# Traffic Improvements Linear Consulting Botanical Garden Detailed Design Document

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**University of British Columbia**

**CIVL 446**

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# CIVIL 446

# Traffic Improvements

## Linear Consulting Botanical Garden

## Detailed Design Document


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## Table of Contents

## Executive Summary


This report will provide detailed design analysis pertaining to traffic flow, parking facilities and a pedestrian overpass.  Major fields of analysis include, structural engineering, open channel flow, traffic engineering and project scheduling and cost estimation.

**Detailed Design Mandate and Requirements**

- Development of detailed Construction Scheduling
- Preliminary Cost Estimation Models for all traffic improvements
- Economic Feasibility, and timelines regarding traffic improvement implementation

**Proposed Traffic Improvements**

- Roundabout Design and Roadway Drainage Initiatives
- Porous Parking Structure with waste water collection systems
- Botanical Garden Pedestrian Overpass

All designs will undergo extensive cost estimation using the RSMeans method of cost estimation. All models will be developed using AutoCAD, Google Sketchup and Revit 2014.  Further still, tasks will be modelled after existing comparable projects using scheduling Microsoft Project 2013.

**Assigned Roles**

- Research, models, economic analysis, and construction scheduling of Roundabout – Steven, Iris
- Research models of Roadway Improvements and Drainage– Rayna, Iris
- Research models, economic analysis and construction scheduling of Parking – Saman, Mike
- Research models, economic analysis and construction scheduling of Pedestrian Overpass – Dan, Mike
- Report Compilation, Editing and Final Draft - Mike

# List of Figures



# List of Tables

# 1.0 Introduction

## 1.1 Background

The UBC Botanical Garden is situated on the SW Marine Drive and West 16th corridor. While the serene natural backdrop is among the finest curated botanical gardens in the world, accessibility and patron accessibility are of paramount concern. The UBC Botanical Garden has set forth a mandate of sustainability, improved patron experience and cost effective solutions which will maintain the original objectives of the garden to curate and categorize its countless species of flora and fauna.

From a series of site visits there are three key areas, pertaining to traffic improvements, which require immediate attention. Utilizing expertise in:

- Structural Engineering
- Open Channel Flow
- Transportation Management

Figure 1 - Current Botanical Garden Traffic Corridor

It should be noted that given the limited space within this report, various software outputs were not included, but are readily available in digital or printed media upon request.

## 2.0 Roundabout and Approach Roadway Design

Through consultation with the UBC Botanical Gardens, it became apparent that a traffic-calming device was required at the intersection of Southwest Marine Drive. The Ministry of Transportation provided details for the traffic counts from the years of 1985 to 1993. The counts for this road may be difficult to extrapolate to the present year due to improvements made further south on Southwest Marine Drive. It has been decided for a traffic circle to be utilized at the intersection of Southwest Marine Drive and Stadium Road.

### 2.1 Traffic Circle Design

Using various programs such as *Autodesk AutoCAD 2012 and Google SketchUp,* the final design meets required specifications from the Ministry of Transportation. There are many factors that have to be considered when designing a traffic circle; the following are the major factors for the UBCBG traffic circle:

- Fastest path and vehicle speeds.
- Driver and pedestrian safety.
- Lighting.
- Geometric design elements.
- The potential for UBC Botanical Garden Signage and planting areas to bring more interest for the gardens.

In order to meet requirements for a roundabout design, the following dimensions are summarized below in Table 1 referencing the design manual used.

**Table 740.A Recommended Inscribed Circle Diameter (ICD) Ranges**

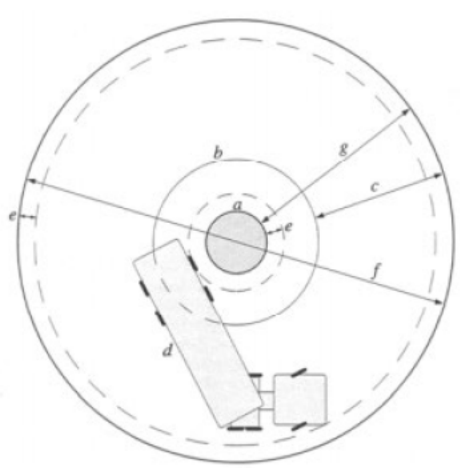
| Site Category | Inscribed Circle Diameter Range* |
|---|---|
| Urban Single Lane | 37 – 46 m |
| Urban Double Lane | 46 – 67 m |
| Rural Single Lane | 40 – 61 m |
| Rural Double Lane | 53 – 76 m |

* Assumes approximately 90-degree angles between entries and no more than four legs.

Table 1- TAC Recommended Inscribed Circle Diameter (ICD) Ranges

The Inscribed Circle Diameter was the key design feature, and limited most of the consecutive design aspects. A large raised center island was accommodated to allow the UBC Botanical Gardens to have entrance signs as shown in Figure 5. If heavy plants and signage materials are used, the scope for this design may have to be increased to allow for a geotechnical survey to be completed to ensure plant species used will not conflict with asphalt or other roadway elements.

**Figure 740.A Required Turning Widths**
(from "Roundabout Design Guidelines" Ourston Roundabout Engineering 2001)




LEGEND

*a* Raised central island.
*b* Low profile mountable apron.
*c* Remaining circulatory roadway width, 1.0-1.2 times the maximum entry width.
*d* Design vehicle.
*e* 1 meter clearance minimum.
*f* Inscribed circle diameter (ICD).
*g* Width between curbs.

NOTE: Splitter islands should not protrude into the inscribed circle if the roundabout is designed tightly as illustrated here, allowing only the minimum width g.

| Inscribed Circle Diameter (f) (metres) | Design Vehicle WB-20 (g) (metres) |
|---|---|
| 79.2 | 7.2 |
| 73.2 | 7.5 |
| 67.1 | 7.8 |
| 61.0 | 8.1 |
| 57.9 | 8.4 |
| 54.9 | 8.7 |
| 51.8 | 9.0 |
| 48.8 | 9.3 |
| 45.7 | 9.8 |
| 42.7 | 10.1 |
| 39.6 | 11.1 |
| 36.6 | 12.2 |
| 33.5 | 13.7 |
| 30.5 | ** |
| 29.0 | ** |

** Design Vehicle requires larger ICD

Figure 2 - TAC Required Turning Widths

| Legend | Required (m) | Design (m) |
|---|---|---|
| a) Raised Central Island Diameter | - | 25 |
| b) Low Profile Apron Diameter | - | 35 |
| c) Roadway Width | 7.2 | 8.5 |
| d) Design Vehicle | 8.7 | 8.7 |
| e) 1m Clearance | 1 | 1 |
| f) Inscribed Circle Diameter | 53 - 76 | 54.9 |
| g) Width Between Curbs | - | 13.4 |

Table 2 - Summary of Traffic Circle Design

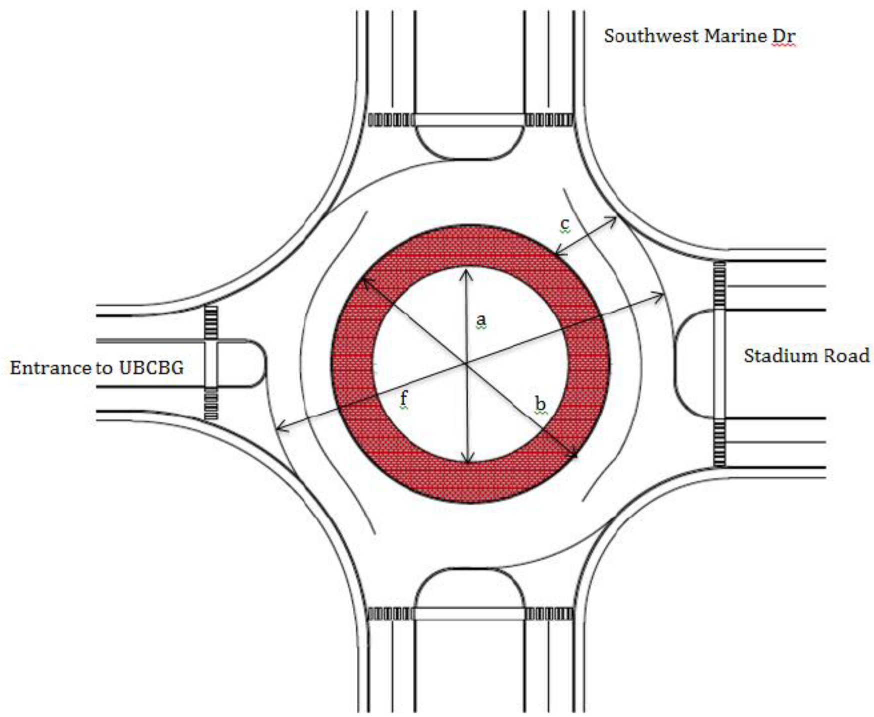



Figure 3 - AutoCAD Plan View of Traffic Circle

## 2.2 Cost of Traffic Circle

A simple cost breakdown has been performed, the construction is similar to previously completed traffic circles on West 16th Avenue as referenced in Table 3 below.

| Roundabout | Construction Cost | Percent Similar |
|---|---|---|
| Westbrook Mall | \$ 300 000 | 25% |
| East Mall | \$ 300 000 | 25% |
| SW Marine Drive | \$ 400 000 | 50% |
| | **Total Cost** | \$ 350 000 |
| | **Adjusted for year** | \$ 394 272 |

Table 3- Traffic Circle Cost

Figure 4 - Traffic Circle Entrance on Southwest Marine Drive

## 2.3 Drainage System Improvement

The roundabout construction could create complex carriageway cross –falls, and as a result, it can be extremely difficult to correctly capture standing water. Ineffective water drainage results in ponding at roundabouts, which can be particularly hazardous to pedestrians,cyclists, and vehicles. Linear Consulting proposes the following solutions to mitigate this issue. The overview of the proposed drainage system plan at the proposed roundabout is shown below, with the ACO KerbDrain system indicated in purple and French Drainage system indicated in yellow:

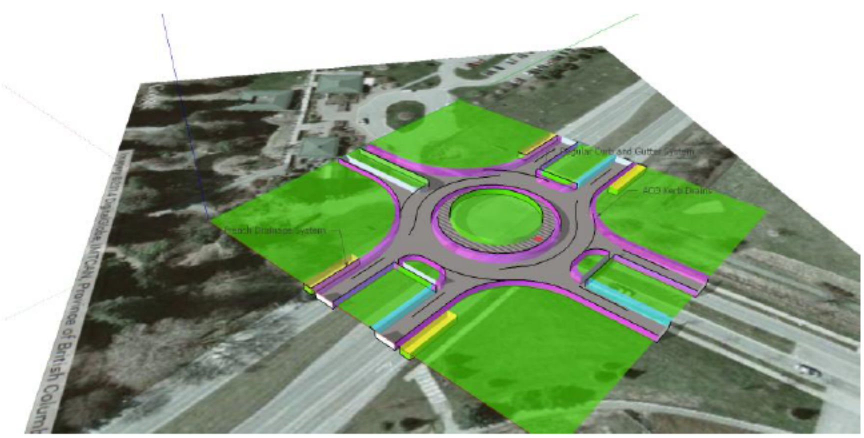


Figure 5 - Proposed Location for ACO KerbDrain Install

In order to maximize the performance of the ACO KerbDrain system, it is recommended that the installation occur at the outer edge of the circulatory roadway and the central island. In comparison to a traditional drainage system Linear Consulting has summarized the main advantages of ACO KerbDrain system in the table below.

| | CONVENTIONAL DRAINAGE SYSTEM | ACO KERBDRAIN SYSTEM |
|---|---|---|
| IMPACT RESISTANCE | Regular | 50% higher |
| CONSTRUCTION STYLE | Multiple systems | One piece construction |
| HIGH CHEMICAL RESISTANCE | X | √ |
| COST EFFECTIVENESS | X | √ |
| SAFE IN USE | X | √ |
| HIGH CAPACITY DRAINAGE PERFORMANCE | X | √ |
| AESTHETIC APPEARANCE | X | √ |

Table 4 - ACO KerbDrain Comparison

The appearance of ACO Kerbdrain System constructed in a roundabout is shown below:

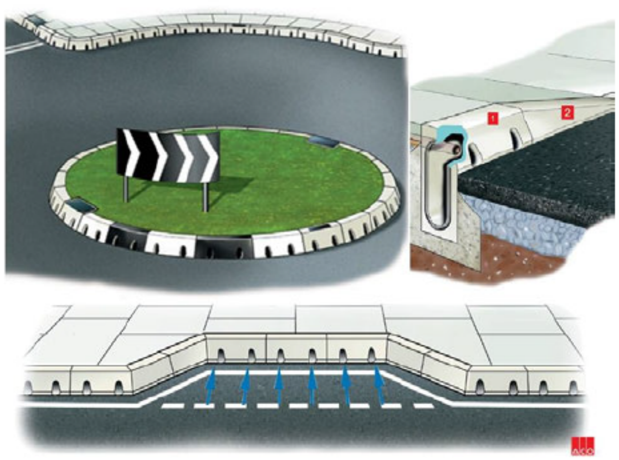


Figure 6. ACO Kerbdrain System at Roundabout

Primary investigation on the current drainage system on SW Marine Drive has led to the conclusion of the need for a more appropriate drainage system. In order to improve current drainage performance and protect the surrounding vegetation, Linear Consulting proposes to install the French Drainage System on both side of SW Marine Drive before and after entrance of  the roundabout region. A section view of French Drainage System is shown below.

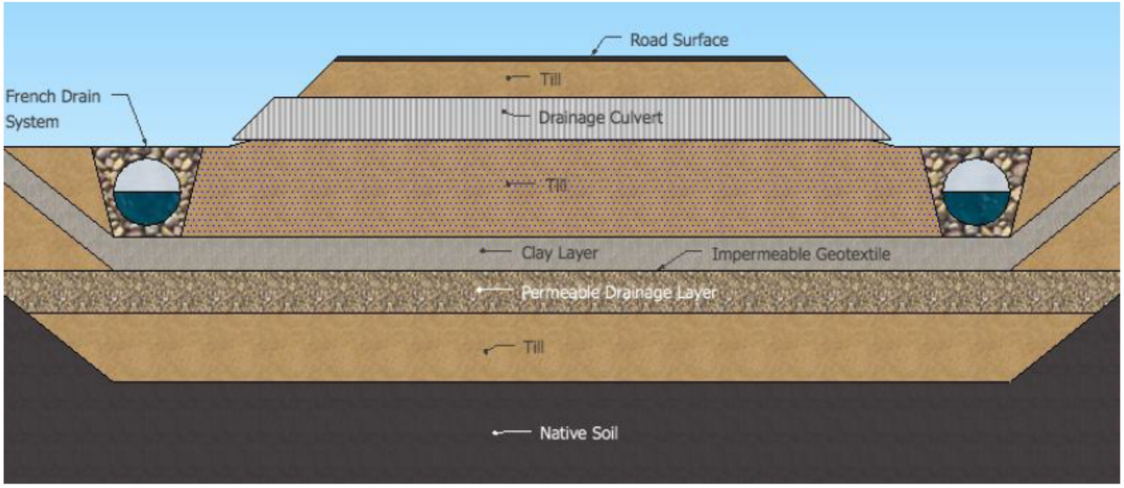



Figure 7. Cross Section of French Drainage System

# 3.0 Parking Lot Improvements

## 3.1 Increase in Capacity of Parking

Different modification options were considered to increase the capacity of the existing surface parking at the UBC Botanical Garden. One major constraint, however, limited the ability of the design team to practice many of the possible modification options. According to the garden coordinators and based on UBC rules, the parking dimensions cannot be increased beyond the current dimensions. As a result all modifications are focused within the existing layout of the parking. The first modification that is made to increase the parking capacity is removal of the sizable grass strip in the middle of the parking as it does not provide any parking space utility.

The other major modification is the change in the angle of the parking stalls. Through research it was found that reduction in the angle of parking stalls results in lower required isle width (Creative Transportation Solutions, 2005). The decrease in the angle of parking stalls results in increased ease of use and better accessibility for the patrons as well. The width of the aisle ways is also dependent on whether the isle ways will be used for one-way or two-way car travel. In order to maximize capacity of the parking, the modified isle ways will be used for one-way travel of cars only. To appropriately accommodate this modification, extra signage will be installed within the parking to guide the parking users.

The District of North Vancouver provides a detailed guideline that specifies the standard parking stall dimensions and also the required isle widths corresponding to different parking stall angles (Creative Transportation Solutions, 2005). Using this guideline and considering different recommended parking stall angles, the angle of 60 degrees was determined to result in the maximum increase in the capacity of the parking. All the modification made to the existing parking facility will result in increase of parking capacity from 84 cars to 102 cars. The plan view of the modified parking is provided in Figure 8.

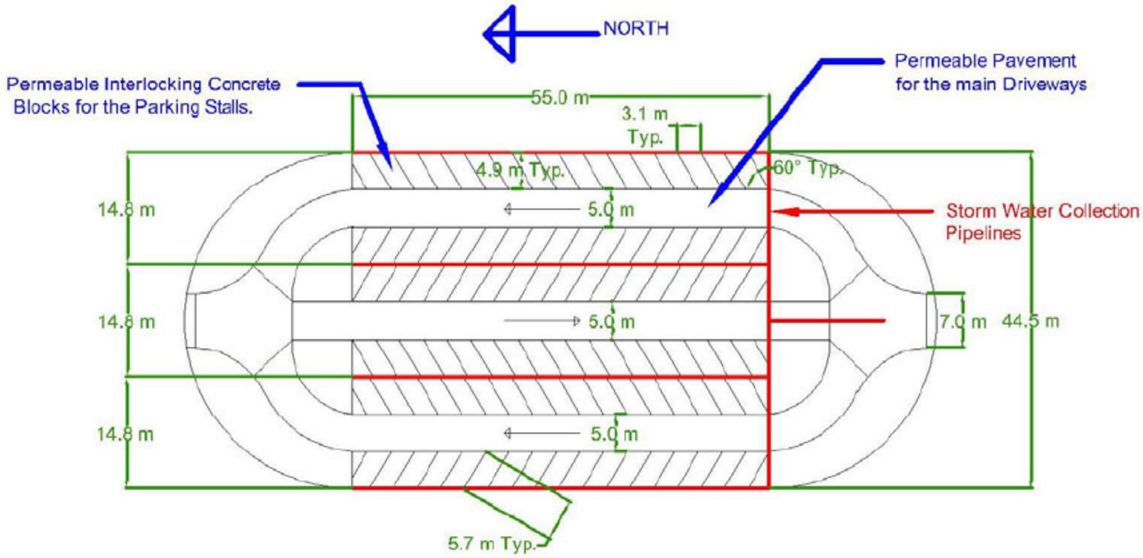



Figure 8: Plan view of the modified parking

### 3.1 The Storm water Collection system

The large surface area of the existing parking facility (approximately 2450 $m^2$) provides a great opportunity for the collection of water from precipitation. The design of the storm water collection system for the parking utilizes two major systems for collecting water. These include a primary and a secondary system. Further still, A two system design increases reliability and durability of the overall system.

### 3.2 Primary Water Collection System

The primary water collection system makes use of porous materials for the surface and four perforated pipes that will be laid longitudinally underneath the parking. Figure 8 also shows the layout and location of the perforated pipes in the parking. In addition to the use of porous materials for the surface, highly permeable gravel will replace the original layer of top soil. This will allow for efficient and easy flow of water to the collecting pipes. Flow of water in the ground will be guided towards the perforated pipes by using impermeable layers of fabric. Figures 9 and 10 show the details of this design at the North and the South cross sections of the parking respectively. The layer of soil under the perforated pipes will need to be tested for criteria such as texture classification, moisture content, bearing capacity and permeability (Metro Vancouver, 2012).

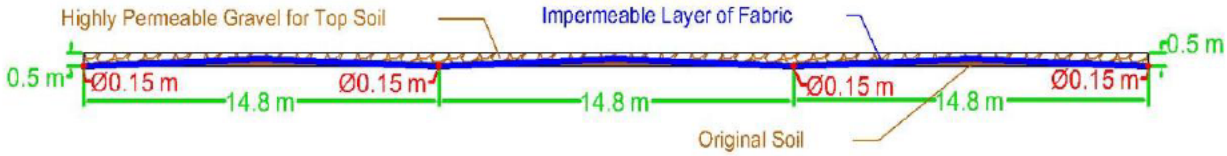



Figure 9: Cross-section of the modified parking at the North Side

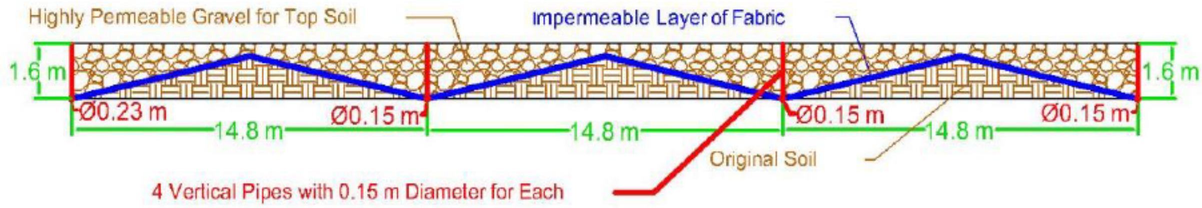



Figure 10: Cross-section of the modified parking at the South side

Two different types of porous materials will be used for the parking surface. Interlocking concrete pavements will be used for the parking stalls and permeable pavement system will be used for the aisle ways. In the interlocking concrete pavement system, the space between the concrete shells will be filled with gravel and this allows for flow of the precipitation water to the underlying layers. In addition to providing permeability, the interlocking concrete pavements contribute to the aesthetics of the parking as they allow for growth of vegetation between the concrete shells. However, as they are not suitable for use in locations with high level of vehicle traffic, they are only used for the parking stalls. The pavement system for the aisle ways will make use of specialized plastic grids that will also be filled with gravel to allow for infiltration of water into the underlying layer of gravel. Figures 11 and 12 show examples of the interlocking concrete pavement and permeable pavement systems respectively.

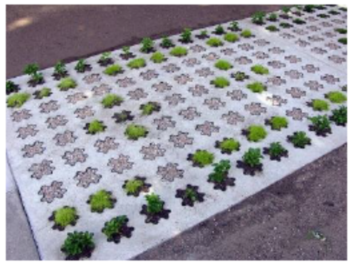


Figure 11: Interlocking concrete pavement (Green Innovations, 2010)

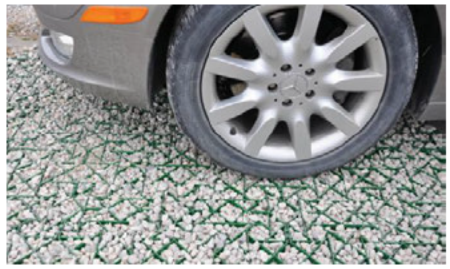


Figure 12: Permeable pavement system (Green Innovations, 2010)

Based on the data obtained during the period from 1981 to 2010 by Environment Canada, the maximum daily precipitation for the Vancouver area is 104.6 mm/day (Environment Canada, 2010). Applying an assumed safety factor of 1.5 and considering the tributary area of each of the longitudinal perforated pipes shown in Figure 11, the flow of 63.9 m^3/day and 127.7 m^3/day are expected for each of the side and interior pipes respectively. The guidelines related to design of porous pavement systems in the city of Vancouver specify a minimum diameter of 0.15 m for the perforated pipes (Metro Vancouver, 2012). In order to satisfy the maximum water flow demand and comply with the design guidelines for the city of Vancouver, a diameter of 0.15 meters is selected for each of the perforated PVC pipes. With the slope of 3 % for all of the 4 pipes and considering the maximum expected flow, 1.65 meters of energy in the form of head are calculated for each of the pipes. The calculated energy heads correspond to the energy at the South end of the perforated pipes where water will be delivered to a main pipe to be transferred to a storage tank or to the garden’s proposed water management system. The positive energy heads at the end of the pipes ensure the easy flow of water towards the South side of the parking without the need for a pump. Please refer to Appendix B for details of the calculations for water flow demands and also energy calculations.

## 3.3 Secondary Water Collection System

For the secondary water collection system the surface of the parking will be sloped at 1% from North to South. Four vertical PVC pipes each with a diameter of 0.15 m will be placed at the South side of the parking to deliver water from surface to the main pipes. This system will help to direct the storm water to the main drainage pipes when the primary system becomes overloaded and prevents flooding to take place at the parking. Furthermore, the guidelines require the inclusion of a secondary overflow system and a minimum surface slope of 1% for porous pavement systems in Vancouver (Metro Vancouver, 2012). Figure 13 provides an elevation view of the modified parking cross section.

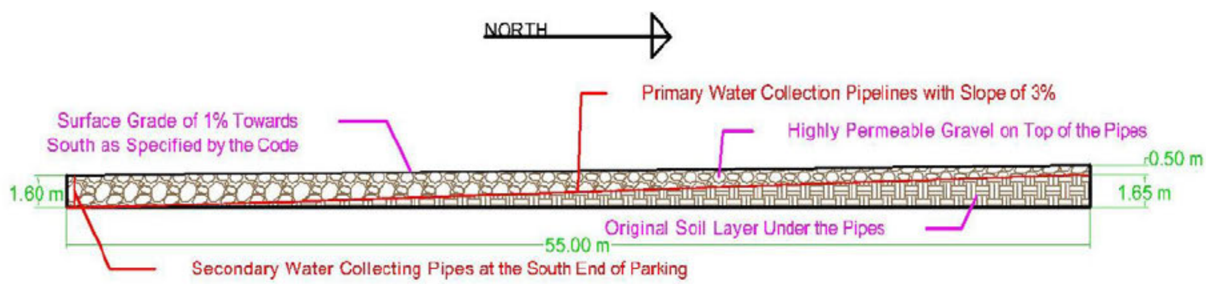



Figure 13: Side cross-section of the modified parking design

## 3.1 Cost Estimation & Scheduling

Utilizing RSMeans Cost estimation, and Sigma Enterprise cost analysis, the parking lot is estimated to cost approximately $121,566. Due to space limitations, it should be noted that the Figure 14 is a sample output of the overall cost estimate. As seen in Figure 15, the costs are divided primarily towards labor, as standardized componentry keeps costs low. The implementation of the improvement plan proposed for the parking at the UBC Botanical Garden will involve four major phases. Figure 16 demonstrates the proposed construction schedule in more details. In total, the parking installation is expected to have a construction time line of approximately 3 months

| Pos | Text | Num | Category | Unit | Quantity | Unit Cost | Cost | Total UC | Total Cost | Reg. |
|---|---|---|---|---|---|---|---|---|---|---|
| | **My Estimate** | | | | | | **116,251.00** | | **116,251.00** | |
| | **Total supplement (0.00% of 128,819.00)** | | | | | | | 0.00 | | 0.00 |
| 1. | **Ashpalt Removal** | | | | | | **52,099.20** | | **52,099.20** | |
| 1.1. | **Demolish, remove pavement & curb, remove bituminous pavement, 3" thick, excludes hauling and disposal fees** | | | **S.Y.** | **10,000** | **4.02** | **40,200.00** | **4.02** | **40,200.00** | **1** |
| 1.1.1. | Common Building Laborers Outside Foreman | CLABO | Labor | Hours | 0.0116 | 37.10 | 0.43 | 37.10 | 0.43 | 1 |
| 1.1.2. | Common Building Laborers | CLAB | Labor | Hours | 0.0232 | 35.10 | 0.81 | 35.10 | 0.81 | 1 |
| 1.1.3. | Equipment Operators, Light Equipment | EQLT | Labor | Hours | 0.0116 | 44.75 | 0.52 | 44.75 | 0.52 | 1 |
| 1.1.4. | Equipment Operators, Medium Equipment | EQMD | Labor | Hours | 0.0116 | 46.55 | 0.54 | 46.55 | 0.54 | 1 |
| 1.1.5. | Backhoe Loader, 48 H.P. | 015433200450 | Equipment | Days | 0.0014 | 334.00 | 0.48 | 334.00 | 0.48 | 1 |
| 1.1.6. | Hyd.Hammer, (1200 lb.) | 015433200486 | Equipment | Days | 0.0014 | 178.40 | 0.26 | 178.40 | 0.26 | 1 |
| 1.1.7. | F.E. Loader, W.M., 4 C.Y. | 015433204730 | Equipment | Days | 0.0014 | 611.00 | 0.89 | 611.00 | 0.89 | 1 |
| 1.1.8. | Pvmt. Rem. Bucket | 015433500740 | Equipment | Days | 0.0014 | 59.00 | 0.09 | 59.00 | 0.09 | 1 |
| 1.2. | **Demolish, remove pavement & curb, remove bituminous pavement, 3" thick, excludes hauling and disposal fees** | | | **S.Y.** | **2,960** | **4.02** | **11,899.20** | **4.02** | **11,899.20** | **1** |
| 1.2.1. | Common Building Laborers Outside Foreman | CLABO | Labor | Hours | 0.0116 | 37.10 | 0.43 | 37.10 | 0.43 | 1 |
| 1.2.2. | Common Building Laborers | CLAB | Labor | Hours | 0.0232 | 35.10 | 0.81 | 35.10 | 0.81 | 1 |
| 1.2.3. | Equipment Operators, Light Equipment | EQLT | Labor | Hours | 0.0116 | 44.75 | 0.52 | 44.75 | 0.52 | 1 |
| 1.2.4. | Equipment Operators, Medium Equipment | EQMD | Labor | Hours | 0.0116 | 46.55 | 0.54 | 46.55 | 0.54 | 1 |
| 1.2.5. | Backhoe Loader, 48 H.P. | 015433200450 | Equipment | Days | 0.0014 | 334.00 | 0.48 | 334.00 | 0.48 | 1 |
| 1.2.6. | Hyd.Hammer, (1200 lb.) | 015433200486 | Equipment | Days | 0.0014 | 178.40 | 0.26 | 178.40 | 0.26 | 1 |
| 1.2.7. | F.E. Loader, W.M., 4 C.Y. | 015433204730 | Equipment | Days | 0.0014 | 611.00 | 0.89 | 611.00 | 0.89 | 1 |
| 1.2.8. | Pvmt. Rem. Bucket | 015433500740 | Equipment | Days | 0.0014 | 59.00 | 0.09 | 59.00 | 0.09 | 1 |
| 2. | **Berm Removal** | | | | | | **24,570.00** | | **24,570.00** | |
| 2.1. | **Structural excavation for minor structures, bank measure, sandy soil, pits to 6' deep, hand** | | | **B.C.Y.** | **700** | **35.10** | **24,570.00** | **35.10** | **24,570.00** | **1** |

Figure 14 - RSMeans Cost Estimation

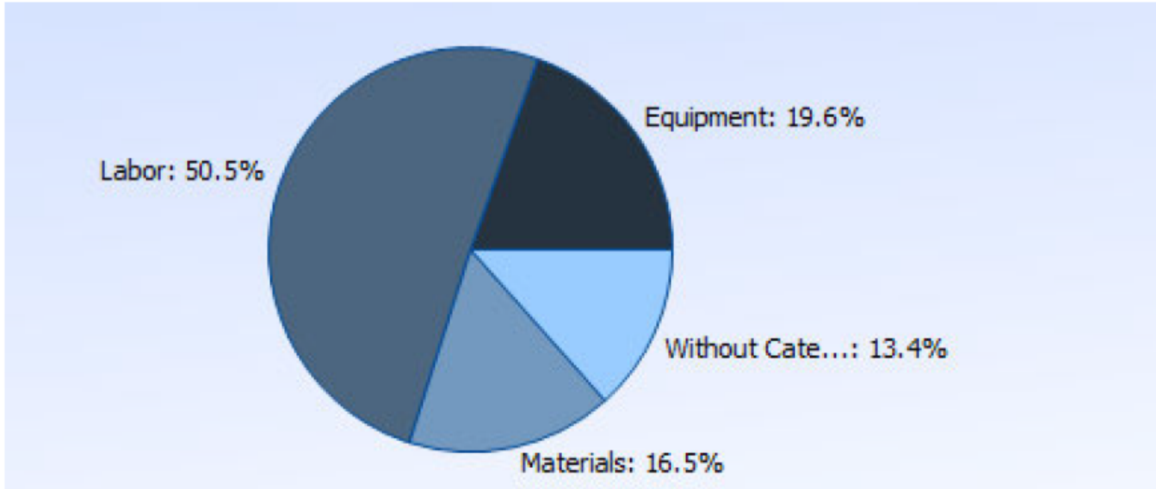
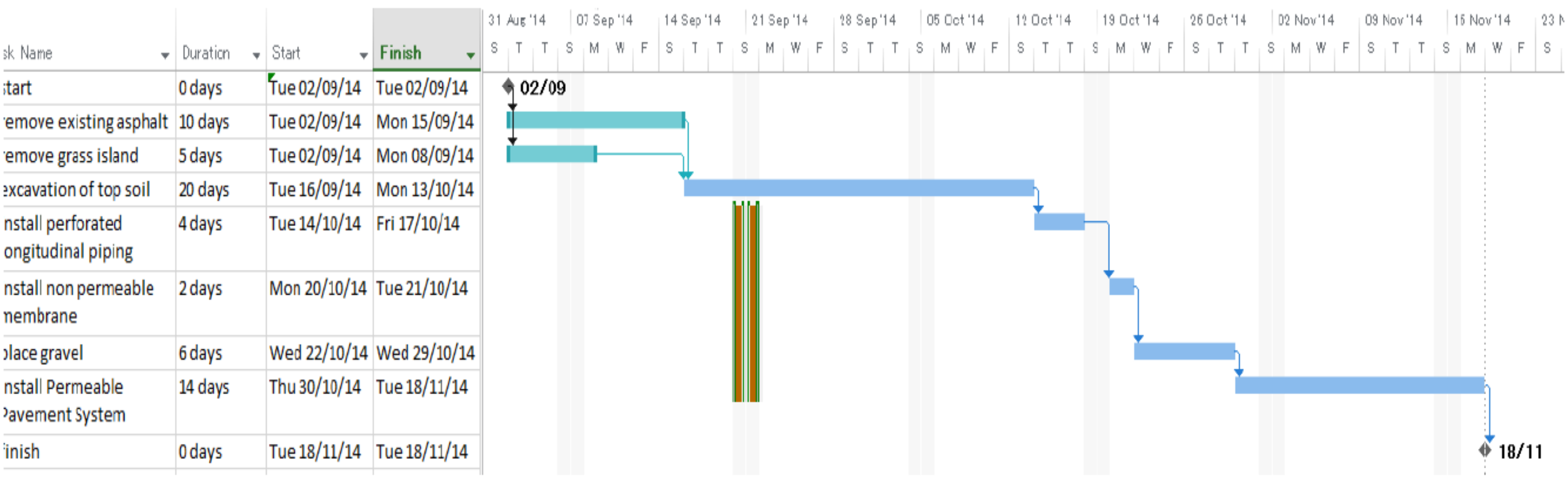



**Figure 15 - Costing Breakdown**



| ;k Name | Duration | Start | Finish |
|---|---|---|---|
| ;tart | 0 days | Tue 02/09/14 | Tue 02/09/14 |
| emove existing asphalt | 10 days | Tue 02/09/14 | Mon 15/09/14 |
| emove grass island | 5 days | Tue 02/09/14 | Mon 08/09/14 |
| xcavation of top soil | 20 days | Tue 16/09/14 | Mon 13/10/14 |
| nstall perforated ongitudinal piping | 4 days | Tue 14/10/14 | Fri 17/10/14 |
| nstall non permeable nembrane | 2 days | Mon 20/10/14 | Tue 21/10/14 |
| place gravel | 6 days | Wed 22/10/14 | Wed 29/10/14 |
| nstall Permeable Pavement System | 14 days | Thu 30/10/14 | Tue 18/11/14 |
| inish | 0 days | Tue 18/11/14 | Tue 18/11/14 |



**Figure 16- Parking Lot Schedule**

## 4.0 Pedestrian Bridge Detailed Design

In keeping with the UBC Botanical Garden's goals of increasing accessibility while establishing the garden as a key landmark on campus, Linear Consulting proposes the installation of a pedestrian overpass to span over the Southwest Marine Drive corridor.  Aside from these top-priority project requirements, Linear Consulting recognizes the need for financial and societal considerations during the preliminary design phase.  Detailed in this section are the justifications guiding the proposed final design, relevant analysis and adapted methodologies, cost estimates, and a construction schedule.

### 4.1 Design Justifications

Linear Consulting Ltd. considered whether or not an installation of a pedestrian bridge to span over Southwest Marine Drive is deemed feasible.  Based on consultations with the garden administrative staff and multiple site-visit observations, it is recommended that a pedestrian bridge should be installed with the expected benefits, as summarized below:

| Design Justification | Description |
|---|---|
| Increased accessibility between the north and south garden areas. | The only real access between both garden grounds is through the Moon Gate tunnel, which is deemed inefficient by the garden staff. |
| Increased pedestrian safety. | Coupled with the proposed detailed roundabout design, the pedestrian overpass is expected to provide for an extra measure of safety against vehicular traffic. |
| Increased garden attraction. | At present, the garden entrance lacks the ability to attract needed attention. |
| Increased revenue. | The general population would be more inclined to visit the garden as a result from the increase in garden attraction. |

Table 5 - Design Justification Table

It is expected that the design and construction would be a large financial undertaking for the botanical garden. However, it should be noted that given the long term benefits as described previously, initial investments are expected to be recouped eventually.

A comprehensive approach was undertaken to select the pedestrian bridge design to span over the Marine Drive corridor. Multiple design charrettes were conducted, where key advantages and disadvantages to typical bridge design type were brainstormed, with guidance from external research material in the form of precedent studies and literature such as the Bridge Engineering Handbook (Chen, Duan, 2000).

| Bridge Design Type | Advantages | Disadvantages |
|---|---|---|
| Arch | • Architecturally pleasing.<br>• Minimal obstruction of view.<br>• Prefabricated components-ease of construction.<br>• Minimal traffic impact.<br>• Relatively simple design. (mainly compressive forces).<br>• One of a kind structure at UBC. | • Steel members need to be fairly large<br>• Horizontal foundation loads |
| Suspension | • Architecturally pleasing.<br>• Good for long spans. | • Supporting column can only be built in the median. Building support columns at the ends will lead to unwanted removal of flora and trees.<br>• Column construction will significantly obstruct traffic for long periods of time. |
| Truss | • Relatively simple design (mainly axial forces).<br>• Prefabricated components. | • Obstructs overall view.<br>• Not architecturally pleasing |
| Cable-stayed | • Architecturally pleasing.<br>• Good for very long spans. | • Unnecessary, given the estimated span length of approximately 70m.<br>• Not economically viable given the relatively short span. |

Table 6 - Bridge Design Characteristics

It was determined that the preferred design type is the steel arch bridge. Apart from the design being a modern architecturally pleasing structure, the prefabricated components reduce the overall construction time and impact on vehicular traffic. Additionally, the very nature of this bridge design eliminates the need for a structural support to be installed at the road median since the structural load is directly transferred to the arches. Lastly, UBC-Vancouver has yet to have a structure of this kind on campus. Thus, the installation of an arch structure aids in establishing UBCBG as an iconic landmark on campus.

### 4.2 Analysis and Design Methodologies

Linear Consulting Ltd. specializes in a wide variety of engineering services including structural engineering. Our structural engineering division is comprised of motivated individuals from UBC's civil engineering department. Their structural engineering academic background coupled with their past experience with the most current structural software such as SAP200 has led to the bridge design which will be described shortly.

The analysis and design was accomplished through an iterative procedure that involved both hand calculations and the use of structural engineering software. Outlined below in Table 7 are the steps taken in designing the overall structure. All calculations were performed in accordance with provincial and national design standards and regulations. Limitations on the scope and time to perform detailed designs led the team to focus only on the steel arches, tension cables, and concrete slab with edge beams. Linear Consulting is willing to provide a detailed design on approach ramps and foundations, pending the successful reward of the contract from UBCBG. Furthermore, only summarized results from the analysis and design phase. Computer output is readily available upon request.

| Design Steps | Description |
| --- | --- |
| 1. Review of the winning conceptual pedestrian overpass designs from CIVL 445. | • Applicable design features were extracted from the winning groups to be considered in the final proposed design. |
| 2. Conduct precedent studies and gather | • Different bridge designs from around the world |

| | |
|---|---|
| reference material. | were reviewed.<br>• CSI Reference Manual aided in computer modelling. |
| 3. Establish basic design dimensions. | • Using Google Earth. Overall deck length, L=70m.<br>• As per BC MoTI provisions, minimum deck width=2.5m, design set for 3 m.<br>• As per BC MoTI provisions, minimum road clearance=5.5m, design set for 5.5 m. |
| 4. Establish necessary design provisions, regulations, and standards. | • Concrete components were designed as per CSA A23.3-04.<br>• Steel components were designed as per CSA S16-09 (Done in SAP2000 software).<br>• Loading conditions were gathered from CSA S6-06.<br>• Restrictions and design regulations were dictated by BC MoTI's Bridge Standards and Procedures Manual. |
| 5. Define necessary loading conditions and load combinations. | • Specific loading guidelines, load factors, and their corresponding load combination equations were extracted from CSA S6-06. (See Appendix A) |
| 6. Perform hand calculations for concrete slab and edge beams. | • SAP2000 feature: dead load calculations are automatically done through the dead load multiplier. For SAP2000 to size the steel members, the overall load from the concrete deck must be established. The overall load includes the deck dead load, dictated by the size of the concrete components.<br>• Sizing of the concrete deck was performed using the defined loading conditions and combinations and CSA A23.3-04. |
| 7. Establish a simplified model on SAP2000. | • Frame material and area section properties were defined in SAP2000. |
| 8. Verify the validity of the simplified model. | • Arbitrary loads were imposed on the concrete deck and the software analysis was done. Hand calculations based on tributary widths were compared to SAP2000 output (see Appendix A) |
| 9. Modify the final computer model. | • The computer model was modified to emulate the final bridge design (i.e. angle the arch members out – will be discussed shortly). |
| 10. Define load values and load combinations | • The calculated load values (dead load, live load, wind load) were imposed on the structure.<br>• As per CSA S6-06, three wind pressure directions were defined (vertical down, horizontal, vertical up).<br>• Load combinations were extracted from CSA S6-06 and manually imported into the computer model. |

| | |
|---|---|
| 11. Run software analysis. | • Governing load combination used for concrete calculations: 1.1 DL +1.7LL<br>• Governing load combinations used to design steel arches: 1.5DL+1.4LL+0.5W |
| 12. Design frame sections. | • Design preferences were set to CSA S16-09 for the steel arches.<br>• Deflection limits were enabled. |
| 13. Change member sizes, if applicable. Repeat steps 11 and 12. | • Member sizes were increased if the calculated stresses in SAP2000 were more than recommended. |

Table 7- Detailed Design Process

Expected member loading values from the governing load combination with corresponding capacities are included below. The maximum loading values and capacities for the steel arches were determined from the SAP2000 software. Loading values and capacities for the concrete deck were all determined through hand calculations as per CSA A23.3-04.

| COMPONENT | CAPACITY | MAX FORCE IN ANY MEMBER |
|---|---|---|
| HSSx14x0.375 | Axial=2557kN<br>Moment=331kNm<br>Shear=1784kN | Axial=1600kN<br>Moment=150kNm<br>Shear=26 kN |
| 1" steel cables. | Axial=200kN | Axial=165kN |
| Concrete slab | Moment=25.08kNm<br>Shear=81kN (all from concrete) | Moment=11.54kNm<br>Shear=15.39kN |
| Concrete Edge beam | Moment=92.11kNm<br>Shear=200kN | Moment=68.14kNm<br>Shear=50kN |

Table 8- Component Capacities

Detailed concrete deck and steel cable size hand calculations are included in Appendix A for further reference. Hand calculations verifying the validity of the computer model are also included in Appendix A.

The following figures below illustrate a graphical representation of a typical analysis and design output from SAP2000. Relevant quantitative software output is included in Appendix A for reference purposes.

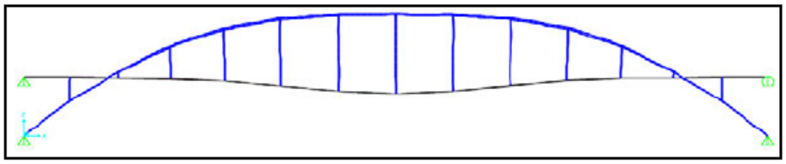


Figure 17- SAP2000 Load Capacity

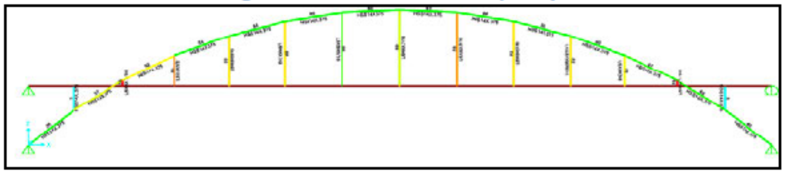


Figure 18 - SAP2000 Member Sizing

## 4.3 Final Design

Specific design features that were determined are presented below. As previously stated, all calculations and design determinations were based off provincial and national regulations, provisions, and standards. Further design work for the shallow footing to support the steel arches and the approach ramps needs to be performed.

### 4.3.1 General Structure Layout.

The bridge superstructure spans over a 70 m distance from the primary garden entrance to the northeastern corner of the Southwest Marine Drive and Stadium Road intersection. The deck is located 5.5m above the existing road elevation, while the arch apex is 13m above the road elevation. The proposed layout can be seen in the figures below.

Figure 20 - Proposed Bridge Location

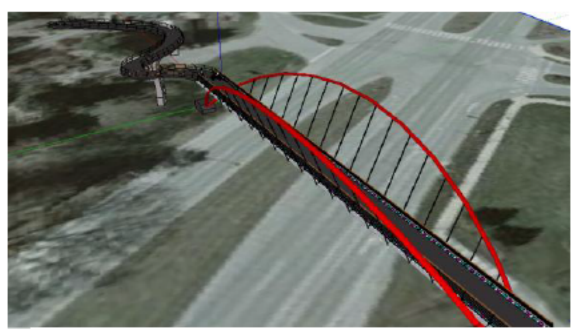


Figure 19 - Bridge Model Overlay

An approach ramp is to be included on the primary entrance-side of the bridge to get up to the minimum 5.5 m grade as per BC MoTI provisional standards. It was determined that the existing height of the northern end of the bridge (Figures 20 & 21) is already 5.5 m above the ground. In order to save costs, the span is oriented such the northern end spans to that 5.5m height, eliminating the need for a ramp to get up to that minimum height. Additionally, the steel arches are oriented outwards by 13° for a more aesthetically pleasing structure.

Figure 21 - Bridge Design Detail

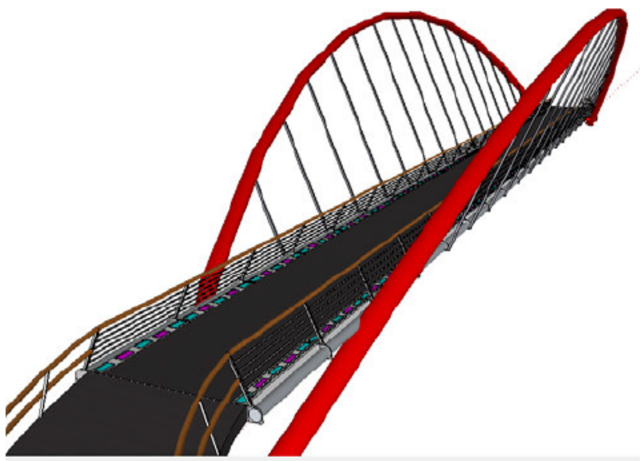


Figure 22 - Bridge Design Isometric View

### 4.3.2 Determined structural member sizes.

Based on the conducted analysis design, Linear Consulting Ltd. recommends the following sizes and dimensions to form the proposed pedestrian overpass. The concrete deck is to be constructed with prefabricated modules to be transported on-site for assembly. The deck modules include a 150mm thick concrete slab attached to two 250mm x 450mm edge beams on either side to increase stiffness and limit deflections.

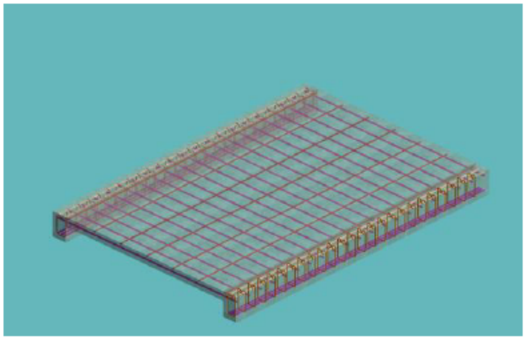
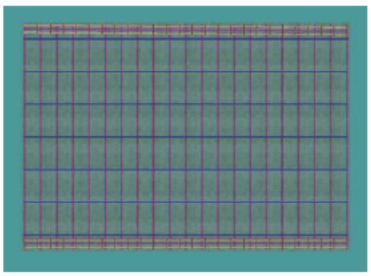
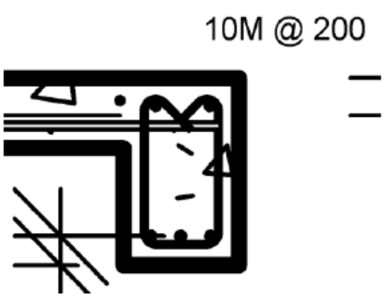
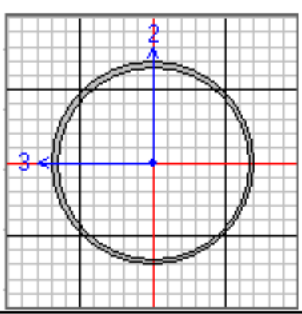
| Structural component | Sizes |
|---|---|
| Prefabricated concrete deck module<br> | • Module Length, L=5500mm.<br>• Total module width, w=3500mm.<br>• Module slab thickness, hs=150mm.<br>• Module beam height, h=400mm. |
| Slab detail<br> | • Slab longitudinal length = 5500mm.<br>• Slab transverse width = 3000mm.<br>• Slab thickness = 150mm<br><br>• Reinforcement:<br>  ○ 15M@250 main flexural reinforcement (transverse direction)<br>  ○ 15M@500 temperature and shrinkage. |
| Edge beam detail<br> | • Beam longitudinal length = 5500mm.<br>• Beam depth = 400mm.<br>• Beam width = 250mm.<br><br>• Reinforcement:<br>  ○ 3-20M main flexural (longitudinal direction)..<br>    ▪ 40mm cover.<br>  ○ 2-20M top anchor bars.<br>  ○ 10M@200 shear reinforcement.<br>    ▪ 135° hooks at anchor bars. |
| HSS14x0.375 Steel arch detail.<br> | • Length = 5450mm.<br>• Outer diameter = 14” (356mm).<br>• Thickness = 0.375” (9.74mm). |
| Steel cables | • Length = varies, thickness=1” (25.4mm) |

Figure 23- Structural Component Details

## 4.4 Cost Estimates

For the Botanical Garden Pedestrian Overpass a series of cost estimation methods were utilized to provide a comprehensive cost estimate for the installation, operation and maintenance of this visually stunning structure. It should be noted that recuperation and feasibility analysis of the installation of this relatively large scale structure are outside of the scope of Linear Consulting's detailed design analysis package.

### 4.4.1 Preliminary Costing

As a pedestrian overpass construction of this scale has not been undertaken on the UBC Endowment lands, preliminary cost estimates were derived from a number of reputable sources. Once preliminary designs, and overpass dimensions were properly sized out, industry experts were contacted to provide rough cost figures which could be used to further refine the detailed RSMeans cost estimation model. Given a preliminary costing of "approximately $8000 to $10000 per square meter of deck area depending on soil conditions and special bridge features" (Jiang, 2014) We estimated that the bridge would cost approximately $2.1 million as seen in Table 9. The preliminary costing information was obtained using estimation spreadsheets obtained from the New York Transportation Authority, and served as the basis for subsequent in depth analysis.

| | | | | |
|---|---|---|---|---|
| **TOTAL BRIDGE COST** | | | | |
| **$ / ft² SB AREA =** | $3,556 | | | |
| Shoulder Break Area (ft²) | | 384 X Cost / ft² $3,556 = BRDGE ONLY COST | | $1,366,000 |
| | **Contingencies** | Remove existing bridge | | $0 |
| | | Work Zone Traffic Control (WZTC) | | $50,000 |
| | | Detour structure | | $0 |
| | | Channel work | | $10,000 |
| | | Slope protection, other than for channel work | | $10,000 |
| | | Utilities | | $20,000 |
| | | Aesthetics (e g. Form liners, decorative railing, lights & stone facades) | | $10,000 |
| | | MSE for abutments. Specified "Plain" $53, "As Shown" $102 per ft² of MSE | | |
| | | Overhead (e g.Construction office, computer software & hardware, office supplies) | | |
| | | Input as decimal for anticipated year of letting: | | |
| **Simple Inflation Rate For SFY** | | 11/12 to 12/13 - 3 0% 12/13 to 13/14 - 3.0% 13/14 to 14/15 - 3 0% | | 1.400 |
| | | **TOTAL BRIDGE SHARE** (Includes additional 4 % for mobilization) | = $ | 2,134,496 |
| rev.4/2014 | | | | |

Table 9 - Prelimnary Pedestrian Overpass Cost

### 4.4.2 RS Means Cost Estimation

Using the MasterFormat RSMeans Cost Estimation, the pedestrian bridge was broken down into its individual components and unitized for a comprehensive pricing for the project. Due to economic constraints, costing information was obtained from the RSMeans General Unit Library which did not contain specific heavy construction items. However, adjustments were made to account for location, general contractor markups, and use of prefabricated components. As seen in table 9 below, with the finer granularity and annual costing information provided by RSMeans, the determined cost of the pedestrian bridge was approximately $1.84 million.

| | | | |
|---|---|---|---|
| Subtotal | | 1,396,053.39 | 0.00 |
| General Contractor's Markup on Subs | 20.00% | $0.00 | $0.00 |
| Subtotal | | $1,396,053.39 | $0.00 |
| General Conditions | 10.00% | $139,605.34 | $0.00 |
| Subtotal | | $1,535,658.73 | $0.00 |
| General Contractor's Overhead and Profit | 20.00% | $307,131.75 | $0.00 |
| Subtotal | | $1,842,790.48 | $0.00 |
| Grand Total | | $1,842,790.48 | |

Table 10 - RS Means Costing Information for the Pedestrian Bridge

## 4.5 Scheduling/Implementation Alternatives

The pedestrian overpass makes extensive use of prefabricated componentry and has similar construction to a variety of existing bridges in the Lower Mainland. As a result, the Botanical Garden Overpass preliminary design, testing, site work, right of way determination and construction support provided by Linear Consulting should occur within the time span of 1 year as seen below in Figure 25. Further still, the scheduling was confirmed through referencing the Massachusetts Department of Transportation typical bridge design schedules. The tie-back method is recommended for the construction of the bridge. Allowing temporary piers with cables to support the arches during construction minimizes the overall impact on traffic during construction (Chen & Duan, 2000).

| Name | Start |
|---|---|
| Contract/Project Management | Tue 14/01/14 |
| Project Milestones | Tue 14/01/14 |
| Issue Design Contract NTP | Tue 14/01/14 |
| PM Submits Documents for Advertising | Wed 08/04/15 |
| Prepare for Advertise | Sat 11/04/15 |
| Advertise Construction Contract | Sat 18/04/15 |
| Bid Opening | Wed 17/06/15 |
| Issue Construction Contract NTP | Sun 16/08/15 |
| Scope of Service Complete | Mon 15/08/16 |
| Project Development | Tue 14/01/14 |
| Bridge Design | Tue 14/01/14 |
| Environmental | Tue 14/01/14 |
| Right of Way | Tue 14/01/14 |
| Right of Way Milestones | Tue 14/01/14 |
| Preliminary ROW Plans | Tue 14/01/14 |
| Prepare Preliminary ROW Plans | Tue 14/01/14 |
| Submit Prelimnary ROW Plans | Wed 12/02/14 |
| Review Preliminary ROW Plans | Thu 13/02/14 |
| Prepare Preliminary ROW Plans Rev1 | Fri 14/03/14 |
| Submit Prelimnary ROW Plans Rev1 | Fri 14/03/14 |
| Review Preliminary ROW Plans Rev1 | Fri 14/03/14 |
| Layout Plans and Order of Taking | Sat 15/03/14 |
| Perform Title Exams (All TPI) | Sat 15/03/14 |
| Conduct Interviews (All TPI) | Wed 14/05/14 |
| Perform Appraisals (All TPI) | Sun 13/07/14 |
| Prepare Layout Taking Plans & Instrument | Sat 15/03/14 |
| Submit Layout TPIs | Tue 13/05/14 |
| Review Layout TPIs | Thu 11/09/14 |
| Prepare Layout Taking Plans & Instrument Rev | Sat 11/10/14 |
| Submit Layout TPIs Rev 1 | Sun 09/11/14 |
| Review Layout TPIs Rev 1 | Mon 10/11/14 |
| Approve Acquisitions | Wed 10/12/14 |
| Relocation | Wed 10/12/14 |
| Final ROW Plans | Mon 17/08/15 |
| Prepare Final ROW Plans | Mon 17/08/15 |
| Submit Final ROW Plans | Sun 23/08/15 |
| Review Final ROW Plans | Mon 24/08/15 |
| Construction Engineering | Mon 17/08/15 |
| Construction Support Services | Mon 17/08/15 |
| Construction Support Services | Mon 17/08/15 |

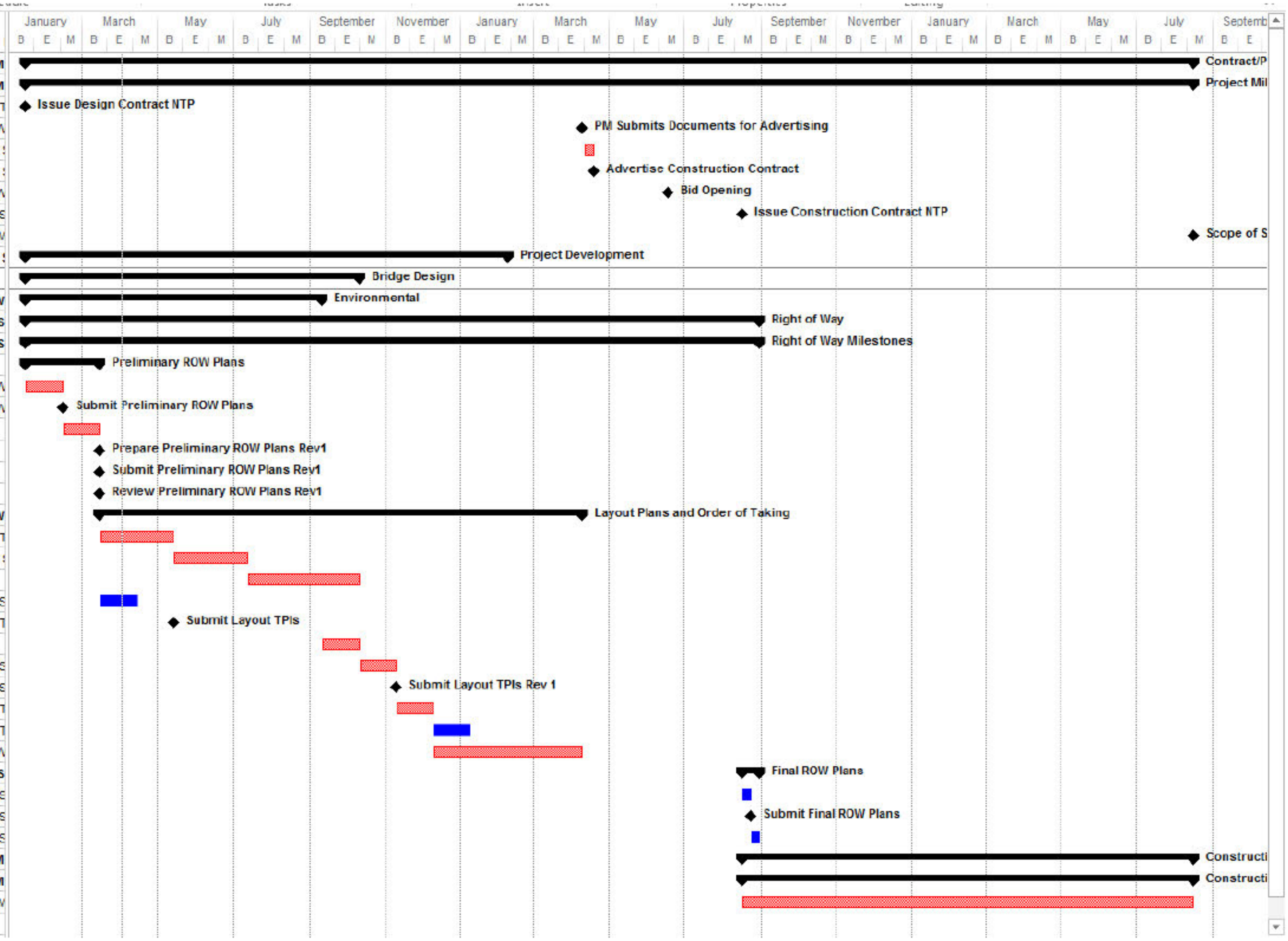



**Figure 24 - Pedestrian Overpass Design Schedule**

## 5.0 Conclusion

Linear Consulting is confident that with the commencement of the traffic improvement initiative, the UBC Botanical Garden will drastically improve the user experience while contributing positively to the social and economic landscape of the surrounding area. Comprehensive detailed design of the Pedestrian Bridge and Parking Lot provide an effective and sustainable solution to a number of the Botanical Garden's current issues.  Given the extensive costs and time demands of this project, it is clear that all implementations should be phased pending funding and resource availability. Moreover, the installation of these landmarks will provide the UBC Botanical Garden with curb side recognisability as well as improved land use and usability.  Thus, it is Linear Consulting's strong recommendation that all of these proposed improvements be further designed and implemented.

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## Appendix A – Bridge Sample Calculations

**CONCRETE SLAB DESIGN:**

Flexural Design:

As determined:

- Specified span length = 70000mm (Google Earth ruler tool)
- Specified slab width = 3000mm (As per 1.5.2.3 of BC MoTI Bridge Standards and Procedures Manual Volume 1, minimum width = 2000mm).

$$L_1 / L_2 = 70000 / 3000 = 23.33 > 2 \therefore \boldsymbol{One - way\ slab\ analysis.}$$

- As per CSA A23.3-04, consider a unit strip of width b = 1000mm (along the longitudinal bridge direction).
- Chosen concrete strength f'c = 30 MPa.
- Concrete unit weight $\gamma = 24\ \frac{kN}{m^3}$.

1) Estimate slab thickness.
   As per Cl. 9.8.2.1 CSA A23.3-04, it is prescribed that $h \geq l_n/20$ satisfies CSA A23.3-04 deflection requirements (No detailed deflection calculations needed).

$$h = \frac{3000mm}{20} = 150mm.$$

2) Factored bending moment, $M_f$:
   - As per 3.6 of BC MoTI Bridge Standards, dead load calculations must include at least 50mm of asphalt overlay.
   - Chosen asphalt unit weight = 16 kN/m$^3$ (Source:http://www.rjcsolutions.com/calculators/grav-den.htm)
   - As per CL. 3.1 CSA S6-06, snow loads are not specified because in normal circumstances, the occurrence of considerable snow load will cause a traffic load reduction.
   - As per CL. 3.10.2.3, the vertical wind case, Fv, shall be taken to act up or down.
   - It was assumed that other loads specified in CSA S6-06 (E, P, K, V, S, EQ, F, A, H) can be neglected, given the time and scope limits on the design process.

Dead load, DL = slab self weight + asphalt overlay + railings.

$$Specified\ DL = 24\frac{kN}{m^3} * 0.15m * 1m + 16\frac{kN}{m^3} * 0.05m * 1m + 2 * 1.2\frac{kN}{m} * \frac{1m}{3m} = 5.2\ \frac{kN}{m}$$

Live Load, LL = pedestrian load (CL. 3.8.9 CSA S6-06)

$$1.6\ kPa \leq LL = 5.0 - \frac{s}{30} \leq 4.0\ kPa; s = 70, total\ loaded\ length, m$$

$$1.6kPa \leq 2.67kPa \leq 4.0kPa\ \therefore \boldsymbol{OK}!$$

Wind Load, WL (CL. 3.10.2.3 CSA S6-06)

$$F_v = qC_eC_gC_v$$

$$q = 480\ Pa\ (Table\ A3.1.1\ CSA\ S6-06\ for\ a\ return\ period\ of\ 50\ years.$$

$$C_e = 1.1\quad Table\ 3.8\ \&\ CL. 3.10.1.4\ CSA\ S6-06\ .$$

$$C_g = 2.5\quad CL. 3.10.1.3\ CSA\ S6-06\ .$$

$$C_v = 1.0$$

$$F_v = 1.32\ kPa.$$

From Table 3.2 (Load combinations), the worst ULS combination, excluding E, P, K, V, S, EQ, F, A, H:

$$ULS\ Combo\ 1 = 1.1DL + 1.7LL = 1.1 * 5.2 + 1.7 * 2.67 = 10.26\ \frac{kN}{m}. \leftarrow \boldsymbol{GOVERNS}!$$

$$ULS\ Combo\ 2 = 1.1DL + 1.4LL + 0.5WL = 1.1 * 5.2 + 1.4 * 2.67 + 0.5 * 1.32 = 10.12\ \frac{kN}{m}.$$

Factored bending moment design, $M_f$:

$$Simply\ supported: M_f = \frac{w_f l_n^2}{8} = \frac{10.26kN/m * 3m^2}{8} = 11.54\ kNm.$$

3) Effective depth, d:
   - Table 17 of CSA A23.3 specifies a minimum cover of 40mm for slabs with exposure classes F-1, F-2, S-1, S-1 (freeze thaw).

$$d \cong h - \ cover + 10mm \ = 150 - \ 40 + 10 \ = 100mm.$$

4) Required area of tension reinforcement, $A_s$:
   - Using the Direct Procedure as outlined in "Reinforced Concrete Design," by Brzev & Pao, and setting $M_r$ equal to $M_f$:

$$A_s = \frac{\alpha_1 \emptyset_c f'_c b}{\emptyset_s f_y}(d \pm \sqrt{d^2 - \frac{2M_r}{\alpha_1 \emptyset_c f'_c b}}; +`ve\ can\ be\ ignored$$

$$A_s = \frac{0.895 \quad 0.65 \quad 30 \quad 1000}{0.85 \quad 400}(100 - \sqrt{100^2 - \frac{2 \ 11.54x10^6}{0.895 \quad 0.65 \quad 30 \quad 1000}}$$

$$A_s = 351.44mm^2 \ \therefore \boldsymbol{Try\ 15M\ bars\ A_b = 200mm^2}$$

5) Required bar spacing (CL. 7.4.1.2 CSA A23.3-04):

$$s \leq A_b \frac{1000}{A_s}$$

$$s \leq \ 200mm^2 \ \frac{1000}{351mm^2} = 570mm \ \rightarrow \boldsymbol{Set\ s = 250mm}$$

$$A_s = A_b \frac{1000}{s} = 800mm^2.$$

6) Confirm max tension reinforcement requirement is satisfied (CL. 10.5.2 CSA A23.3-04)

$$\rho = \frac{A_s}{bd} = \frac{800mm^2}{1000mm * 150mm} = 0.0053$$

$$\rho_b \cong \frac{f'_c}{1000}\ for\ Grade\ 400\ steel$$

$$\rho_b \cong \frac{30}{1000} = 0.027 \geq \rho = 0.0053 \ \therefore \boldsymbol{OK}!$$

7) Confirm minimum reinforcement requirement is satisfied (CL. 7.8.1 CSA A23.3-04)

$$A_{s,min} = 0.002A_g$$

$$A_{s,min} = 0.002 \ 1000 * 150 \ = 300mm^2$$

$$A_s = 800mm^2 \geq A_{s,min} \therefore \boldsymbol{OK}!$$

8) Maximum bar spacing (CL. 7.4.1.2 CSA A23.2-04)

$$s_{max} = \min \ 3h, 500 \ = \min \ 3 * 150,500 \ = 450mm$$

$$s = 250mm \leq s_{max} \ \therefore \boldsymbol{OK}!$$

9) Moment Resistance, $M_r$.
   - Actual effective depth, d: $d = 150mm - 40mm - \frac{15mm}{2} = 102.5mm$

$$a = \frac{\emptyset_s f_y A_s}{\alpha_1 \emptyset_c f'_c b} = \frac{0.85 \quad 400MPa \quad 800mm^2}{0.895 \quad 0.65 \quad 30MPa \quad 1000mm} = 15.59mm$$

$$M_r = \emptyset_s f_y A_s \ \ d - \frac{a}{2} \ \ = \ 0.85 \ \ 400MPa \ \ 800mm^2 \ \ \ 102.5mm - \frac{15.59mm}{2} \ \ = 25.08\ kNm.$$

$$M_r = 25.08kNm \geq M_f = 11.54kNm \ \therefore \boldsymbol{OK}! \ \ \boldsymbol{USE\ 15M@200\ FOR\ TENSION\ REINF.}$$

10) Crack control parameter, z (CL. 10.6.1 CSA A23.3-04)
   - Distance from centroid of tension reinforcement to concrete tension face

$$d_c = h - d = 150mm - 102.5mm = 47.5mm$$

   - Effective tension area per bar

$$A = 250mm * 2 \ 47.5mm \ \ = 23{,}750mm^2.$$

- Stress in steel reinforcement under service load level

$$f_s = 0.6f_y = 0.6 * 400MPa = 240MPa$$

$$\therefore z = f_s \sqrt[3]{d_c A} = 240MPa \sqrt[3]{47.5mm * 23{,}750mm^2} = 24{,}984.1\frac{N}{mm}.$$

$$z \leq 25{,}000\frac{N}{mm} for\ exterior\ exposure\ \therefore \boldsymbol{OK}!$$

11) Shrinkage and Temperature reinforcement (CL. 7.8.1 & CL. 7.8.3 CSA A23.3-04)
   - $A_{s,min} = 0.002A_g = 300mm^2$
   - $s_{max} = \min(3h, 500) = 500mm$
   - Required bar spacing $s \leq A_b\frac{1000}{A_s} = 200mm^2\frac{1000}{300mm^2} = 666.7mm \geq s_{max}\ \therefore \boldsymbol{set\ s = 500mm.}$

   $$A_s = A_b\frac{1000}{s} = 200mm^2\frac{1000}{500mm} = 400mm^2 \geq A_{s,min} = 300mm^2$$

   $$\therefore \boldsymbol{OK!\ \ USE\ 15M@500\ FOR\ TEMPERATURE\ AND\ SHRINKAGE\ REINFORCEMENT.}$$

Shear Design:

1) No significant tensile stresses caused by axial loads, Simplified Method as per CSA A23.3-4 can be used (CL. 11.3.6.3 CSA A23.3-04).
2) Factored shear force, $V_f$
   - For a simply supported beam subjected to a uniform distributed load, the shear force at the supports are:

   $$V_f = \frac{w_f l_n}{2} = \frac{(10.259\frac{kN}{m})(3m)}{2} = 15.39kN.$$

   - NOTE: There are no demand reductions at a distance $d_v$ away from the supports due to the lack of increased support compressive strength.
3) Concrete shear resistance (CL. 11.3.4 CSA A23.3-04)
   - Effective depth, d = 102.5mm (from flexural design, 15M bars with 40mm cover).
   - Effective shear depth, $d_v$: $d_v = \max(0.9d, 0.72h) = 108$mm.
   - As per CL. 11.3.6.2 CSA A23.3-04, for slab thicknesses less than 350mm, $\beta = 0.21$.
   - Use normal density aggregates, $\lambda = 1.0$

   $$V_c = \emptyset_c \lambda \beta \sqrt{f'_c} b_w d_v$$

   $$V_c = (0.65)(1.0)(0.21)(\sqrt{30MPa})(1000mm)(108mm) = 80.75kN$$

   $$V_c = 80.75kN \geq V_f = 15.39kN\ \therefore \boldsymbol{OK}!$$

   $$\therefore \boldsymbol{TRANSVERSE\ REINFORCEMENT\ IS\ NOT\ REQUIRED.}$$

**EDGE BEAM DESIGN:**

Flexural Design:

12) Estimate beam dimensions.
   As per Cl. 9.8.2.1 CSA A23.3-04, it is prescribed that $h \geq l_n/16$ satisfies CSA A23.3-04 deflection requirements

   $$h = \frac{3000mm}{20} = 150mm.$$

   $$Typically, b = \frac{h}{2} = 200mm.$$

   $$Sec. 5.4.1\ dictates\ b \geq 250mm, \therefore set\ b = 250mm.$$

13) Factored bending moment, $M_f$:
   Slab contributions:

- DL (excluding railings) = $24\,{}^{kN}{}_{m^3} * 0.15m + 16\,{}^{kN}{}_{m^3} * 0.05m = 4.4kPa.$
- LL = 3.6 kPa.
- WL (vertical) = 1.32 kPa.
- Slab tributary width = 3m/2 = 1.5m.

Dead load, DL = beam self weight + slab reaction force + railings.

$$Specified\ DL = \ 24\ \frac{kN}{m^3} * 0.25m * 0.4m \ + \ 4.4\frac{kN}{m^2} * 1.5m \ + \ 1.2\frac{kN}{m} \ = 10.2\ \frac{kN}{m}$$

Live Load, LL = pedestrian load (CL. 3.8.9 CSA S6-06)

$$Specified\ LL = 2.6\ kPa * 1.5m = 4\ \frac{kN}{m}$$

Wind Load, WL (CL. 3.10.2.3 CSA S6-06)

$$Specified\ WL = 1.32\ kPa * 1.5m = 1.98\ \frac{kN}{m}$$

From Table 3.2 (Load combinations), the worst ULS combination, excluding E, P, K, V, S, EQ, F, A, H:

$$ULS\ Combo\ 1 = 1.1DL + 1.7LL = 18.02\ \frac{kN}{m}. \leftarrow \boldsymbol{GOVERNS}!$$

$$ULS\ Combo\ 2 = 1.1DL + 1.4LL + 0.5WL = 17.81\ \frac{kN}{m}.$$

Factored bending moment design, $M_f$:

$$Simply\ supported{:}\ M_f = \frac{w_f l_n^2}{8} = \frac{10.26kN/m * 3m^2}{8} = 68.14\ kNm.$$

14) Estimate Effective depth, d:
    - Assuming 1 layer: $d \cong h - 70 = 400 - 70 = 330mm.$

15) Required area of tension reinforcement, $A_s$:
    - Using the Direct Procedure as outlined in slab design calcs.

$$A_s = \frac{0.895 \quad 0.65 \quad 30 \quad 1000}{0.85 \quad 400}(330 - \sqrt{330^2 - \frac{2\ 68.14x10^6}{0.895 \quad 0.65 \quad 30 \quad 1000}}$$

$$A_s = 665.44mm^2\ Table\ 5.1\ of\ text\ recommends\ 20M\ or\ 25M\ for\ beam\ size.$$
$$Choose\ 3 - 20M\ A_s = 900mm^2 \geq 665.44mm^2$$

16) Confirm maximum tension reinforcement requirement is satisfied (CL. 10.5.2 CSA A23.3-04)

$$\rho = \frac{A_s}{b * d} = \frac{900}{250 * 330} = 0.019 \geq \rho_b = 0.027 \quad \text{for f'c} = 30\ \text{MPa} \quad \therefore \mathbf{OK}!$$

17) Actual effective depth, d:
    - Table A.2 Brzev and Pao pg. 914 of text: For exposure classes F-1, F-2, S-1, S-2 (freeze and thaw prone conditions), cover=40mm.
    - $S_{min}$ for 20M, assuming 10M trans. reinf. max(1.4$d_b$, 1.4agg, 30)=30mm – using $d_b$=20mm and aggregate size of 20mm.

$$s = \frac{250mm - 2 * 40mm - 2 * 10mm - 2(\frac{d_b}{2})}{3\ bars - 1} = 65mm > s_{min} \ \therefore \boldsymbol{OK}!$$
$$d = h - cover - trans.reinf. - \frac{d_b}{2} = 340mm.$$

18) Confirm minimum reinforcement requirement is satisfied (CL. 10.5.1.2 CSA A23.3-04)

$$A_{s,min} = \frac{0.2\ \sqrt{f_c'}b_t h}{f_y} = \frac{0.2 * \ \sqrt{30} * 250mm * 400mm}{400\ MPa} = 273.8\ mm^2$$
$$A_s = 900mm^2 \geq A_{s,min} \therefore \boldsymbol{OK}!$$

19) Moment Resistance, $M_r$.

$$a = \frac{\emptyset_s f_y A_s}{\alpha_1 \emptyset_c f'_c b} = \frac{0.85 \quad 400MPa \quad 900mm^2}{0.895 \quad 0.65 \quad 30MPa \quad 250mm} = 77.97mm$$

$$M_r = \emptyset_s f_y A_s \left(d - \frac{a}{2}\right) = (0.85)(400MPa)(900mm^2)\left(340mm - \frac{77.97mm}{2}\right) = 92.11\ kNm.$$

$$M_r = 92.11kNm \geq M_f = 68.14kNm$$

$$\therefore \boldsymbol{OK}! \quad \boldsymbol{USE\ 3-20M\ with\ 40mm\ cover\ FOR\ TENSION\ REINF.}$$

20) Crack control parameter, z (CL. 10.6.1 CSA A23.3-04)

- Effective tension area for all bars: $A_e = b * 2(ds) = 30{,}000mm^2$

$$d_s = h - d = 400mm - 340mm = 60mm$$

- Effective tension area per bar: $A = \frac{A_e}{N} = \frac{30000mm^2}{3\ bars} = 10{,}000mm^2$
- Stress in steel reinforcement under service load level

$$f_s = 0.6f_y = 0.6 * 400MPa = 240MPa$$

$$\therefore z = f_s \sqrt[3]{d_c A} = (240MPa)\sqrt[3]{47.5mm * 23{,}750mm^2} = 20{,}242.4\frac{N}{mm}.$$

$$z \leq 25{,}000\frac{N}{mm} for\ exterior\ exposure\ \therefore \boldsymbol{OK}!$$

Shear Design:

4) No significant tensile stresses caused by axial loads, Simplified Method as per CSA A23.3-4 can be used (CL. 11.3.6.3 CSA A23.3-04).

5) Factored shear force, $V_f$

$$V_f = \frac{w_f l_n}{2} = \frac{(18.02\frac{kN}{m})(5.5m)}{2} = 50\ kN.$$

- NOTE: There are no demand reductions at a distance $d_v$ away from the supports due to the lack of increased support compressive strength.

6) Concrete shear resistance (CL. 11.3.4 CSA A23.3-04)

- Effective depth, d = 340 mm (from flexural design, 15M bars with 40mm cover).
- Effective shear depth, $d_v$: $d_v = \max(0.9d, 0.72h) = 306\text{mm}.$
- Assume trans. reinf. is less than minimum prescribed, $\beta = \frac{230}{1000+d_v}$=0.1761
- Use normal density aggregates, $\lambda = 1.0$

$$V_c = \emptyset_c \lambda \beta \sqrt{f'_c} b_w d_v = (0.65)(1.0)(0.17611)\left(\sqrt{30MPa}\right)(250mm)(300) = 47.965\text{kN}$$

$$V_c\ is\ pretty\ close\ to\ V_f = 50kN\ \therefore \boldsymbol{Provide\ some\ reinforcement\ just\ in\ case}.$$

7) Steel shear resistance (CL. 11.3.5.1 CSA A23.3-04)

$$V_r = V_s + V_c; setting\ V_r\ to\ V_f\ and\ rearranging:$$

$$V_s = 50 - 47.964 = 2.0355\ kN$$

$$s_{req'd} = \frac{\emptyset_s A_v f_y d_v cot\theta}{V_s} = \frac{(0.85)(2 * 100mm^2)(400MPa)(306mm)\cot(35)}{2.035 * 10^3 N} = 14{,}600mm$$

8) Stirrup spacing as per CL. 11.2.8.2 & CL. 11.3.8.1 CSA A23.3-04

$$s_{max} = \frac{A_v f_y}{(0.06)\left(\sqrt{f_c'}\right) b_w} = \frac{(2 * 100mm^2)(400MPa)}{(0.06)\left(\sqrt{30}\right)(250mm)} = 973.73mm.$$

- As per CL. 11.3.3 $V_{r,max} = 0.25\emptyset_c f_c' b_w d_v = 0.25 * 0.65 * 30 * 250 * 306 = 373\ kN.$
- Since $V_f = 50\ kN < \frac{V_{r,max}}{2}, then\ s_{max} = \min(600, 0.7d_v, 973.73mm)$

$$\therefore \boldsymbol{set\ s = 200mm.}$$

9) Minimum required shear reinforcement (CL. 11. 2.8.2 CSA A23.3-04)

$$A_{v,min} = 0.06\left(\sqrt{f_c'}\right)\frac{b_w s}{f_y} = 0.06 * \left(\sqrt{30}\right) * \frac{250 * 200}{400} = 41.08mm^2$$

$$A_v = 2 * 100 = 200mm^2 \geq A_{v,min} \therefore \boldsymbol{OK}!$$

10) New shear resistance

$$V_r = V_c + V_s = V_c + \frac{\emptyset_s A_v f_y d_v cot\theta}{s} = 47.9kN + \frac{(0.85)(200)(400)(306)(\cot 35)}{200 * 1000} = 200\ kN$$

$$V_f \leq V_r \leq V_{r,max}\ \therefore \boldsymbol{OK}!$$

## Appendix B – Surface Parking Sample Calculations

Average maximum daily precipitation: q= 104.6 mm/day = 0.1046 m/day (Environment Canada, 2010)
Total area of the parking: $A = (55m) * (44.5m) = 2448\, m\text{^}2$
Tributary area for the side pipes: $A_s = 7.4\, m * 55\, m = 407\, m\text{^}2$
Tributary area for the interior pipes: $A_i = 14.8\, m * 55\, m = 814\, m\text{^}2$
Maximum water flow for the side pipes: $Q_s = A_s * q = 407\, m^2 * 0.1046\, \frac{m}{day} = 42.6\, m^3/day$
Maximum water flow for the interior pipes: $Q_i = A_i * q = 814\, m^2 * 0.1046\, \frac{m}{day} = 85.1\, m^3/day$
To be more conservative and to improve safety of design we apply an assumed factor of safety of (FS) 1.5 to the calculated water flow values.
Maximum factored water flow for the side pipes: $Q_{sf} = Q_s * FS = 42.6\, \frac{m^3}{day} * 1.5 = 63.9\, m^3/day$
Maximum factored water flow for the interior pipes: $Q_{if} = Q_i * FS = 85.1 \frac{m^3}{day} * 1.5 = 127.7\, m^3/day$

**Calculations for Energy Head at the South end of the Pipes:**

Assuming a temperature: $T = 10°\mathrm{C}$ for design
Kinematic viscosity: v=1.306*10^(-6) (m^2/sec) at 10°C (Houghtalen, Akan, & Hwang, 2010)
Gravitational acceleration: g= 9.81 m^2/sec is used

**Energy head for the side pipes:**

$$Q_{sf} = 63.9 \frac{m^3}{day} * \frac{1\, day}{24\, hr} * \frac{1\, hr}{3600\, sec} = 7.396 * 10^{-4}\, (m^3/day)$$

Selected diameter of each side pipe: $D_s = 0.15\, m = 150\, mm$
Calculation of friction in the pipe:

Flow velocity: $v = \frac{Q_{sf}}{A_{pipe}} = \frac{7.396*10^{-4}(\frac{m^3}{day})}{\frac{\pi}{4}*(0.15\, m)\text{^}2} = 0.042\, m/sec$

Reynolds Number: $R_e = \frac{D_s * v}{v} = \frac{0.15m\ *(0.042\frac{m}{sec})}{1.306*10^{-6}\, (\frac{m^2}{sec})} = 4824$

Roughness height for PVC pipes: e=0.0015 mm (Houghtalen et al., 2010)
Relative Roughness: $\frac{e}{D_s} = \frac{0.0015\, mm}{150\, mm} = 1 * 10\text{^} - 5$
Using the Moody diagram (Houghtalen et al., 2010): Friction factor: $f = 0.038$
Conservatively assume that pipe cross section will be filled with water for the entire length of the pipe:
Frictional head loss: $h_f = \frac{0.0826 * f * L * Q_{sf}^2}{D_s^5}$ where L is the pipe length equal to 55 m

$$h_f = \frac{0.0826 * 0.038 * (55\, m) * (7.396 * 10^{-4}\, m\text{^}3/sec)^2}{(0.15m)^5} = 1.24 * 10^{-3}\, m$$

Bernoulli Equation:
Where point 1 represents the North end of the pipe and point 2 represents the South end:

$$h_1 + \frac{P_1}{\gamma} + \frac{v_1^2}{2g} = h_2 + \frac{P_2}{\gamma} + \frac{v_2^2}{2g} + h_f$$

Define $E = \frac{P}{\gamma} + \frac{v^2}{2g}$ as the energy head and substitute into the original Bernoulli equation:

$$h_1 + E_1 = h_2 + E_2 + h_f$$

Where:
$h_1 = -0.5\, m$
$E_1 = 0; since\ P_1 = 0\ and\ v_1 = 0\ at\ the\ North\ end\ of\ the\ pipe$
$h_2 = -2.15\, m$
Solving for $E_2$: (energy head for each pipe )

$$E_2 = h_1 + E_1 - h_2 - h_f = -0.5m - -2.15m - 1.24 * 10^{-3}m = 1.65\, m$$