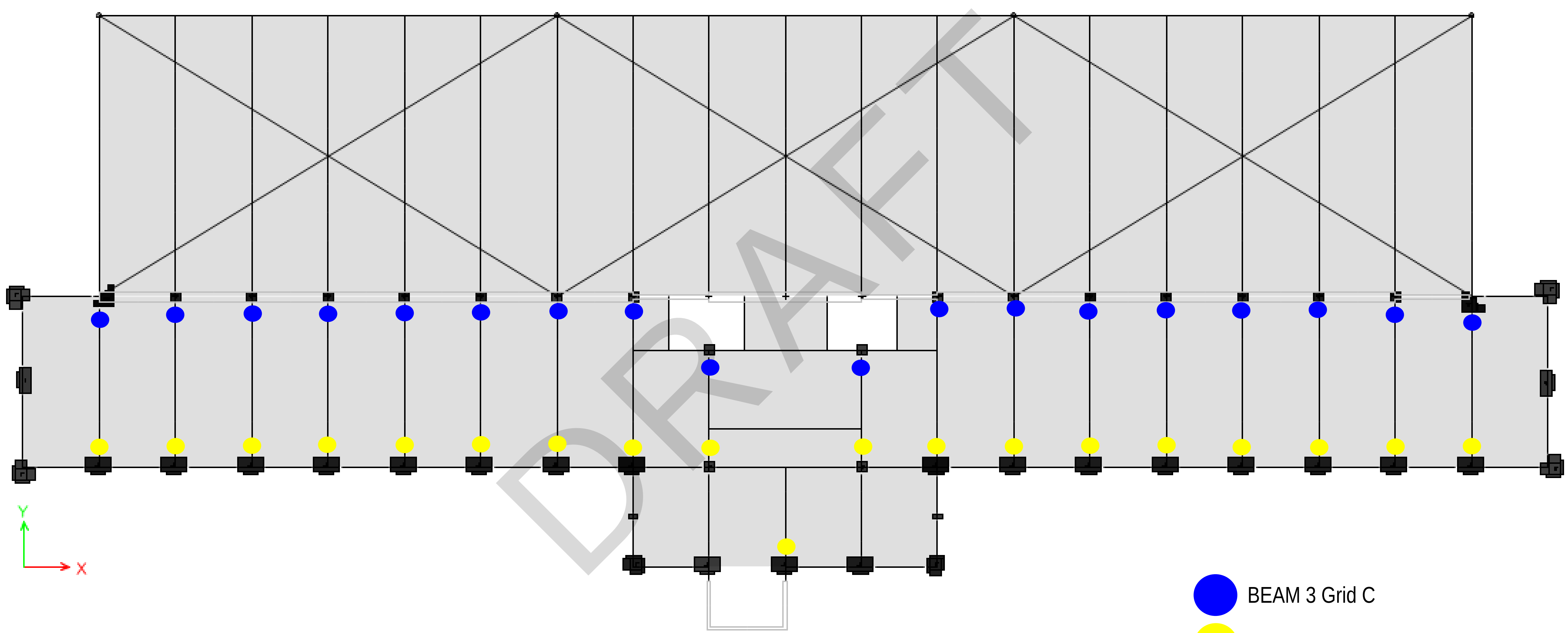
LINKS YIELDING
STEP 0 ALL PUSHES
STOREY 1



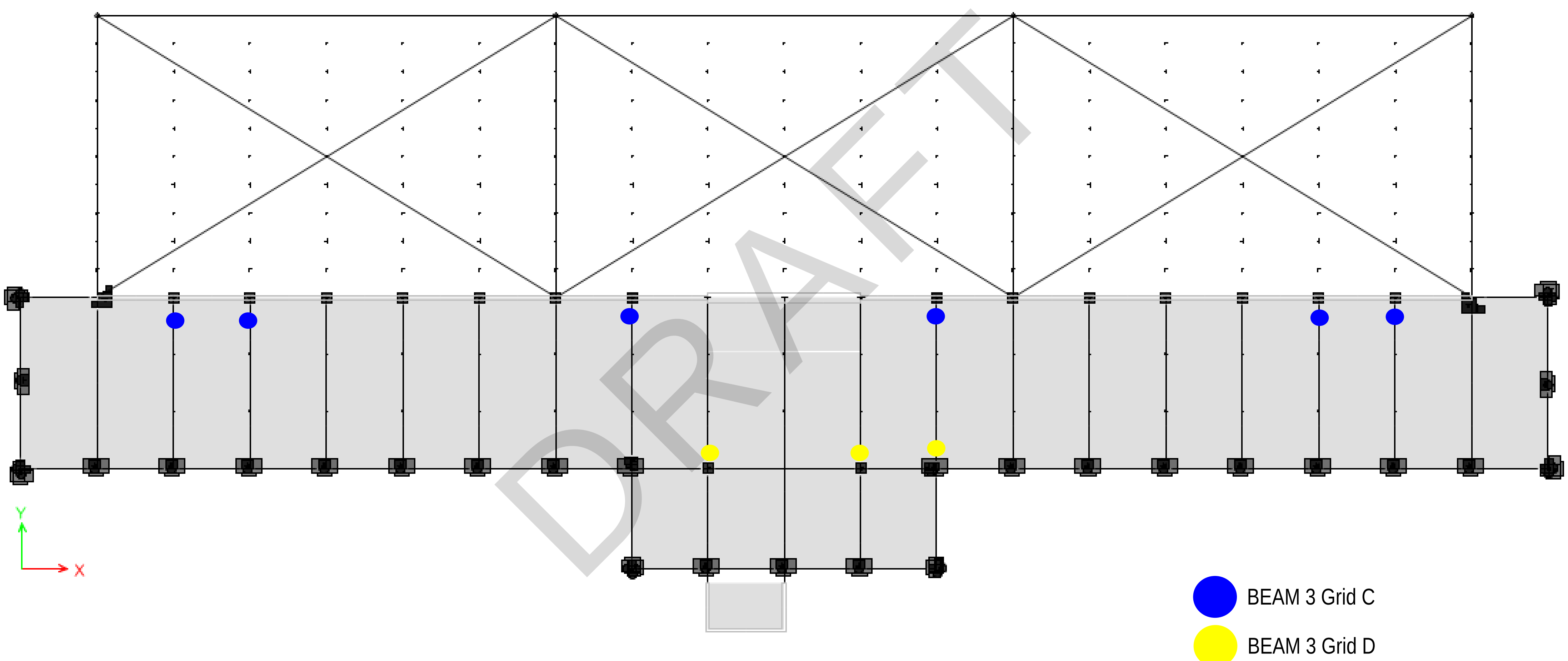
LINKS YIELDING
STEP 0 ALL PUSHES
STOREY 2



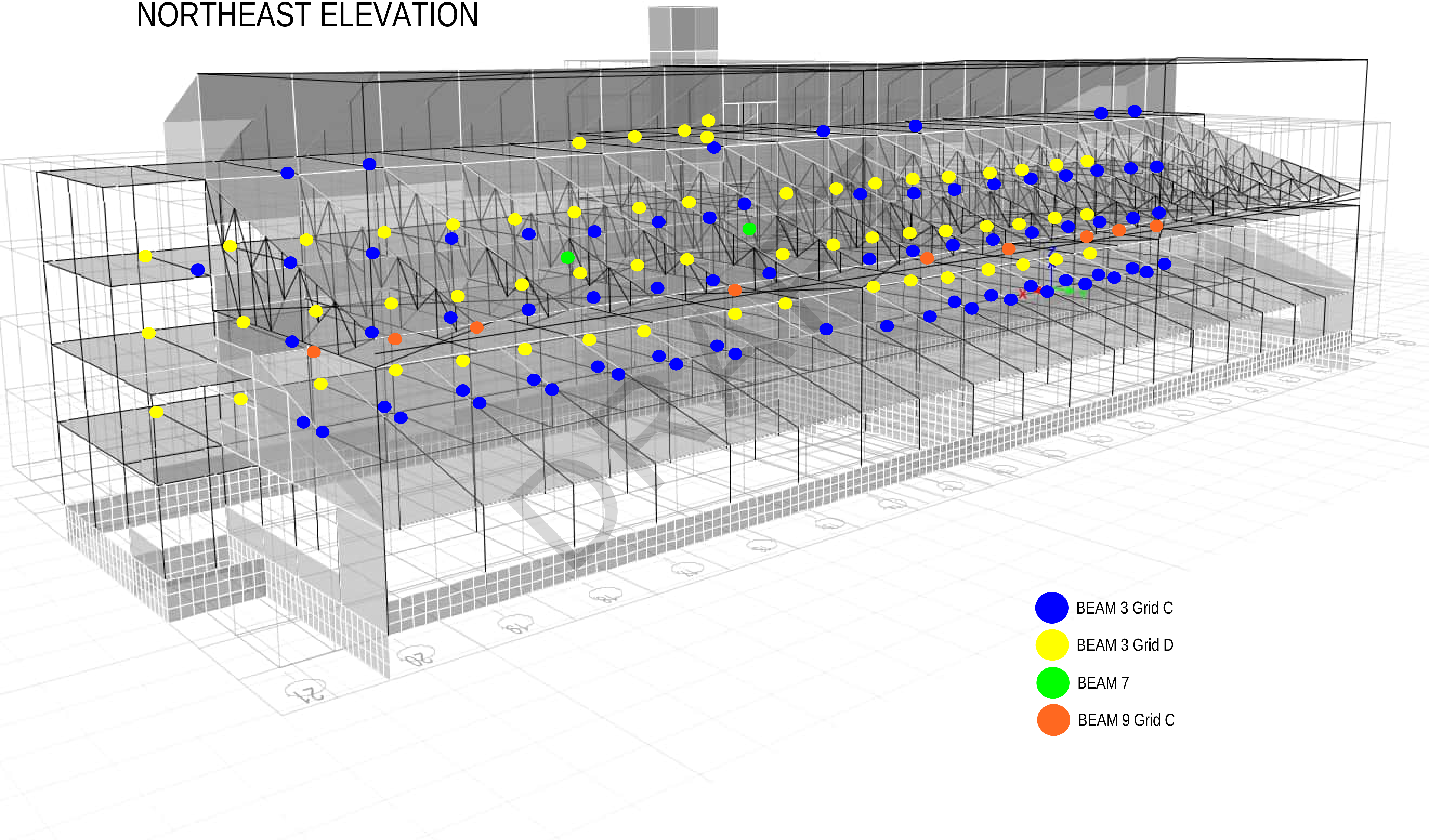
LINKS YIELDING
STEP 0 ALL PUSHES
STOREY 3



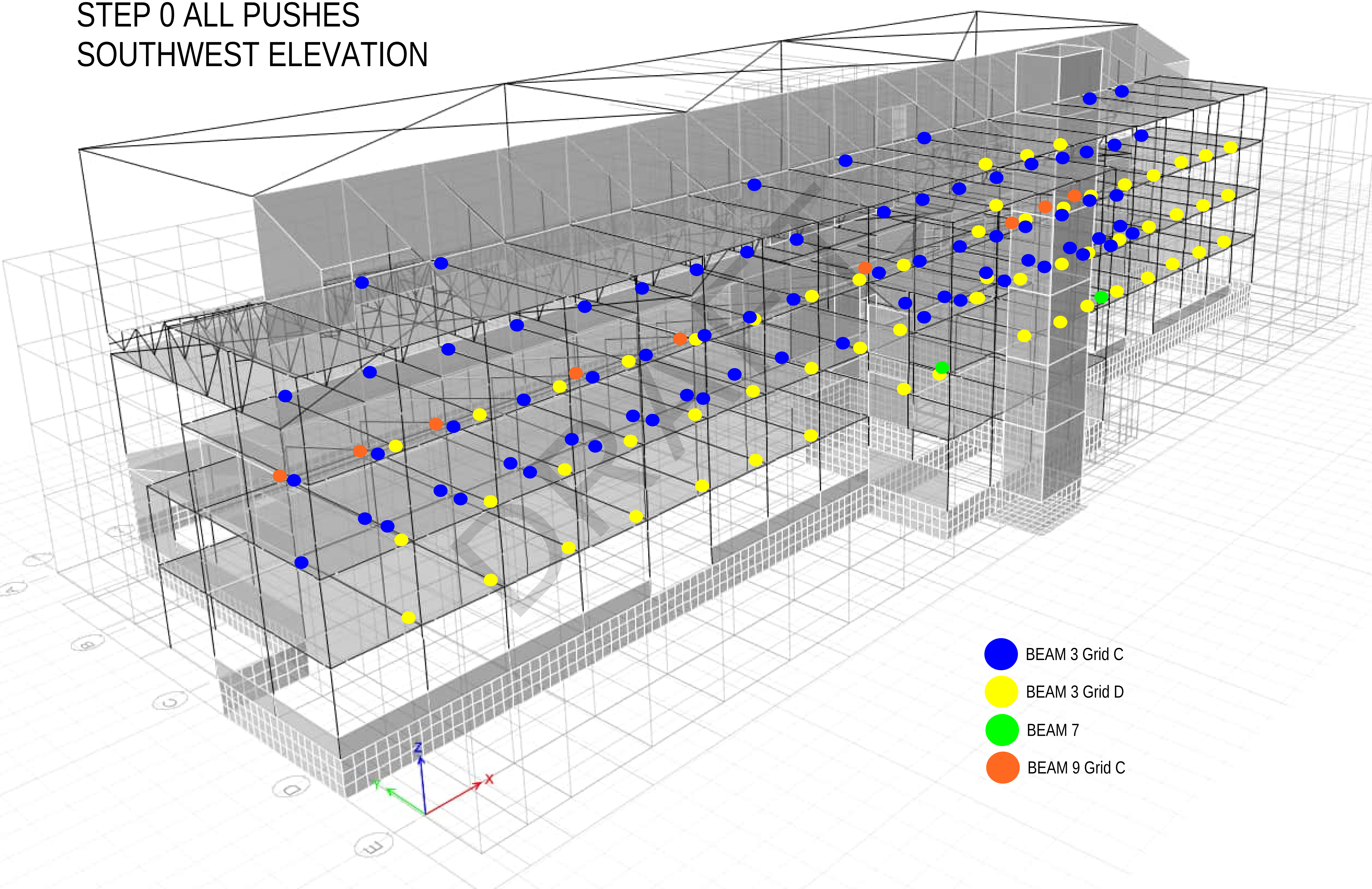
LINKS YIELDING
STEP 0 ALL PUSHES
STOREY 4



LINKS YIELDING
STEP 0 ALL PUSHES
NORTHEAST ELEVATION



LINKS YIELDING
STEP 0 ALL PUSHES
SOUTHWEST ELEVATION



DRAFT

Appendix D

Minutes of Initial Scope Meetings

AECOM

Record of conversation

Project: GNS - ADDITIONAL ANALYSIS MTG - AECOM & TT.
Date: 16.09.15.
Subject: DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD,
Time: 10:00 - 13:00.
Participants (AECOM): NIK LICHTER, ANDREW MCMENAMIN,
File/ref no
Participants (client/other): MARK FERFOLJA.

Distribution: (AS ABOVE) + NIC TODD, CHRIS KAHANEK.

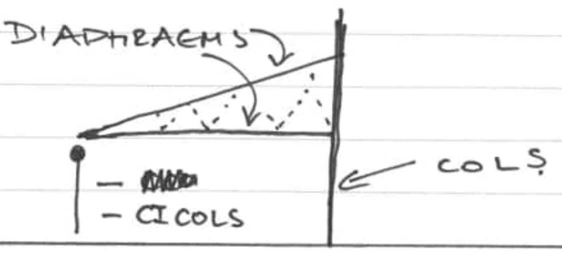
GROUND RULES FOR MEETING.

- GENERAL INTRODUCTIONS & AGENDA.
- ESTABLISHED THAT TO PROGRESS GNS POSITION AECOM & TT TO TRY & ESTABLISH CONSENSUS. WHERE CONSENSUS NOT POSSIBLE AECOM TO ADOPT TT POSITION BUT NOTE DISAGREEMENT.

AECOM & TT AGREE 3D RSA

- 3D RSA. (YES) - AGREED.
  - HOW BUILDING BEHAVES.
  - MOST CRITICAL ELEMENTS.
  - SOME D/C RATIOS.
  - SPOT CHECKS OF MOST CRITICAL ELEMENTS.
  - SENSE OF TORSION.
  - MAINLY 1st MODE RESPONSE
  - D/C RATIO'S TARGETED (DW).
  - MODEL (MINOR TWEAKS ± 7S -- IF NEEDED).
  - TRUSSES AS DIAPHRAGMS.

TRUSS & STAND 'PROXY' IS OK.



SOMETHING THAT REPRESENTS MASS & STIFFNESS CORRECTLY.

SUMMARY: ① AGREED THAT 3D RSA TO BE COMPLETED. ② MINOR TWEAKS OF GEOMETRY ARE ACCEPTABLE (IF NEEDED) ③ NOT NECESSARY TO MODEL ROOF TRUSS SYSTEM OR UPPER STAND BUT DO REQUIRE PROXY FOR MASS & STIFFNESS OF ROOF & STAND. ④ DO SAY ½ DOZ "TARGETED" D/C RATIOS TO DEVELOP 'FEEL' FOR COLUMN & BEAM BEHAVIOUR.

AECOM

Record of conversation

Project GNS – ADDITIONAL ANALYSIS MTG – AECOM & TT.   Date 16.09.15.

Subject   Time 10:00 – 13:00.

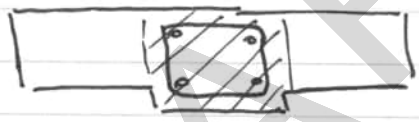
Participants (AECOM) DAVID WEBSTER, ALBERTO CUEVAS, NIK RICHTER   File/ref no

Participants (client/other) CRAIG OLDFIELD, ANDREW MCMENAMIN,   Page 2. of 7.

Distribution MARK FERFOLJA.

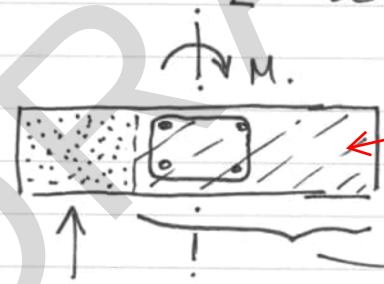
↑ (AS ABOVE) + NIC TODD (DLNZ) & CHRIS KAMANEK (TT).

- FOUNDATION ELEMENTS.
  - SPRINGS & GAP ELEMENTS.
  - TT OK WITH CURRENT MODELLING.
- SECTION PROPERTIES.



↑ CORE ONLY TAKEN.

-



INCLUDE THIS BIT.

IGNORE THIS BIT.

I was under the impression we agreed on the basis for this approach? i.e., that the increased flexural strength is real and may result in shear failure at a lower drift level.

TT/AECOM AGREE TO DISAGREE.

RAW CONCRETE DATA TO TT.

- MATERIALS   STEEL $f_y = 297MPa$. (TESTED).

CONC $f_c =$

AECOM TO SEND 'SENSITIVITY' ANALYSIS FOR FRAME.

FOR CONCRETE STRENGTH SEND PARAGRAPH ON OUR SENSITIVITY ANALYSIS & RAW CRUSHING RESULT.

SUMMARY | (1) FOUNDATION MODEL AS PER THAT ALREADY UNDERTAKEN BY AECOM (2) COLUMNS MODELLED AS CUSTOM SECTIONS ACCOUNTING FOR COMPRESSION AREA OF UNREINFORCED CONCRETE. NOTED THAT AECOM DO NOT AGREE WITH THIS METHOD AS UNREINFORCED CONCRETE AREAS NOT 'BOUND' TO CORE CONCRETE ELEMENTS. (3) AGREED $f_y = 297MPa$ (REINFORCING STEEL), USE DORMAN LONG VALUE OF 203MPa UNLESS ALBERTO CUEVAS ~~[illegible]~~ INDICATES OTHERWISE. (4) SEND RAW CONCRETE DATA & SENSITIVITY TO ALBERTO.

AECOM

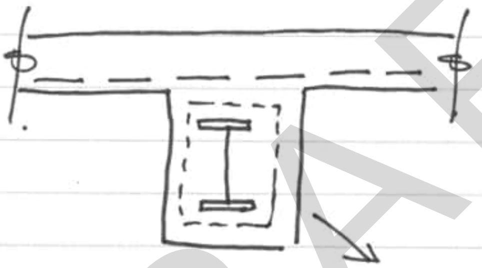
Record of conversation

Project	GNS - ADDITIONAL ANALYSIS MTG - AECOM & TT.	Date	16.09.15.
Subject	DAVID WEBSTER, ALBERTO CUEVAS, NIK RICHTER,	Time	10:00 - 13:00.
Participants (AECOM)	CRAIG OLDFIELD, ANDREW MCMENAMIN,	File/ref no	
Participants (client/other)	MARIC FERFOLJA.	Page	3 of 7.
Distribution	(AS ABOVE) + NIC TODD, CHRIS KAMANEK.		

- STRUCTURAL STEEL (MIDDLESBOROUGH).
  - USE AECOMS NUMBER UNLESS ALBERTO SAYS OTHERWISE. ALBERTO TO ADVISE TODAY.

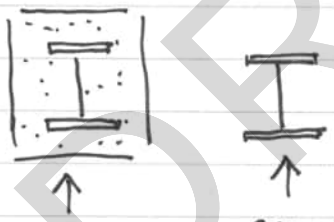
TOP OF STEEL BEAM EXPOSED FROM TOP OF SLAB & NO SHEAR STUDS PRESENT

-



SECTION AS FOUND ONSITE.
NO SHEAR STUDS.

I - SECTION FULLY EFFECTIVE. (AECOM ASSUMPTION.)

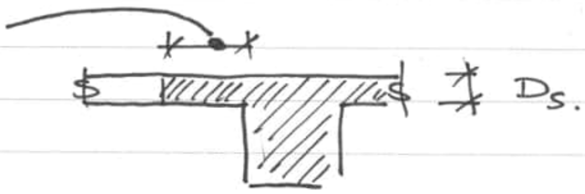


STIFFNESS STRENGTH.

(AECOM ASSUMPTION.)

EFFECTIVE FLANGE OUTSTAND TO BE $2D_s$.

- FLANGE OUTSTANDS = $2D_s$



SUMMARY ① EFFECTIVE FLANGE WIDTH = $2D_s$. ② NO SHEAR STUDS OBSERVED, CONCRETE REALLY ONLY ENCASEMENT.

AECOM

Record of conversation

Project: GNS – ADDITIONAL ANALYSIS MTG – AECOM & TT. | Date: 16.09.15.

Subject: DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD | Time: 10:00 – 13:00.

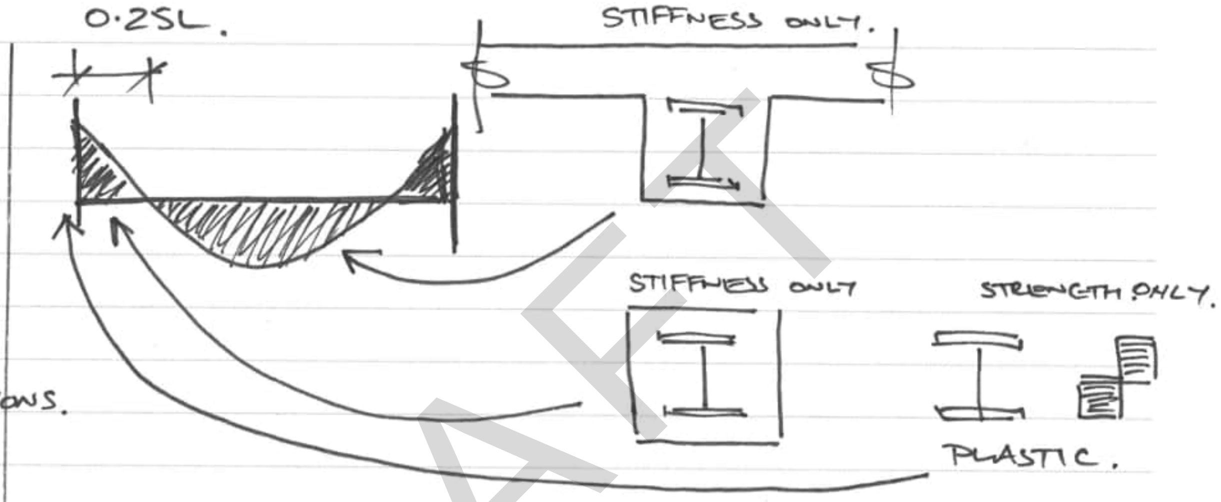
Participants (AECOM): NIK LICHTER, ANDREW MCMENAMIN, MARK

File/ref no

Participants (client/other): FEZFOLVA.

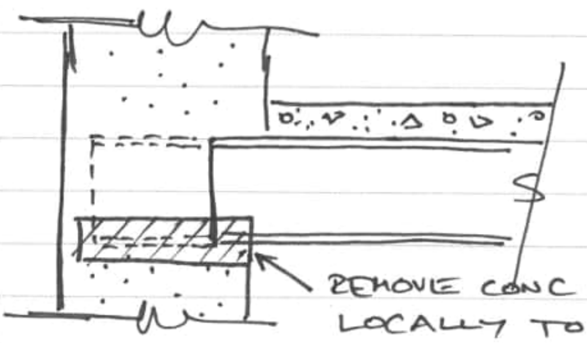
Distribution: (AS ABOVE) + NIC TODD, CHRIS KAMANEK.

AECOM TO USE DIFFERENT SECTION PROPERTIES FOR BEAM STIFFNESS AS PER GRAVITY BMD COMP. LOCATIONS.



- COLUMN TO BEAM CONNECTION (CONCRETE ENCASED).

- BEAM / COL JOINT.
  - PROP ✓
  - ABRUPT CHANGE GEOMETRY.
  - 4 – 6 LOCATIONS (SAY).



EXTENSION OF STEEL BEAM INTO EXISTING CONCRETE COLUMN IS UNKNOWN.

REMOVE CONC LOCALLY TO BETTER UNDERSTAND JOINT.

SUMMARY: (1) AECOM TO VARY BEAM STIFFNESS @ 0.25L & 0.75L LOCATIONS. (2) EFFECTIVE FLANGE = 2Ds. (3) PLASTIC SECTION CAPACITY OF STEEL BEAM @ COLUMN JUNCTION TO BE USED. (4) AECOM TO INVESTIGATE BEAM COLUMN JOINT IN MORE DETAIL.

AECOM

Record of conversation

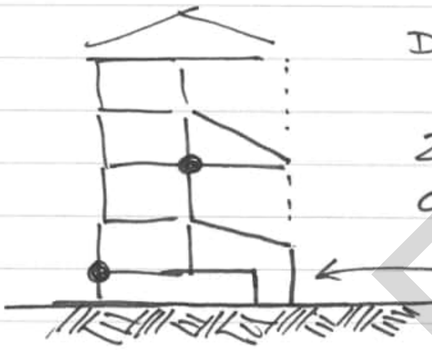
Project: GNS - ADDITIONAL ANALYSIS MTG - AECOM & TT
Date: 16.09.15.

Subject: DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD,
Time: 10:00 - 13:00.

Participants (AECOM): NIK RICHTER, ANDREW MCMENAMIN, MARK
File/ref no

Participants (client/other): FERFOLJA.
Page 5. of 7.

Distribution: (AS ABOVE) + NIC TODD, CHRIS KAMANEK.

TESTING: TWO LOCATIONS ON 3 CONSECUTIVE? FRAMES = 6 LOCATIONS.	•  DEMOLITION OF B/C JOINT. 2 LOCATIONS ON 3 COLUMN LINES. THESE RAKED MEMBERS & SUPPORT (LATERAL).
MODEL SEMI-RIGID DIAPHRAGM.	• SLAB AS A RIGID DIAPHRAGM. – MODEL DIAPHRAGM AS A COARSE SHELL ELEMENT. DIAPHRAGM AS SEMI RIGID.
AT THIS STAGE RAMPS & STAIRS ARE SEPERATE UNLESS MASS IS SIGNIFICANT. IF SO, 'LOLLIPOP' MODEL USED TO INCLUDE EFFECTS.	• RAMPS & STAIRS – IGNORE STAIRS & RAMPS. – ADVISE TT OF RAMP MASS VS BLDG MASS & TT TO ADVISE IF NECESSARY TO INCLUDE. – COLLECTIVE POSITION ON OUTCOMES FOR STAIRS & RAMPS TO BE DECIDED & AGREED @ LATER STAGE. CURRENTLY MAJOR FOCUS ON BUILDING ONLY. •

SUMMARY: (1) 6 OFF LOCATIONS FOR BEAM/COLUMN INVESTIGATION. AECOM WILL NEED TO PROP BLDG FOR THIS. (2) USE SEMI RIGID DIAPHRAGM. (3) STAIRS & RAMPS MAY/MAY NOT BE INCLUDED. UNLIKELY TO OFFER CONSIDERABLE MASS OR STIFFNESS. AECOM TO ADVISE TT OF CONTRIBUTION.

AECOM

Record of conversation

Project: GNS - ADDITIONAL ANALYSIS MTG - AECOM & TT
Date: 16.09.15.

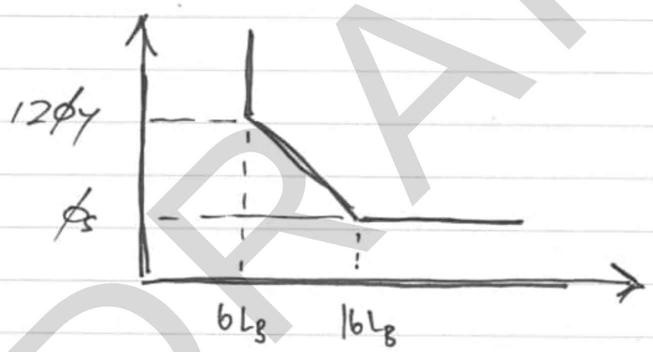
Subject: DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD
Time: 10:00 - 13:00.

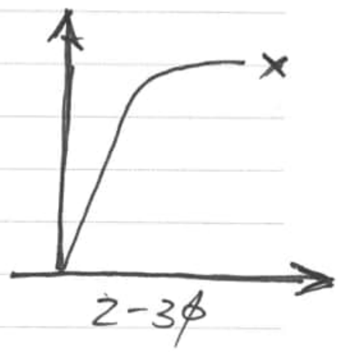
Participants (AECOM): NIK RICHTER, ANDREW MCMENAMIN, MARK
File/ref no

Participants (client/other): FELFOLJA.
Page: 6. of 7.

Distribution: (AS ABOVE) + NIC TODD, CHRIS KAMANER.

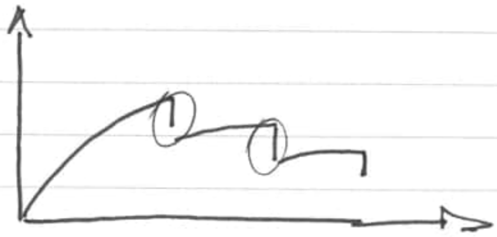
- DISPLACEMENT BASED PUSH-OVER.
- FEMA HINGES FOR BEAM.
- MOMENT CURVATURE FOR COLUMNS?
- O/S SHORT LIST OF REFERENCES (ALBERTO).





- AC TO PROVIDE PAPERS.
- ASC
  - NON LINEAR LINKS. ← ADOPT THIS METHODOOLOGY.

CAREFUL NOT TO MIX ANALYSIS & RETROFIT. AECOM TO DETERMINE % NBS.



SUMMARY: (1) ALBERTO TO PROVIDE PAPERS FOR MODELLING OF COLUMN HINGES. (2) AECOM TO INCORPORATE PAPERS TO MODEL NON LINEAR LINKS @ BEAM COLUMN JOINT. (3) FORCE DEFORMATION CURVES TO BE DISCUSSED @ FUTURE MTG. (4) ENSURE (OR AT LEAST BE COGNISCANT) THAT ANALYSIS & RETROFIT ARE SEPERATE AT THIS STAGE. AECOM CONTRACTED TO DETERMINE %NBS.

AECOM

Record of conversation

Project CNS - ADDITIONAL ANALYSIS MTG - AECOM & TT | Date 16.09.15.
Subject DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD | Time 10:00 - 13:00.
Participants (AECOM) NIK RICHTER, ANDREW MCMENAMIN, MARK | File/ref no
Participants (client/other) FERFOLJA. | Page 7. of 7.
Distribution (AS ABOVE) + NIC TODD, CHRIS KAHANEK.

- AC DISPLACEMENT COMPATABILITY.
- STILL RELIANT ON ASSUMPTIONS.
- SCRUM CONCEPT INTRODUCED. TO BE EXPLAINED IN FURTHER DETAIL GOING FORWARD.

END OF MEETING.

POST MEETING NOTE

- AECOM TO INCLUDE PΔ EFFECTS.

SUMMARY | (1) AECOM ENGAGED TO DETERMINE A %NBS WHICH IS HEAVILY RELIANT ON 'QUESTIONNABLE' ASSUMPTIONS. (2) POST MTG - AECOM TO INCLUDE PΔ EFFECTS.

AECOM

Record of conversation

Project: GNS – ADDITIONAL ANALYSIS MTG.
Date: 18.09.15.

Subject: POST MEETING NOTES / CLARIFICATIONS / ACTIONS.
Time: 13:30.

Participants (AECOM):
File/ref no:

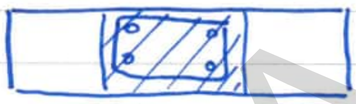
Participants (client/other): DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD, NIK RICHTER, ANDREW MCMENAMIN, (NIC TODD & CHRIS KAHANEK)
Page: 1 of 3.

Distribution:

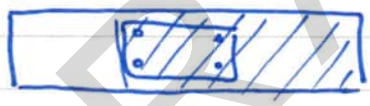
THE FOLLOWING ITEMS ARE POST MTG TOPICS, CLARIFICATIONS ETC.
THESE NOTES TO BE READ IN CONJUNCTION WITH NOTES DATED 16.09.15.

CLARIFICATION.

COLUMN MODELLING (STIFFNESS & STRENGTH).
- NOTING DAVID WEBSTER'S COMMENTS (TT), FROM 17.09.15
- ADDITIONAL CONSIDERATIONS AROUND STIFFNESS POST MEETING.

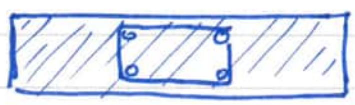


[STRENGTH] [FLEXURE]
- ORIGINALLY PROPOSED BY AECOM.



[STRENGTH] [FLEXURE]
- PROPOSED BY TT.
- AECOM DISAGREE, BUT ACCEPT

* NO CONSIDERATION FOR STIFFNESS & SHEAR *.



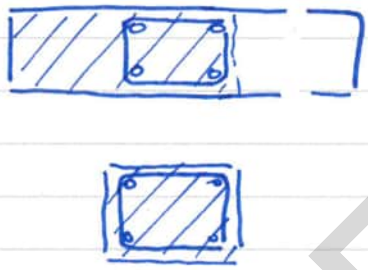
[STIFFNESS]
- $I_{cr} \simeq 0.2 \rightarrow 0.5 I_g$
- I_{cr} TO BE DETERMINED USING CRACKED SECTION ANALYSIS.

* THESE ARE POST MTG NOTES * READ IN CONJUNCTION WITH MTG MINUTES (RAW NOTES) DATED 16.09.15 *.

AECOM

Record of conversation

Project GNS - ADDITIONAL ANALYSIS MTG. Date 18.09.15.
Subject POST MEETING NOTES / CLARIFICATIONS & Time 13:30.
Participants (AECOM) ACTIONS. File/ref no
Participants (client/other) DAVID WEBSTER, ALBERTO CUEVAS, NIK RICHTER
Distribution CRAIG OLDFIELD, ANDREW MCMENAMIN (NIC TODD + CHRIS KAMANEK).



AECOM TO CHECK BOTH CASES FOR SHEAR CAPACITY.

ACTION. - AECOM TO SEND RAW CONCRETE COMP DATA TO THORNTON TOMASETTI.

ACTION. - AECOM TO EMAIL RESULTS OF SENSITIVITY ANALYSIS (CONC STR. AS VARIABLE) TO TT.

ACTION. - AECOM TO ARRANGE FOR INVESTIGATION OF BEAM COLUMN JOINT IN 6 (TOTAL) LOCATIONS ON GNS FRAMES. PROP BEAMS & HIDE PROPS.

ACTION. - AECOM TO ADVISE MASS OF RAMPS & STAIRS VS BLDG MASS & ~~[illegible]~~ DETERMINATION TO BE MADE ON WHETHER TO INCLUDE IN ANALYSIS.

ACTION. - ALBERTO TO SEND THROUGH PAPERS ON MODELLING OF BEAM COLUMN JOINT.

* THESE ARE POST MTG NOTES * READ IN CONJUNCTION WITH MTG MINUTES (RAW NOTES) DATED 16.09.15 *.

AECOM

Record of conversation

Project GNS - ADDITIONAL ANALYSIS MTG.
Date 18.09.15.
Subject POST MEETING NOTES / CLARIFICATIONS / ACTIONS.
Time 13:30.
Participants (AECOM) DAVID WEBSTER, ALBERTO CUEVAS, NIK
File/ref no
Participants (client/other) RICHTER, CRAIG OLDFIELD, ANDREW
Page 3 of 3.
Distribution MCMENAMIN (NIC TODD + CHRIS KAMANEK).

ACTION. - AECOM TO CONSIDER PΔ EFFECTS.

ACTION. - AECOM & TT TO RECONVENE & DISCUSS / CLOSE OUT ITEMS LISTED ABOVE AS WELL AS GRAVITY & LIVE LOAD REQUIREMENTS & ANALYSIS (IE WHICH BITS GET MODELLED + LEVEL REQD FOR MODELLING).

* THESE ARE POST MTG NOTES * READ IN CONJUNCTION WITH MTG MINUTES (RAW NOTES) DATED 16.09.15 *.

AECOM

Record of conversation

Project	Canterbury Jockey Club	Date	24/09/15
Subject	Grand National Stand - Additional analysis #2	Time	0900
Participants (AECOM)	A McMenamin, N Richter, C Oldfield, Mark F.	File/ref no	60439900
Participants (client/other)	C Kahanek, A Cuevas	Page	1 of
Distribution	As above pluss AECOM PM team		

CK: NZI very focussed on collapse hazard + mitigating this to make building not EQ prone. TT has advised NZI that the key points are:

1. Displacement based design of retrofit.
2. Respect strength of building as is.

↳ AECOM to discuss with PM + CJC.

↳ TT to discuss with NZI.

1. Progress since last meeting

- Structural steel yield strength: 230 MPa agreed.
  ↳ Not looking to evaluate a range of values for multiple parameters leading to a sensitivity evaluation of overall response.

- Concrete compressive strength:
  ↳ Higher f'c may increase shear strength enough to suppress shear failure mechanisms + thus relieve EQ proneness.
  ↳ AECOM to run analysis on a shear dominated frame with the higher concrete strength to determine whether the hierarchy of failure is changed.

- Column modelling
  ↳ AECOM has not reviewed the papers referenced by TT (references received yesterday).
  ↳ AECOM to review + comment by next meeting.

DRAFT

AECOM

Record of conversation

Project		Date	
Subject		Time	
Participants (AECOM)		File/ref no	
Participants (client/other)		Page	2 of
Distribution			

- Mass of ramps & stairs: Agree to use lumped mass ~~that~~ for determining overall structural response and agree that the precise mass has little effect. Agree that the ramps & stairs need to be modelled separately to determine their detailed response.

- P-Δ effects: agree to include.

- Beam column joint investigation: will occur next week.
  ↳ AECOM to confirm timing to TT, as TT would like to be present. Site restrictions ⇒ 4 locations initially + 2 more if high variability.

2. Gravity & wind analysis

- Load combinations: as per AS/NZS 1170.
  ↳ TT to confirm NZU is aware that this work is not EQ related.
  ↳ AECOM to confirm fee for analyses.
- Roof trusses: agree that investigation to determine member sizes is required as part of EQ investigation.

3. Next meeting Wed 30/09 afternoon
  ↳ AECOM to send invitations.

AECOM

Record of conversation

Project CNS - METHODOLOGY ANALYSIS MEETING.

Date THURSDAY 1/10/15.

Subject

Time 09:00 - 11:00.

Participants (AECOM) DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD, NIK RICHTER, MARK FERFOLJA.

File/ref no

Participants (client/other)

Page 1 of 4.

Distribution AS ABOVE + NIC TODD, CHRIS KAMANEK, ANDREW MCMENAMIN.

- SHEAR DOMINATED FRAME.
- f'_c = 25MPa, FRAME GRID Ⓒ. ANALYSIS 15 & 25 MPa.
- REANALYSED @ 25MPa. 11 – 18. % NBS.

  12 – 21 % NBS.

  EFFECT OF ↑ CONC. STRENGTH.
- TT TO VALIDATE MODEL & RESULTS FROM SAP 17.01 (~~AECOM~~ TT TO CHECK MODEL & VALIDATE % NBS OUTCOMES).
- DW: (HAND CALCS BY TT TO CHECK. EXPECT SIMILAR BUT NOT NECESSARILY SAME RESULTS).
- MF: GRAVELS FOR CONCRETE SOURCED FROM ONSITE @ RICCARTON PARK.

EXPOSURE OF BEAM COLUMN JOINT MAY DICTATE ANALYSIS DIRECTION.

- 'SHELF' CONCRETE STRENGTH DISCUSSION UNTIL AFTER BEAM/COLUMN JOINT EXPOSED. THIS ITEM OPEN.
- 'CAST IRON' COLUMNS.

3 LOCATIONS, NOT IN SAME PLANE.

MEASURE WALL THICKNESS.

ORGANISE THIS WHILST ~~DURING~~ DOMINION STILL ONSITE LOOKING @ OTHER AREAS.

- ASSESS LOSS OF SECTION DUE TO CORROSION.

SUMMARY ① CONC STRENGTH HAS SMALL EFFECTS ON %NBS. ② TT WILL VALIDATE MODELS WITH HAND CALCS. UNLIKELY TO PRODUCE SAME RESULT BUT SHOULD PRODUCE SIMILAR RESULT. ③ INVESTIGATION OF 'CAST IRON' COLUMNS, COLUMN CONNECTIONS & LOSS OF STEEL SECTION REQUIRED.

AECOM

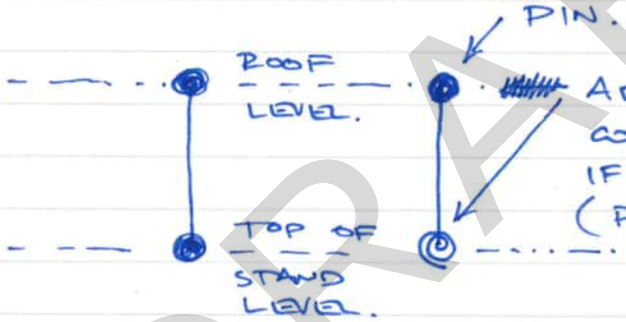
Record of conversation

Project	GNS – METHODOLOGY ANALYSIS MEETING	Date	THURSDAY 1/10/15.
Subject		Time	09:00 – 11:00.
Participants (AECOM)	DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD, NIK RICHTER,	File/ref no	
Participants (client/other)	MARK FERFOLJA.	Page	2. of 4.
Distribution	AS ABOVE + NIC TODD, CHRIS KAMANEK, ANDREW MCMENAMIN.		

NOTE THAT CONC MAY BE VERY ~~[illegible]~~ POOR INSIDE TUBE DEPENDING ON FILLING METHOD.

AECOM/DLNZ TO DISCUSS INTRUSIVE WITH CJC & DOMINION REGARDING DEMO ~~[illegible]~~ TO REVEAL COLUMN CONNECTION.

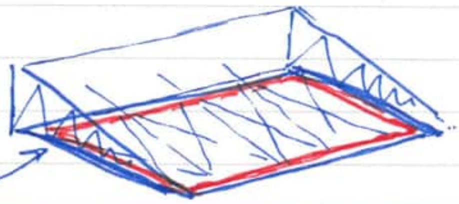
- f_{tu} = 100MPa.
- f_{cu} = 490MPa. } HISTORICAL DATA
- MF – CONCRETE TUBE FILLED FROM 5.0– HEIGHT LIKELY TO BE SEGREGATED ETC.
- REMOVE TIMBER FROM BOTTOM OF TOP COLUMN
- MODELLING COLUMNS



AECOM MAY CONSIDER SPRINGS IF FIXITY EXISTS (PARTIAL FIXITY)?

- SENSITIVITY ANALYSIS AROUND THE PIN/PIN & FIX/FIX COLUMN ENDS.
- BEAM/COL JOINT – DOING SHORING PLANS & FINALISING DETAILS. AECOM WILL ADVISE TT WHEN JOINTS AVAILABLE.
- ~~ADVISE~~ AECOM TO ADVISE BREAK UP OF COSTS ACTION ON MF.
- AECOM MODELLING:

RED DIAPRAGM ONLY



SUMMARY ① SENSITIVITY OF COLS RE PIN/PIN, PIN/FIX, FIX/FIX ~~[illegible]~~ TO BE UNDERTAKEN TO UNDERSTAND BEHAVIOUR. ② PROPPING PLANS & CONTRACTOR TIMING REGARDING BEAM/COL JOINTS TO BE ADVISED. ③ AECOM TO ADVISE BREAKUP OF $160K MODELLING FEE WRT INTRUSIVE WORKS. ④ ROOF TO BE MODELLED AS 'EQUIVALENT' DIAPHRAGM.

AECOM

Record of conversation

Project: GNS – METHODOLOGY ANALYSIS MEETING

Date: THURSDAY 1/10/15.

Subject:

Time:

Participants (AECOM): DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD, NIK RICHTER,

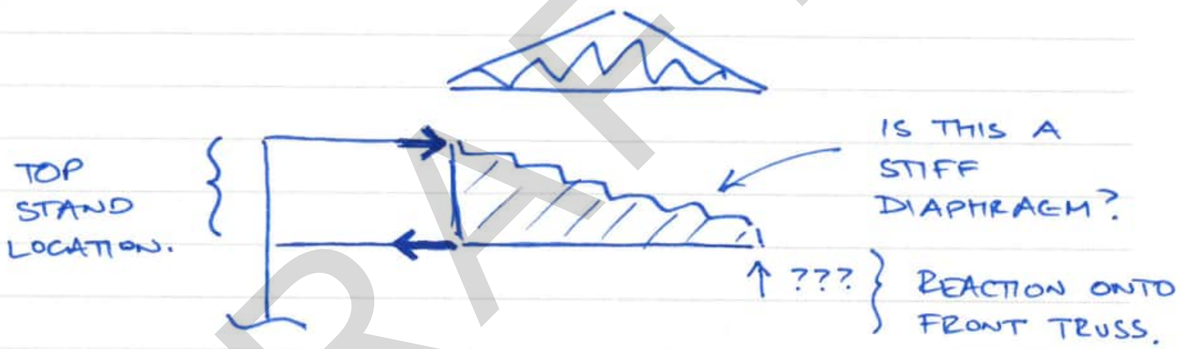
File/ref no:

Participants (client/other): MARK FERFOLJA.

Page 3. of 4.

Distribution: AS ABOVE + NIC TODD, CHRIS KAMANEK, ANDREW MCMENAMIN.

- DW: SMEAR MASS APPROXIMATELY CORRECTLY & USE LINKS TO TIE MASS TO STRUCTURE (PROVIDE REASONABLE REPRESENTATION).



- DW: TOP STAND – MODEL TRUSSES OR AN AGREED EQUIVALENT. MODEL SOME FORM OF EQUIVALENT 'PROXY' MEMBERS.
- BOTTOM STAND: RAKED STRINGERS SUPPORTING BLEACHERS NO DIAPHRAGM PRESENT.
- BOTTOM STAND: 1 TEST PIT @ GRID B & GRID A IE, TWO TEST PITS IN TOTAL. A7 & A8 & B8.
- REVIEW OF PAPERS
  - WHAT SORT OF ROTATION CAPACITY PRESENT?

AECOM TO LIAISE WITH DOMINION & [redacted] CJC/DLNZ REGARDING TRIAL PIT LOCATIONS. AECOM TO ADVISE IT ONCE COMPLETE.

SUMMARY: ① TOP STAND TO BE MODELLED USING A PROXY FOR MASS + STIFFNESS. ② FOUNDATION CONDITIONS SOUGHT ON GRIDS A + B TO DETERMINE SPRING SUPPORTS.

AECOM

Record of conversation

Project GNS METHODOLOGY ANALYSIS MEETING — Date THURSDAY 1/10/15.

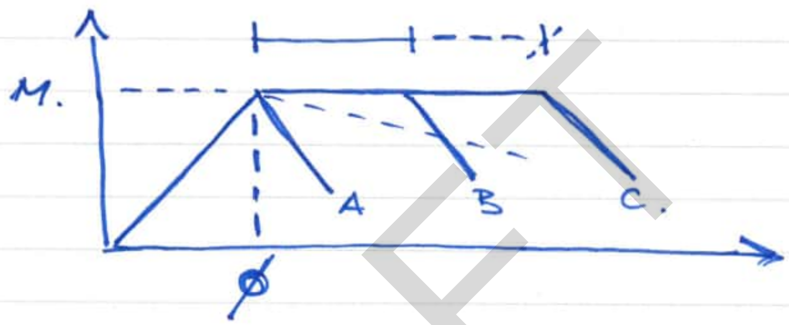
Subject — Time

Participants (AECOM) DAVID WEBSTER, ALBERTO CUEVAS, CRAIG OLDFIELD, NIK RICHTER, — File/ref no

Participants (client/other) MARK FERFOLJA. — Page 4 of 4.

Distribution AS ABOVE + NIC TODD, CHRIS KAMANEK, ANDREW MCMENAMIN.

WHAT IS THIS PLATEN LENGTH???

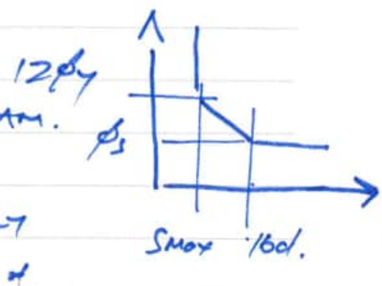


AECOM TO SEND THROUGH PRIESTLEY REFERENCE.

- PRIESTLEY & CALVI
- AISC 41 13 – ABSENCE OF MORE INFO.

END OF MEETING.

- POST MTG DISCUSSION:
  - INCLUSION OF PRIESTLEY DIAGRAM.



- NEXT MEETING? RELEVANCE, WILL RELY HEAVILY ON B/C JOINT EXPOSURE. DELAY & HAVE ONSITE ONCE B/C JOINT EXPOSED (ANTICIPATED TO BE NEXT WEEK).

SUMMARY (1) PROVIDE PRIESTLEY DIAGRAM (2) MOVE/POSTPONE NEXT TT MEETING TO ALIGN WITH B/C JOINT EXPOSURE 'EAR MARKED' FOR NEXT WEEK. B/C JOINT DATA PIVOTAL TO OUTCOMES.

Lawrence, Kit

From:	Cuevas, Alberto <ACuevas@ThorntonTomasetti.com>
Sent:	Monday, 19 October 2015 4:53 p.m.
To:	Richter, Nik
Subject:	GNS analytical model

Hi Nik,

Thanks for joining me to the site visit last Friday. Regarding what we discussed about the GNS modeling, as I mentioned, I still think it is worth if you go ahead creating the model for the 3D RSA before getting all the missing information from site (or finishing updating the dwgs). You could even group the different elements (ie, groups named: chords, diagonals, etc) so that you can easily redefine/assign the properties once they are known just by selecting the elements by group names. I am not 100% aware of the schedule but it's better if we stick to it as much as possible and the best way is by making some progress with the model, which is key for the final outcome.

Regards,

Weidlinger and Thornton Tomasetti have merged (read more)

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