Invercargill City Council

Surrey Park Grandstand

Detailed Seismic Assessment Report

Contact Details

Name: Andrew Davidson

12 Moorhouse Avenue
PO Box 1482, Christchurch Mail Centre,
Christchurch 8140
New Zealand

Telephone: +64 4 471 7200
Mobile: +64 21 228 6080

Document Details:

Date: 21/01/2019
Reference: 6-VI043.00
Status: Final

Prepared By

Julian Maranan
Senior Engineer Structural

Reviewed By

Andrew Davidson
Senior Structural Engineer/CPEng 1013636
Team Leader

Approved for Release By

Jamie Lester
Principal Structural Engineer/CPEng 260347
Work Group Manager

Contents

Executive Summary 1

1 General 4

1.1 Scope 4

1.2 Compliance 4

1.3 Means of Compliance 6

1.4 Limitation of the Report 6

2 Background Information 7

2.1 Source of Geotechnical Data 7

2.2 Source of Building Data 7

3 The Site 8

3.1 Topography 8

3.2 Regional Geology 9

3.3 Seismicity 9

3.4 Ground Condition 10

4 The Structure 11

4.1 Building Description 11

4.2 Primary Gravity Load Resisting System 11

4.3 Primary Lateral Load Resisting System 11

4.4 Site Inspection and Building Condition 11

4.5 Alterations 13

4.6 Assumptions 13

5 Detailed Seismic Assessment 14

5.1 Analysis Methodology 14

5.2 Gravity Loads 14

5.3 Earthquake Loads 14

6 Results of the Detailed Seismic Assessment 17

6.1 Steel Portal Frame (Bleacher) 17

6.2 Steel Portal Frame (Canopy) 17

6.3 Steel Frame - South Elevation 17

6.4 Steel Brace - North Wall 18

6.5 Roof Bracing - North Wall 18

6.6 Foundation 18

6.7 Concrete Block Veneer 18

6.8 Bleacher Support....18
6.9 Summary of Assessed Seismic Capacity....19
6.10 Critical Structural Weakness (CSW)....20

7 Conclusion and Recommendations....21

Appendix A: Assessment Summary Report Template....22

Appendix B: Record Drawings....26

Appendix C: 67%NBS Strengthening Concept....27

List of Figures

Figure 3-1 : Aerial View of the Site (source: Google Maps)....8
Figure 3-2 : Invercargill Geological Map (source: GNS)....9
Figure 3-3 : Map of South Island of New Zealand showing the tectonic plate boundary (source: www.es.govt.nz)....10
Figure 3-4 : Liquefaction Susceptibility Invercargill City (source: GNS Report dated January 2012)....10
Figure 5-1 : Graphical Presentation of the 3D Analysis Model....14

List of Tables

Table 1 : Summary of Assessed Seismic Capacity (%NBS)....1
Table 6-1 : Assessed Seismic Capacity (%NBS) of Key Elements. 19

Document History and Status

Revision	Date	Author	Reviewed by	Approved by	Status
0	26/11/2018	JPM	AD	JL	Draft for Client Comment
1	21/01/2019	JPM	AD	JL	Final

Revision Details

Revision	Details
1	Status changed to Final, no changes to the report.

Executive Summary

Background

This report presents the Detailed Seismic Assessment (DSA) of the Surrey Park Grandstand located in Invercargill, New Zealand. The building has been assessed at Importance Level 3 (IL3). The results of the assessment are presented and discussed to provide information and options to the Invercargill City Council.

Building Description

The Surrey Park Grandstand is approximately 55m long and approximately 8m wide. It is effectively a two-level building consisting of two structural forms, each with a significantly different stiffness that will affect how the building performs under lateral loading from wind and earthquake loads. The top roof section is a flexible lightweight structure and open on one side while the lower bleacher support section is more rigid consisting of heavy concrete encased structural sections and precast concrete bleachers.

The lower section of the grandstand seating area is constructed of inclined steel portal frames running transversely which support precast concrete bleachers. The legs of the portal frames are encased in concrete to three-quarter height on the south wall and up to the knee on the north wall. Bracing is provided by steel cross bracing in the north internal wall and the south wall. The cross bracing in the south wall does not extend to ground level but terminates above the windows on the south side. There is infill hollow concrete masonry between the portal legs below the windows. This masonry has a bond beam along the top edge but otherwise is not reinforced or filled. The north wall is low beneath the front set of bleachers and consists of a reinforced concrete retaining wall. The north wall of the lower seating section of the grandstand is short and rigid while the south wall is taller with cross bracing that does not extend to ground level and hollow masonry veneer that will provide minimal lateral restraint.

Assessment Results

The seismic demands were calculated using an equivalent static analysis (ESA) procedure in accordance with the New Zealand loading code (NZS1170.5) Part 5 – Earthquake Design Actions.

The seismic capacities have been assessed in accordance with the guideline document titled "The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments", dated July 2017. The earthquake rating is based on the building being Importance Level 3 (IL3).

The assessed seismic capacities of the primary structural elements, based on the respective failure mode, are presented in the table below.

Table 1 : Summary of Assessed Seismic Capacity (%NBS)

Location	Element	%NBS (IL3, ULS)	Mode of Failure/Remarks	Direction
Canopy Structure	M16 Cross Bracing	40	Axial tension	E-W
	2 ½" ID Pipe Strut	85	Axial compression	E-W
	M20 wall cross bracing	55	Axial tension, interior bays <67%NBS and end bays >67%NBS	E-W
	Roof Beam (8"x5 ¼" x 17# UB)	100	Flexure	N-S
	Rear Wall Column (10"x5 ¾"x21# UB)	70	Combined axial and bending	N-S

	Roof strut (5"x2" RSC) along the top of the south wall	70	Axial compression	E-W
	4"X4" Canopy Columns	67	Combined axial and bending	E-W
Bleacher Structure	Beam (14"x6 ¾"x30#UB)-Steel Portal Frame	80	Combined axial and bending	N-S
	Column (14"x6 ¾"x30#UB)-Steel Portal Frame	60-90	Combined axial and bending. Eight columns <67%NBS.	N-S
	M19 Bleacher cross bracing	30-64	Axial capacity in tension. The cross bracing at the end bays have capacity < 34%NBS and the rest have capacity >34% but <67%NBS.	E-W
	Bleacher Strut (2.5ID Pipe)	45	Axial capacity in compression. The struts located at the braced bays have a capacity of 45%NBS and the rest of the struts have capacity > 67%NBS.	E-W
	6" unreinforced concrete block partition walls	30 50	Out-of-plane loads. In-plane	N-S
South Wall Frame	6" unreinforced concrete block – south wall	25	In-plane shear capacity. Out-of-plane capacity is > 67%NBS.	E-W
	M22 vertical cross bracing	25	Axial capacity in tension; all braced bays	E-W
	Strut (7"x4" RSJ) along the top of the portal frame	64-80	Axial capacity in compression. Struts in interior bays < 67%NBS.	E-W
	Strut (6"x3" RSJ), along mid-height of portal column	45-67	Axial capacity in compression. Struts on the braced bays have a capacity of >34%NBS and the rest are >67%NBS.	E-W
North Wall Frame	Strut (7"x4" RSJ)	35	Axial compression. Involves 6 interior struts from grids 4-10.	E-W
	6" unreinforced concrete block wall	30	Out-of-plane and in-plane	E-W
Foundation	3'x3'x10" RC square footing	40	Bearing, structural capacity is > 67%NBS	N-S/E-W
	2.5'x2.5'x10" RC square footing	40	Bearing, structural capacity is > 67%NBS	N-S/E-W

The outcome of this assessment indicates that the existing Surrey Park Grandstand has a seismic capacity of less than 34%NBS (IL3) in the north-south (transverse direction) and east-west direction (longitudinal direction). The assessed seismic capacity is governed by the in-plane and out-of-plane resistance of the unreinforced infill concrete block and M20 cross bracing which is considered the critical structural weaknesses (CSW's) of this building. Failure of these elements will not lead to a progressive collapse because of the available alternative load path (frame action); however, collapse of the infill walls may potentially pose a threat to the life safety of the building occupants.

The assessed capacity is based on the undamaged state of the building and the steel connections were assumed to be adequate to develop the member capacity. While deterioration is evident and requires repair, from what is visible it is not considered that it is substantial enough to notably affect the building's seismic capacity; however, confirmation of this is required.

WSP Opus recommends strengthening the building to at least 67%NBS. The required strengthening works shall include the following:

- Replace the cross bracing with a section of higher tensile capacity. Use of equal angles (EA) is considered a feasible replacement for inadequate cross bracing.
- Replace the existing infill concrete block walls with reinforced concrete block construction. Provide continuous strip footing anchored to the existing slab on grade and footing. Note that this solution will also improve the capacity of the existing foundation.
- Replace the struts with a section of higher capacity. Use of square hollow section (SHS) is considered a feasible replacement option for struts.
- Install fly bracing on the roof frames to ensure that the frames' flexural capacity will not be governed by flange buckling.

We have prepared a conceptual sketch of the proposed strengthening with preliminary sizes for scoping and costing purposes. This is presented in Appendix C. The sizes, quantities, and arrangement shall be finalized during the detailed design.

Furthermore, cleaning and painting of all steelwork are recommended to manage future durability of the structure. The accurate extent of this work can be determined following the completion of the detailed investigations outlined below.

Prior to the detailed design, we recommend that an invasive site investigation is conducted to further check the following:

- Concrete block wall construction. Need to confirm or otherwise if the existing concrete block work is unreinforced;
- Corrosion. This will determine the extent of corrosion, corrosion-induced structural damage, and an evaluation of the building's residual life if required. This should include a factual report and recommendations outlining the repair options.
- Site-specific geotechnical investigation and interpretative report. This is necessary to confirm ground conditions and potential geotechnical hazards.

1 General

1.1 Scope

WSP Opus has been commissioned by the Invercargill City Council (ICC) to carry out a Detailed Seismic Assessment (DSA) of the existing Surrey Park Grandstand located in Invercargill. The building is considered to have a high consequence of failure as it will affect crowds and therefore has been assessed as an Importance Level 3 (IL3) building in accordance with the New Zealand Building Code (NZBC).

ICC has previously engaged WSP Opus to review the Structural Engineering Report for the Surrey Park Grandstand completed by Kensington Consulting Ltd. (KCL) in February 2013. The report provided a score of 22%NBS based on Initial Evaluation Procedure (IEP), which KCL upgraded to the order of 34%NBS due to a more detailed assessment of the seismic capacity. The outcome of the WSP Opus review, based on an IEP, indicated that the Surrey Park Grandstand had a seismic capacity of 30%NBS(IL3) and was considered to be a potentially Earthquake Prone Building (EPB).

The purpose of the DSA is to determine the performance of the building and compare it with the requirements provided in the NZBC for a new building. The comparison is expressed as a percentage of the New Building Standard (%NBS). The DSA also aims to confirm or otherwise that the Surrey Park Grandstand fulfils the requirements for the Territorial Authority to consider the building to be an EPB in terms of the Building Act 2004.

The scope includes the following tasks:

- A desktop review of relevant available information;
- Walk through non-intrusive site inspection of the building;
- Detailed structural assessment to determine the percentage of the New Building Standard (%NBS) of the building based on Importance Level 3 (IL3);

Assessment of the seismic capacity of the non-structural components of the building is not included in this DSA. Likewise, assessment of the geotechnical hazards such as liquefaction, lateral spreading, soil instability, and soil deformability are not part of the scope.

This report describes the methodology used in the assessment and presents the results of the DSA.

1.2 Compliance

This section contains a summary of the requirements of the various statutes and authorities that control activities in relation to earthquake-prone buildings within the country at present.

1.2.1 Building Act 2004

This section incorporates the new Building (Earthquake-prone Buildings) Amendment Act 2016 that came into effect from 1 July 2017. Several sections of the Building Act are relevant when considering structural requirements. The following provides an overview of sections typically relevant to the assessment of existing buildings.

Section 112 - Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to the alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 133AB – Meaning of earthquake-prone buildings

This section defines a building or a part of a building as earthquake-prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. The respective territorial authority determines whether a building or part of a building is earthquake-prone.

Section 133AE Meaning of priority building

This section provides the definitions of priority buildings which generally includes buildings that are needed during emergency response; occupied for educational purposes as well as any unreinforced masonry building on busy thoroughfares with falling hazards. It also applies to buildings on strategic routes.

Section 133AF Role of territorial authority in identifying certain priority building

The section requires the territorial authorities within the medium and high seismic risk zones to identify priority buildings in their districts.

Section 133AG Territorial authority must identify potentially earthquake-prone buildings

This section requires the territorial authorities to identify buildings or parts of buildings that are potentially earthquake-prone within the set time frame for each seismic risk area starting from 1 July 2017, which is the commencement date on which this section of the Act comes into force.

Section 133AH Territorial authority must request engineering assessment of potentially earthquake-prone buildings

For buildings that are identified as potentially earthquake-prone, the TA must ask the building owner to provide an engineering assessment.

Section 133AI Obligations of owners on receiving a request for engineering assessment

Upon notified by the TA, the building owner must obtain an engineering assessment of the building and submit to the TA within 12 months.

Section 133AK Territorial authority must determine whether the building is earthquake-prone

Based on the engineering assessment, the TA must determine whether the building or a part of the building is earthquake-prone. If it is earthquake prone, the TA must issue an EPB notice and publish the building information on the EPB register.

Section 133AM Deadline for completing seismic work

This section defines the time frame that the owner of the building subject to EPB notice has to complete seismic strengthening work. The time frame depends on the seismic risk zone and whether the building is considered a priority building or not and can range between 7.5 years and 35 years.

1.2.2 The Seismic Assessment of Existing Buildings

The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments July 2017 (the Guidelines) provides a technical basis for engineers to carry out seismic assessments of existing buildings within New Zealand. The Guidelines support seismic assessments for a range of purposes. It informs local authority's decision-making on whether or not a building is earthquake-prone in terms of the Building Act 2004 and for property risk identification generally.

The principal outcome of a seismic assessment using the Guidelines is a rating expressed as a percentage of the new building standard that applies to an equivalent new building on the same site.

The Guidelines supersede the previous assessment guidance published in 2006 by NZSEE (The Assessment and Improvement of the Structural Performance of Buildings in Earthquakes).

1.3 Means of Compliance

The detailed seismic assessment of the existing building has been undertaken in compliance with the following standards and guidelines:

- AS/NZS1170, Parts 0,1, and 5 Structural Design Actions
- NZS3101:2006 Concrete Structures Standard
- NZS3404:1997 Steel Structures Standard
- The Seismic Assessment of Existing Buildings (sub-Part C where relevant):
  - Part A- Assessment, Objective and Principles;
  - Part C – Detailed Seismic Assessment
    - C1 – General Issues
    - C2 – Assessment Procedures and Analysis Techniques
    - C3 – Earthquake Demands
    - C4 – Geotechnical Considerations
    - C5 – Concrete Buildings
    - C6 – Structural Steel Buildings
    - C7 – Moment Resisting Frames with Infill Panels
    - C8 – Unreinforced Masonry (URM) Buildings
    - C9 – Timber Buildings
    - C10 – Secondary Structural and Non-structural Element

1.4 Limitation of the Report

The structural assessment is based on the limited information provided by the building owner/representative and on a limited visual inspection of the building. No destructive tests or specific geotechnical investigations have been undertaken at this stage.

The analysis and assessment are based on the building in its undamaged state. However, there may be deficiencies in the building that could cause the capacity of the building to be reduced. In this case, the current capacity of the building may be lower than that stated in this report.

Despite the use of best national and international practice, the analysis outcomes are affected by a certain level of uncertainty due to some assumptions and simplifications made during the assessment process. These include:

- Simplifications introduced in the analysis and numerical modeling of the existing structure, including boundary conditions (e.g. foundation fixity, soil stiffness, etc.);
- Assessment of material strength based on the indication provided in The Seismic Assessment of Existing Buildings guidelines;
- Approximation in the assessment of the post-yield behavior of structural elements;
- Assessment is based on the undamaged condition of the building;
- Site soil condition and properties.

The assessment is limited to Section B1 Structure of the New Zealand Building Code (NZBC), and no assessment of the compliance of other sections of the NZBC has been undertaken. Also, the assessment considers earthquake loads only; other loads such as wind and snow are not covered.

2 Background Information

2.1 Source of Geotechnical Data

No specific geotechnical investigation has been carried out for this assessment. The geotechnical information used as a basis in the assessment was adopted from published reports.

2.2 Source of Building Data

The Detailed Seismic Assessment (DSA) has been carried out based on the information available in the record drawings and information gathered during the site inspection.

2.2.1 Original Documentation

- Incomplete architectural and structural drawings, September 1964, Moir New & Jenkins Consulting Civil and Structural Engineers, and Smith Rice Lawrence & Mollison
- Initial Seismic Assessment Review and Condition Assessment, 16 August 2018, WSP Opus

2.2.2 Site Inspection

A visual non-intrusive site inspection was undertaken by WSP Opus on the on 15th of October and 6th of November 2018. The site inspections included a walk-through observation of the interior and exterior of the building.

The site inspection confirmed the general layout of the building as illustrated in the structural drawings. A minor discrepancy with the record drawings was noted in the building and considered to have no significant impact on the DSA.

3 The Site

3.1 Topography

The Surrey Park Grandstand is sited on a generally flat area with no tributaries in the immediate vicinity of the site. The site is situated approximately 3km west of the Waihopai River and approx. 900m north of Otepuni Stream.

Entry to the site is through Isabella Street via Surrey Park Rd (Figure 3-1).

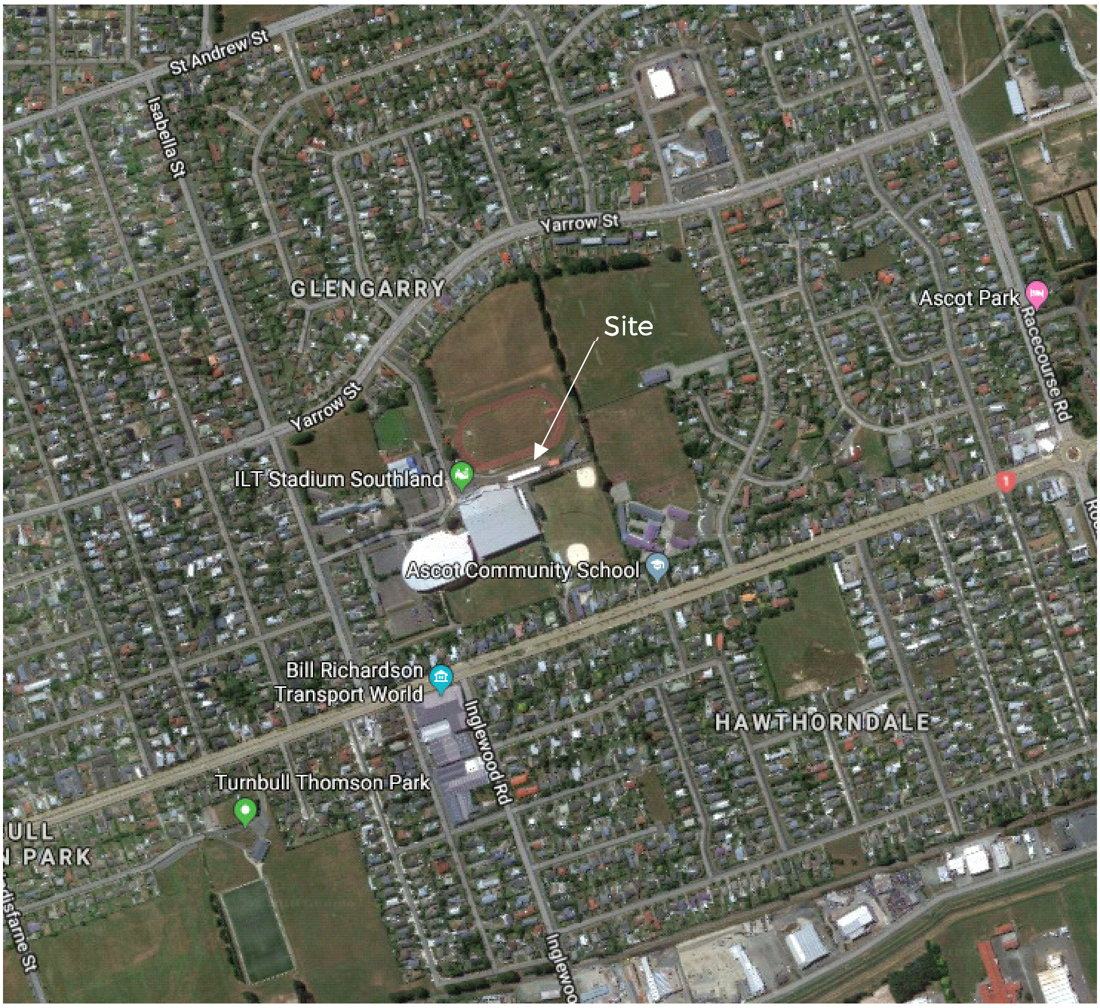


Figure 3-1 : Aerial View of the Site (source: Google Maps)

3.2 Regional Geology

Invercargill City is located adjacent to the New River Estuary. It is for the most part built on Holocene estuarine silts and river alluvium and older Quaternary gravels. The geological map of Invercargill shows that it is generally underlain with slightly weathered to weathered sand and gravels and weathered clayey sandy gravels, (Figure 3-2).

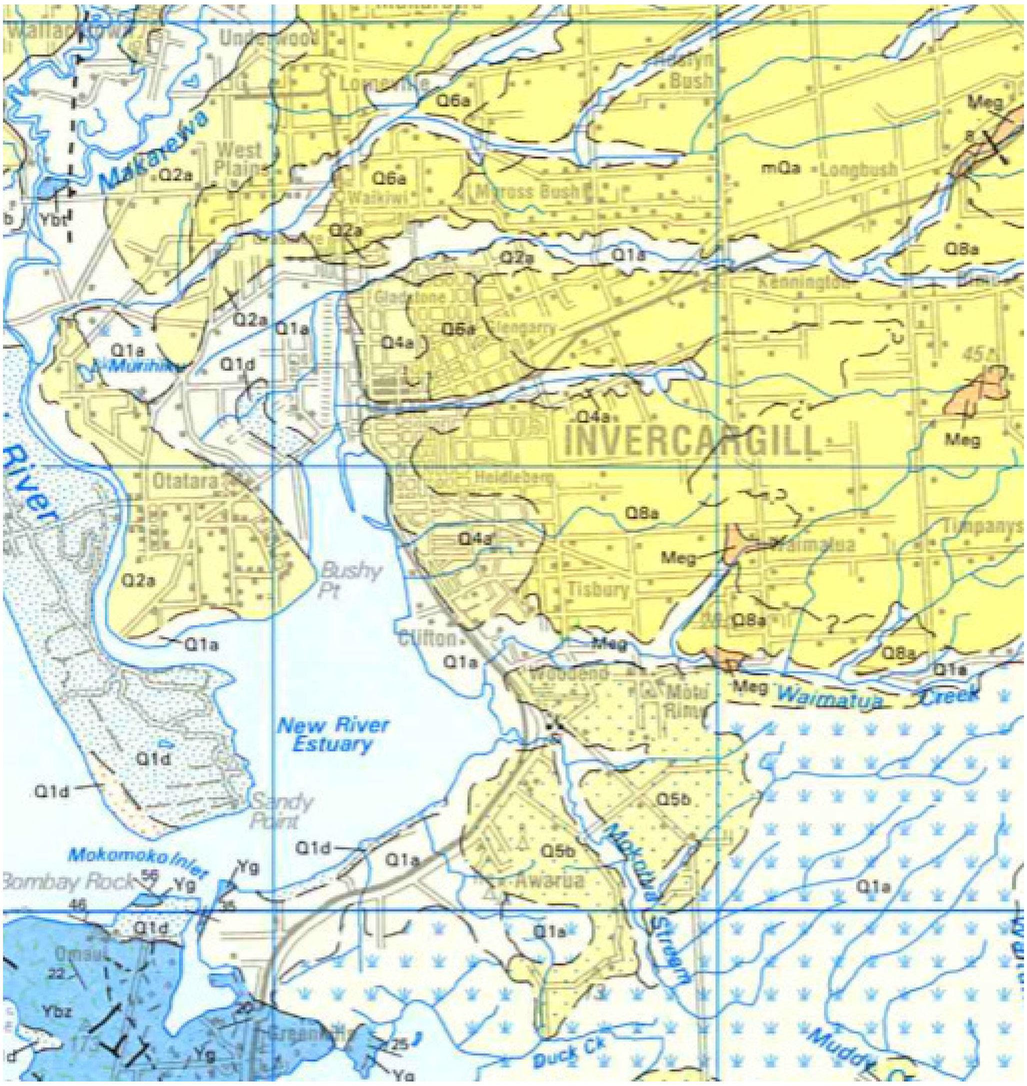


Figure 3-2 : Invercargill Geological Map *(source: GNS)*

The site is situated in an area marked Q8a which is described as weathered clayey sand and gravel in high terraces remnants; weathered clayey sandy gravels in associated fan remnants.

3.3 Seismicity

New Zealand overlaps the boundary between the Australian and Pacific tectonic plates. The movement of the subducting Australian plate and the overlying Pacific plate is the source of frequent earthquakes in the subduction zone. The New Zealand Standards, NZS 1170.5, has assigned Invercargill with a hazard factor (Z) of 0.17g.

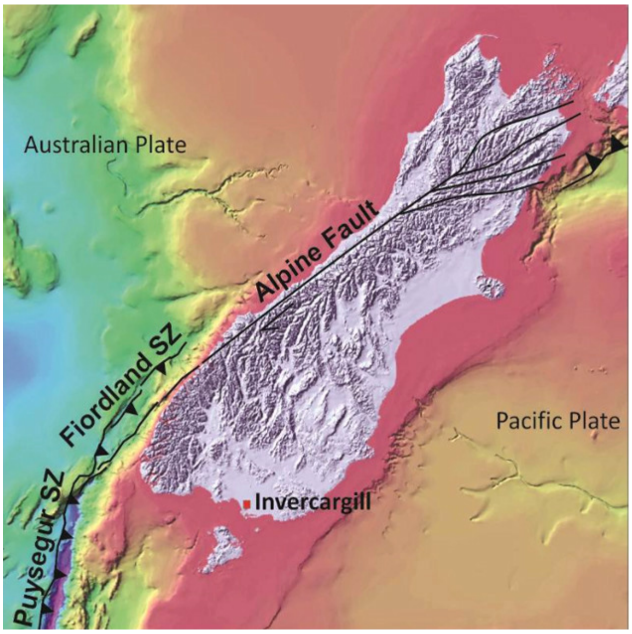


Figure 3-3 : Map of South Island of New Zealand showing the tectonic plate boundary (source: www.es.govt.nz)

3.4 Ground Condition

The Invercargill City lies within an area underlain with Deep or Soft Soil or Very Soft Soil. These soil types, depending on the magnitude and distance of the earthquake source, are susceptible to ground shaking amplification and liquefaction.

Based on the GNS report titled "Amplified ground shaking and liquefaction susceptibility, Invercargill City", dated January 2012, the site appears to be situated on an area of Deep or Soft Soil, which is considered consistent with a Class D soil in accordance with NZS 1170.5. The GNS report also indicates that the site is within an area of low to negligible liquefaction susceptibility (Figure 3-4).

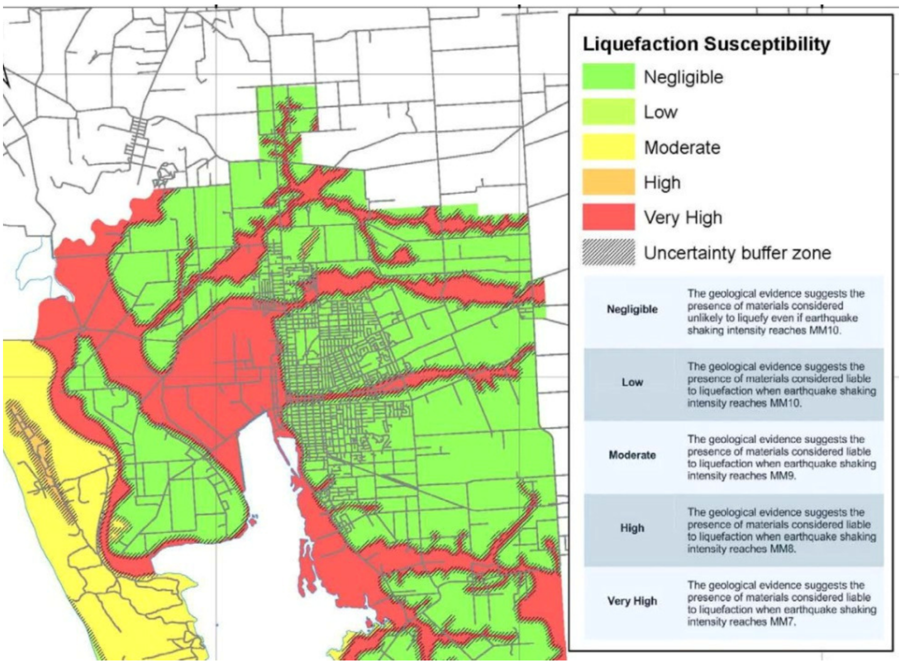


Figure 3-4 : Liquefaction Susceptibility Invercargill City (source: GNS Report dated January 2012)

4 The Structure

4.1 Building Description

The Surrey Park Grandstand has a footprint area of 55m x 8m wide and has a seating capacity of approx. 850. The building was built circa 1964 and is still currently in use.

It is effectively consisting of two structural forms, each with a significantly different stiffness that will affect how the building performs under lateral loading from wind and earthquake loads. The upper portion of the building (roof section and bleacher walls) comprises lightweight construction. The lower part, bleacher support section, is of more rigid construction consisting of concrete encased structural steel sections, precast concrete bleachers, and unreinforced concrete block walls.

The lower section of the grandstand seating area is constructed of inclined steel portal frames running transversely which support precast concrete bleachers. The legs of the portal frames are encased in concrete to three-quarter height on the south wall and up to the knee on the north wall. Bracing is provided by steel cross bracing in the north internal wall and the south wall. The cross bracing in the south wall does not extend to ground level but terminates above the windows on the south side. There is infill hollow concrete masonry between the portal legs below the windows. This masonry has a bond beam along the top edge but otherwise is not reinforced or filled. The north wall is low beneath the front set of bleachers and consists of a reinforced concrete retaining wall. The north wall of the lower seating section of the grandstand is short and rigid while the south wall is taller with cross bracing that does not extend to ground level and hollow masonry veneer that will provide minimal lateral restraint.

4.2 Primary Gravity Load Resisting System

The primary gravity load resisting elements comprise roof steel frames, precast concrete bleachers, and steel portal frames.

The gravity loads on the roof are supported by angular (4x3 UA) steel purlins which transfer the loads to the steel frames which then transfer the loads to the foundation.

The precast concrete bleachers support the gravity loads and transfer them to the steel portal frames which then transfer the loads to the foundation.

4.3 Primary Lateral Load Resisting System

The building's primary lateral load resisting system (LLRS) comprises steel portal frames in the north-south direction and steel bracing in conjunction with infill concrete block walls in the east-west direction. These LLRS provides overall building's stability against the lateral loads (wind and seismic).

The roof bracing system (tension-only and struts) acts as a flexible diaphragm and transfers the roof inertial forces to the rear wall (south wall) in the east-west direction. In the north-south direction, the inertial forces on the roof are resisted by the cantilever action of the steel frames.

4.4 Site Inspection and Building Condition

A site visual inspection was carried out by WSP Opus Engineers on 15th October and 6th November 2018. The inspection involved a walk-through observation of the interior and exterior of the building. Minor discrepancies between the structure and the record drawings were noted during the inspection, as follows:

- Eccentricity between the vertical bracing and concrete block wall exist at the south elevation;
- Some of the interior concrete blocks wall shown on the record drawings do not exist.
- The vertical bracing at the north wall (3"x1/4" flat) do not exist and appear to have been replaced with a concrete block wall. 6" unreinforced concrete block walls were observed on five bays with each bay has door opening.

It is apparent that the building has substantial corrosion to structural steel members. Some of the secondary steel sections that support the bleacher seating were observed with corrosion (Photo 1). Cracking to the encasement of the steel portal frame were observed; presumably caused by corrosion development triggered by water ingress on the top of concrete encasement (Photo 2). The level of corrosion at this stage is not considered to be significant in terms of reducing the seismic capacity at this stage. However, the affected steel requires cleaning back and repainting to ensure that the level of corrosion does not cause further deterioration that may lead to an issue of structural stability. Where damaged the concrete encasement of the steel columns should be removed with the structural steel inspected prior to a repair solution being specified to remediate. During the inspection it was noted that patch-repair approach has been done in some locations, particularly to the base of the SHS posts supporting the grandstand roof, however, we are unaware of the process followed when this was completed. If corrosion has not been adequately treated, the underlying problem and corrosion will continue that could potentially create risks (durability and stability) to the structure.

Photo 1: Corroded Steel Section

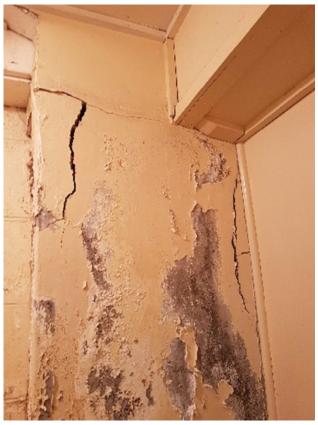


Photo 2: Cracking to Concrete Encasement

The Kensington Consulting has completed an initial structural assessment (ISA) in February 2013 and identified issues on the condition of the building concerning corrosion of the steelwork and cracking of concrete. Minor cracking of the concrete encasement around the main steel portal frames at the north wall was noted. They recommended that the observed cracks be monitored.

In August 2018, the ICC engaged the services of WSP Opus to review the Structural Engineering Report by KCL dated February 2013. As part of the review, WSP Opus conducted a site visual inspection and noted that the condition of the building appeared to have worsened since the KCL report. Significant cracking in the concrete encasement and rusting of steel was observed. For further details of the outcome of initial condition assessment, refer to WSP Opus report titled "Initial Seismic Assessment Review and Condition Assessment", dated 16 August 2018.

It was clear from our inspection that substantial deterioration still exists. Although some repairs appear to have been completed at some locations, the implemented repairs appear to be cosmetic.

4.5 Alterations

Concrete block partition walls have been added to the building since the original construction. There has also been the addition of access stairs to the front of the stadium at the east and west ends of the grandstand. No details of these alterations were available.

4.6 Assumptions

The following assumptions were made during the process of this assessment. These assumptions are due to the unclear and/or limitation of information on the building and are based on the best judgment of the author(s). The results of this report are subject to the accuracy of these assumptions:

- The building was constructed in accordance with the information available in the record drawings.
- The soil within the site is adequate to support all the loads imposed by the building to its foundation. The ultimate bearing capacity of soil was assumed to be 300kPa.
- Hidden damage/condition (if any) has no significant impact on the structural performance of the building.

Table 4-1: Material Properties

Year Built	Material	Lower Characteristic Strength	Probable Strength
1964	Concrete	20 MPa	30 MPa
	Reinforcement	230 MPa	300 MPa
	Structural Steel	230 MPa	250 MPa
	Masonry	12 MPa	12 MPa
	Masonry Grout	17 MPa	17 MPa

5 Detailed Seismic Assessment

5.1 Analysis Methodology

The New Zealand standard methodology for assessing the earthquake performance of existing buildings is specified in the guidelines prepared by the Ministry of Business, Innovation, and Employment and the New Zealand Society for Earthquake Engineering jointly with SESOC and NZGS. The force-based methodology described therein was used in this assessment.

The assessment has been carried out using a three-dimensional linear elastic model created in SAP2000. Steel portal frames, tension braces, and struts were modeled as line elements with properties consistent with that of the actual elements. All concrete block walls (exterior and interior partitions) and concrete walls were modeled as equivalent compression braces in accordance with the recommended procedure in the assessment guidelines. These equivalent compression braces are not shown in the figure below to avoid confusion as they look similar to the steel braces (Figure 5-1). Although steel columns underneath the bleachers are encased with concrete, the encasement was not considered in the model because they only prevent flange buckling and has no considerable impact in terms of the overall stiffness of the structure. The mass of the concrete encasement was captured via weight modifiers.

The model was used to simulate the combination of the design gravity loads and the expected earthquake loads.

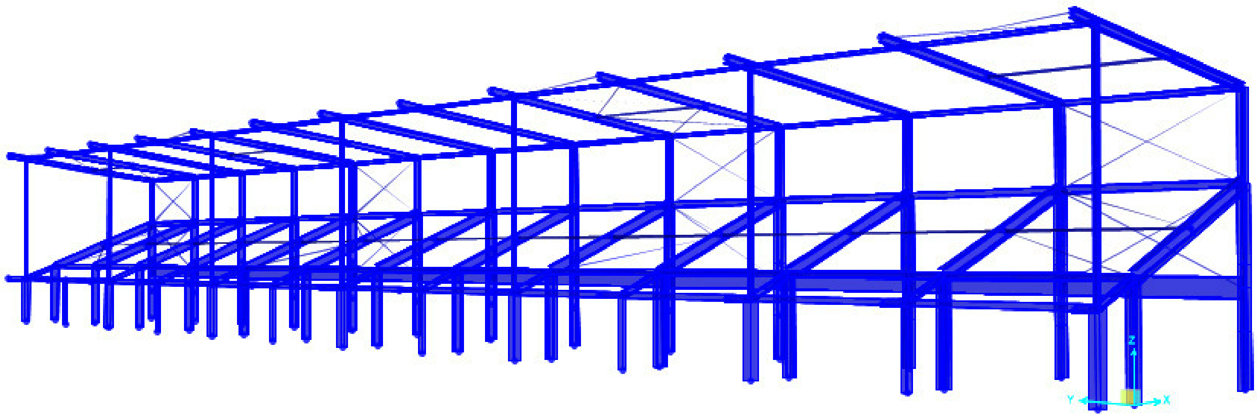


Figure 5-1 : Graphical Presentation of the 3D Analysis Model

5.2 Gravity Loads

The gravity loads include the self-weight of structural, superimposed dead loads, and occupancy live loads. The self-weight of the structural elements is software-generated and is based on the respective material unit weights. The superimposed dead loads and the live load were applied in the model as uniform distributed line loads.

5.3 Earthquake Loads

The expected displacement ductility of the system, building behavior under ground motion, and analysis technique were taken into consideration to determine the best approach in analyzing the earthquake demands. An equivalent static analysis (ESA) procedure has been utilized to determine the seismic demands on the building and the corresponding forces on the key structural elements. This procedure is appropriate for this building because it has a short structural period and its modal response is dominated by lower modes.

5.3.1 Acceleration

Acceleration spectra from the loading code, NZS1170.5, was reproduced and presented below. These include response spectrum curves ULS (1/1000-year event) and SLS1 (1/25-year event) developed based on assumed site soil class of D and the code minimum hazard factor (Z) of 0.30. The unreduced peak accelerations are 1.17g and 0.16g for ULS and SLS1 respectively for a building with a structural period less than or equal to 0.50sec.

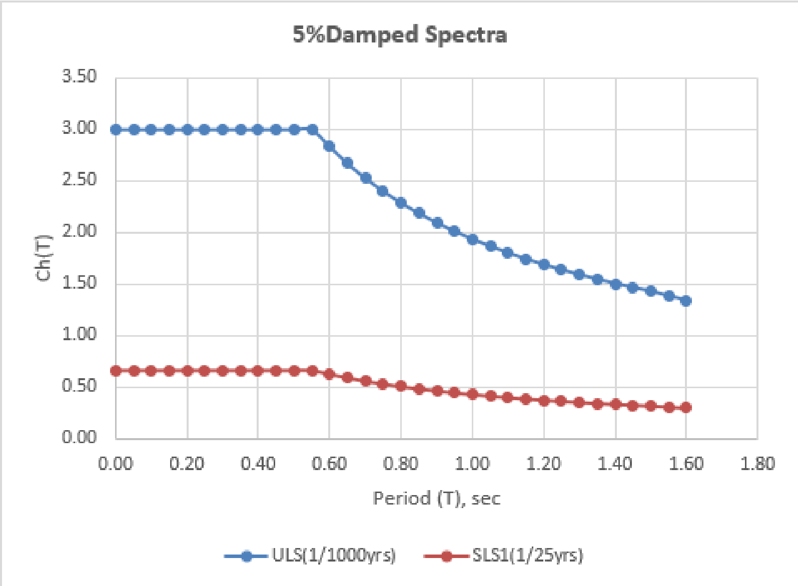


Figure 5-2: Response Spectrum Curve (NZS 1170.5)

5.3.1 Seismic Weight

The seismic mass comprises the permanent weight of the structure and a percentage of the occupancy live load. The seismic weight contributed by occupancy live load was estimated considering the applicable area reduction factor (μa) of 0.50 and a live load combination factor (μe) of 0.30 for the global analysis.

5.3.2 Specific Parameters for Seismic Analysis

The design seismic demand is not solely dependent on the ground acceleration on site. It is also influenced by parameters specific to the building. The table below presents some of these parameters.

Table 5-1: Specific Parameters for Seismic Analysis

Description	ULS (1/1000 years)	SLS1 (1/25 years)
System ductility, μ	1.25	1.0
Structural performance factor, Sp	0.925	0.7
Factored structural ductility, kμ	1.14	1.0
Horizontal Design Coefficient, Cd(T)	0.537	0.09

A system ductility of 125 has been adopted in the assessment because the structural system of the building and its member connections do not have sufficient capacity to resist inelastic deformation.

5.3.3 Seismic Base Shear

The seismic base shear is the product of the seismic weight, the acceleration corresponding to the structural period of the building, and the seismic parameters inherent to the building ductility. The building structural period is less than 0.50 secs in the orthogonal directions, hence the corresponding base shears are 1500 kN (ULS) and 250 kN (SLS1).

6 Results of the Detailed Seismic Assessment

6.1 Steel Portal Frame (Bleacher)

The steel portal frames are made of 14”x6 ¾”x30# UB. These frames are the main gravity support system and provide the main lateral load resisting elements in the north-south direction. The critical seismic demands on these frames were determined using the 3D analysis model and a 2D analysis model. Note that for the 2D model, an area reduction factor for live load was not applied.

The result of the assessment indicates that the portal frames have the following seismic capacities:

- Beams: 80%NBS (IL3) governed by the beam’s flexural capacity;
- Columns: 60-90%NBS (IL3) governed by the combined axial and flexural capacity.

The 60%NBS capacity of the columns is governed by the minor axis bending induced by the eccentricity between the vertical bracing (M20 tension brace) and concrete block walls.

The portal frames are not critical under SLS case.

6.2 Steel Portal Frame (Canopy)

The steel portal frames that support the canopy are made of 10”x5 ¾”x 21#UB (250UB31) column and 8”x5 ¼” x17# UB (200UB25) steel beam. These elements provide the main gravity support for the canopy and transmit the roof inertial loads to the steel portal frame underneath the bleacher. These frames provide lateral stability to the canopy structure in the east-west direction.

The assessed seismic capacity is as follows:

- Beams: 100%NBS (IL3) governed by flexure in combined minor and major axis bending;
- Columns: 70%NBS (IL3) governed by the combined axial and flexural capacity.

6.3 Steel Frame - South Elevation

The resistance of the south elevation frame in the east-west direction is provided by combined action of the tension-only cross bracing, compression struts, and infill concrete blocks.

The cross bracings are typically M22 round bars with upper and lower level horizontal struts made of 7”x4” RSJ and 6”x4” RSJ respectively. The infill walls are unreinforced and unfilled 6” thick concrete blocks with reinforced bond beams along the top.

The result of the assessment indicates the south elevation frames have the following seismic capacities:

- M22 Tension Bracing: 25%NBS (IL3) governed by the axial capacity in tension.
- 7”x4” RSJ strut: 64-80%NBS (IL3) governed by the axial capacity in compression.
- 6”x3” RSJ strut: 45%NBS (IL3) governed by the axial capacity in compression. These struts are located on the braced bay; the rest of the struts have capacity > 67%NBS.

- 6” Infill Concrete Block Wall: 25%NBS (IL3) in-plane shear and >67%NBS(IL3) out-of-plane.

6.4 Steel Brace - North Wall

There are five bays on the north wall with 6” concrete block wall. There are struts on all bay comprise of 7x4RSJ. These walls and struts help provide lateral stability to the bleacher in the east-west direction.

These elements have been assessed with the following seismic capacities:

- Strut (7”x4” RSJ): 35%NBS (IL3), governed by axial compression. These struts are located at five interior bays (grids 4-10); the rest of the struts have a seismic capacity of >67%NBS.
- The 6” unreinforced concrete blocks walls have been assessed with a seismic capacity of 30%NBS against both in-plane and out-of-plane seismic loads.

6.5 Roof Bracing - North Wall

There are two braced-bays on the north wall and these are comprised of M20 tension-only cross bracings and 2 ½” diameter (pipe ID) strut. These braces provide lateral resistance in an east-west direction.

These elements have been assessed with the following seismic capacities:

- Cross Brace (M20 round bar): 45%NBS(IL3) governed by axial capacity in tension.
- Strut (2 ½” ID pipe): 95%NBS (IL3), governed by axial capacity in compression.

6.6 Foundation

The building foundation comprises a reinforced square footing. These include 900x900x250mm and 750x750x250mm reinforced concrete footing.

These footings have been assessed with a seismic capacity of 40%NBS(IL3), governed by the bearing capacity of the supporting soil.

Strength check (shear and flexure) indicates that the footing has a strength greater than the demand.

6.7 Concrete Block Veneer

The unfilled concrete block veneer at the east elevation of the building that has been previously reported to be damaged and unstable was already repaired. This was not considered in the seismic rating of the building because it is a non-structural component and its failure will not affect the structural integrity of the building.

6.8 Bleacher Support

The concrete bleachers are supported directly on the steel portal frames fixed with M20 bolts through 12mm thick steel cleats. The bleacher’s inertial force is transferred to every steel frame in the north-south direction and to the alternate frames in the east-west direction; this is made possible via slotted bolt holes on alternate frames.

The bleacher connections (bolts and cleats) have been assessed with a seismic capacity of greater than 67%NBS. The condition of the bleacher supports should be checked as part of the recommended invasive condition and corrosion assessment.

6.9 Summary of Assessed Seismic Capacity

Table 6-1 : Assessed Seismic Capacity (%NBS) of Key Elements.

Location	Element	%NBS (IL3, ULS)	Mode of Failure/Remarks	Direction
Canopy Structure	M16 Cross Bracing	40	Axial tension	E-W
	2 ½" ID Pipe Strut	85	Axial compression	E-W
	M20 wall cross bracing	55	Axial tension, interior bays <67%NBS and end bays >67%NBS	E-W
	Roof Beam (8"x5 ¼" x 17# UB)	100	Flexure	N-S
	Rear Wall Column (10"x5 ¾"x21# UB)	70	Combined axial and bending	N-S
	Roof strut (5"x2" RSC) along the top of the south wall	70	Axial compression	E-W
	4"X4" Canopy Columns	67	Combined axial and bending	E-W
Bleacher Structure	Beam (14"x6 ¾"x30#UB)- Steel Portal Frame	80	Combined axial and bending	N-S
	Column (14"x6 ¾"x30#UB)- Steel Portal Frame	60-90	Combined axial and bending. Eight columns <67%NBS.	N-S
	M19 Bleacher cross bracing	30-64	Axial capacity in tension. The cross bracing at the end bays have capacity < 34%NBS and the rest have capacity >34% but <67%NBS.	E-W
	Bleacher Strut (2.5ID Pipe)	45	Axial capacity in compression. The struts located at the braced bays have a capacity of 45%NBS and the rest of the struts have capacity > 67%NBS.	E-W
	6" unreinforced concrete block partition walls	30 50	Out-of-plane loads. In-plane	N-S
South Wall Frame	6" unreinforced concrete block – south wall	25	In-plane shear capacity. Out-of-plane capacity is > 67%NBS.	E-W
	M22 vertical cross bracing	25	Axial capacity in tension; all braced bays	E-W
	Strut (7"x4" RSJ) along the top of the portal frame	64-80	Axial capacity in compression. Struts in interior bays < 67%NBS.	E-W
	Strut (6"x3" RSJ), along mid-height of portal column	45-67	Axial capacity in compression. Struts on the braced bays have a capacity of >34%NBS and the rest are >67%NBS.	E-W
North Wall Frame	Strut (7"x4" RSJ)	35	Axial compression. Involves 6 interior struts from grids 4-10.	E-W
	6" unreinforced concrete block wall	30	Out-of-plane and in-plane	E-W
Foundation	3'x3'x10" RC square footing	40	Bearing, structural capacity is > 67%NBS	N-S/E-W
	2.5'x2.5'x10" RC square footing	40	Bearing, structural capacity is > 67%NBS	N-S/E-W

6.10 Critical Structural Weakness (CSW)

In accordance with the seismic assessment guidelines, the lowest scoring structural weakness determined from a DSA is considered to be a critical structural weakness (CSW). The concrete infill block walls at the south wall and the M20 cross bracing at the bleacher and south wall locations have been assessed with the lowest seismic score of less than 34%NBS (IL3) and therefore considered the CSW's of this building. Failure of these elements will not lead to a progressive collapse because of the available alternative load path (frame action); however, collapse of the infill walls may potentially pose a threat to the life safety of the building occupants.

7 Conclusion and Recommendations

The seismic capacities have been assessed in accordance with the guideline document titled "The Seismic Assessment of Existing Buildings -Technical Guidelines for Engineering Assessments", dated July 2017. The earthquake rating is based on the building being Importance Level 3 (IL3).

The outcome of this assessment indicates that the existing Surrey Park Grandstand has a seismic capacity of less than 34%NBS (IL3) in the north-south (transverse direction) and east-west direction (longitudinal direction). The assessed seismic capacity is governed by the in-plane and out-of-plane resistance of the unreinforced infill concrete block and M20 cross bracing which is considered the critical structural weaknesses (CSW's) of this building. Failure of these elements will not lead to a progressive collapse because of the available alternative load path; however, collapse of the infill walls may potentially pose a threat to the life safety of the building occupants.

The assessed capacity is based on the undamaged state of the building and the steel connections were assumed to be adequate to develop the member capacity. While deterioration is evident and requires repair, from what is visible it is not considered that it is substantial enough to notably affect the building's seismic capacity; however, confirmation of this is required.

WSP Opus recommends strengthening the building to at least 67%NBS. The required strengthening works shall include the following:

- Replace the cross bracing with a section of higher tensile capacity. Use of equal angles (EA) is considered a feasible replacement for inadequate cross bracing.
- Replace the existing infill concrete block walls with reinforced concrete block construction. Provide continuous strip footing anchored to the existing slab on grade and footing. Note that this solution will also improve the capacity of the existing foundation.
- Replace the struts with a section of higher capacity. Use of square hollow section (SHS) is considered a feasible replacement option for struts.
- Install fly bracing on the roof frames to ensure that the frames' flexural capacity will not be governed by flange buckling.

We have prepared a conceptual sketch of the proposed strengthening with preliminary sizes for scoping and costing purposes. This is presented in Appendix C. The sizes, quantities, and arrangement shall be finalized during the detailed design.

Furthermore, cleaning and painting of all steelwork are recommended to manage future durability of the structure. The accurate extent of this work can be determined following the completion of the detailed investigations outlined below.

Prior to the detailed design, we recommend that an invasive site investigation is conducted to further check the following:

- Concrete block wall construction. Need to confirm or otherwise if the existing concrete block work is unreinforced;
- Corrosion. This will determine the extent of corrosion, corrosion-induced structural damage, and an evaluation of the building's residual life if required. This should include a factual report and recommendations outlining the repair options.
- Site-specific geotechnical investigation and interpretative report. This is necessary to confirm ground conditions and potential geotechnical hazards.

Appendix A

Appendix A: Assessment Summary Report Template

A1 Building Information

Gatehouse Building	
Building Name/Description	Surrey Park Grandstand
Street Address	
Territorial Authority	Invercargill City Council
Year of Design (approx.)	1964
No. of Storeys	2-Storey
Gross Floor Area (m2)	Ground Floor: approx. 472 m2 Bleacher Floor: approx.472 m2
Building Use:	Grandstand
Construction Type	▪ Foundation system: Reinforced concrete foundation beams ▪ Ground Floor: Concrete slab-on-grade, unreinforced concrete block walls ▪ Bleacher: precast slab/seating, steel portal frames ▪ Roof: Lightweight roof cladding supported by cold-formed steel purlins and steel frames
Adjacent Structures	None
Key Features of Ground Profile and Identified Geohazards	No specific geotechnical assessment carried out for this building.
Previous Strengthening/ Significant Alterations	None. Although there is a number of concrete block partition walls that have been added at some point.
Heritage Issues/Status	None
Other Relevant Information	None

A2 Assessment Information

Assessment Information	
Consulting Practice	WSP Opus
CPEng Responsible, including: • Name • CPEng Number • A statement of suitable skills and experience in the seismic assessment of existing buildings	Jamie Lester CPEng # 260347 CPEng Practice Area includes seismic assessment
Documentation available:	▪ Architectural/Structural drawings (refer to Appendix B)
Geotechnical Report(s)	No specific Geotech report for this building
Date(s) Building Inspected and Extent of Inspection	15 October 2018 and 06 November 2018, visual and non-intrusive
Description of any Structural Testing Undertaken and Results Summary	None
Previous Assessment Reports	None
Other Relevant Information	None

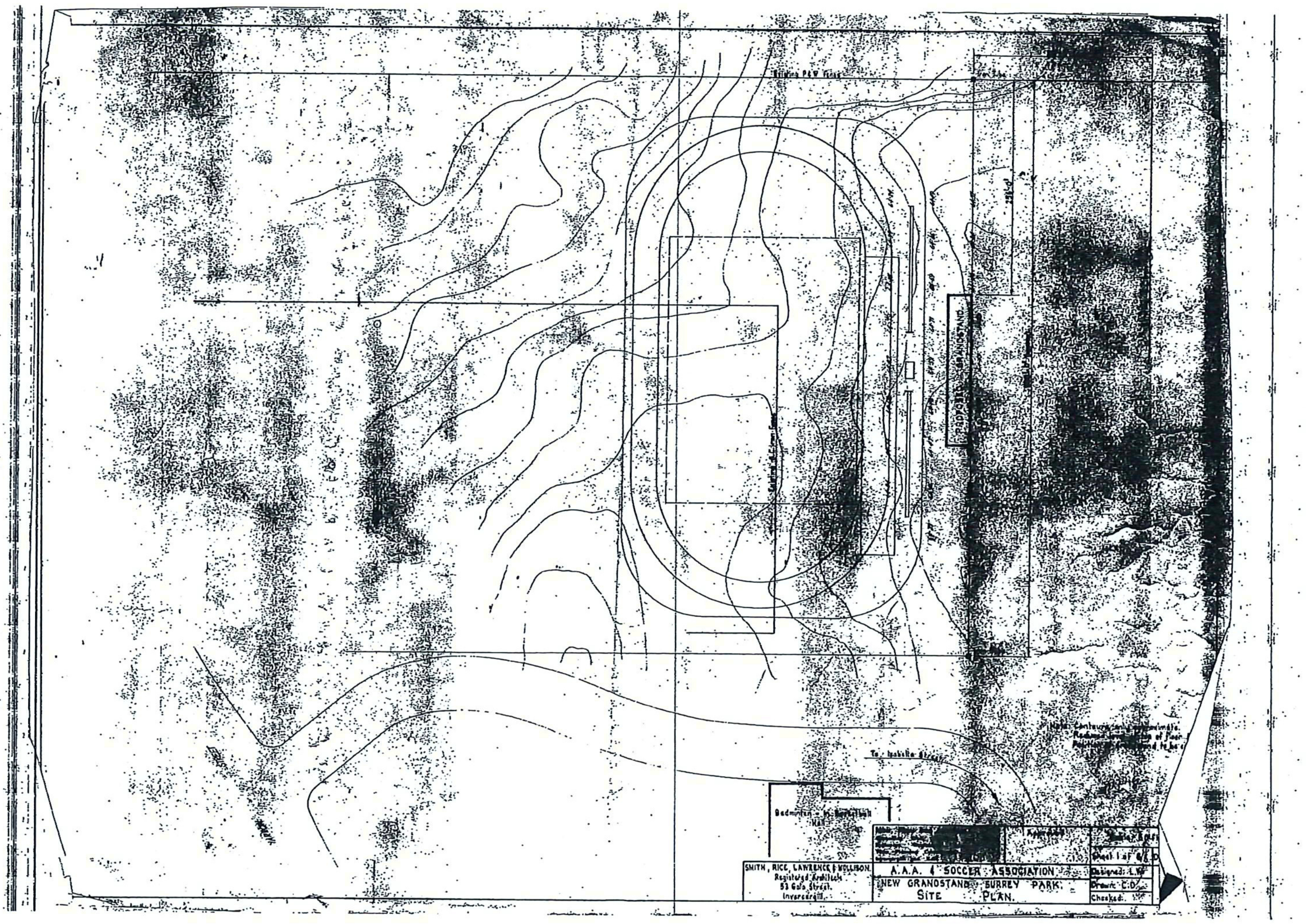
A3 Summary of Engineering Assessment Methodology and Key Parameters Used

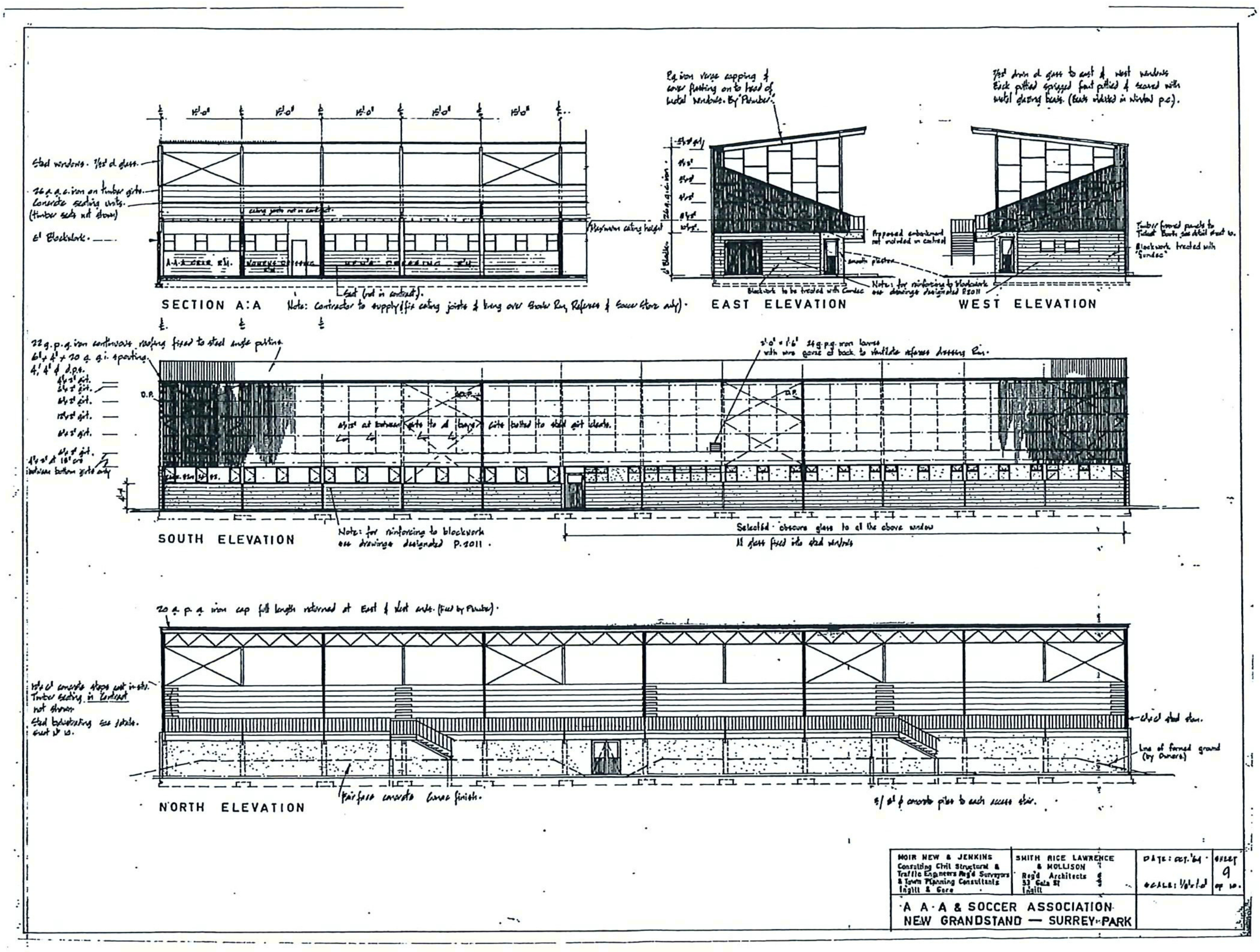
Methodology and Key Parameters	
Occupancy Type (s) and Importance Level	Grandstand Importance Level 3 (ULS)
Site Subsoil Class	D (assumed, based on the desktop review)
For an ISA:	NA
Summary of how Part B was applied, including: Key parameters such as μ, Sp and F factors Any supplementary specific calculations	NA
For a DSA:	
Summary of how Part C was applied, including: the analysis methodology(s) used from C2 other sections of Part C applied	Equivalent Static Analysis (ESA) Sections of Part C used include Sections C1, C2, C3, C5, C6, C7, and C8.
Other Relevant Information	NA

Appendix B

Appendix B: Record Drawings

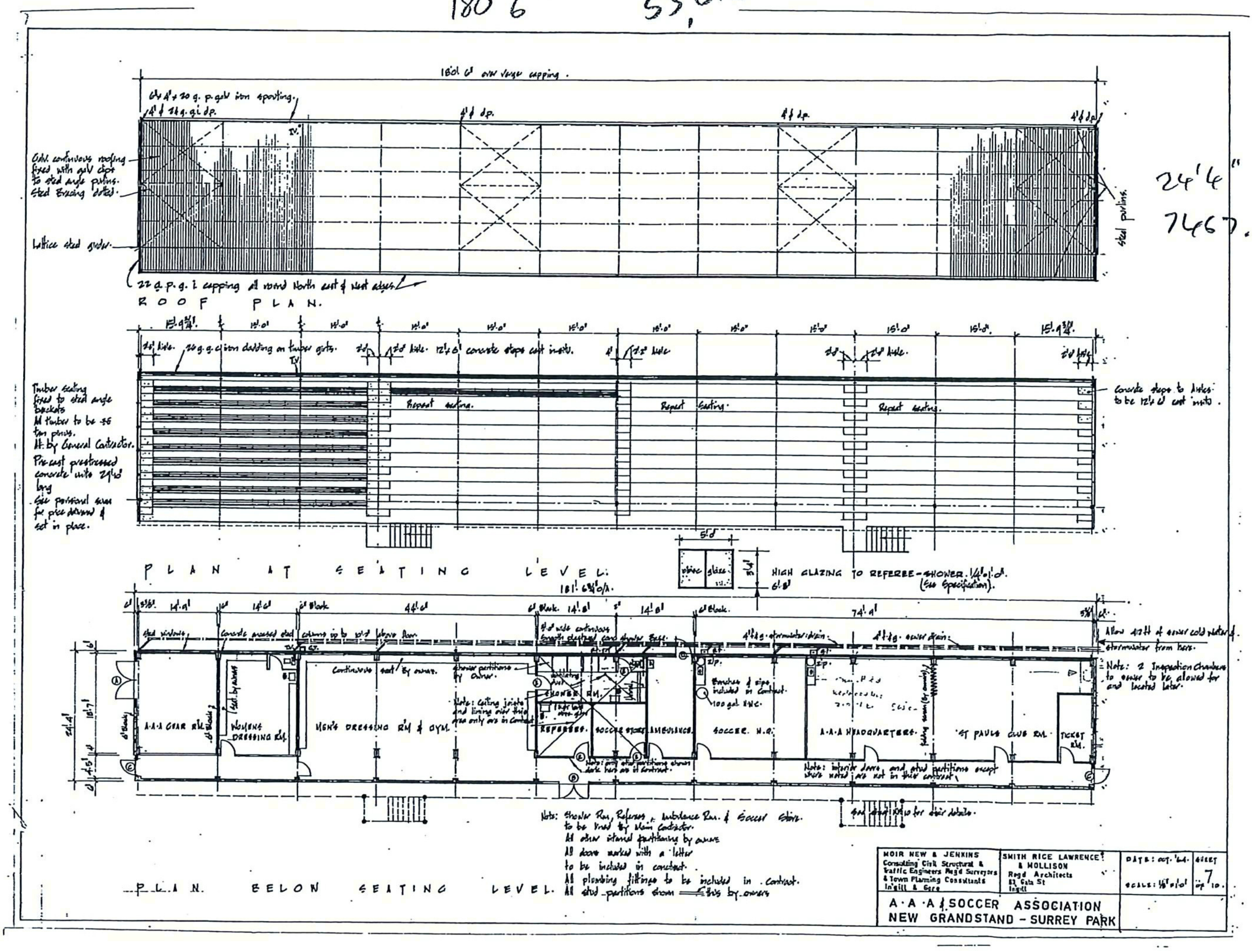
<u>Record Drawings</u>

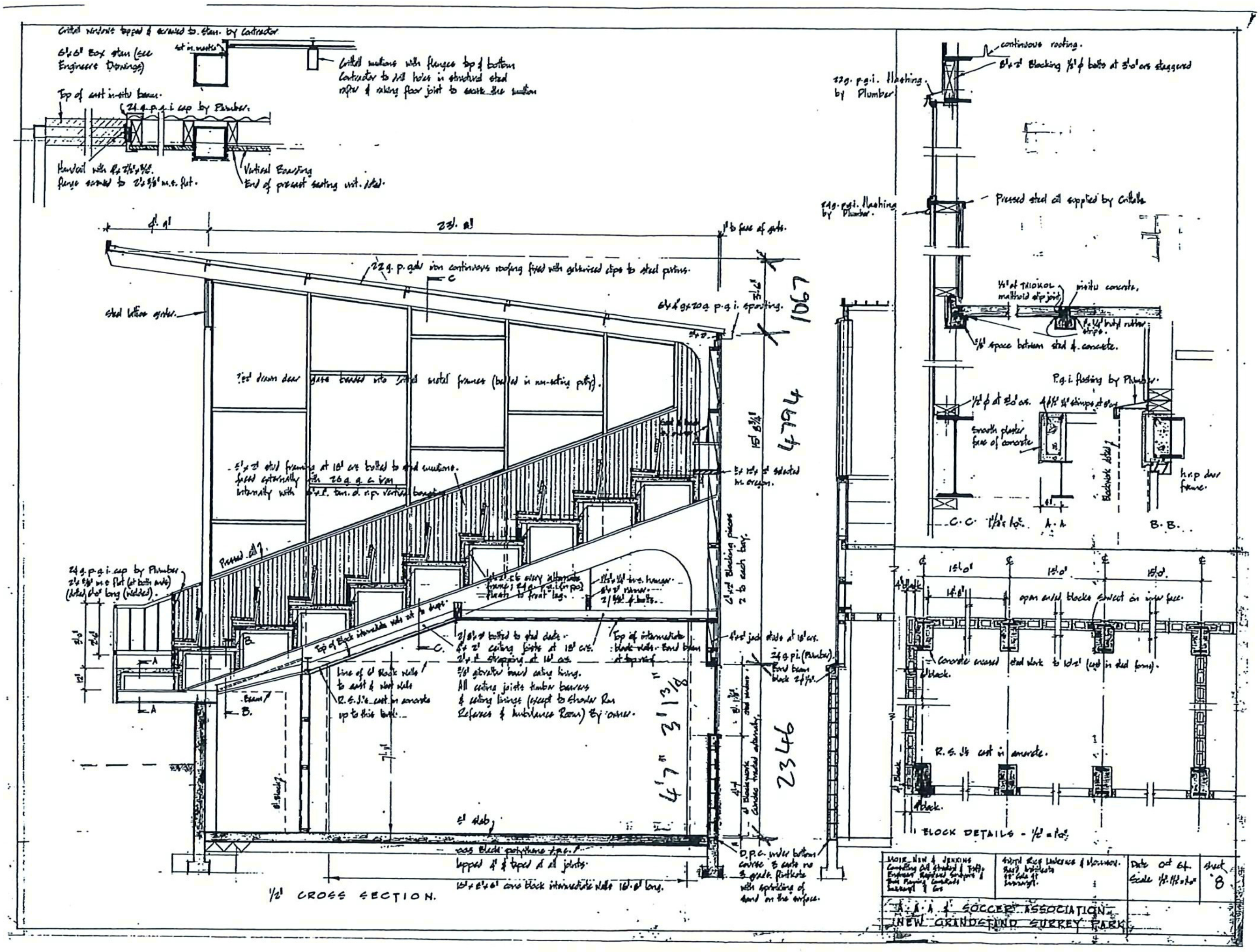


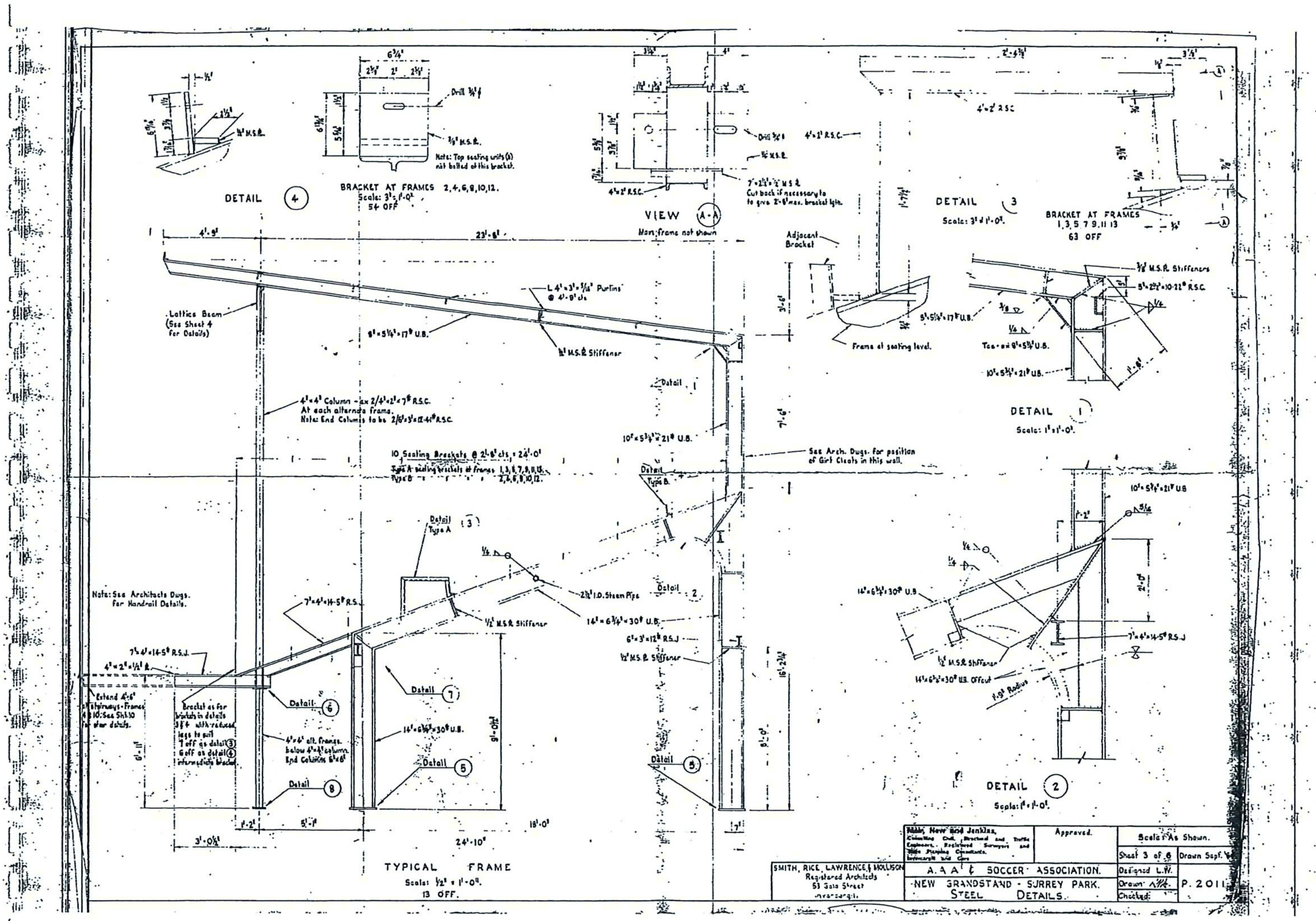

SECTION A:A
EAST ELEVATION
WEST ELEVATION
SOUTH ELEVATION
NORTH ELEVATION
MOIR NEW & JENKINS
Consulting Civil Structural & Traffic Engineers Reg'd Surveyors & Town Planning Consultants
Ingill & Gore
SMITH RICE LAWRENCE & MOLLISON
Reg'd Architects
53 Gala St
Ingill
DATE: OCT. '64
SCALE: 1/8" = 1'-0"
SHEET 9 OF 10.
A A A & SOCCER ASSOCIATION
NEW GRANDSTAND — SURREY PARK

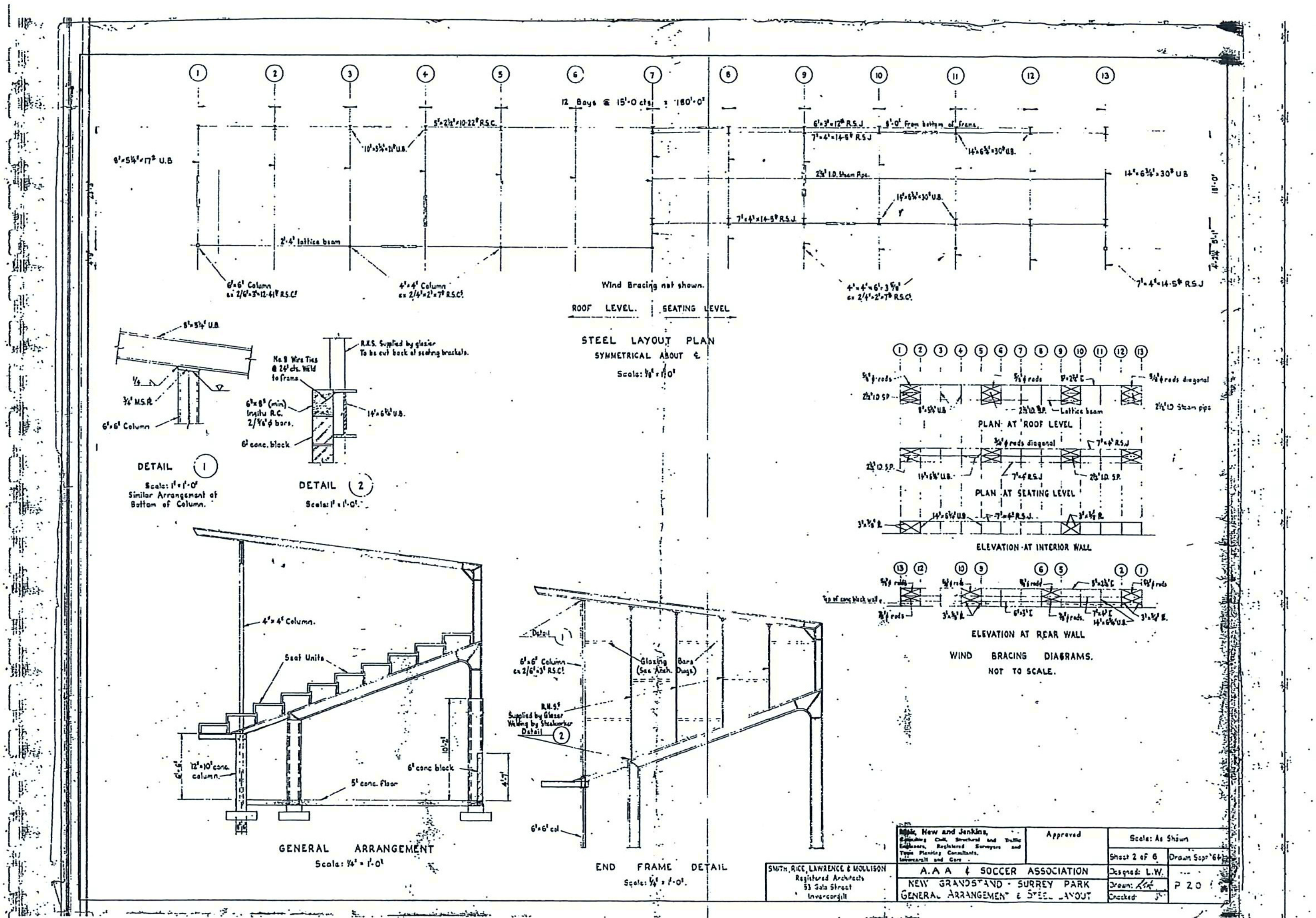
180'6" 55,016

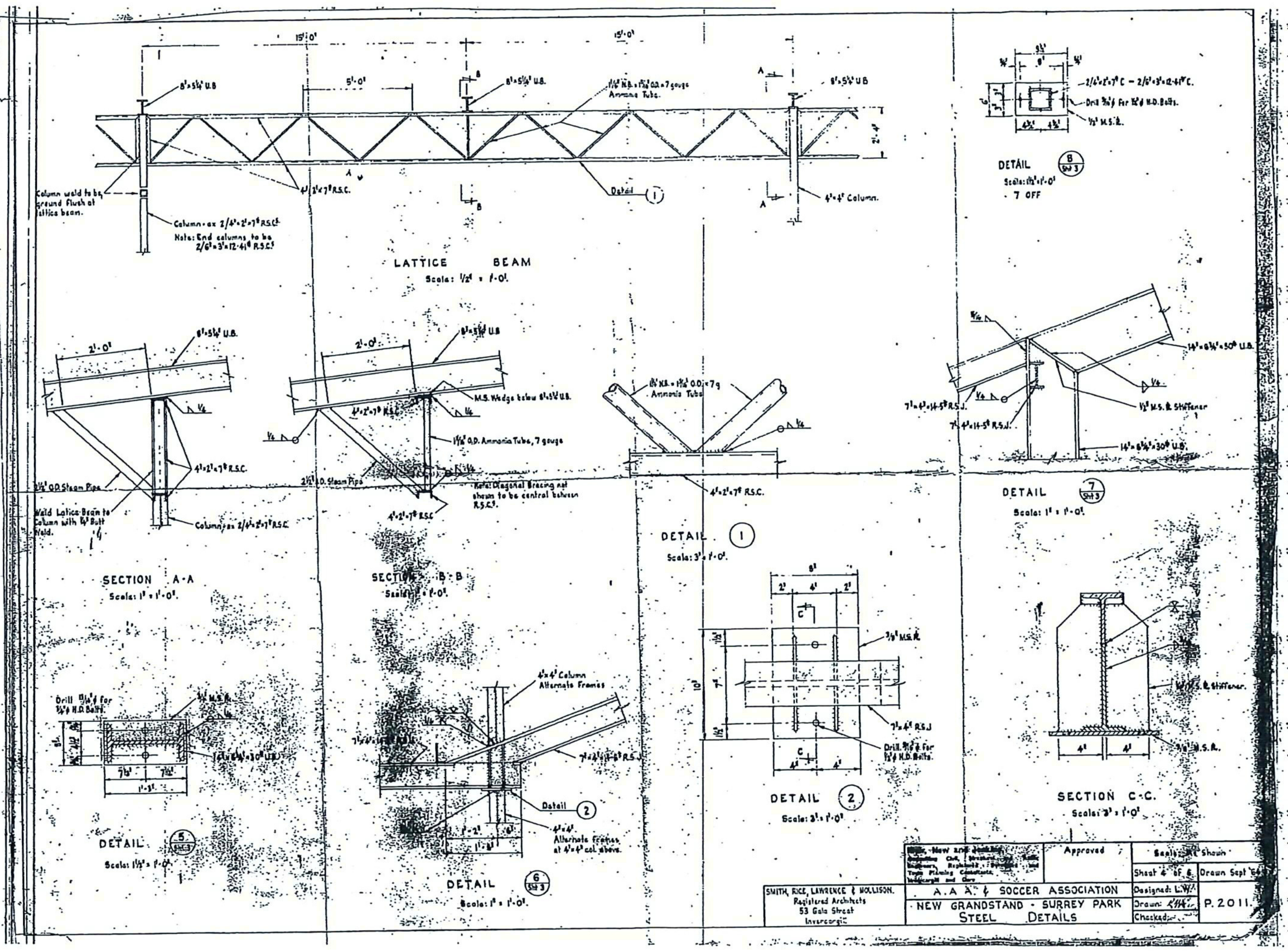
24'4" 7467.



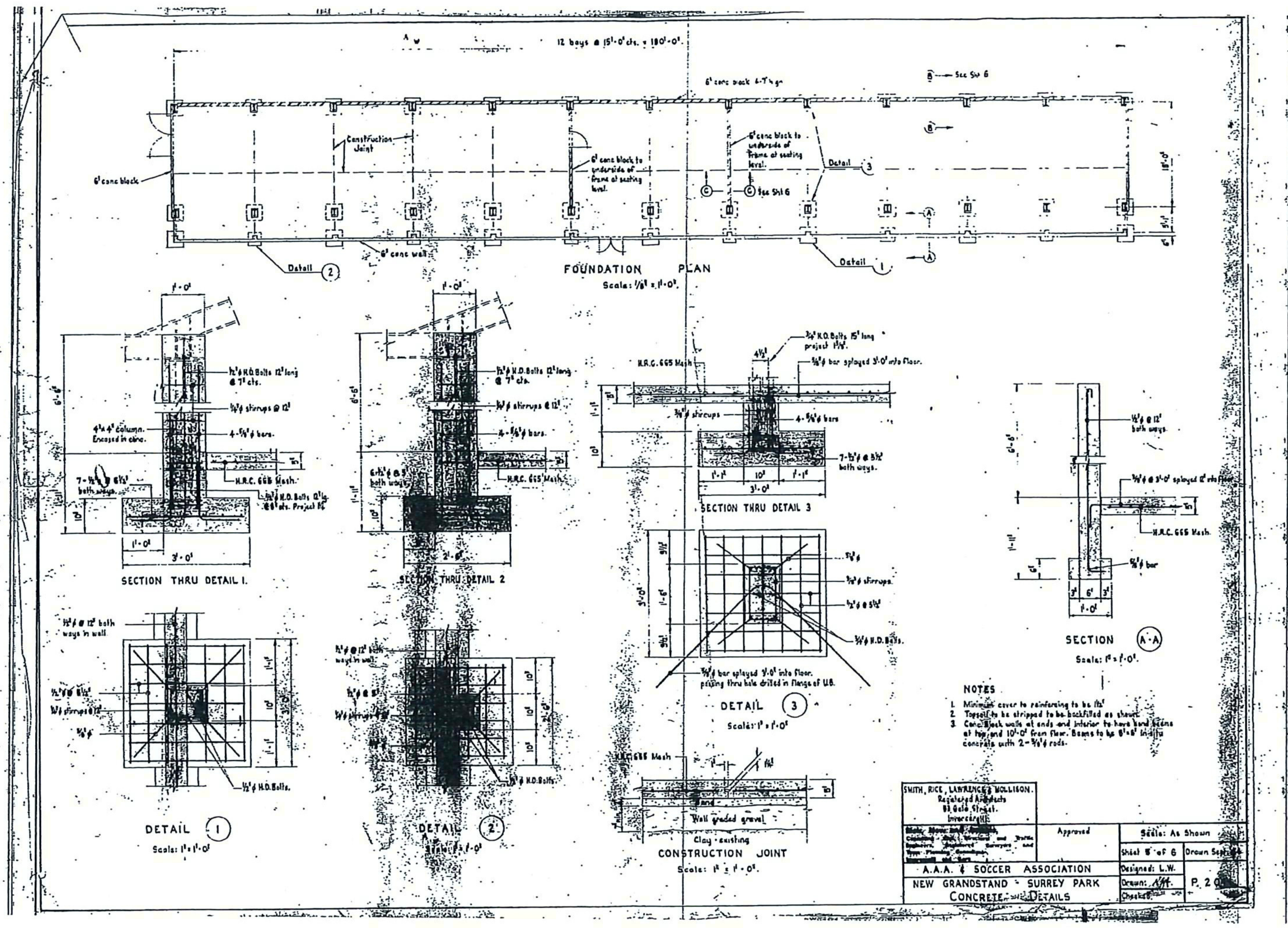

½" CROSS SECTION.
BLOCK DETAILS - ½" = 1'0"
Pressed steel sill supplied by Crittalls
continuous roofing.
R.S.J's cast in concrete.
C.C
A.A
B.B.
Date Oct 64.
Sheet 8
A.A.A. SOCCER ASSOCIATION
NEW GRANDSTAND SURREY PARK

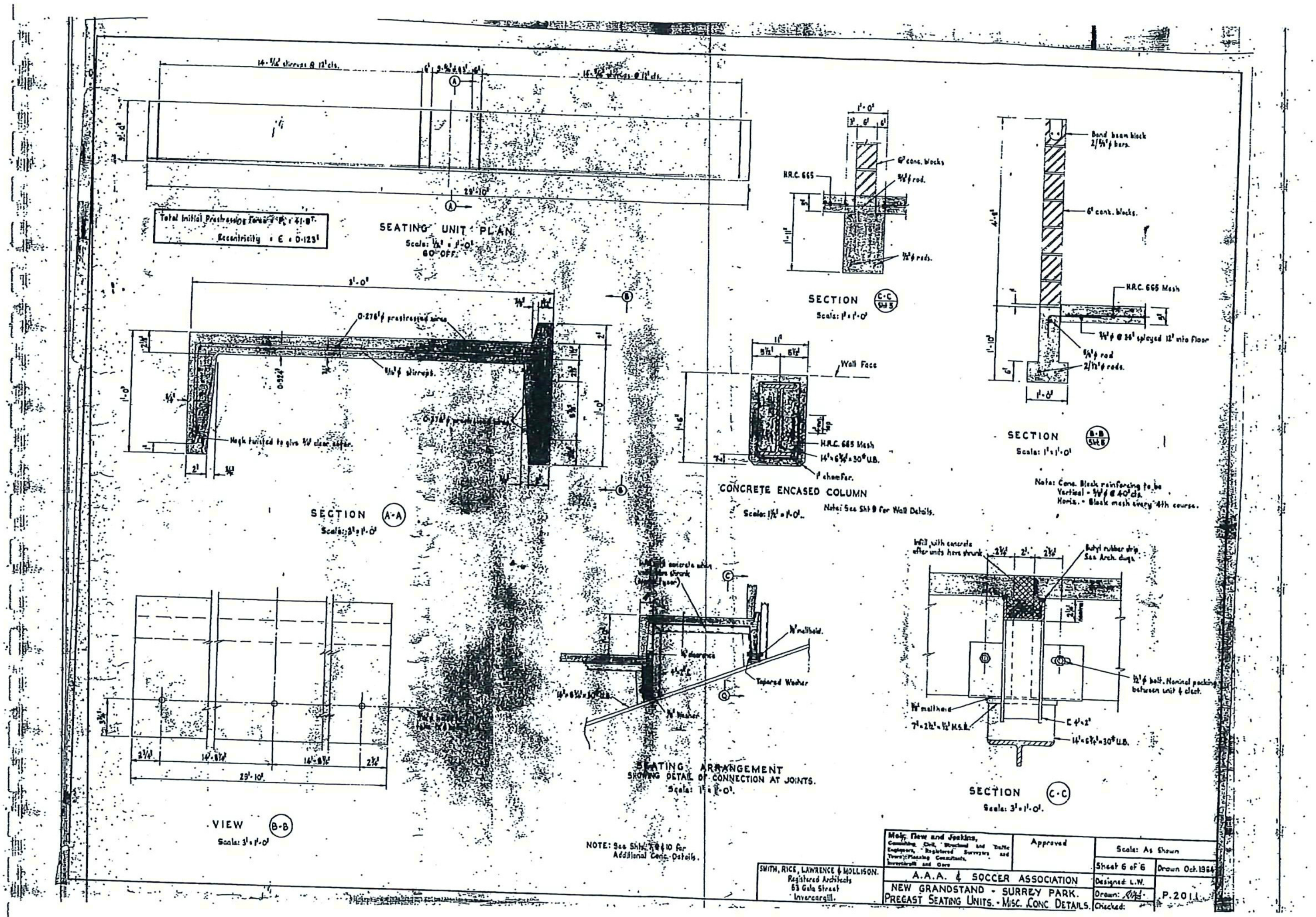

DETAIL 4
BRACKET AT FRAMES 2, 4, 6, 8, 10, 12.
Scale: 3" = 1'-0".
54 OFF
Note: Top seating units (s) not bolted at this bracket.
VIEW A-A
Mon. frame not shown
DETAIL 3
Scale: 3" = 1'-0".
BRACKET AT FRAMES 1, 3, 5, 7, 9, 11, 13
63 OFF
Adjacent Bracket
Frame at seating level.
Lattice Beam (See Sheet 4 for Details)
DETAIL 1
Scale: 1" = 1'-0".
See Arch. Dwgs. for position of Girt Cleats in this wall.
Note: See Architects Dwgs. for Handrail Details.
DETAIL 2
Scale: 1" = 1'-0".
TYPICAL FRAME
13 OFF
Detail 5
Detail 6
Detail 7
Detail 8
Smith, New and Jenkins,
Approved.
Scale: As Shown.
Sheet 3 of 6
SMITH, RICE, LAWRENCE & MOLLISON
Registered Architects
53 Gala Street
A.A.A. & SOCCER ASSOCIATION.
NEW GRANDSTAND - SURREY PARK.
STEEL DETAILS.
Designed L.W.
Checked:
P. 2011


12 Bays @ 15'-0 cts = 180'-0"
Wind Bracing not shown.
ROOF LEVEL.
SEATING LEVEL
STEEL LAYOUT PLAN
SYMMETRICAL ABOUT ℄
DETAIL 1
Similar Arrangement at Bottom of Column.
DETAIL 2
PLAN AT ROOF LEVEL
PLAN AT SEATING LEVEL
ELEVATION AT INTERIOR WALL
ELEVATION AT REAR WALL
WIND BRACING DIAGRAMS.
NOT TO SCALE.
4" x 4" Column.
Seat Units
5" conc. floor
6" conc. block
GENERAL ARRANGEMENT
END FRAME DETAIL
SMITH, RICE, LAWRENCE & MOLLISON
Registered Architects
53 Sala Street
Invercargill
New and Jenkins,
Approved
Scale: As Shown
Sheet 2 of 6
Drawn Sept '64
A.A.A & SOCCER ASSOCIATION
NEW GRANDSTAND - SURREY PARK
GENERAL ARRANGEMENT & STEEL LAYOUT
Designed: L.W.
Drawn:
Checked:
P 20

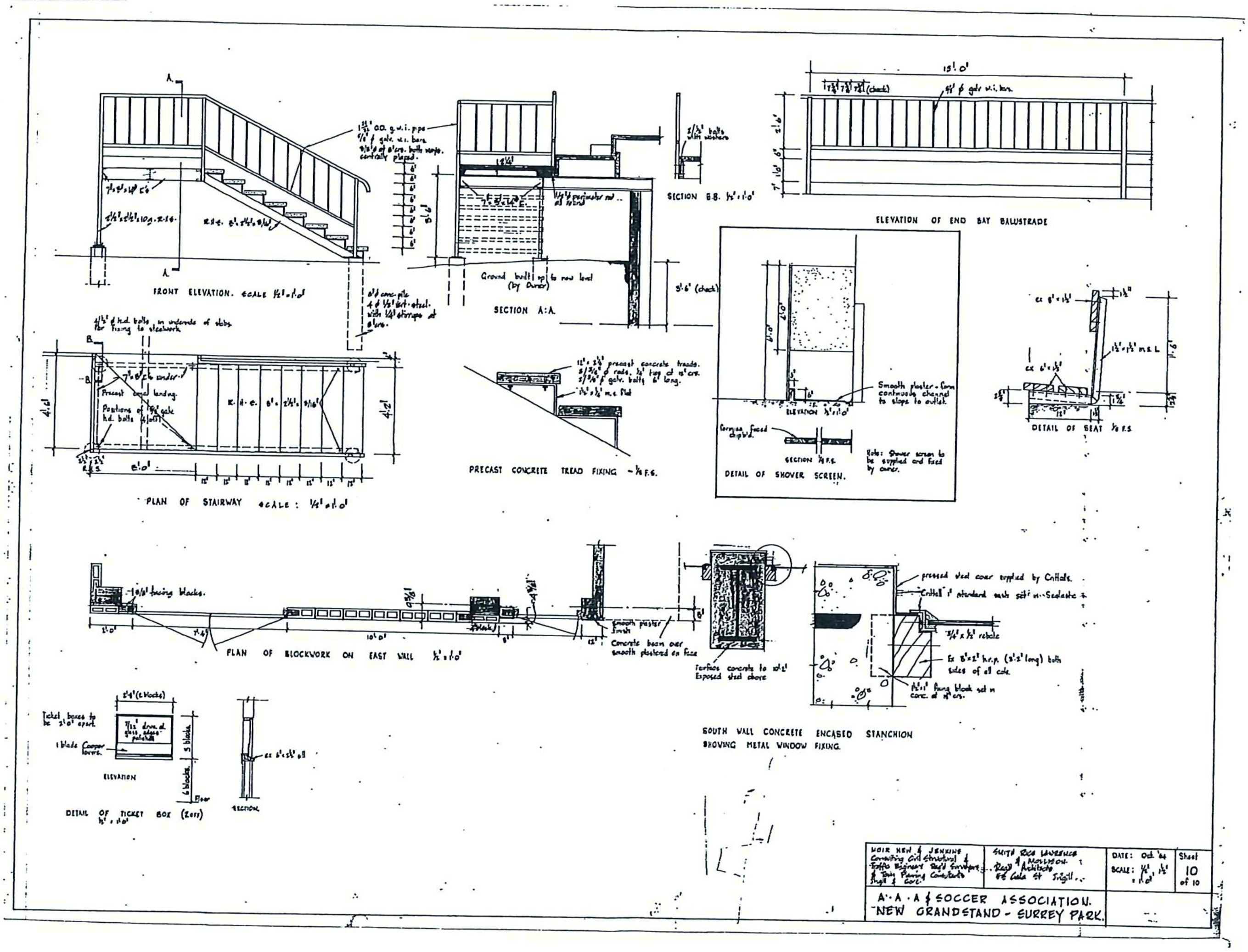


LATTICE BEAM
Scale: 1/2" = 1'-0"
DETAIL 8
7 OFF
SECTION A-A
Scale: 1" = 1'-0"
SECTION B-B
DETAIL 1
Scale: 3" = 1'-0"
DETAIL 7
Scale: 1" = 1'-0"
DETAIL 5
DETAIL 6
Scale: 1" = 1'-0"
DETAIL 2
Scale: 3" = 1'-0"
SECTION C-C
Scale: 3" = 1'-0"
Approved
Scale: As Shown
Sheet 4 of 6
Drawn Sept '64
SMITH, RICE, LAWRENCE & MOLLISON.
Registered Architects
53 Gala Street
Invercargill
A.A.A. & SOCCER ASSOCIATION
NEW GRANDSTAND - SURREY PARK
STEEL DETAILS
Designed: L.W.
Drawn:
Checked:
P. 2011.


12 bays @ 15'-0" cts. = 180'-0"
FOUNDATION PLAN
Scale: 1/8" = 1'-0"
Detail 1
Detail 2
Detail 3
6" conc block
6" conc wall
Construction Joint
See Sht 6
SECTION THRU DETAIL 1.
SECTION THRU DETAIL 2
SECTION THRU DETAIL 3
N.R.C. 665 Mesh
4-5/8" ф bars.
DETAIL 1
Scale: 1" = 1'-0"
DETAIL 2
DETAIL 3
Scale: 1" = 1'-0"
CONSTRUCTION JOINT
Scale: 1" = 1'-0"
Clay - existing
SECTION A-A
Scale: 1" = 1'-0"
NOTES
1. Minimum cover to reinforcing to be 1½"
2. Topsoil to be stripped to be backfilled as shown.
3. Conc. Block walls at ends and interior to have bond beams at top and 10'-0" from floor. Beams to be 8"x8" in-situ concrete with 2-⅜"ф rods.
SMITH, RICE, LAWRENCE & MOLLISON.
Registered Architects
Approved
Scale: As Shown
Sheet 5 of 6
A.A.A. & SOCCER ASSOCIATION
Designed: L.W.
NEW GRANDSTAND - SURREY PARK
CONCRETE DETAILS
Checked:



SEATING UNIT PLAN
SECTION A-A
VIEW B-B
SEATING ARRANGEMENT
SHOWING DETAIL OF CONNECTION AT JOINTS.
CONCRETE ENCASED COLUMN
Note: See Sht 8 for Wall Details.
SECTION C-C
SECTION B-B
SECTION C-C
Scale: 3" = 1'-0".
Note: See Shts. 7, 9 & 10 for Additional Conc. Details.
SMITH, RICE, LAWRENCE & MOLLISON.
Registered Architects
63 Gala Street
Invercargill.
Approved
Scale: As Shown
Sheet 6 of 6
Drawn Oct. 1964
A.A.A. & SOCCER ASSOCIATION
Designed: L.W.
NEW GRANDSTAND - SURREY PARK.
PRECAST SEATING UNITS. - MISC. CONC. DETAILS.
Checked:
P.2011


FRONT ELEVATION. SCALE ½" = 1'0"
SECTION A:A.
SECTION B.B. ½" = 1'0"
ELEVATION OF END BAY BALUSTRADE
PLAN OF STAIRWAY SCALE : ½" = 1'0"
PRECAST CONCRETE TREAD FIXING
DETAIL OF SHOWER SCREEN.
DETAIL OF SEAT
PLAN OF BLOCKWORK ON EAST WALL ½" = 1'0"
SOUTH WALL CONCRETE ENCASED STANCHION
SHOWING METAL WINDOW FIXING.
DETAIL OF TICKET BOX
ELEVATION
SECTION
Ground built up to new level (by Owner)
Precast conc. landing.
Sheet 10 of 10
A.A.A. & SOCCER ASSOCIATION.
NEW GRANDSTAND - SURREY PARK.

Appendix C

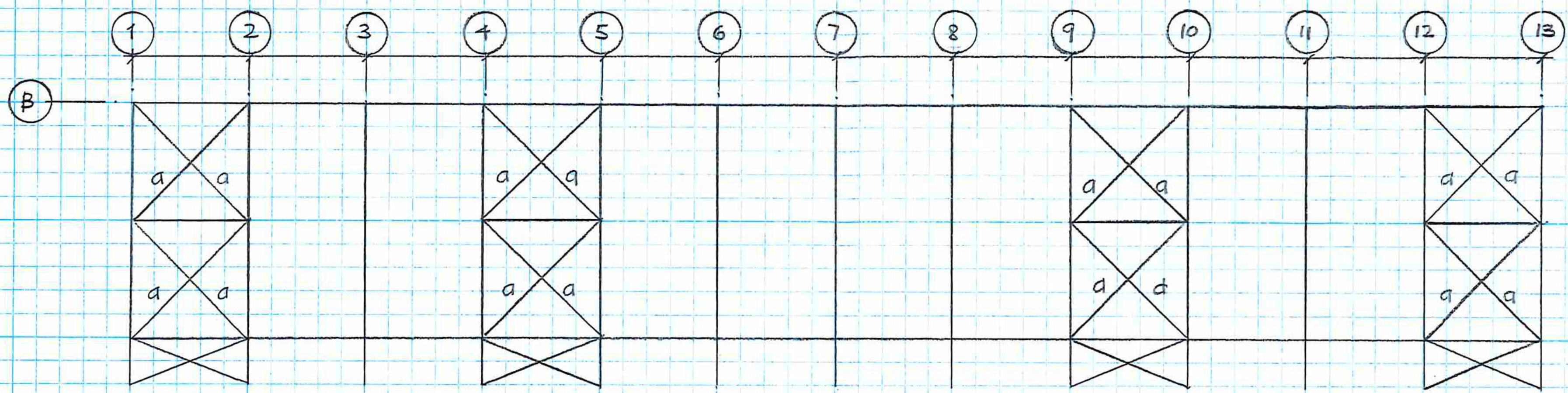
Appendix C: 67%NBS Strengthening Concept

Calculation Sheet

Job: SURREY PARK GRANDSTAND - DSA 67%NBS (IL3) CONCEPT STRENGTHENING

Design: JPM
Date: 08/11/2018
Checked: AD
Date: 08/11/2018

Office: CH
Job No.: 6-V1043.00
Page No.:



ROOF FRAMING PLAN

NEW MEMBER SCHEDULE

MARK	SECTION	STEEL GRADE
a	M20 STEEL ROD	300

LEGEND:

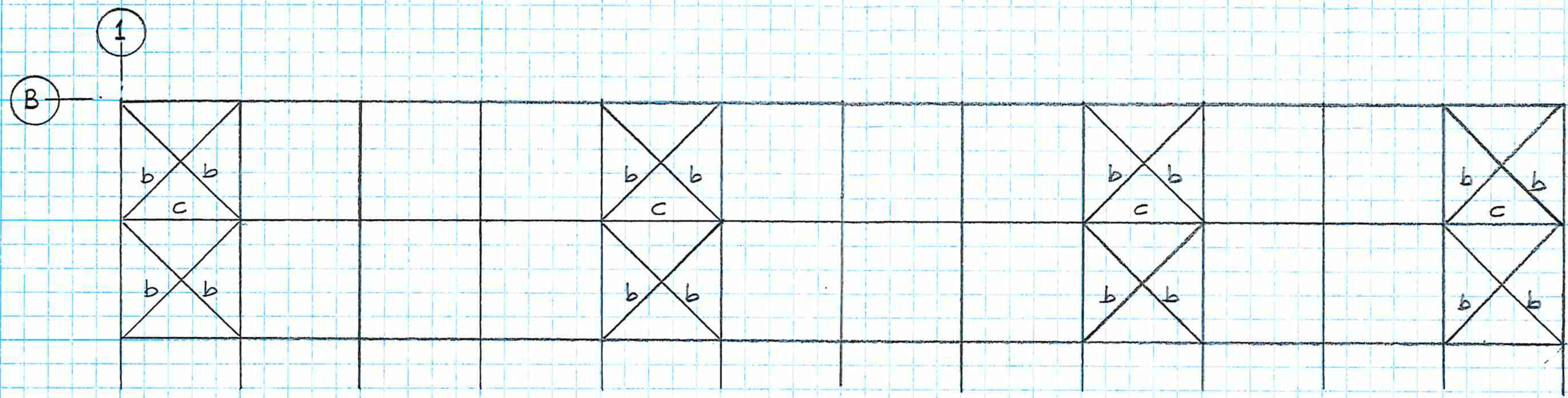
———	EXISTING MEMB TO REMAIN
(a) ———	EXISTING MEMB TO BE REPLACED

Calculation Sheet

Job SURREY PARK GRANDSTAND - DSA 67% NBS (IL3) CONCEPT STRENGTHENING

Design JPM
Date 08/11/2018
Checked AD
Date 08/11/2018

Office CH
Job No. 6-V1043.00
Page No.



BLEACHER FRAMING PLAN

NEW MEMBER SCHEDULE

MARK	SECTION	GRADE
b	75X6 EA	300
c	89X6 SHS	350

LEGEND:

EXISTING MEMB. TO REMAIN

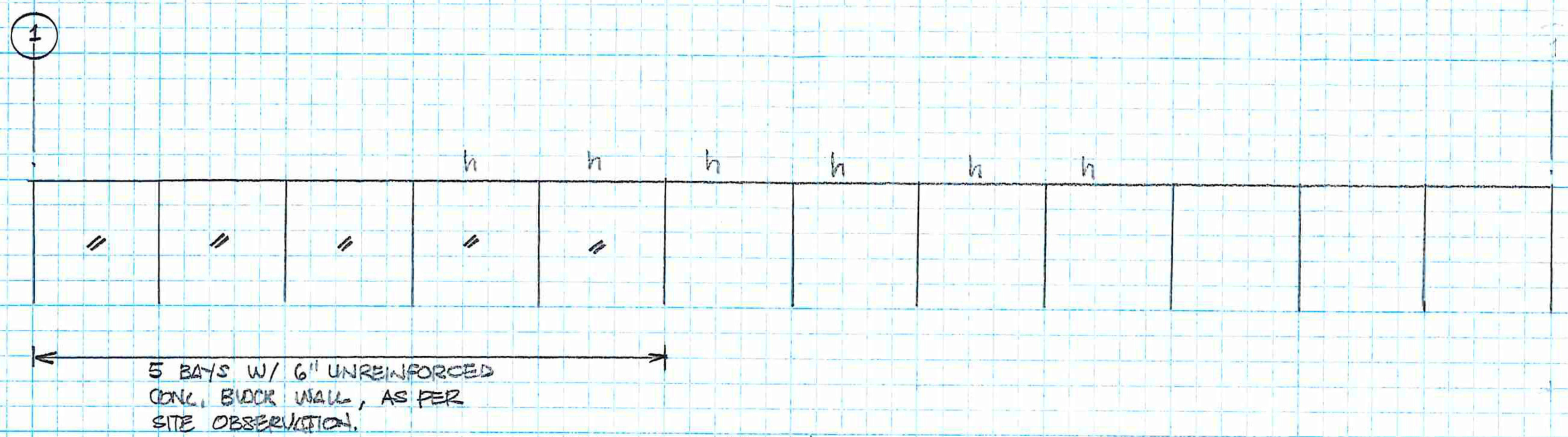
EXISTING MEMB. TO BE REPLACED

Calculation Sheet

Job SURREY PARK GRANDSTAND 67% NBS (IL3) CONCEPT STRENGTHENING

Design JPM | Date 08/11/18 | Checked AD | Date 08/11/18

Office CH | Job No. | Page No.



NORTH WALL ELEVATION
GRID A

NEW MEMBER SCHEDULE

MARK	SECTION	STEEL GRADE
h	125x6 SHS	350

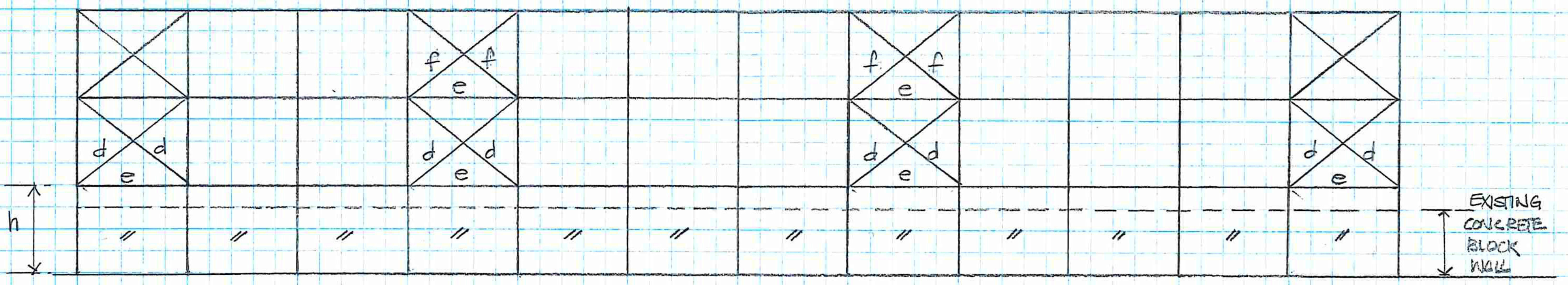
LEGEND:

———	EXISTING MEMB. TO REMAIN
(h)	EXISTING MEMB. TO BE REPLACED
//	EXISTING 6" CONC. BLOCK WALL TO BE REMOVED & REPLACE WITH REINF. CONC. BLOCK WALL

wsp

Calculation Sheet

Job: SURREY PARK GRANDSTAND 67% NBS (IL3) CONCEPT STRENGTHENING
Design: JPM
Date: 08/11/18
Checked: AD
Date: 08/11/18
Office: CH
Job No.
Page No.



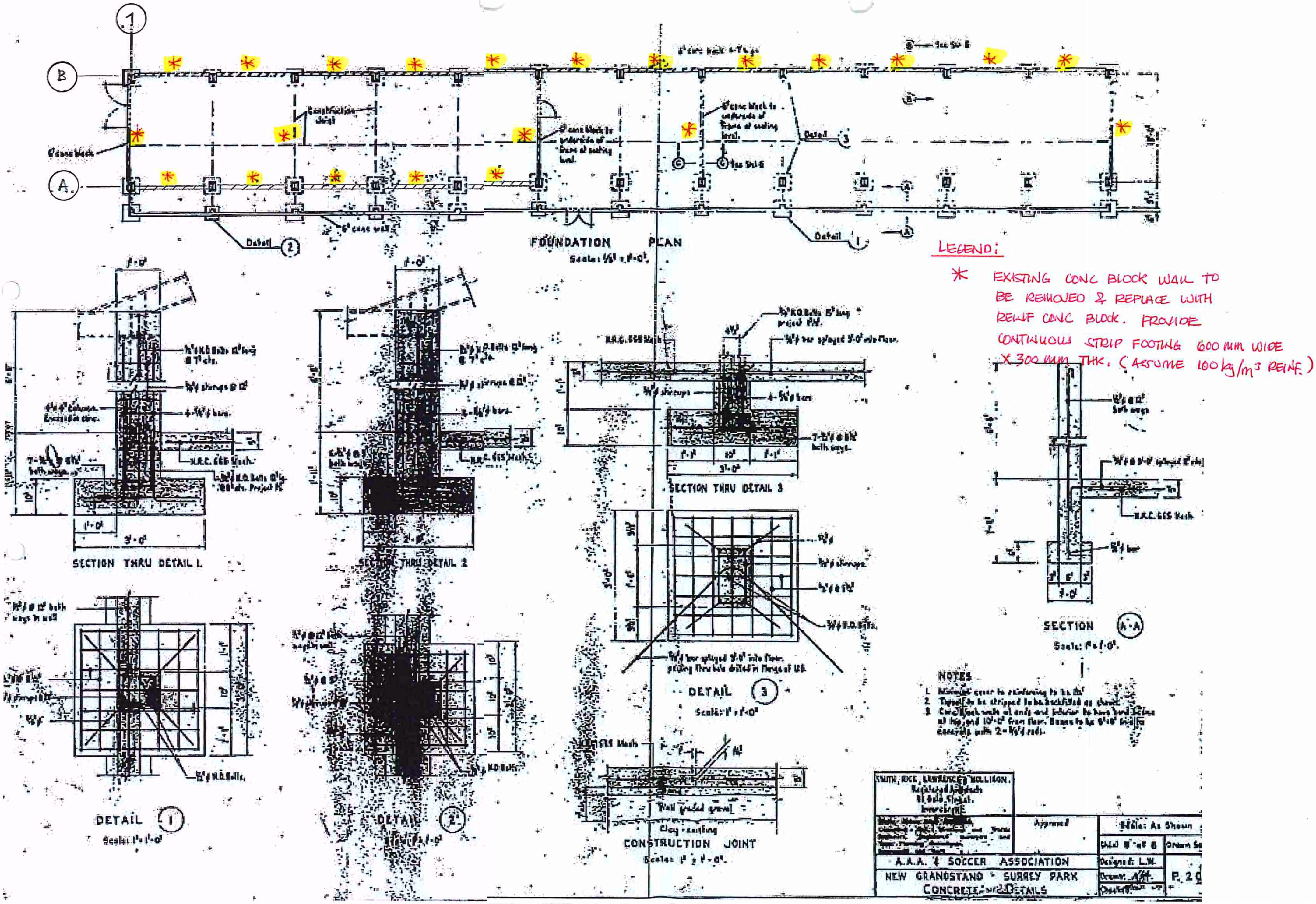
SOUTH ELEVATION
GRID B

NEW MEMBER SCHEDULE

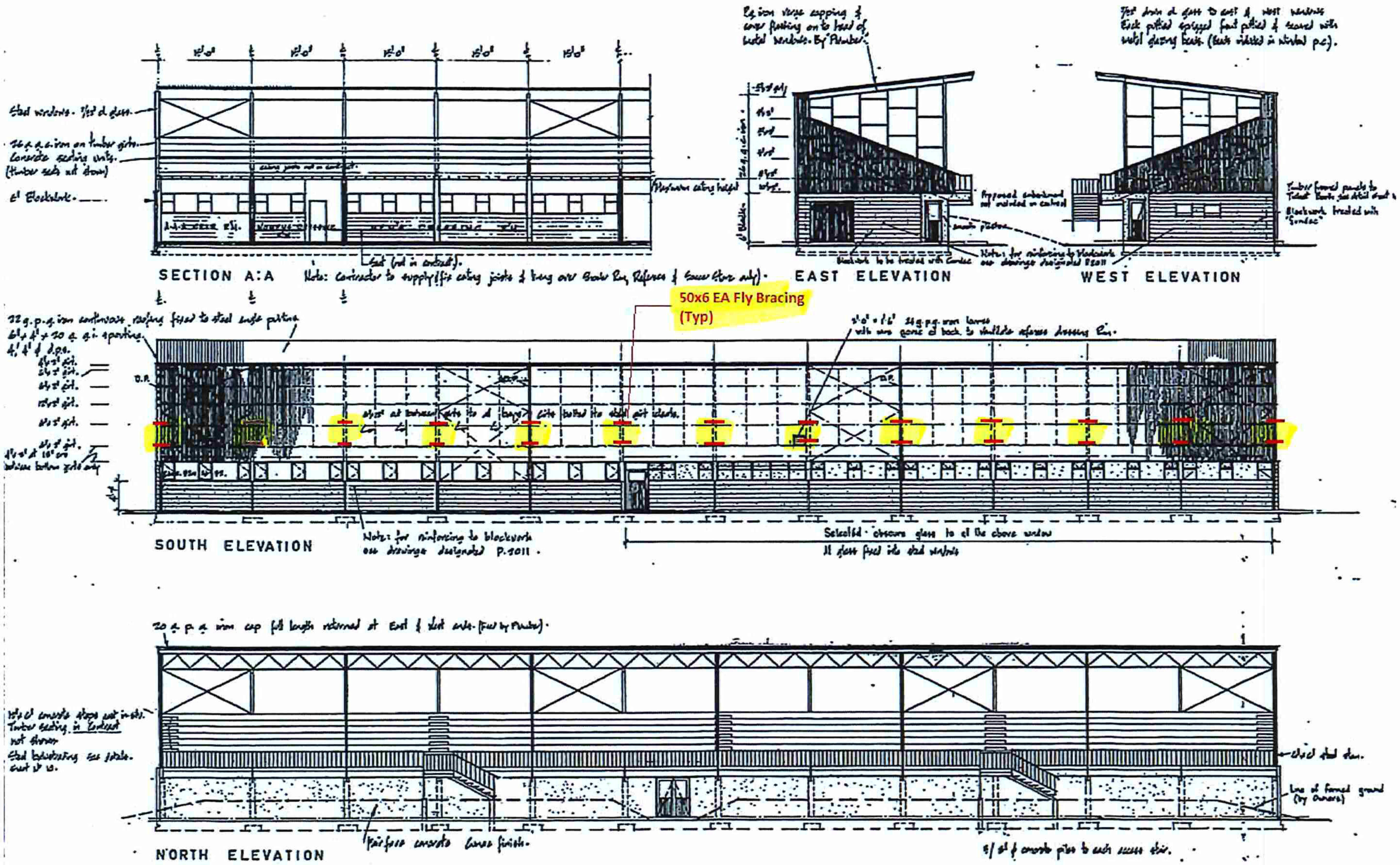
MARK	SECTION	STEEL GRADE
d	75X8 EA	300
e	100X6SHS	350
f	M20 ROD	300
g	90X8 EA	300

LEGEND:

(a)	MEMB TO BE REPLACED
	EXISTING MEMBER TO REMAIN
(g)	NEW MEMBER
//	(E) CONCRETE BLOCK WALL TO BE REPLACED WITH A REINFORCED CONC. BLOCK WALL. EXTEND THE WALL TO HEIGHT "h", SEE ELEV.



FOUNDATION PLAN
Detail 1
Detail 2
Detail 3
SECTION THRU DETAIL 1
SECTION THRU DETAIL 2
SECTION THRU DETAIL 3
DETAIL 1
DETAIL 2
DETAIL 3
CONSTRUCTION JOINT
SECTION A-A
NOTES
LEGEND:
* EXISTING CONC BLOCK WALL TO BE REMOVED & REPLACE WITH REINF CONC BLOCK. PROVIDE CONTINUOUS STRIP FOOTING 600 mm WIDE X 300 mm THK. (ASSUME 100 kg/m3 REINF.)
SMITH, RICE, LAWRANCE & MOLLISON
A.A.A. & SOCCER ASSOCIATION
NEW GRANDSTAND - SURREY PARK
CONCRETE DETAILS
Scale: As Shown


SECTION A:A
EAST ELEVATION
WEST ELEVATION
50x6 EA Fly Bracing
(Typ)
SOUTH ELEVATION
NORTH ELEVATION

www.wsp-opus.co.nz