NSW Health Infrastructure

The Children's Hospital at Westmead Stage 2 Redevelopment

Multi-Storey Car Park Structural Design Report SSDA

CHW-ST-RPT-00006

01 | 20 January 2021

Job number
271985-00

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Document Verification

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Job title	The Children's Hospital at Westmead Stage 2 Redevelopment	**Job number**	271985-00
Document title	Multi-Storey Car Park Structural Design Report SSDA	**File reference**	
Document ref	CHW-ST-RPT-00006		

Revision	**Date**	**Filename**	CHW-ST-RPT-00006		
Issue 01	20 Jan 2021	**Description**	Issue 01		
			Prepared by	Checked by	Approved by
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		Signature			
		Filename			
		Description			
			Prepared by	Checked by	Approved by
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		Signature			
		Filename			
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Contents

1 Introduction

1.1 Purpose of Structural Report

This report is written primarily for the support of State Significant Development Application (SSDA) as part of the project planning requirements. It will also be useful for:

- Engineers or building professionals who are constructing the structure;
- Owners or tenants of the finished building;
- Other members of the design team; and
- Engineers making alterations to the structure in the future.

The report has a number of purposes:

- It contains a description of the project, the site and the structural works;
- It lists the assumptions about structural materials, loadings and the structural performance criteria; and
- It sets out and describes the principle methods of analysis and justifications that will be used in the structural calculations in subsequent design stages.

It should be noted that this document describes works associated with the permanent condition of the structure only. All non-permanent works are considered temporary works and are considered outside Arup's scope of services. All temporary works shall be designed and certified by a registered person and same shall be made known to Arup and the client team prior to undertaking any works on the project.

2 General Information

2.1 Scope of Proposed Works

The scope of proposed works includes:

- Demolition of The Lodge
- Construction of a new Multi-Storey Car Park (MSCP).
    - 8-storey carpark will provide approximately 1000 car parking spaces for staff and visitors
    - Vehicular access from Labyrinth Way and / or Redbank Road
    - A split-level approach to the MSCP to respond to the natural ground level
- Ancillary retail facilities
- Road works:
        - Realignment of Redbank Road with vehicular access connection to MSCP

- Tree removal
- Associated landscape works

The MSCP is being designed to be constructed in a single stage yet car parking will be staged operationally to come on-line with parking demand across the Precinct:

- The first stage of car parking operation would provide replacement car parking for the demolished P17 car park. There would be no net increase of parking on site under this stage.
- The second stage of car parking operation to serve the growth in hospital activity associated with the future PSB would only come on-line operationally with the PSB SSDA consent becoming operational, specifically at occupation. This would provide growth of around 280 additional spaces in line with hospital activity projections until 2031.

2.2 The Project

Arup have been engaged by Health Infrastructure (The Client) to provide Structural engineering services for the new Multi-Storey Car Park (MSCP) proposed on the corner of Redbank Rd and Labyrinth Way. The carpark composition consists of an 8-storey, split deck development. The car park will provide approximately 1000 car parking bays for both visitors and staff. Figure 1 delineates the architectural intent of the MSCP.

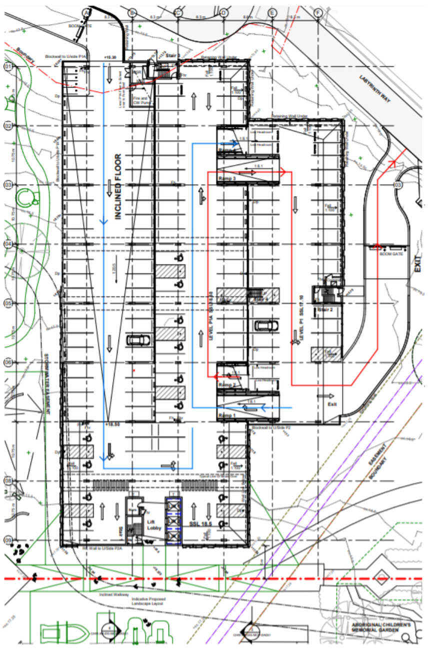

Figure 1: MSCP Architectural Layout

2.3 The Site

The new MSCP will be situated on the site of the existing old Ronald McDonald House. As shown in Figure 2, the proposed site is bounded by Redbank Road, Labyrinth Way and the existing Children's Hospital at Westmead.

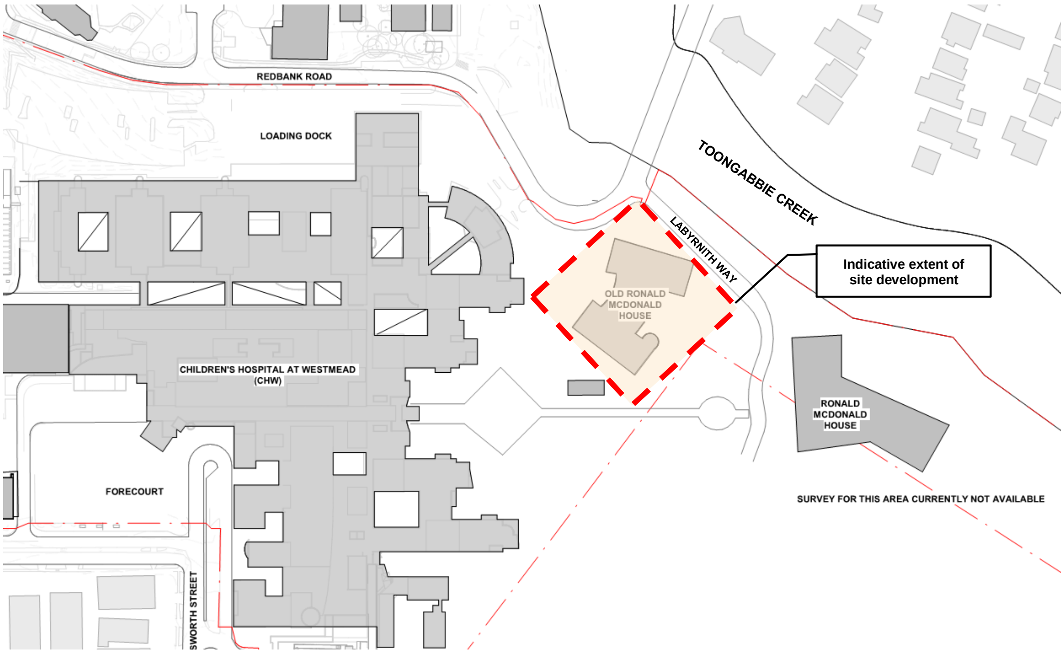

Figure 2: Site Location

2.4 Site Topology

The existing ground level of the site is generally higher to the south-west (Hawkesbury Road), falling towards Toongabbie Creek. Refer to the Civil documentation for the extent of the ground plane usage and associated extent of cut and fill.

2.5 Site Geology

The expected site geology has been based on the interpretation of the JK Geotechnics report 33303Brpt2 Geotechnical Investigation (Draft) (6th November 2020). This report highlighted that the site geology is consistent with the expected regional geology and previous geotechnical assessments conducted within the Westmead precinct. The geological profile has been summarised in Table 1.

Table 1: Typical Geotechnical Profile

Component	Depth	Allowable Bearing Pressure	Allowable Shaft Adhesion
Topsoil/Fill	Depth varies across the site from a depth of 0.2m to 5.9m.	N. A	N. A
Class V Bedrock	Depth varies across the site from 0.6m to 8.2m	1000 kPa	100 kPa
Class IV Bedrock	Depth varies across the site from 1.8m to 8.2m	1200 kPa	120 kPa
Class III Bedrock	Depth varies across the site from 2.6m to 9.2m	4000 kPa	400 kPa
Class II Bedrock	Depth varies across the site from 3.4m to 9.7m	8000 kPa	800 kPa

2.6 Contamination

In July 2020, a detailed site inspection was conducted by JBS&G to delineate the extent of asbestos impacted fill, as well as characterise non-asbestos contaminates across the site. The investigation found that soils were impacted by asbestos-containing materials (ACMs) between a depth of 0.5m – 4m. Due to the depth and location variability of the ACM, as well as the historical filling of the site during the main CHW development, JBS&G states that remediation/ management for asbestos is required.

3 Design Criteria and Performance

3.1 Codes and Standards

The following codes and standards are to be used through the project design phase:

AS/NZS 1170.0: 2002	Structural design actions – Part 0: General Principles
AS/NZS 1170.1: 2002	Structural design actions – Part 1: Permanent, imposed and other actions
AS/NZS 1170.2: 2011	Structural design actions – Part 2: Wind actions
AS1170.4: 2007	Structural design actions – Part 4: Earthquake actions in Australia
AS3600: 2018	Concrete structures
AS3700: 2018	Masonry structures
AS4100: 1998	Steel structures
NCC (BCA): 2019	National Construction Code (Building Code of Australia)

3.2 Design Life

The structure will be designed for a 50-year design life.

3.3 Building Importance Level

The multistorey carpark (MSCP) is categorised as an Importance Level 2 building.

The importance level is used for calculation of design recurrence intervals for wind and seismic events, these design events for safety are summarised in Table 2.

Table 2: Lateral Loading Probability of Exceedance References for MSCP

Component	Annual Probability of Exceedance
Wind loading	1: 500
Earthquake loading	1: 500

3.4 Design Floor Loadings

3.4.1 Structure Self Weight

The structure self-weight is calculated explicitly in the structural design.

Refer to section 4 of this report for specific material properties.

3.4.2 Superimposed Dead and Imposed Live Loads

The proposed design loadings in accordance with Section 3 of AS/ NZS 1170.1:2002 are listed in the table below.

Table 3: Structural Design Floor Loading Allowances

Area	**Superimposed Dead Load (SDL)**	**Live Load (LL)**
Carparking and ramps	0.5 kPa finishes & services	2.5 kPa
Plant Areas	2.0 kPa finishes 0.5 kPa ceiling and services	As calculated for relevant use. 5.0 kPa minimum
Stairs and corridors	0.5 kPa ceiling and services	4.0 kPa

3.4.3 Live Load Reduction

Per AS1170.1 live load reduction is not permitted in car park structures and will not be used in the design.

3.4.4 Façade/ Cladding Dead Loads

A 1kPa façade dead load has been considered in the preliminary design. This load may be amended in the following design stages once the façade design develops.

3.5 Serviceability

3.5.1 Deflection Limits

The Post Tensioned (PT) and reinforced concrete elements have been sized to limit deflections to acceptable limits defined in AS3600:2018 Table 2.3.2. These are summarised in Table 4. Where not explicitly described in Table 4 below, deflection limits for reinforced concrete and structural steelwork have been designed generally in accordance with AS3600 and AS4100 respectively. The deflection limits summarised in Table 4 have been adopted in the design of the structural elements.

Table 4: Deflection limits

Element	Deflection limit under Total Load	
	Spans	Cantilevers
Beams and Slabs		
Generally	L/250 or 30mm max.	L/250 or 25mm max.
Live load only	L/360	L/180
Supporting articulated masonry	L/500 (incremental)	L/250 (incremental)
Supporting un-jointed masonry	L/1000 (incremental)	L/500 (incremental)
Beams and Slab Supporting Cladding Elements		
Transfer structures (cumulative at location of element transferred) (Live Load only)	L/1000 or 12mm max.	L/500 or 12mm max.
Heavy Façade (Brick)	L/1000 or 10mm max.	L/500 or ±10mm max.
Light Façade (Glazing)	L/750 or 13mm max.	L/750 or ±13mm max.
Interstorey drift under wind	H/500 or 0.2%H (SLS)	
Interstorey drift under seismic	H/66 or 1.5%H (ULS)	
Pile Settlement	1% of pile diameter	
Differential Pile Settlement	Grid spacing/1000 or 10mm whichever is least.	
Horizontal Pile Movement	H/500	

The deflection limits described in the above table are calculated under the following load combinations:

Deflection Type	Combination	Time
Initial Deflection (ID)	$G + SDL_1$	Before loading
Short Term Deflection (ST)	$G + SDL_1 + 0.7Q$	At first loading
Long Term Deflection (LT)	$G + SDL_1 + SDL_{extra} + 0.4Q +$ Creep + Shrinkage	30 years
Incremental Deflection (Inc)	LT - ID	

G : Self weight.
SDL_1 : Superimposed dead load on floor including hobs and upstands
SDL_{extra} : Facade, and services area load

3.6 Durability

Concrete covers are to be in accordance with AS 3600. The durability requirements of AS 3600 has been applied to all reinforced and PT concrete. It is proposed that the minimum concrete strength f'_c shall be 40MPa and concrete cover shall be a minimum of 30mm. However, structural requirements for certain elements may increase concrete strengths above the minimum required for durability.

3.7 Structural Fire Resistance

Fire resistance levels for structural elements shall be determined in accordance with the BCA and any subsequent approved relaxations. Fire resistance levels are to be confirmed by the BCA Consultant

3.7.1 Concrete

Concrete covers are to be in accordance with AS3600: 2009 Section 5. The fire rating of blockwork walls is to be shown on the architect's schedule.

3.7.2 Structural Steelwork

A fire engineered approach will be undertaken to minimise or alleviate the requirement for passive protection to structural steel wherever possible. Where required, fire protection to structural steel may be provided using fire rated cladding, vermiculite (or similar) spray, or intumescent paint.

3.8 Structural Robustness

The structure will be designed to provide load paths to the foundation for forces generated by all types of actions from all parts of the structure, for the minimum actions as given in Clauses 6.2.2 to 6.2.5 of AS 1170.0: 2002. The key minimum actions are discussed in the proceeding sections.

3.8.1 Minimum Resistance

The structure will be designed to have a minimum lateral resistance equivalent to 1.0% of $(G + \Psi cQ)$ for each level, applied simultaneously at each level for a given direction.

3.8.2 Minimum Lateral Resistance of Connections and Ties

All parts of the structure are interconnected. Connections will be designed to be capable of transmitting 5% of the value of $(G + \Psi cQ)$ for the connection under consideration.

3.9 Tolerances and Movements

Tolerances for concrete and structural steel will be considered in accordance with the relevant Australian Standard, unless noted otherwise in the structural, services or architectural specifications.

The movements of the structure will be calculated in accordance with the relevant Australian Standard(s) based on the design loadings. The structural movements expected to act on the structure include the items described in Table 5 below.

Table 5: Structural Movements

Movement	
Settlement:	1% of footing width at allowable bearing pressure for spread footings.
Heave	either absolute or differential
Temperature range:	Exposed structure +65°C and -10°C from mean temp 20°.
Shrinkage:	Floor slabs - design shrinkage <500 microstrain. (Final values to be agreed with concrete suppliers)
Vertical structure:	Design shrinkage <500 microstrain.
Creep:	Floor slabs and vertical structure.
Elastic shortening:	Vertical structure and elements under prestressing.

Loading arising from restrained or partially restrained creep and shrinkage will be designed for in accordance with the joint locations described on the structural drawings.

Additionally, the effect of column shortening due to axial loads and construction methodology will affect the heights between floors and will be considered in the design and construction of the columns and walls.

4 Materials

The following structural materials are used. Typical values for the properties of these materials are listed. These values are to be adjusted where appropriate.

4.1 Reinforced Concrete

4.1.1 Concrete

The concrete properties are assumed to be:

• Concrete density:	24.5 kN/m3
• Young Modulus E (MPa):	30.1 kN/mm2 for f'c = 32 MPa
(short term)	32.8 kN/mm2 for f'c = 40 MPa
	34.8 kN/mm2 for f'c = 50 MPa
	37.4 kN/mm2 for f'c = 65 MPa
• Coefficient of thermal expansion:	10 x 10-6 per ºC
• Basic shrinkage strain UNO:	650 x 10-6 at 8 weeks for S Grades
	850 x 10-6 at 8 weeks for N Grades
• Basic creep factor:	Table 3.1.8.3 of AS 3600
• Poisson's ratio:	0.2

4.1.2 Reinforcement

The bar reinforcement and mesh in accordance with AS 1302, AS 1303, AS 1304 and AS/NZS 4671 shall be:

• Plain bars (R250N)	fsy = 250 MPa
• Deformed bars (D500N)	fsy = 500 MPa
• Steel Wire, plane and deformed	fsy = 500 MPa
• Welded wire fabric	fsy = 500 MPa
• Modulus of elasticity	E = 200,000 MPa

4.1.3 Tendons

All prestressing strands shall be 12.7 or 15.2mm diameter super grade stress relieved strands to AS 1311 with an ultimate tensile strength of 184kN or 250kN respectively. Relaxation to be a 2.5% maximum after 1000 hrs at 70% of breaking load.

Stressing Procedure:

- Stress each individual strand of all tendons to 25% U.T.S at approximately 24 hrs (fcp= 7MPa minimum after completion of pour);
- Jack tendons to 85% U.T.S. and lock off;
- Jack central tendons first and work progressively outwards on each side. (alternating); and
- Fully stress tendons when concrete attains f'c =22 MPa for 12.7mm dia. strands (mono), f'c =25MPa for 15.2mm dia. strands or 12.7mm dia. strands (multi)

4.2 Structural Steel

4.2.1 Steel Grades

The steelwork properties are assumed to be:

- Modulus of elasticity: 200,000 MPa
- Poisson's ratio: 0.3
- Co-efficient of thermal expansion: 11.7 x 10-6 per °C
- Steelwork density: 7850 kg/m3

Steel grades are to be as follows:

Member Type	Standard	Grade (MPa)
Rolled sections	AS 3679	300
Flat Products	AS 1594	250
Plates and Floor Plates	AS 3678	250
CHS	AS 1163	350
RHS and SHS	AS 1163	350
Cold-formed sections	AS 1397	450
Bar	AS 3679	250

4.3 Masonry / Brickwork

- Block grade 15
- Characteristic Strength f'uc = 15 MPa.
- Core fill grout f'c = 15 MPa

5 Structural Design of Multi-Storey Car Park

5.1 Building Configuration

The proposed structure is an 8-storey building, each floor covering a plan area of approximately 3950m^2. The column grid is typically 10.75m in the North- South direction, which allows 4 car spaces per bay in the slab span direction. In the East-West direction the grid is typically 8.3m in the span between columns and 4.445m cantilevers at the ends of each floorplate. The floor to floor height is typically 2.8m.

The current arrangement has a step in the plan of the structure and incorporates a split level.

The proposed superstructure consists of a system of post-tensioned band beams and one-way spanning post-tensioned slabs. Band beams span the shorter direction to effectively support the cantilever on the end beam span.

5.2 Foundations

Based on the foundation loads and existing ground conditions described within the JK Geotechnics report 33303Brpt2 Geotechnical Investigation (Draft) (6th November 2020), the most suitable foundation type used for this building is reinforced concrete bored piles. Pile caps will be eliminated such that columns bear directly onto the pile, refer to Figure 3 for details. There will be sufficient tolerance between the pile and column over, thus eliminating timely and costly construction of the pile cap element. The area under the stability lift cores will have a denser number of piles and a pile cap will be required to transfer the line loads from the wall elements into the individual pile elements of the foundation zone. It is assumed the piles will be bored from the existing ground level. Above the pile trim level, the pile will continue to L01 and formed with conventional column formwork such as Formatube. This portion of the foundation will be surrounded by backfill beneath the L01 slab.

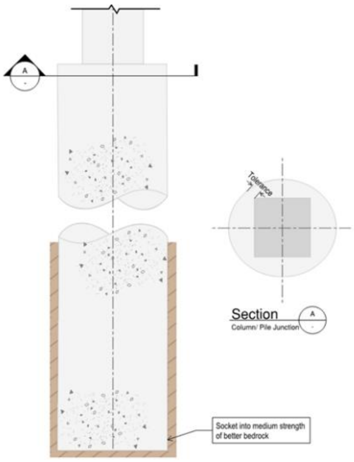

Figure 3: Typical Bored Pile Section & Plan

5.3 Retention

To minimise the extent of material to disposed of offsite, the bulk earthworks has been balanced between both the PSB and MSCP development site. Consequently, a significant volume of material is required to be distributed over the MSCP development site, resulting in the need for retention around the Northern and Eastern portion of the site. The required retaining walls range from 1.0 – 4.5m in height. Due to the heterogeneous nature of the existing ground condition a piled retaining wall is the most appropriate retention system, as recommended by JK Geotechnics. In the temporary case this piled retaining wall is cantilevered, while in the permanent case the wall is tied back to the suspended structure. Consequently, the temporary case governs the design. To accommodate the large cantilevers, buttresses were introduced mid span to stiffen the retaining walls and provide three-sided support, thus minimising the retaining wall thickness. A 10kPa construction surcharge has been assumed in the retention design.

5.4 Floor Framing

The proposed floor framing is one-way spanning post-tensioned beams, 450x2400, which accommodate the end cantilevers and perpendicular one-way spanning post-tensioned slabs. The internal slabs are typically 160mm deep and the end spans are 190mm deep.

It is assumed that the flat part of the suspended floors will be poured first, with the ramp structure following behind floor by floor. The ramps are proposed to utilise a permanent doweled movement joint at their base and tied in with a standard construction joint detail at their tops. This is to limit the amount of restraint forces between the split decks and allow the stressed structure to “breathe”.

The lift and stair cores located to the exterior of the floorplate will provide an in-plane restraint of the slabs. The upper deck will be restrained between the north and south cores. As such, a temporary movement joint is proposed towards the middle of the floorplate in east-west direction. This will minimise restraint forces by allowing the floors to move towards the outer cores when they are stressed. It also relieves the initial major component of temperature, shrinkage and creep forces. The lower deck is isolated from the upper deck, this allows the floor to move and shrink independently. Consequently, no movement joints are required in this floorplate.

5.5 Columns

Vertical loads will be supported by a combination of the reinforced concrete columns and walls. Based on the current architectural layouts, it is unlikely that the building will require any significant column transfer structures.

Blade columns have been adopted in the design to minimise the structural width between car parking spaces. The position of these columns aligns with the car position and limitations on car door positions.

5.6 Lateral Stability

The lateral stability system will be formed by a combination of lift and stair cores. The core walls provide stability in both orthogonal directions.

It is likely for an 8-storey building the core walls will be jump formed, this is a typical construction methodology that will be economical for this building type and height.

The importance level discussed in Section 3.3, will be used for the calculation of design recurrence intervals for wind and earthquake events. The following table summarises the relevant loading codes that will be used in the design:

Structural Robustness	In accordance with AS 1170.0
Wind loading	In accordance with AS 1170.2
Earthquake loading	In accordance with AS 1170.4

For this form of structure, the stability design is driven by:

- The notional loading requirement of AS 1170.0:2002 Section 6.2.2. This requires 1.0% of the gravity service load applied horizontally at each level.
- The seismic design requirement of AS 1170.4: 2007.
- The wind loading requirements of AS 1170.2: 2011.

5.6.1 Wind Loading

In absence of the wind tunnel testing results, the following overall design parameters apply:

- Region : A2
- Importance Level (BCA Table B1.2a): : 2
- Annual probability of exceedance: : 1:500
- Regional wind speeds:
  - ULS – V500 : 45m/s
  - SLS - V25 : 37m/s
- Terrain category (direction dependant) : Category 3

5.6.2 Seismic Loading

Earthquake loading applied to the structural elements will be assessed in accordance with AS 1170.4: 2007. The following design parameters apply:

- Importance Level (BCA Table B1.2a): : 2
- Annual probability of exceedance: : 1:500

- Probability Factor, k_p : 1.0
- Hazard Factor, Z : 0.08
- Site Sub-Soil Class : C_e

  (To be confirmed during Geotechnical Investigations)
- Earthquake Design Category, EDC : II
- Structural System : Limited Ductile Structural Walls
- Structural Ductility Factor, μ : 2
- Structural Performance Factor, S_p : 0.77

Utilising the parameters listed above, a 3D finite element analysis was conducted using GSA. A modal analysis was conducted to determine the natural frequency of the building, under this modal analysis 90% of the buildings mass was participating. Following the modal analysis, a response spectrum analysis was conducted to achieve a more accurate representation of the base shear and shear distribution through the building during a seismic event. The response spectrum curve used in the analysis is shown in Figure 4.

There is minimal frame action in the north-south direction due to the column orientation. Thus, the lateral loads imposed on the structure are resisted by the shear core walls and ultimately the piled foundations under the cores. This results in local uplift forces on the piles which is resisted by skin friction between the pile and Class II rock. To achieve adequate skin friction capacity, the socket length into Class II rock has been increased from the compression piles.

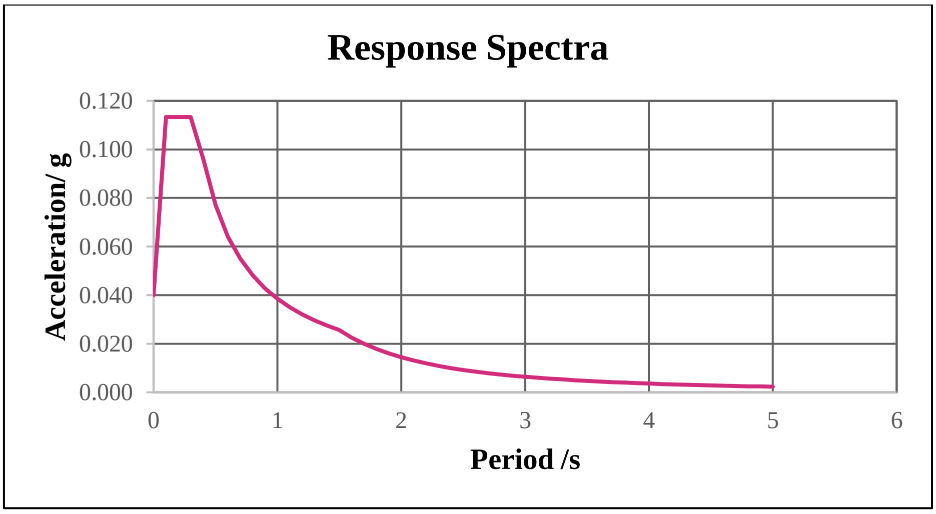

Figure 4: Response Spectra Curve

5.7 Façade

At this stage the Architect is proposing the use of a lightweight perforated aluminium façade that will be bolt connected to the floor structure and span floor-to-floor.

5.8 Vehicle Barriers

A w-beam guardrail will be provided around the perimeter of the building for vehicles on the slab edges.

5.9 Future Proofing

No future proofing strategies or contingencies have been included in the design of the MSCP.