PLANNING AND DESIGN OF EXPRESS LANES CONNECTING TWO HIGHWAYS IN MIAMI, FLORIDA, USA

(Capstone Project)

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Declaration

We hereby declare that this report entitled “Planning and Design of Express Lanes Connecting Two Highways in Miami, Florida, USA” is the result of our own project work except for quotations and citations which have been duly acknowledged. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

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List of Abbreviations

CIP – Cast in Place

FDOT – Florida Department of Transportation

MSE – Mechanically Stabilized Earth

SSD – Stopping Side Distance

SDG – Structural Design Guide

CHAPTER 1: INTRODUCTION

1.1 Background

Rising traffic and congestion in urban areas increases a demand for long bridges and interchanges. However, construction and designing such bridges evolve the infrastructure due to being permanently affected by safety, environment and budget. Therefore, there is a variety of bridge construction types that are chosen depending on the location and several parameters such as feasibility, safety maintainability, cost, and simplicity of construction. Over 60 years, precast prestressed concrete girders have been widely used all over the USA owing to their low life-cycle cost, endurance and modularity. Usually, this type of girders is used for full length simply supported bridges. Most of prestressed beam bridges use precast pre-tensioned girders and cast-in-place (CIP) deck. According to the Wilast Amorn (2008), cost of the curved, precast, prestressed concrete girder is 40 % lower than original steel plate girder, where the 30 % of the cost is spent for the pre-tensioning. The price of the CIP deck and precast concrete deck is almost the same, so the overall difference in price is 25% in favor of the precast, precast, prestressed concrete girder.

The output and effectiveness of a precast, prestressed concrete beam bridge is highly affected by the design and construction performance. This implicates a combination of the variety of design improvements instead of introducing them separately. The main impediments for the designers and fabricators are the transportation, erection and service loads. In order to provide the most convenient precast girder transportation, three lengths were chosen for all bridge spans, which are small enough to meet simple support requirements and will not disturb observation for the driver. The bridge spans and their lengths are reported in table 3.1. This report:

- Plans concept plan and general design of the flyover for congested interchange in Miami.
- Estimates geometric design of the path
- Conducts structural and geotechnical analyses
- Executes construction management plan for the project

1.2 Significance

According to the Urban Mobility Report (2012), Miami is ranked as first in Florida and eleventh nationally by rate of congestion in the USA. One of the most congested interchanges in Miami is located at the crossroads of I-75 and Palmetto Expressways/SR 826. Implementation of expressway will provide an alternative of fast and comfortable drive, thus decrease the traffic congestion at the interchange of I-75 and SR 826 by 30%. Based on other expressways in Florida State it is predicted that 15000 of vehicles will pass the proposed expressway which is roughly 30% of existing daily traffic of the same route. Decrease in the travel time has also indirect impact on overall economy of the region in terms of travel time saving, accident decrease, pollutant emissions decrease and excess fuel consumption decrease. The economic benefit of this bridge can be estimated by multiplication of average number of people in each vehicle by the time saved by using this road instead of previous and by the gross national product (GNP) of USA per working hour. This makes construction of such tolled bridges a sustainable business (Branco and de Brito, 2004).

1.3 Objectives and Scope

This project aims to offer solution for reducing congestion and providing additional travel options by constructing the tolled expressway. The project has three parts for designing, which are the design of the flyover, additional road in the middle of I-75, and expansion of Palmetto Corridor, which leads to the constriction of the nearby Peter Pike's canal that will require retaining structure.

The scope of the project comprises the conceptual and general design of the project, structural and geotechnical design of supporting elements, technical drawings of the main components, financial analysis of the project, and the project management of the construction.

Total length of the project is 7.43 km, 490 m of which belong to bridges. Design includes three simply supported, precast, prestressed, concrete I-beam bridges, 15 Mechanically Stabilized Earth walls, and a steel sheet pile. There are two lanes for both direction along the expressway, and one toll station for both directions. Design speed is 112 km/h (70 mph), and speed limit on the expressway is 96 km/h (60 mph).

1.3.1 Location

Location of the SR 826/Palmetto Expressway and I-75 intersection is shown in figure 1.1a, while figure 1.1b shows three sections of the project. In order to illustrate the locations of certain works, the path on the highway has been colored in orange blue and yellow, where

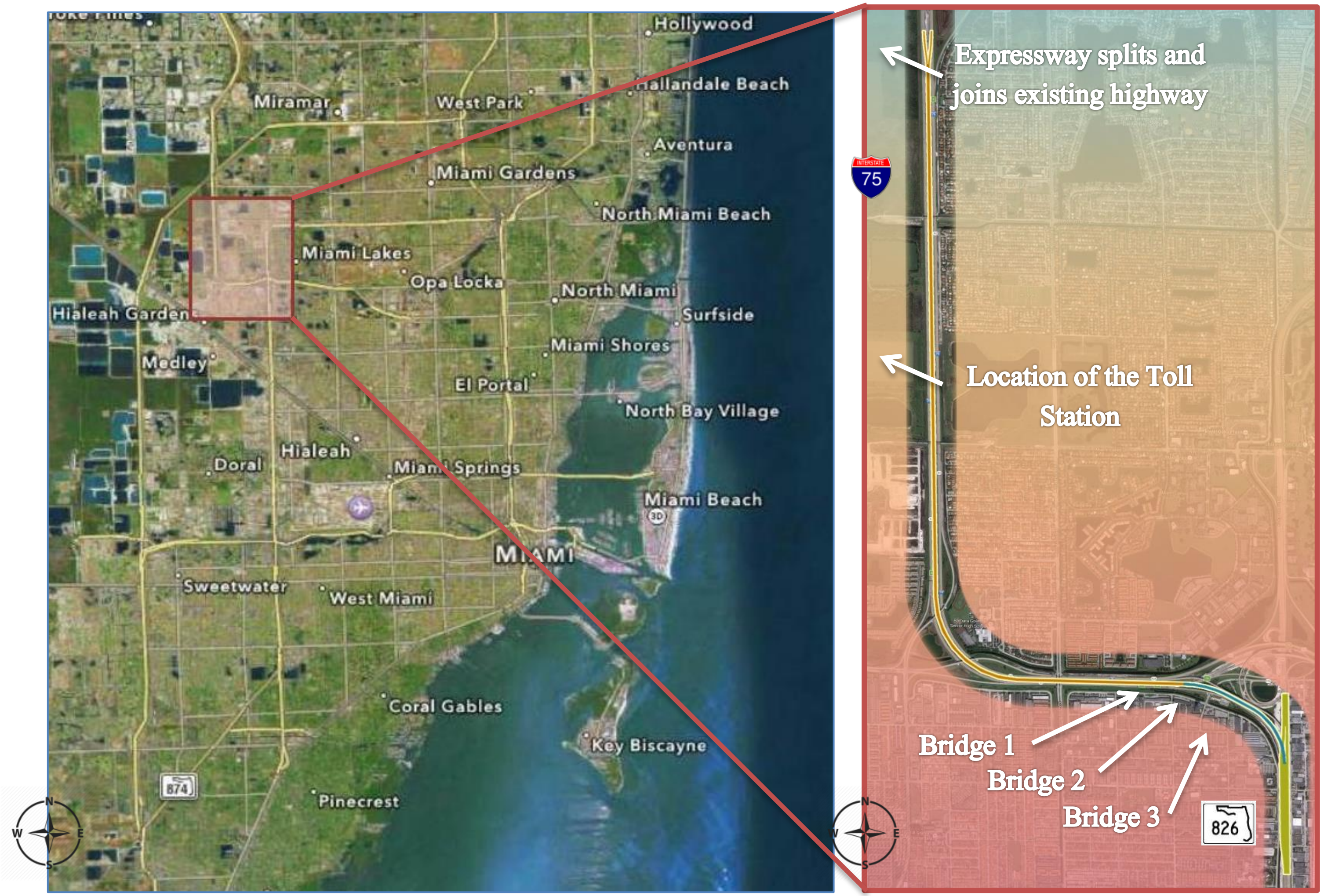
Section A does not require expansion of the existing road.

Section B requires construction of bridges and MSE walls.

Section C requires expansion of the existing road.

The first and third bridges shown on the figure 1.1b are symmetrical, while bridge two is half-straight and half-curved.

The conceptual drawings of three bridges, and split of expressway and its connection to existing highway are illustrated in Appendix B.



(a) (b)

Figure 1. 1 Location of the Project on the Map

1.4 Outline

Chapter 1 is an introduction to this report. Chapter 2 is a literature review of highway design and construction standards, geometric design of the highway, simply supported precast, prestressed concrete I-beam bridges, prestressed design options, MSE walls, sheet piles, and foundation types. Chapter 3 contains a structural analysis of the superstructure and substructure of the bridges. Chapter 4 presents the geotechnical analysis of retaining structures. Chapter 5 comprises the financial analysis of the project. Chapter 6 is a project management plan of the construction, and Chapter 7 is a summary of the project with conclusion.

CHAPTER 2: LITERATURE REVIEW

2.1 Definitions of Key Terms and Concepts

Anchorage – A device generally used to enable the tendon to impart and maintain prestress in concrete (Maity D.).

Post-tensioning – A method of prestressing concrete by tensioning the tendons against hardened concrete. In this method, the prestress is imparted to concrete by bearing.

Pretensioning – A method of prestressing concrete in which the tendons are tensioned before the concrete is placed. In this method, the concrete is introduced by bond between steel and concrete (Maity D.).

Sight Distance – The length of the road that is seen to a driver ahead at any certain time (Garber, 2015: 84).

Stopping Side Distance – The minimum sight distance required for a driver to stop the vehicle after catching sight of the object (Garber, 2015: 84).

Tendon – A stretched element used in a concrete member of structure to impart prestress to the concrete (Maity D.).

2.2 Highway Design and Construction Standards

- A Policy on Geometric Design of Highways and Streets AASHTO (2011), often called "The Green Book". This book covers the functional design of roads and highways including such things as the layout of intersections, horizontal curves and vertical curves.
- AASHTO LRFD Bridge Design Specifications is the base bridge design manual that is used in the US (2012).
- Florida Department of Transportation (FDOT) Structures Manual provides engineering and detailing standards, criteria, and guidelines to designers and detailers who design structures for the Florida Department of Transportation (2016).

2.3 Geometric Design

Geometric design is the determination of the dimensions of the highways. In this project, main parts such as vertical and horizontal curves, cross sections, and climbing lanes were considered. The characteristics of driver, vehicle, and road are major factors affecting the geometric design of the highway. For example, the stopping sight distance, depending on the driver, vehicle, and road characteristics, affects the length of vertical curves and radius of the curvature. The purpose of the geometric design is to ensure safe and smooth traffic.

2.3.1 Design of the Alignment

Design of the geometrical alignment consists of vertical and horizontal alignments. The alignment depends on the chosen design speed. There are two significant conditions for safe geometrical alignment. Firstly, it should be consistent, without any unexpected changes, while the second is the conformity of vertical and horizontal alignments.

2.3.1.1 Vertical Alignment

The vertical alignment is comprised of straight lines known as grades, which are joined by curves. Selection of grades and length of the curves constitute the vertical alignment design. Properly selected vertical alignment provides gradual transition from one grade to another while vehicles shift from highway to bridge and conversely.

There are two types of sag: *crest* and *sag*, which are shown on the figure 2.1. There are two cases that affect design length: first is when the SSD is greater than the length of the vertical curve, and second is when the length of the vertical curve is greater than SSD. They are shown as figures 2.2 and 2.3.

In this project the SSD is less than the length of vertical curve. Therefore, minimum length of *crest* vertical curves is calculated by equation 2.1.

$$\text{Lmin} = \frac{AS^2}{658} \quad (\text{for } S < L) \qquad (2.1)$$

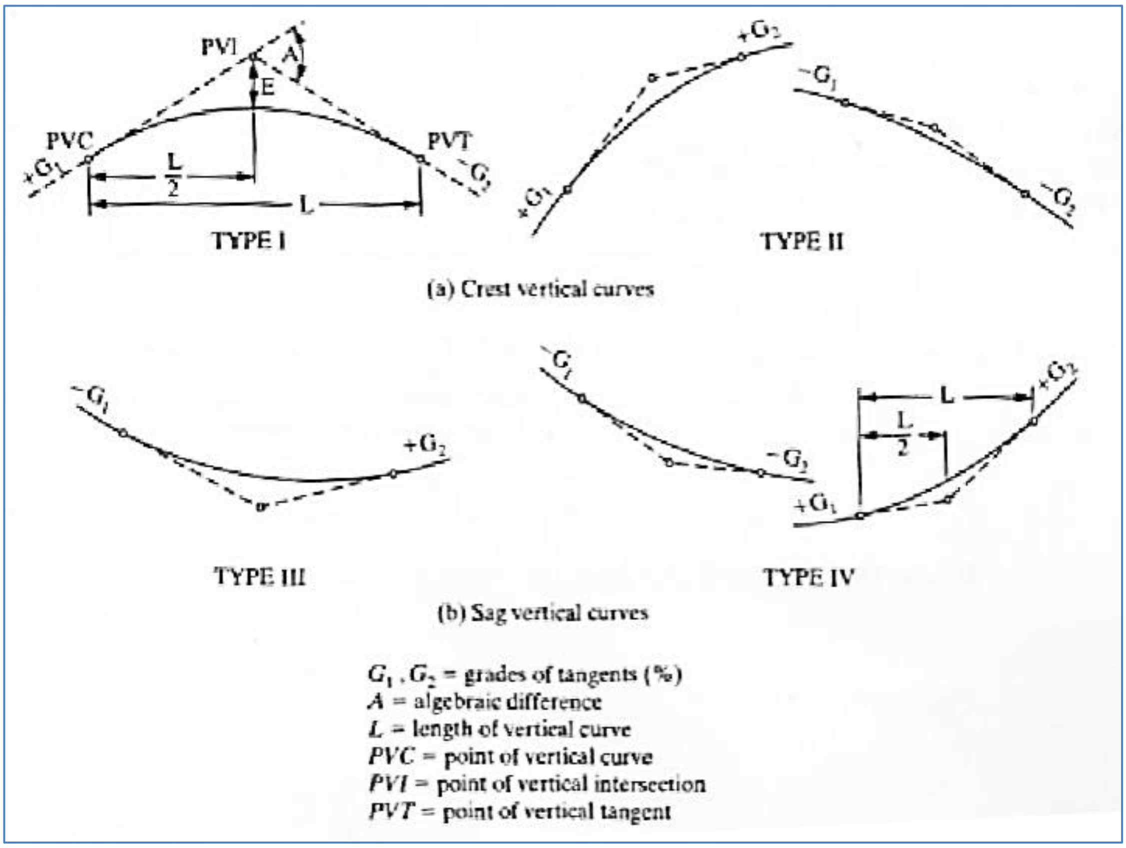
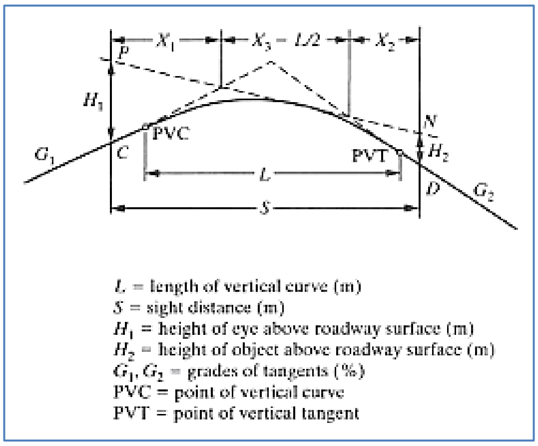


Figure 2. 1 Types of Vertical Curves

Length of the *sag* vertical curves is dependent on four criteria: SSD provided by the headlight, comfort criterion, appearance criterion, which are calculated by equations 2.2 – 2.4, and drainage at the lowest point criterion.



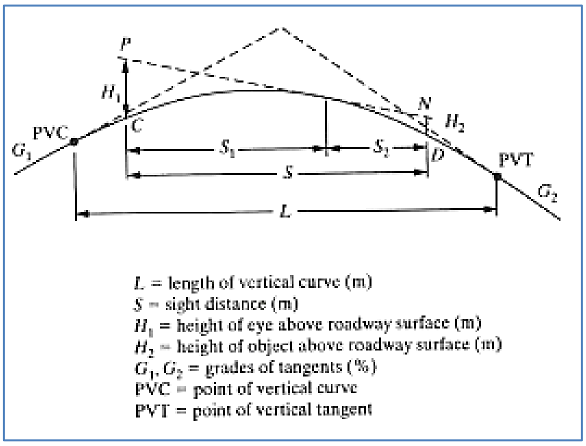


Figure 2. 2 Sight Distance on Crest Vertical Curve (S > L)

Figure 2. 3 Sight Distance on Crest Vertical Curve (S < L)

$$L_{min} = \frac{AS^2}{120+3.5S} \quad (\text{for } S < L) \tag{2.2}$$

$$L_{min} = \frac{Au^2}{395} \tag{2.3}$$

$$L_{min} = 30A \tag{2.4}$$

In equations 2.1 to 2.5 A = geometric difference between grades, S = sight distance, u = design speed in mi/h.

The drainage criterion is that 0.35% slope should be provided within 15 m of the lowest point of a curve.

Substituting values of grade and design speed, which is 3.5%, crests and sag lengths were estimated as 295 m and 205 m, which in sum give 500 m.

2.3.1.2 Horizontal Alignment

Horizontal alignment is comprised of straight sections known as tangent, which are joined by curves. These curves are circle segments that have radii which will provide flowing movement for traffic. The design of the horizontal alignment includes the determination of the minimum radius and length of the curve. Types of horizontal curves are simple, compound, reversed and spiral. In order to estimate the minimum radius of the curvature, equation 2.5 can be used.

$$R = \frac{u^2}{127(e+fs)} \tag{2.5}$$

Here, R = minimum radius (m), u = design speed (km/h), e = superelevation (m/m), fs = coefficient of side friction (Garber, 2015: 771, 788-802).

Substituting superelevation and coefficient of side friction, which were respectively chosen as 10%, and 0.114, radii of the curved part of the second bridge, and curvature of the first and third bridges were determined to be 460 m.

2.4 Simply Supported Precast Concrete I-Beam Bridge

Simply supported bridge decks are not continuous over three or more supports and are statically determinate structures. Theoretically talking, "When the main girders of abridge are supported by a movable hinge at one end and a fixed hinge or roller at the other end then such bridges are called simply supported bridges".

Precast concrete is a concrete that has been made in advance by casting concrete in special reusable mold, cured in required environment, and transported to the construction site. This reduces construction time, while using concrete instead of steel decreases the cost. We have chosen girder type I-94, which is one of the common precast girder type used in the simply supported bridges.

2.5 Prestressed Design Options

Prestressing the concrete is the process of inducing the internal stresses in order to withstand potential tensile stresses within concrete resulting from loads. Prestressed concrete can withstand higher tension and compression compared to typical concrete. Compressive stresses of prestressed concrete can be reached either by pre-tensioning or post tensioning. In pre-tensioning tension will be given to reinforcement before concrete is placed. While in post-tensioning the steel is tensioned after the installation of the concrete. It is usually used in cast in place concrete and large bridge girders (Hamilton and Charles, 2016).

Advantage of Prestressed Concrete:

1. High strength concrete and steel utilization enables lighter weights and thin shapes of the girders in comparison with reinforced concrete elemnents.
2. Fully prestressing ensures effectiveness of the entire section by eliminating tensile stresses under working loads.
3. Prestressing creates counter-balances for dead loads by eccentric prestressing

4. Because of the presence of compressive stresses prestressing ensures better resistance to shear forces
5. High strength concrete and absence of cracks prolongs durability
6. Lower weight of prestressed elements allows longer spans and less budget
7. Factory output is allowable
8. Prestressed members pass the check before usage.
9. Before the crash, prestressed concrete deflects significantly, so it gives sufficient signal
10. Fatigue strength of the prestressed steel is higher because of small deformations in steel. This is advantageous for dynamically loaded structures.

Disadvantages of Prestressed Concrete:

1. Requires specially trained workers
2. Initial equipment cost is high
3. Requires experienced engineers
4. However, prestressing makes concrete brittle
5. Prestressed concrete has lower fire resistance.

Prestressed concrete can be classified depending on different features of prestressing, design and construction.

2.5.1 Pre-Tensioning

In pre-tensioning the tendons are tensioned before placing a concrete. They kept anchored and are released only after the concrete is hardened. Finally, the concrete is ready to use.

2.5.2 Post-Tensioning

The process, where the tendons are tensioned after hardening of the concrete is called post-tensioning. They are place into concrete at certain places with special covering and are pulled or tensioned against the outer edges of the concrete.

2.6 Retaining Structures

2.6.1 Mechanically Stabilized Earth Retaining Wall

MSE wall is cost-effective earth-retaining structure that is constructed with reinforced soil. It consists of precast segmental blocks or panels, horizontal reinforcing elements including steel strips, polymeric grids and geotextile sheets. By placing tensile reinforcing elements in the soil, the strength of the soil is improved significantly such that the system becomes self-supporting (Garber, 2015: 754-789).

Advantages of MSE wall:

- Ease of construction
- Flexible construction that can tolerate movement
- Low cost

According to FHWA there are 9 steps for designing MSE walls are as follows (2009).

2.6.2 Sheet Pile

Sheet piling is an earth retention and excavation support technique that retains soil, using steel or other sheet sections with interlocking edges. Thus it creates continuous wall. There are several types of sheet piles:

- Wooden sheet piles
- Precast concrete Sheet piles
- Prestressed concrete sheet piles
- Steel sheet piles

Steel sheet piling uses steel sheet sections with interlocking edges in order to retain soil. Therefore, steel sheet piles should be placed to design depth along estimated location or seawall alignment. Basically, Sheet pile walls mainly used to sustain excavations for below grade parking structures, basements and to design seawalls and bulkheads. In addition to that, permanent steel sheet pile provides long serviceability.

Sheet Pile Design Steps

2.6.3 Foundation types

Types of foundations:

- Shallow
- Deep foundations
  - Drilled shafts
  - Driven piles
    - Concrete piles
    - Steel piles
    - Timber piles
    - Composite

CHAPTER 3: STRUCTURAL ANALYSIS AND DESIGN

3.1 Structural Analysis

3.1.1 General Criteria

General layouts and input parameters necessary for bridge design are given in this section.

3.1.1.1 Bridge Geometry

Horizontal Profile

In this section general horizontal geometry of bridges is demonstrated as plan view. Bridges that are designed in this project are curved. However, AASHTO LRFD Bridge Design Specifications (2012) allows considering the curved bridges as straight if the central angle in one span is less than 12 degrees. Central angles for proposed bridge spans are demonstrated in Table 3.1. It can be seen that central angles do not exceed 12 degrees.

Table 3. 1 Determination of Maximum Central Angles

Bridges	Curvature Radius, m	Length of Span 1, m	Length of Span 2, m	Length of Span 3, m	Length of Span 4, m	Maximum Central Angle, degree
Bridge 1	460	50	60	50	50	7.47
Bridge 2	460	30	60	30	–	7.47
Bridge 3	460	50	60	50	–	7.47

Vertical Profile

This section includes general elevation view of bridges. The height of the bridges is approximately 7.5 m, which are provided as profile view in technical drawings.

3.1.1.2 General Input Parameters

Overall bridge length: $L_{bridge} = 170m$

The maximum bridge design: $L_{span} = 60m$

Structure beam type: Florida-I 96 Beam (FIB-96)

Unit weight of concrete: $\gamma_{conc} = 23.565\ {}^{kN}/_{m^3}$

Modulus of elasticity for reinforcing steel: $E_s = 2 * 10^6 MPa$

3.1.2 LRFD Criteria: Limit States

This section provides the LRFD design criteria necessary for bridge component design. The LRFD describes the limit states that satisfy design conditions. There are 4 limit states proposed by LRFD.

- Strength Limit State: "Load combinations which ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life. Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained" (AASHTO LRFD Bridge Design Specifications, 2012).
- Extreme Event Limit State: "Load combinations which ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle, or ice flow, possibly under scoured conditions. Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge" (AASHTO LRFD Bridge Design Specifications, 2012).
- Service Limit State: "Load combinations which place restrictions on stress, deformation, and crack width under regular service conditions" (AASHTO LRFD Bridge Design Specifications, 2012).
- Fatigue Limit State: "Load combinations which place restrictions on stress range as a result of a single design truck occurring at the number of expected stress range cycles. It is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge" (AASHTO LRFD Bridge Design Specifications, 2012).

3.1.3 Superstructure Design

3.1.3.1 Dead load

During the designing dead loads, they should be separated into two categories. They are dead load of components (DC) and dead load of wearing surfaces and utilities (DW). Dead load of components is a gravity load and is a weight of slab, sidewalk, rail, haunch, and beam. Dead load of wearing surfaces and utilities is a weight of future wearing surfaces, mainly weight of asphalt.

Before beginning of dead load design, term “x” should be identified. “x” represents any point along the length of designing girder.

Interior girder

Design moments and shears for DC dead load

The first step to design dead loads of interior girder is to calculate weights of girder, deck slab, includes haunch and milling surface, stay-in-place forms, and traffic railing barriers. These weights are calculated using the equations 3.1 – 3.3.

Weight of beam:

$$w_{BeamInt} = A_{nc} * \gamma_{concrete} \tag{3.1}$$

Weight of the deck slab, which includes haunch and milling surface:

$$w_{SlabInt} = \left((t_{slab} + t_{mill}) * BeamSpacing + h_{buildup} * b_{tf}\right) * \gamma_{concrete} \tag{3.2}$$

Weight of stay-in-place forms:

$$w_{FormsInt} = (BeamSpacing - b_{tf}) * \rho_{forms} \tag{3.3}$$

The weight of traffic railing barriers are constant, and will be considered regarding what type of barrier will be used.

The next step is to calculate moments and shears, which were occurred due to mentioned weights and are calculated using Equations 3.4, 3.5.

Moment:

$$M_{type\ of\ structure} = \frac{w_{type\ of\ structure} * L_{design}}{2} * x - \frac{w_{type\ of\ structure} * x^2}{2} \tag{3.4}$$

Shear:

$$V_{type\ of\ structure} = \frac{w_{type\ of\ structure} * L_{design}}{2} - w_{type\ of\ structure} * x \tag{3.5}$$

After calculation of the moment and shear for each type of structure, the sum of moments and sum of shears should be found.

Design moments and shears for DW dead load

As DW dead load includes future wearing surfaces and utilities, again the weight of these components should be calculated. Equation 3.6 is used to calculate it.

Weight of future wearing surface:

$$w_{fwsInt} = \text{Beam Spacing} * \rho_{fws} \tag{3.6}$$

After, using equations for moment and shear, moment and shear will be calculated.

Exterior Girder

Designing of dead loads for exterior girder is almost the same as for interior girder. There are only two differences. The first one is in calculating the weight of deck slab, includes haunch and milling surface. Also, calculating DW dead load is different. Namely, the weight of future wearing differs. These two differences are shown in Equations 3.7 and 3.8.

Weight of the deck slab that includes haunch and milling surface:

$$w_{SlabInt} = \left((t_{slab} + t_{mill}) * (\text{Overhang} + \frac{\text{BeamSpacing}}{2}) + h_{buildup} * b_{tf}\right) * \gamma_{concrete} \tag{3.7}$$

Weight of future wearing surface:

$$w_{fwsInt} = \left(\text{Overhang} - d_{\text{from centerline to exterior beam}} + \frac{\text{BeamSpacing}}{2}\right) * \rho_{fws} \tag{3.8}$$

All other steps are same. The order of calculation follows as in interior beam design.

3.1.3.2 Live load

The distribution of the live load into the girder should be identified before design of live load. The equations for distribution factors are created by conducting works in the NCHRP Project 12-26 and the results were compared with 3-dimensional bridge analysis and field measurements to ensure that equations give accurate results. Distribution factor equations already include multiple presence factors. There are some conditions, which are provided by AASHTO, to be able to use distribution factors:

- The range of girder spacing (S) should be between 3.5 feet and 16.0 feet
- Slab thickness (t_s)should be between 4.5 inches and 12 inches
- The length of design beam (L_{design})should be between 20 feet and 240 feet

- Number of girders is not less than 4
- Longitudinal stiffness parameter(K_g) should be between 10,000 in^4 and 7,000,000 in^4

If given limitations are not met, refined method of analysis is used and diaphragms shall be required. However, if one or more of the parameters may be slightly outside the given range, the equation can be considered as a valid.

Equations 3.9 and 3.10 are used to calculate moment distribution factors for interior and exterior girdersrespectively.

$$g_{m,Interior} = 0.075 + (\frac{S}{9.5})^{0.6} + (\frac{S}{L_{design}})^{0.2} + (\frac{K_g}{12*L_{design}t_s^3})^{0.1} \quad (3.9)$$

$$g_{m,Exterior} = g_{m,Interior} * \left(0.77 + \frac{d_e}{9.1}\right) \quad (3.10)$$

The shear distribution factors for interior and exterior girders are calculated using Equations 3.11 and 3.12.

$$g_{V,Interior} = 0.36 + \frac{S}{25} \quad (3.11)$$

$$g_{V,Exterior} = g_{V,Interior} * \left(0.6 + \frac{d_e}{10}\right) \quad (3.12)$$

In order to calculate maximum live load moment and shear, “HL-93 Design Truck” term is used. HL-93 Design Truck a hypothetical Live Load Model proposed by AASHTO for analysis of bridges. This model is used to prescribe the set of live load that will occur during extreme load effect. Design Truck consists of front and two rear axles. The weights of front axle and two rear axles are 35kN and 145 kN, respectively. Distance between rear axle and front is 4.3m. Moreover, the variation of the two rear axles is between 4.3m to 9.0m to design the worst case. The distance from tire to tire in one axle is 1.8m for all axles. Figure 3.6 shows two positions of the axle loads of an HL-93 Design Truck to calculate the live load moments and shears.

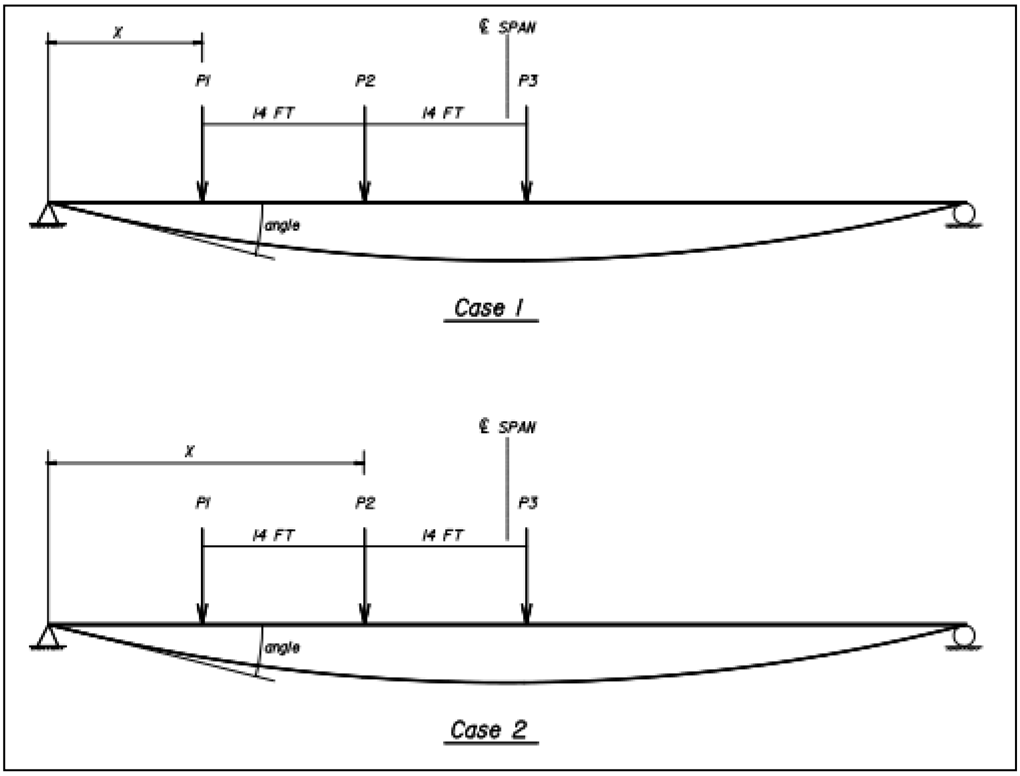


Figure 3. 1 Positioning of HL-93

Thus, there are two cases of positioning HL-93 Design Truck and the different equations are used to estimate moment and shear. Moment and shear for case 1 could be found by equations 3.13 and 3.14, while equations 3.15 and 3.16 are applied for case 2. After calculation moments and shears, the maximum values for both should be taken.

$$M_{truck1}(x) = P1 * \frac{(L_{design} - x)}{L_{design}} * x + P2 * \frac{(L_{design} - x - 14ft)}{L_{design}} * x + P3 * \frac{(L_{design} - x - 28ft)}{L_{design}} * x \quad (3.13)$$

$$V_{truck1}(x) = P1 * \frac{(L_{design} - x)}{L_{design}} + P2 * \frac{(L_{design} - x - 14ft)}{L_{design}} + P3 * \frac{(L_{design} - x - 28ft)}{L_{design}} \quad (3.14)$$

$$\mathrm{M_{truck2}(x)} = \mathrm{P1} * \frac{(\mathrm{L_{design}} - \mathrm{x})}{\mathrm{L_{design}}} * (\mathrm{x} - 14\mathrm{ft}) + \mathrm{P2} * \frac{(\mathrm{L_{design}} - \mathrm{x})}{\mathrm{L_{design}}} * \mathrm{x} + \mathrm{P3} * \frac{(\mathrm{L_{design}} - \mathrm{x} - 14\mathrm{ft})}{\mathrm{L_{design}}} * \mathrm{x}$$

(3.15)

$$V_{truck2}(x) = P1 * \frac{-(x - 14ft)}{L_{design}} + P2 * \frac{(L_{design} - x)}{L_{design}} + P3 * \frac{(L_{design} - x - 14ft)}{L_{design}} \quad (3.16)$$

AASHTO also requires determining design lane load. The design lane load composes of 0.64 klf uniformly distributed load in the longitudinal direction. Also, this load should be uniformly distributed over a 10 ft. width. A dynamic load allowance should not include force impact resulted from the design lane load.

After calculation moments and shears for dead and live loads using distribution factors, the controlling girder should be chosen between exterior and interior. Thus, further calculations will be done only for this girder. Controlling means that more moment and shear are applied to this girder than the second one. So, designing controlling girder and applying design outcomes to second girder ensures that it will not fail.

3.1.3.3 Loss of prestress

Loss of prestress for post-tensioned member can be defined as the sum of the instantaneous loss and time-dependent loss. The instantaneous loss is the losses occurred due to elastic shortening, friction and anchorage set. Elastic shortening (Δf_{pES})could be found by multiplying sum of the concrete stress at the centroid of the presstressing tendons and weight of member, where maximum moment applied(f_{cgp}) to ratio of modulus of elasticity of the presstressing steel(E_p) and modulus of elasticity of the concrete (E_{ci}) (Equation 3.17). Equation 3.18 is used to calculate friction losses. The anchorage set value should be taken greater than stress, which able to control the stress in prestressing steel at transfer. The assumed anchorage value should be mentioned during calculation process.

$$\Delta f_{pES} = \frac{N-1}{2N} * \frac{E_p}{E_{ci}} * f_{cgp} \tag{3.17}$$

Here, N-number of prestressing tendons.

$$\Delta f_{pf} = f_{pj} * \left(1 - e^{-\mu(\alpha+0.04)}\right) \tag{3.18}$$

Time-dependent losses (Δf_{pt}) are losses due to shrinkage, creep, and relaxation. AASHTO LRFD provides three methods to estimate time-dependent losses:

- a refined estimate
- the approximate lump sum estimate
- the background necessary to perform a rigorous time-step analysis

The different methods used for different girder types. Since the designing girder is a post-tensioned, a refined estimate method should be used. The refined estimate method divides time-dependent losses to two segments. First segment is the losses of creep, shrinkage, and relaxation in time between transportation and deck placement. The second one is losses between deck placement and final time. Then these two values are summed, which gives the total time-dependent loss.

3.1.3.4 Flexure Design

The first step of the flexure design part is to identify compressive and tensile stress limits. These limits consist of two parts, stress limits at transfer and final stress limits.Stress limits at transfer calculate before losses, while final stress limits calculate after losses. According to AASHTO specifications, the compressive stress limit at transfer for post-tensioned concrete components should be $0.6 * f_{ci}^{'}$. The value of tension stress limitat transfer is shown in Table 3.2. As designing bridge is not segmentally constructed bridge, and has presstressed steel, the tension stress is equal to $0.24\sqrt{f_{ci}^{'}}$.

Table 3. 2 Tensile Stress Limits in Prestressed Concrete before losses (ibid)

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	• In precompressed tensile zone without bonded reinforcement	N/A
	• In areas other than the precompressed tensile zone and without bonded reinforcement	$0.0948\sqrt{f'_{ci}} \leq 0.2$ (ksi)
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5f_y$, not to exceed 30 ksi.	$0.24\sqrt{f'_{ci}}$ (ksi)
	• For handling stresses in prestressed piles	$0.158\sqrt{f'_{ci}}$ (ksi)
Segmentally Constructed Bridges	Longitudinal Stresses through Joints in the Precompressed Tensile Zone	
	• Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of $0.5f_y$; with internal tendons or external tendons	$0.0948\sqrt{f'_{ci}}$ maximum tension (ksi)
	• Joints without the minimum bonded auxiliary reinforcement through the joints	No tension
	Transverse Stresses through Joints	
	• For any type of joint	$0.0948\sqrt{f'_{ci}}$ (ksi)
	Stresses in Other Areas	
	• For areas without bonded nonprestressed reinforcement	No tension
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of $0.5 f_y$, not to exceed 30 ksi.	$0.19\sqrt{f'_{ci}}$ (ksi)
	Principal Tensile Stress at Neutral Axis in Web	
	• All types of segmental concrete bridges with internal and/or external tendons, unless the Owner imposes other criteria for critical structures	$0.110\sqrt{f'_{ci}}$ (ksi)

Final compressive stress limit is the sum of the permanent loads, effective prestress, and temporary loads, which are in process of shipping and handling. AASHTO LRDF simplifies this sum by approaching following formula:$0.6\varphi_w f'_c$. φ_w is taken as 1.0 for solid sections. Moreover, AASHTO suggests using the value of $0.19\sqrt{f'_{ci}}$ as final tensile stress limit for post-tensioned concrete.

After calculation of the stress limits, top and bottom girder stresses should be evaluated. If top and bottom girder stresses is less than limit stresses, the designing of the bridge should continue. Otherwise, assumed girder size and strand arrangement should be changed.

In the next step, the factored flexure resistance should be calculated and compared with maximum flexure moment in the same section to check the ability of the section to carry applied moment. In order to evaluate factored flexure resistance, nominal flexure resistance is used.

The maximum and minimum reinforcement should be checked. To satisfy maximum reinforcement condition, the ratio of the distance between the neutral axis and the compressive face to distance from extreme compression fiber to the centroid of the prestressing tendons should be less than 0.42. The minimum reinforcement could be taken so that, the factored flexure resistance should be at least equal to lesser of 1.2 times cracking moment, which is total moment acting to the girder during modulus of rapture is equal to maximum tension stress, and 1.33 times factored maximum flexure moment.

3.1.3.5 Shear Design

The first step of shear design is to determine effective shear depth and effective width of the web. Effective shear depth is a distance between the resultants of the tension and compression forces occurred by flexure, and measured perpendicular to neutral axes. The value should be at least equal to the greater of 0.72 times total depth of girder or 0.9 times distance from center of the prestressed steel to extreme compressive fiber. Effective width of the web is a distance between the resultants of the tension and compression forces occurred by flexure, and measured parallel to neutral axes.

Then, nominal shear resistance should be calculated. It is lesser of the sum of the shear resistance provided by the concrete, the shear resistance provided by the transverse reinforcement, and excessive shear cracking (equation 3.19) and equation 3.20.

$$\mathrm{V_n} = \mathrm{V_c} + \mathrm{V_s} + \mathrm{V_p} \tag{3.19}$$

$$V_n = 0.25f_c' * b_v * d_v \qquad (3.20)$$

Minimum transverse reinforcement and maximum spacing for transverse reinforcement should be determined. Minimum transvers reinforcement depends on compressive strength of concrete and yield strength of the reinforcement. While, maximum spacing varies with nominal shear resistance.

Then, factored bursting resistance should be checked. Factored bursting resistance is a load, which could withstand tearing, to not being rupture reinforcements by pulling them apart. The bursting resistance should be equal or greater that 4% of prestressed force.

3.1.3.6 Deck slab design

In process of deck slab designing, not only the dead and live loads at service and strength limit states must be considered. The AASHTO LRFD (5th Edition, 2010) demands to consider vehicular collision with the railing system at the extreme event limit state. Since at the extreme event limit design goal is to prevent collapse of structural component, allowing the damage of any structural components, the resistance factor is taken as 1.0.

There are two methods for deck slab design recognized by AASHTO LRFD (5th Edition, 2010). The first method is Traditional Design Method, and second is Empirical Design Method. The traditional design method is known as more flexible, because it can typically be employed in any situation. At the same time, there are some limitations based on bridge behavior and deck geometry in the empirical design method.

The traditional design method is based on designing concrete decks as transverse strips as a flexure member. The truck axle load is supposed to be supported by transverse strip of the deck, which is supported by girders. In order to simplify designing of the bridge, girders are considered as a rigid supports. The width of equivalent strips (interior strip) provided by The AASHTO LRFD is shown in Table 3.3. As it can be seen, the width of an equivalent strip depends on type of used deck; direction of primary strip relative to traffic, and the sign of moment at given region (2012).

Table 3. 3 Equivalent Strips (AASHTO, 2012)

Type of Concrete Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (inch)
Cast-in-place	Overhang Either Parallel or Perpendicular	45.0 + 10.0X +M: 26.0 + 6.6S -M: 48.0 +3.0S
Cast-in-place with stay-in-place concrete formwork	Either Parallel or Perpendicular	+M: 26.0 + 6.6S -M: 48.0 +3.0S
Precast, post-tensioned	Either Parallel or Perpendicular	+M: 26.0 + 6.6S -M: 48.0 +3.0S
S = spacing of supporting components (ft) +M = positive moment -M = negative moment X = distance from load to point of support (ft)		

The Empirical Design Method is method of deck design based on testing. The laboratory testing shows that the transfer of applied loads on the deck to the supporting components takes place through arching action in the deck, while tradition design assumes that they transfers through shears and moments. However, the following limitation should be satisfied in order to be able to use this method:

- The supporting components are made of steel and/or concrete;
- The deck is fully cast-in-place and water cured;
- The ratio of effective length to design depth does not exceed 18.0 and is not less than 6.0;
- Core depth of the slab is not less than 4.0 in.;
- The effective length does not exceed 13.5 ft;
- The minimum depth of the slab is not less than 7.0 in., excluding a sacrificial wearing surface where applicable;

Our team decided to use the traditional design method to design bridge. The main reason is not having ability to conduct testing which empirical method requires. The minimum requirements for thickness of the deck and overhang are 7 and 8 inches, respectively. However, it is recommended to take approximately ¾” to 1” thicker than the deck thickness. The type of parapet is Type-F. The all standardized dimensions by

National Highway Institute are shown in Figure 3.7. Moreover, general values of concrete parapet are listed below:

Mass per unit length = 650 lb/ft

Width at base = 1 ft.- 8 ¼ in.

Moment capacity at the base, Mc/length = 17.83 k-ft/ft

Parapet height, H = 42 in.

Length of parapet failure mechanism, L_c = 235.2 in.

Collision load capacity, R_w = 137.22 k

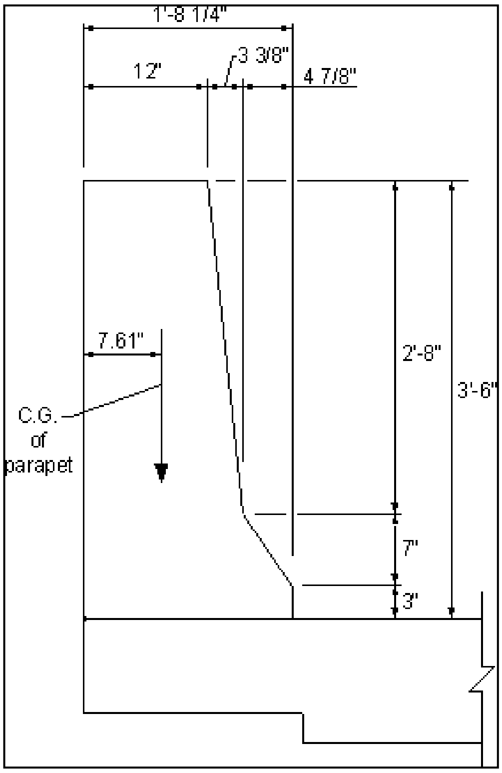


Figure 3. 2 Parapet Cross Section

Dead Load Moment

The positive and negative moments produced by dead loads for a unit deck strip are computed by equation 3.21. However, this equation is not appropriate for calculation of dead load positive and negative moments for the overhang.

$$M = \frac{wl^2}{c} \tag{3.21}$$

Here, c – constant, typically taken as 10 or 12

Live Load Moment

AASHTO provides table to determine positive and negative moments per unit width of deck for live load by beam spacing (S) and distance from center line of girder to design section for negative moment. This table was constructed by modeling the deck as a beam supported on the girders. Table 3.4 is shown below. However, there some limitations and assumptions that should be considered in order to use this table. The limitations and assumptions are followings:

- The method of calculating moments is equivalent strip method.
- For the distances, which are not shown in the table, interpolation may be used.
- In order to use this table, the decks should be supported at least by three beams and the distance between the centerlines of the exterior beams of not less than 14.0 ft.
- The maximum values of moments were taken in the interior regions of the slab.
- Minimum distance from center of wheel to the inside face of parapet = 1 ft
- Minimum distance between the wheels of two adjacent trucks = 4 ft.

Table 3. 4 Maximum Live Load Moments per Unit Width, kip-ft/ft (ibid)

S	Positive Moment	Negative Moment						
		Distance from CL of Girder to Design Section for Negative Moment						
		0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4’ –0”	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4’ –3”	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4’ –6”	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4’ –9”	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5’ –0”	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5’ –3”	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5’ –6”	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5’ –9”	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6’ –0”	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6’ –3”	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6’ –6”	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6’ –9”	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7’ –0”	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7’ –3”	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7’ –6”	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7’ –9”	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8’ –0”	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8’ –3”	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8’ –6”	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8’ –9”	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9’ –0”	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9’ –3”	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9’ –6”	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9’ –9”	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10’ –0”	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10’ –3”	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10’ –6”	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10’ –9”	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11’ –0”	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11’ –3”	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11’ –6”	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11’ –9”	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12’ –0”	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12’ –3”	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12’ –6”	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12’ –9”	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13’ –0”	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13’ –3”	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13’ –6”	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13’ –9”	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14’ –0”	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45
14’ –3”	9.14	12.46	11.59	10.72	9.85	8.99	7.38	6.58
14’ –6”	9.25	12.67	11.81	10.94	10.08	9.21	7.57	6.72
14’ –9”	9.36	12.88	12.02	11.16	10.30	9.44	7.76	6.86
15’ –0”	9.47	13.09	12.23	11.37	10.51	9.65	7.94	7.02

Design for positive moment in the deck

The determination of the reinforced is dependent on the maximum positive moment in the deck. Typically, the center of bay experiences the maximum positive moment for interior bays. The location of the maximum positive moment for the first bay depends on the overhang length and the value and distribution of the dead load. However, all deck bays are reinforced in the same way.

In order to calculate required area of reinforcement per unit width, the following equations 3.22 – 3.24 should be used.

$$k = \frac{M_u}{b{d_e}^2} \tag{3.22}$$

$$r = 0.85 * \frac{f_c}{f_y} - \sqrt{1.0 - \frac{2k}{0.85f_c}} \tag{3.23}$$

$$A_s = r * d_e \tag{3.24}$$

Here, d_e= effective depth from the compression fiber to the centroid of the tensile force in the tensile reinforcement (in.) = total thickness – bottom cover – ½ bar diameter – integral wearing surface.

After calculating required area, the spacing should be found.

$$\text{Spacing} = \frac{\text{Bar area}}{\text{Required area for reinforcement}}$$

Then several checks should be done. Namely, they are check depth of compression block, check if the section is over-reinforced, and check for cracking under Service I Limit State.

Design of the overhang

The purpose of designing overhang is to withstand an axial tension force due to vehicular collision. The designing steps provided by AASHTO LRFD are shown below:

- Determine strip width for overhang
- Determine rail momentresistance at its base and railing load resistance
- Design overhang reinforcement
- Detailreinforcement

3.1.4 Substructure Design

This section of the project contains the structural design of substructure components. The proper connection between substructure and superstructure is dependent on the type and geometry of bridge. For instance, flexible substructure components may be integrated to the superstructure if they are supported by single row of piles. However, stiff substructure components need additional elements as expansion bearings between the substructure and superstructure to lower the design load values in the substructure components.

3.1.4.1 Dead Load and Live Load

Dead Load and Live Load are considered as in Superstructure Design part in order to simplify the design procedure.

3.1.4.2 End Bent Design

End bent of a bridge is the abutments located at each end of a bridge. The primary functions of the abutment are to carrying vertical loads and earth retaining at the bridge ends. There are different types of abutments that may be used in practice (AASHTO LRFD Bridge Design Specifications, 2012). These include:

- Integral Abutment
- Stub Abutment
- Partial-Depth Abutment
- Full-Depth Abutment

Integral abutment may be selected as the most appropriate abutment type for this project based on the following desirable objectives:

- Long-term serviceability of the structure
- Cost efficiency of construction
- Minimum maintenance requirements
- Safety issues and developed aesthetics

Integral abutment is supported on a deep or spread foundation and rigidly connected to the superstructure component. Generally, foundation of integral abutments is supported by single line of concrete or steel piles. The usage of single line of piles as a support decreases the abutment stiffness. Thus, parallel translation between bridge longitudinal axis and abutment may be reached. It allows abandoning use of moveable bearings and expansion joints. It happens because compression in superstructure is transferred to the rigidly attached integral abutment support piles to resist the earth pressure.

However, elimination of expansion joints may cause variety of issues related to shrinkage, creep and thermal expansion (AASHTO LRFD Bridge Design Specifications, 2012). AASHTO LRFD Bridge Design Specifications (2012) report that "Integral abutments shall be designed to resist and/or absorb creep, shrinkage and thermal deformations of the superstructure".

Integral abutment can be designed by performing following design steps:

1. Dead load and live load identification for abutment components
2. Determination of controlling limit states and load factors

3. Pile compressive strength check; and determination of spacing and number of piles
4. Pile cap design; and check for shear and flexure
5. Backwall design; and check for shear and flexure
6. Approach slab design and check for flexure

Components of end bent that will be considered in this section:

- Pile cap
- Pile

Geometry

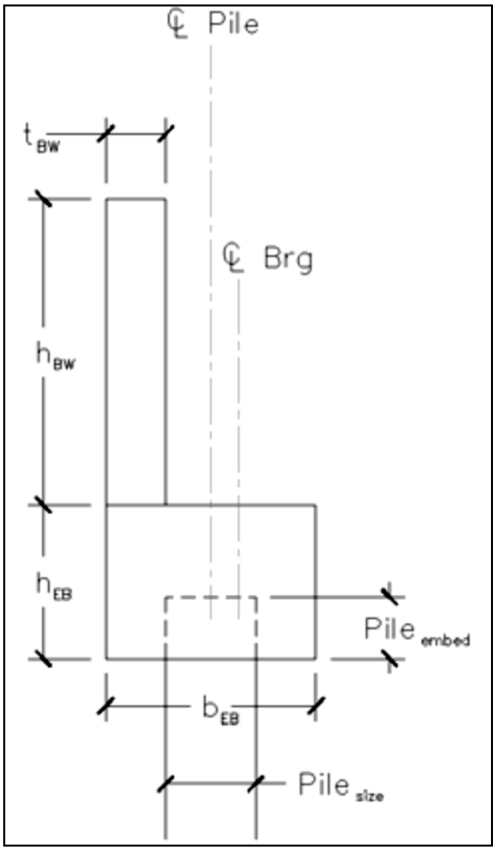


Figure 3. 3 Typical geometry of Integral abutment

Input parameters for abutment design:

Depth of end bent cap: h_{EB} = 3.5 ft (0.915 m)

Width of end bent cap: b_{EB} = 4.5 ft (1.220 m)

Length of end bent cap: L_{EB} = 20.5 m

Height of back wall: h_{BW} = 9.5 ft (2.740 m)

Backwall design width: L_{BW} = 1 ft (0.305 m)

Thickness of back wall: t_{BW} = 1 ft (0.305 m)

Pile Embedment Depth: $Pile_{embed}$ = 1 ft (0.305 m)

Pile Size: $Pile_{size}$ = 24 in (0.610 m)

Pile Cap Design

Both interior and exterior girder reactions are necessary in order to design abutment pile caps. In reality distribution of traffic through the bridge roadway lanes is not equal. However, as a simplification, the traffic load is assumed to present on all roadway lanes and distributed equally through the girders of the bridge. Thus, total load from dead load and live load on the pile cap may be transferred to the abutment piles equally.

Pile Design

Integral abutment may be founded on friction piles or end bearing piles. Typical piles that can be used include H-piles, steel pipe piles filled with concrete and prestressed concrete piles. In this project we take into consideration 24 feet square prestressed concrete piles as an input design data.

3.1.4.3 Intermediate Bent or Pier Design

Pier Design starts from selecting optimum pier type depending on location, superstructure geometry and cost consideration. The common pier types include single column, multi-column and solid wall type. Based on the site conditions as roadway undergoing the bridge, the single column pier is selected for this project. The LRFD specifications do not require exact maximum and minimum dimensions for single column pier components which includes footing, column and pier cap. Thus, the designer has to select preliminary dimensions for the pier components based on the past experience. Once the preliminary dimensions were selected, the next stage is the determination of dead loads and live loads applied to the pier. Then additional load effects as braking force, wind load from superstructure, vertical wind load, wind load on vehicles and wind load on substructures have to be computed. Following, all force effects have to be combined and analyzed. Then, based on the calculated loads pier cap, pier column and footing is designed.

CHAPTER 4: STRUCTURAL DESIGN

4.1 General Notes

Overall bridge length: $L_{bridge} = 170m$

The maximum bridge design: $L_{span} = 60m$

Skew angle: $Skew = 0\ deg$

Structure beam type: Florida-I 96 Beam (FIB-96)

Number of beams: $N_{beams} = 11$

Beam spacing: 1.83m

Deck overhang at End Bent and Pier: 1.1m

Unit weight of concrete: $\gamma_{conc} = 23.57\ {}^{kN}/_{m^3}$

Modulus of elasticity for reinforcing steel: $E_s = 2 * 10^6 MPa$

Ultimate tensile strength for prestressing tendon: $f_{pu} = 1861.58MPa$

Modulus of elasticity for prestressing tendon: $E_p = 196{,}5 * 10^3 MPa$

4.1.1 LRFD Criteria

In order to design bridge components four LRFD design criteria are used. They are dynamic load allowance, resistance factors, limit states, and strength limit states.

Dynamic Load Allowance [LRFD 3.6.2]

The static load of the design truck or tandem will be multiplied by an impact factor, while for centrifugal and braking forces impact factor will not be considered.

Impact factor for fatigue and fracture limit states: $IM_{fatigue} = 1.15$

Impact factor for all other limit states: $IM = 1.33$

Resistance Factors [LRFD 5.5.4.2]

Flexure and tension of reinforced concrete: $\phi = 0.90$

Flexure and tension of prestressed concrete: $\phi' = 1.00$

Shear and torsion of normal weight concrete: $\phi_V = 0.90$

Limit States [LRFD 1.3.2]

According to LRFD specifications, a limit state is a condition beyond which bridge components do not satisfy design conditions. There are four limit states obtained by LRFD: strength limit states, extreme event limit states, service limit states, and fatigue limit states.

Strength Limit States

Local and global strength and stability can be designed to withstand the specified statistically significant load combinations that a bridge could experience during its design life. These designed load combinations are called strength limit states. However, under strength limit state significant depletion and structural faults can happen, but general structural integrity will persist.

Extreme Event Limit States

Minimum load combinations for structural inviolability during natural disasters or vessel or vehicle collision are defined as extreme event limit states. This is a unique case, where return period is higher than bridge design life.

Service Limit States

Service limits are stress, deformation, and crack width standards in accordance with normal service state.

Fatigue Limit States

Fatigue limit is loads constraining stress limitations calculated as a passage of a single design truck at expected amount of times. It is aimed to eliminate crack increase under number of load applications during the bridge design life.

Table 4. 1 Load Factor for Permanent Loads, γp

Type of load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments *DC*: Strength IV only		1.25 1.50	0.90 0.90
DD: Downdrag	Piles, α Tomlinson Method Piles, λ Method Drilled Shafts, O'Neill and Reese (1999) Method	1.40 1.05 1.25	0.25 0.30 0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest • *AEP* for anchored walls		 1.50 1.35 1.35	 0.90 0.90 N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments • Rigid Buried Structure • Flexible Buried Structures ○ Metal Box Culverts and Structural Plate Culverts with Deep Corrugations ○ Thermoplastic Culverts ○ All others		 1.00 1.35 1.30 1.35 1.5 1.3 1.95	 N/A 1.00 0.90 0.90 0.9 0.9 0.9
ES: Earth Surcharge		1.50	0.75

4.1.2 FDOT Criteria

General Requirements

General [SDG 1.1]

The design life for bridge structures is 75 years.

Criteria for Deflection and Span-to-Depth Ratios [SDG 1.2]

According to SDG 1.2, either LRFD 2.5.2.6.3 or 2.5.2.6.2 should be fulfilled. Since superstructure depth does not satisfy LRFD 2.5.2.6.3 conditions, LRFD 2.5.2.6.2 should be satisfied. According to this specification, the deflection limit is span divided by 800 for vehicular load, and span divided by 300 on cantilever arms.

Environmental Classifications [SDG 1.3]

There are three types for environment classification: "Slightly", "Moderately" or "Extremely" aggressive.

Environmental classification for Florida:

Environmental classification for superstructure: $Environment_{super} = "Slightly"$

Environmental classification for substructure: $Environment_{sub} = "Moderately"$

Concrete and Environment [SDG 1.4]

The concrete cover for every bridge component depends on either the environmental classification or the bridge length [SDG 1.4].

Concrete cover for the deck:

$$cover_{deck} = \begin{cases} 0.0508m \ if \ L_{bridge} < 30.48 \ meters \\ 0.0635\text{m otherwise} \end{cases} = 0.0635m$$

[SDG 4.2.1]

Concrete cover for substructure not in contact with water:

$$\text{cover}_{\text{sub}} = \begin{cases} 0.1016\text{m if Environment}_{\text{sub}} = \text{"Extremely"} \\ 0.0762\text{m otherwise} \end{cases} = 0.0762\text{m}$$

Table 4. 2 Minimum 28-day compressive strength of concrete components

Class	Location	Compressive strength
II (bridge Deck)	CIP Bridge Deck Approach Slabs	$f_{c.slab} = 31.03MPa$
IV	CIP Substructure	$f_{c.sub} = 37.92MPa$
V (Special)	Concrete Piling	$f_{c.pile} = 41.37MPa$
VI	Prestressed Beams	$f_{c.beam} = 58.60MPa$

Correction factor for Florida lime rock coarse aggregate: $K_1 = 0.9$

Unit Weight of Florida lime rock concrete: $w_{c.limerock} = 2322{,}67 \, {}^{kN}/_{m^3}$

Moduli of elasticity for slab, beam, superstructure, and piles are calculated using the equation 4.1.

$$\text{E}_{\text{c.type}} = 33000\text{K}_1 * \left(\frac{\text{w}_{\text{c.limerock}}}{{}^{\text{kN}}/_{\text{m}^3}}\right)^{1.5} * \sqrt{\text{f}_{\text{c.type}} * \text{MPa}} \qquad (4.1)$$

Thus, modulus of elasticity for slab, beam, superstructure, and piles are 23,986.87 MPa, 32,963.84 MPa, 26,517.24 MPa, and 27,696.25 MPa respectively.

Loads and Load Factors

Dead loads [SDG 2.2]

Weight of future wearing surface is obtained using the equation 5.2.

$$\rho_{fws} = \begin{cases} 0.718\, {}^{kN}/_{m^2} & \text{if } L_{bridge} < 30.48 \text{ meters} \\ 0\, {}^{kN}/_{m^2} & \text{otherwise} \end{cases} = 0\, {}^{kN}/_{m^2} \tag{4.2}$$

[SDG 4.2.1]

Using 0.0127 m as value of t_{mill}, weight of the sacrificial milling surface can be estimated (equation 5.3).

$$\rho_{mill} = t_{mill} * \gamma_{conc} = 0.3\, {}^{kN}/_{m^2} \tag{4.3}$$

[SDG 4.2.2.A]

Weight of traffic railing barrier: $w_{barrier.ea} = 0.569\ {}^{kN}/_{m}$

Weight of traffic railing median barrier: $w_{medium.bar} = 0.658\, {}^{kN}/_{m}$

Weight of compacted soil: $\gamma_{soil} = \ 15.07\, {}^{kN}/_{m^3}$

Weight of stay-in-place metal forms: $\rho_{forms} = 0.957\, {}^{kN}/_{m^2}$

Barrier / Railing Distribution for Beam-Slab Bridges [SDG 2.8 & LRFD 4.6.2.2.1]

In this project all barriers are equally distributed along all the beams of the bridges.

Exterior interior beams design load calculation also include traffic barriers dead loads (equation 5.3).

$$w_{barrier.exterior/interior} = \frac{w_{barrier.ea}}{N_{beams}} * 2 + \frac{w_{median.bar}}{N_{beams}} = 0.18\, {}^{kN}/_{m} \tag{4.3}$$

Wind Loads [SDG 2.4]

Basic wind speed: $V = 67.06\, {}^{m}/_{s}$

Height, superstructure: $z_{sup} = 6.2m$

Height, substructure: $z_{sub} = 2.65m$

Gust effect factor: $G = 0.85$

Pressure coefficient, superstructure: $C_{p.sup} = 1.1$

Pressure coefficient, substructure: $C_{p.sub} = 1.6$

Velocity pressure exposure coefficient (superstructure) calculation is shown as equation 4.4.

$$K_{z.sup} = \max\left(0.85, 2.01 * \left(\frac{z_{sup}}{274.32m}\right)^{0.2105}\right) = 0.905 \tag{4.4}$$

Velocity pressure exposure coefficient (substructure) calculation is shown as equation 4.5.

$$K_{z.sub} = \max\left(0.85, 2.01 * \left(\frac{z_{sub}}{274.32m}\right)^{0.2105}\right) = 0.85 \tag{4.5}$$

Wind pressure (superstructure, Strength III, Service IV) calculation is indicated as equation 4.6.

$$P_{z.sup.StrIII.ServIV} = 1.28 * 10^{-5} * K_{z.sup} * V^2 * G * C_{p.sup} = 0.049 \tag{4.6}$$

Wind pressure (superstructure, Strength V, Service I) calculation is indicated as equation 4.7.

$$P_{z.sup.StrV.ServI} = 1.28 * 10^{-5} * K_{z.sup} * 31.29^2 * G * C_{p.sup} = 0.011 \tag{4.7}$$

Wind pressure (substructure, Strength III, Service IV) calculation is indicated as equation 4.8.

$$P_{z.sub.StrIII.ServIV} = 1.28 * 10^{-5} * K_{z.sup} * V^2 * G * C_{p.sup} = 0.067 \tag{4.8}$$

Wind pressure (substructure, Strength V, Service I) calculation is indicated as equation 4.9.

$$P_{z.sub.StrV.ServI} = 1.28 * 10^{-5} * K_{z.sup} * 31.29^2 * G * C_{p.sup} = 0.015 \tag{4.9}$$

Superstructure Concrete

Yield strength of reinforcing steel: $\mathrm{f_y} = 413.69$ MPa [SDG 4.1]

Deck Slabs [SDG 4.2]

Thickness of sacrificial milling surface can be found from system indicated as equation 5.10.

$$t_{mill} = \begin{cases} 0\text{m} \ \text{ if } L_{bridge} < 30.48 \text{ meters} \\ 0.0127\text{m} \end{cases} = 0.0127\text{m} \quad (4.10)$$

Deck thickness: $t_{slab} = 0.30\text{m}$

The empirical design method is not allowed by SDG 4.2.4.A. Consequently, the conventional method will be used. Using given minimum 0.2 m slab and smaller than 1.83 m overhang and the table SDG 4.2.4.B the reinforcement of minimum transverse top slab of the median barrier and overhang can be obtained. Minimum area of steel per meter will be taken as 846.67 mm^2 and 1,693.33 $mm^2$for the top of the deck slab at median barrier and at edge barriers respectively.

Pre-tensioned Beams [SDG 4.3]

Minimum compressive concrete strength when released is greater than 27.58 MPa or $0.6 * f_{c.beam}$, calculation of which is shown as equation 4.11.

$$f_{ci.beam.min} = \max(27.58 \text{ MPa}, 0.6 * f_{c.beam}) = 35.16\text{MPa} \quad (4.11)$$

Maximum compressive concrete strength when released is the less than 41.37 MPa or $0.8 * f_{c.beam}$, the calculation of which is shown as equation 4.12.

$$f_{ci.beam.max} = \min(41.37 \text{ MPa}, 0.8 * f_{c.beam}) = 41.37 \text{ MPa} \quad (4.12)$$

Any number lying between minimum and maximum values can be chosen. The compressive concrete strength when released is chosen as $f_{ci.beam} = 41.37$ MPa.

Corresponding modulus of elasticity is obtained through equation 4.13.

$$E_{c.beam} = 33000K_1 * \left(\frac{w_{c.limerock}}{\text{kN}/_{\text{m}^3}}\right)^{1.5} * \sqrt{f_{ci.beam} * \text{MPa}} = 27696.25 \text{ MPa} \quad (4.13)$$

Limits for tension in top of the beam with straight bars only when released [SDG 4.3.1.C] is indicated as equations 4.14, 4.15 (Outer 15 percent of design beam, center 70 percent of design beam).

$$f_{top.outer15} = -\sqrt{f_{ci.beam} * \text{MPa}} = -6.41\text{MPa} \quad (4.14)$$

$$f_{top.center70} = \min(-1.38\ \mathrm{MPa}, 7.88 * 10^{-3}\sqrt{f_{ci.beam} * \mathrm{MPa}}) = -1.38\ \mathrm{MPa} \quad (4.15)$$

4.2 Supersturcture

4.2.1 Dead Loads

FDOT Instructions for Design Standards, Index 20010 Series contain characteristics of non-composite Florida I-96 beam.

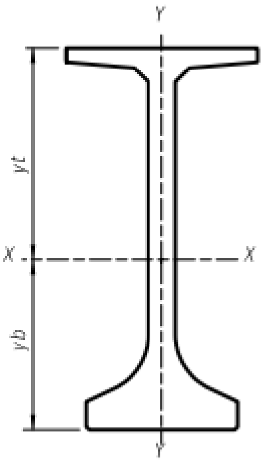


Figure 4. 1 Cross Section of I-beam

Table 4. 3 I-beam Dimensions, Moment of Inertia, and Section Area

Non-composite properties	Dimension	Abbr.	
Moment of Inertia	mm^4	I_{nc}	$0.63*10^{12}$
Section Area	mm^2	A_{nc}	$0.8*10^6$
y_{top}	mm	yt_{nc}	1,350.77
y_{bottom}	mm	yb_{nc}	1,087.63
Depth	mm	h_{nc}	2,438.4
Top flange width	mm	b_{tf}	1,219.2
Top flange depth	mm	h_{tf}	88.9
Width of web	mm	b_w	177.8
Bottom flange width	mm	b_{bf}	965.2
Bottom flange depth	mm	h_{bf}	177.8
Section Modulus top	mm^3	S_{tnc}	$0.47*10^9$
Section Modulus bottom	mm^3	S_{bnc}	$0.58*10^9$

Effective Flange Width [LRFD 4.6.2.6]

The effective flange width of interior beams is calculated using equation 4.16.

$$b_{eff.interior} = \text{Beam Spacing} = 1{,}830\text{mm} \tag{4.16}$$

For exterior beams effective flange width is estimated using equation 4.17. The final value is shown as equation 5.19.

$$b_{\ eff.exterior} = \frac{\text{BeamSpacing}}{2} + \text{Overhang} + \Delta w \tag{4.17}$$

Here, Δw is cross-sectional area of the barrier divided by two times slab thickness (equation 4.18).

$$\Delta w = \frac{A_b}{2t_{slab}} = 430\text{mm} \tag{4.18}$$

$$b_{eff.exterior} = \frac{\text{BeamSpacing}}{2} + \text{Overhang} + \Delta w = 2{,}440\text{mm} \tag{4.19}$$

Transformed Properties

In order to get composite section characteristics the effective flange width of the slab should be converted to the concrete beam characteristics.

Modular ratio between the deck and beam is calculated and represented as equation 4.20.

$$n = \frac{E_{c.slab}}{E_{c.beam}} = 0.728 \tag{4.20}$$

Transformed slab width for interior beams calculation is indicated as equation 5.21.

$$b_{tr.interior} = n * b_{eff.interior} = 1{,}330\text{mm} \tag{5.21}$$

Transformed slab width for exterior beam calculation is indicated as equation 4.22.

$$b_{tr.exterior} = n * b_{eff.exterior} = 1{,}770\text{mm} \tag{4.22}$$

Composite Section Properties

Interior beams

Height of the composite section is the sum of the non-composite beam depth and thickness of the slab: $h = h_{nc} + t_{slab} = 2{,}740$ mm.

Area of the composite section is also the sum of the slab and non-composite beam areas: $A_{Interior} = A_{slab} + A_{nc} = b_{tr.interior}t_{slab} + A_{nc} = 1.2 * 10^6 mm^2$

Distance from the centroid of the beam to extreme fiber in tension is calculated and shown as equation 4.23.

$$y_b = \frac{A_{nc}yb_{nc} + A_{slab}(h_{nc} + \frac{t_{slab}}{2})}{A_{interior}} = 1{,}587.89mm \quad (4.23)$$

Distance from centroid of beam to extreme fiber in compression is a substitution of distance from the centroid of the beam to extreme fiber in tension from beam depth:

$y_t = h - y_b = 1{,}152.11$ mm.

Moments of Inertia calculations are illustrated as equations 4.24, 4.25.

$$I_{slab} = \frac{1}{12} b_{tr.interior} * t_{slab}^3 + A_{slab} * \left(h - \frac{t_{slab}}{2} - y_b\right)^2 = 0.4 * 10^{12} mm^4 \quad (4.24)$$

$$I_{interior} = I_{nc} + A_{nc}(y_b - yb_{nc})^2 + I_{slab} = 1.23 * 10^{12} mm^4 \quad (4.25)$$

Section Moduli for top, beam, and bottom are calculated by equations 4.26-4.28.

$$S_t = \frac{I_{interior}}{y_t} = 1.07 * 10^9 mm^3 \quad (4.26)$$

$$S_{tb} = \frac{I_{interior}}{h_{nc} - y_b} = 1.45 * 10^9 mm^3 \quad (4.27)$$

$$S_b = \frac{I_{interior}}{y_b} = 0.77 * 10^9 mm^3 \quad (4.28)$$

Exterior beams

Height and area of the composite sections are calculated similar to the interior beams' height and area: $h = h_{nc} + t_{slab} = 2740mm$; $A_{Exterior} = A_{slab} + A_{nc} = b_{tr.exterior}t_{slab} + A_{nc} = 1.33 * 10^6 mm^2$.

Distance from centroid of beam to extreme fiber in tension calculation is indicated as equation 4.29.

$$y_b' = \frac{A_{nc}yb_{nc} + A_{slab}(h_{nc} + \frac{t_{slab}}{2})}{A_{exterior}} = 1687.63mm \quad (4.29)$$

Distance from the centroid of beam to extreme fiber in compression is similar to the interior beam calculation*:* $y_t' = h - y_b = 1052.37\ \text{mm}$

Moments of Inertia calculations are illustrated as equations 4.30, 4.31.

$$I_{slab} = \frac{1}{12} b_{tr.exterior} * t_{slab}^3 + A_{slab} * \left(h - \frac{t_{slab}}{2} - y_b'\right)^2 = 0.44 * 10^{12} mm^4 \quad (4.30)$$

$$I_{exterior} = I_{nc} + A_{nc}(y_b' - yb_{nc})^2 + I_{slab} = 1.36 * 10^{12} mm^4 \quad (4.31)$$

Section Moduli for top, beam, and bottom are calculated by equations 4.32-4.34.

$$S_t = \frac{I_{interior}}{y_t'} = 1.29 * 10^9 mm^3 \quad (4.32)$$

$$S_{tb} = \frac{I_{interior}}{h_{nc} - y_b'} = 1.81 * 10^9 mm^3 \quad (4.33)$$

$$S_b = \frac{I_{interior}}{y_b'} = 0.81 * 10^9 mm^3 \quad (4.34)$$

Table 4. 4 Summary of Properties

Composite section properties	Dimension	Abbr.	Interior	Exterior
Effective slab width	mm	$b_{eff.int/ext}$	1,830	2,440 mm
Transformed slab width	mm	$b_{tr.int/ext}$	1,330	1,770 mm
Height of composite section	mm	h	2,740	2,740
Effective slab area	mm^2	A_{slab}	0.4×10^6	0.53×10^6
Area of composite section	mm^2	$A_{int/ext}$	1.2×10^6	1.33×10^6
Neutral axis to bottom fiber	mm	y_b	1,587.89	1,687.63
Neutral axis to top fiber	mm	y_t	1,152.11	1,052.37
Inertia of composite section	mm^4	$I_{int/ext}$	1.23×10^{12}	1.36×10^{12}
Section modulus top of slab	mm^3	S_t	1.07×10^9	1.29×10^9
Section modulus top of beam	mm^3	S_{tb}	1.45×10^9	1.81×10^9
Section modulus bottom of beam	mm^3	S_b	0.77×10^9	0.81×10^9

Dead Loads

Moments and shears are represented as functions of “x”, which is variable belonging to the range between 0 m and L_{beam}. Design check points for the moment and shear are located at points 0 L, 0.05 L, 0.1 L, 0.2 L, 0.3 L, 0.4 L, 0.5 L. According to AASHTO, the value for 0.05 L is referenced as a critical shear section. Since $\rho_{fws} = 0\,{}^{kN}/{}_{m^2}$, the moment and shear for DW load are 0.

Moment and shear for all of the structure types are calculated using equations 4.35 – 4.37.

Moment self-weight of structure type: $M_{type} = \frac{w_{type}L_{beam}}{2} * x - \frac{w_{type}x^2}{2}$ (4.35)

Shear self-weight of structure type: $V_{type} = \frac{w_{type}L_{beam}}{2} - w_{type} * x$ (4.36)

Design Moments and Shears for DC Dead Loads in both internal and external beams

Weight of beam: $w_{beam} = A_{nc} * \gamma_{conc} = 18.85\ {}^{kN}/{}_{m}$ (4.37)

Interior Beams
Deck slab weight includes milling surface (equation 4.38).

$$w_{SlabInt} = (t_{slab} + t_{mill}) * BeamSpacing * \gamma_{conc} = 13.48\,{}^{kN}/{}_{m} \quad (4.38)$$

Weight of stay-in-place forms is calculated and shown as equation 4.39.

$$w_{FormInt} = (BeamSpacing - b_{tf}) * \rho_{forms} = 0.58\,{}^{kN}/{}_{m} \quad (4.39)$$

Weight of traffic railing barriers: $w_{barrier.interior} = 0.18\,{}^{kN}/{}_{m}$

DC Load total is the sum of estimated weights: $w_{DC.Beam.Int} = w_{beam} + w_{SlabInt} + w_{FormInt} + w_{barrier.interior} = 33.09\,{}^{kN}/{}_{m}$.

DC Load Rotation can be calculated using equation 4.40.

$$\theta_{DC.BeamInt} = \frac{(w_{DC.BeamInt} - w_{barrier.Int}) * L_{beam}^3}{24E_{c.beam} * I_{nc}} + \frac{w_{barrier.Int} * L_{beam}^3}{24E_{c.beam} * I_{Interior}} = 0.017\ deg \quad (4.40)$$

Exterior Beams
Similar to the equation 4.38 weight of deck slab, includes milling surface (equation 4.41).

$$w_{SlabExt} = (t_{slab} + t_{mill}) * (Overhang + \frac{BeamSpacing}{2}) * \gamma_{conc} = 14.85\,{}^{kN}/{}_{m} \quad (4.41)$$

Weight of stay-in-place forms for exterior beams is similar to the interior beams calculation also (equation 4.42).

$$w_{FormExt} = \left(\frac{BeamSpacing - b_{tf}}{2}\right) * \rho_{forms} = 0.29\ {}^{kN}/_{m} \tag{4.42}$$

Weight of traffic railing barriers for exterior beams is also $0.18\ {}^{kN}/_{m}$.

DC Load total: $w_{DC.Beam.Ext} = w_{beam} + w_{SlabExt} + w_{FormExt} + w_{barrier.exterior} = 34.17\ {}^{kN}/_{m}$.

DC Load Rotation is calculated and shown as equation 4.43.

$$\theta_{DC.BeamExt} = \frac{(w_{DC.BeamExt} - w_{barrier.Ext}) * L_{beam}^3}{24E_{c.beam} * I_{nc}} + \frac{w_{barrier.Ext} * L_{beam}^3}{24E_{c.beam} * I_{Exterior}} = 0.018\ \text{deg.} \tag{4.43}$$

Summation of moment and shears for every structure type gives total moment and shear, which are shown in table 4.5.

Table 4. 5 Design Moments

Design Moments (kN*m)							
	0L	0.05L	0.1L	0.2L	0.3L	0.4L	0.5L
Location, x=	0	3	6	12	18	24	30
Interior Beam							
Beam	0.00	1611.68	3053.70	5428.80	7125.30	8143.20	8482.50
Slab	0.00	1152.54	2183.76	3882.24	5095.44	5823.36	6066.00
Forms	0.00	49.59	93.96	167.04	219.24	250.56	261.00
Barrier	0.00	15.39	29.16	51.84	68.04	77.76	81.00
Total DC	0.00	2829.20	5360.58	9529.92	12508.02	14294.88	14890.50
Exterior Beam							
Beam	0.00	1611.68	3053.70	5428.80	7125.30	8143.20	8482.50
Slab	0.00	1269.68	2405.70	4276.80	5613.30	6415.20	6682.50
Forms	0.00	24.80	46.98	83.52	109.62	125.28	130.50
Barrier	0.00	15.39	29.16	51.84	68.04	77.76	81.00
Total DC	0.00	2921.54	5535.54	9840.96	12916.26	14761.44	15376.50

Table 4. 6 Design Shears

Design Shears (kN)							
	0L	0.05L	0.1L	0.2L	0.3L	0.4L	0.5L
Location, x=	0	3	6	12	18	24	30
Interior Beam							
Beam	565.50	508.95	452.40	339.30	226.20	113.10	0.00
Slab	404.40	363.96	323.52	242.64	161.76	80.88	0.00
Forms	17.40	15.66	13.92	10.44	6.96	3.48	0.00
Barrier	5.40	4.86	4.32	3.24	2.16	1.08	0.00
Total DC	992.70	893.43	794.16	595.62	397.08	198.54	0.00
Exterior Beam							
Beam	565.50	508.95	452.40	339.30	226.20	113.10	0.00
Slab	445.50	400.95	356.40	267.30	178.20	89.10	0.00
Forms	8.70	7.83	6.96	5.22	3.48	1.74	0.00
Barrier	5.40	4.86	4.32	3.24	2.16	1.08	0.00
Total DC	1025.10	922.59	820.08	615.06	410.04	205.02	0.00

4.2.2 Live Loads

4.2.2.1 Live Load Distribution Factors

Live load on the deck must be distributed to the precast, prestressed beams. Therefore, AASHTO provides factors for the distribution of live load into the beams. Those factors used under special conditions. It should be noted, in case of contradictions with following criteria, a refined method of analysis is required and diaphragms shall be provided:

- Width of deck is constant (OK)
- Number of beams is not less than four ($N_{beams} = 11, so\ OK$)
- Beams are parallel and have approximately the same stiffness (OK)
- The overhang minus the barrier width does not exceed 0.91m (1.1-0.47=0.63<0.91, so OK)
- Curvature in plan is less than the limit specified in Article 4.6.1.2.4 (OK)

4.2.2.2 Moment Distribution Factors

Following parameters estimated in order to define moment distribution factors:

- The distance between center of gravity of non-composite beam and deck:

$e_g = (h - yb_{nc}) - \frac{t_{slab}}{2} = 1.5m$

- Modular ratio between beam and deck: $n^{-1} = 1.374$
- Longitudinal stiffness parameter $K_g = n^{-1} * (I_{nc} + A_{nc}{e_g}^2) = 3.34 * 10^{12} mm^2$
- Distance from centerline of web for exterior beam to barrier:

$$d_e = Overhang - barrier\ width = 0.63m$$

Based on the above parameters, critical values for interior and external beams are defined:

- Distribution factor for moment in interior beams when one design lane is loaded [LRFD 4.6.2.2.2b]:

$$\mathrm{g_{m.Interior1}} = 0.06 + (\frac{\mathrm{BeamSpacing}}{4.27})^{0.4} * \left(\frac{\mathrm{BeamSpacing}}{\mathrm{L_{design}}}\right)^{0.3} * \left(\frac{\mathrm{K_g}}{\mathrm{L_{design}}\mathrm{t_{slab}}^3}\right)^{0.1}$$
$$= 0.35$$

- Distribution factor for moment in interior beams when two or more design lanes are loaded:

$$\mathrm{g_{m.Interior2}} = 0.075 + (\frac{\mathrm{BeamSpacing}}{2.9})^{0.6} * \left(\frac{\mathrm{BeamSpacing}}{\mathrm{L_{design}}}\right)^{0.2} * \left(\frac{\mathrm{K_g}}{\mathrm{L_{design}}\mathrm{t_{slab}}^3}\right)^{0.1}$$
$$= 0.48$$

Comparing both distribution factors for moment in interior beams, the largest one should be chosen. Therefore, distribution factor for moment when two or more design lanes are applied is used $\mathrm{g_{m.Interior}} = 0.48$

- Distribution factor for moment in exterior beams when one design lane is loaded [LRFD 4.6.2.2.2d]:

The calculation of the defined factor should be based on the lever rule, which contains several conversion parameters such as truck load to wheel load (0.5) and for multiple truck presence (1.2).

$$g_{m.Exterior1} = \frac{2 * BeamSpacing - 2d_e - 3}{BeamSpacing} * 0.5 * 1.2 = 0.63$$

- Distribution factor for moment in exterior beams when two or more design lanes are loaded:

$$g_{m.Exterior2} = g_{m.Interior} * \left(0.77 + \frac{d_e}{2.77}\right)^{0.2} = 0.48$$

Similarly, to the interior beams, the largest value should be chosen as critical one. Therefore, distribution factor for moment when one design lane is loaded is used $g_{m.exterior} = 0.63$

4.2.2.3 Shear Distribution Factors

Following calculations performed in order to evaluate critical values for shear distribution factors in interior and external beams:

- Distribution factor for shear in interior beams when one design lane is loaded [LRFD 4.6.2.2.3a]:

$$g_{V.interior1} = 0.36 + \frac{BeamSpacing}{7.62} = 0.6$$

- Distribution factor for shear in interior beams when two or more design lanes are loaded:

$$g_{V.interior2} = 0.2 + \frac{BeamSpacing}{3.66} - (\frac{BeamSpacing}{10.67})^2 = 0.67$$

Comparing both values, the largest one chosen as critical value. Therefore, critical value for distribution factor for shear in interior beams given by $g_{V.Interior} = 0.67$

- Distribution factor for shear in exterior beams when one design lane is loaded [LRFD 4.6.2.2.3b]:

$$g_{V.Exterior1} = \frac{2 * BeamSpacing - 2d_e - 3}{BeamSpacing} * 0.5 * 1.2 = 0.63$$

- Distribution factor for shear in exterior beams when two or more design lanes are loaded:

$$g_{V.Exterior2} = g_{V.Interior} * \left(0.6 + \frac{d_e}{3}\right) = 0.54$$

Similarly, to the interior beams, the largest value should be chosen as critical one. Therefore, distribution factor for shear when one design lane chosen for critical value given by $g_{V.Exterior} = \mathbf{0.63}$

4.2.2.4 Live Load Analysis

The Live Load Model, namely HL-93 Design Truck, is used in order to define maximum live load moment and shear. The main benefit of this model can be represented by prescibing the set of live load that will occur during extreme load effect.

Following input parameters provided for performing HL-93 Design Truck:

- Design Truck consists of front and two rear axles.
- The weights of front axle and two rear axles are 35kN (P3) and 145 kN (P1 and P2)
- Distance between rear axle and front is 4.27m
- The variation of the two rear axles is between 4.3m to 9.0m (worst case)
- The distance from tire to tire in one axle is 1.8m for all axles

Figure 4.2 presented below shows two positions of the axle loads of an HL-93 Design Truck to calculate the live load moments and shears.

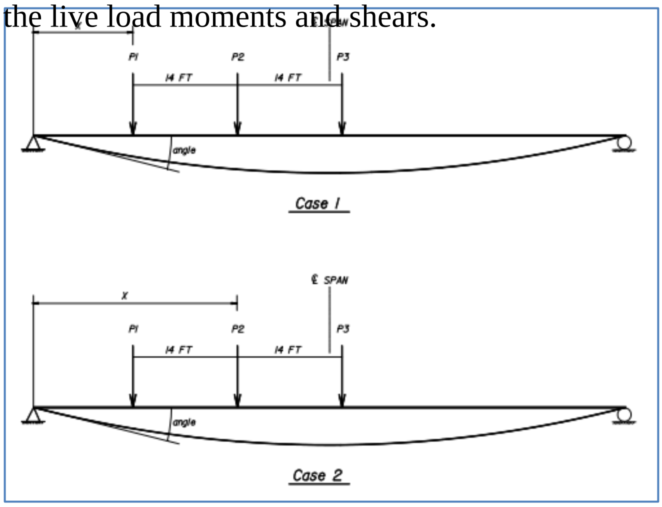


Figure 4. 2 Positioning of HL-93

As it can be clearly seen from the figure 4.2, there are two cases of positioning HL-93 Design Truck. Following estimations of moment and shear performed for both cases:

- **Case 1**

$$M_{truck1}(x) = P1 * \frac{(L_{beam} - x)}{L_{beam}} * x + P2 * \frac{(L_{beam} - x - 4.27)}{L_{beam}} * x + P3 * \frac{(L_{beam} - x - 8.54)}{L_{beam}} * x$$

$$V_{truck1}(x) = P1 * \frac{(L_{beam} - x)}{L_{beam}} + P2 * \frac{(L_{beam} - x - 4.27)}{L_{beam}} + P3 * \frac{(L_{beam} - x - 8.54)}{L_{beam}}$$

- **Case 2**

$$\mathrm{M}_{\mathrm{truck2}}(\mathrm{x}) = \mathrm{P1} * \frac{(\mathrm{L}_{\mathrm{beam}} - \mathrm{x})}{\mathrm{L}_{\mathrm{beam}}} * (\mathrm{x} - 4.27) + \mathrm{P2} * \frac{(\mathrm{L}_{\mathrm{beam}} - \mathrm{x})}{\mathrm{L}_{\mathrm{beam}}} * \mathrm{x} + \mathrm{P3} * \frac{(\mathrm{L}_{\mathrm{beam}} - \mathrm{x} - 4.27)}{\mathrm{L}_{\mathrm{beam}}} * \mathrm{x}$$

$$V_{truck2}(x) = P1 * \frac{-(x - 4.27)}{L_{beam}} + P2 * \frac{(L_{beam} - x)}{L_{beam}} + P3 * \frac{(L_{beam} - x - 4.27)}{L_{beam}}$$

After completing estimation of moments and shears, the maximum values for both should be taken.

Moment and shear induced by the lane load represented by following formulae:

$$M_{lane} = \frac{w_L * L_{beam}}{2} * x - \frac{w_L x^2}{2}$$

$$V_{lane} = \frac{w_L * L_{beam}}{2} - w_L * x$$

In addition, AASHTO requires determination of design lane load. It should be noted, that design lane load composes of $w_L = 9.34\, {kN}/{m}$ uniformly distributed load in the longitudinal direction. Moreover, this load should be uniformly distributed over a 10 ft. width. Finally, a dynamic load allowance should not consist of force impact resulted from the design lane load.

Following formulae perform live load moment and shear for HL-93 truck load (including impact, IM=1.33) and lane load:

$$M_{LLI} = M_{truck} * IM + M_{lane}$$

$$V_{LLI} = V_{truck} * IM + V_{lane}$$

Based on all above equations, following table

Table 4. 7 Design Live Load

Design Live load							
	0L	0.05L	0.1L	0.2L	0.3L	0.4L	0.5L
Location, x=	0	3	6	12	18	24	30
Moments: Interior beam							
Live load + DLA	0.00	1969.43	3725.13	6595.32	8610.57	9770.88	10175.63
Distribution factor	0.48	0.48	0.48	0.48	0.48	0.48	0.48
Design Live load + DLA	0.00	945.33	1788.06	3165.75	4133.07	4690.02	4884.30
Moments: Exterior beam							
Live load + DLA	0.00	1969.43	3725.13	6595.32	8610.57	9770.88	10175.63
Distribution factor	0.63	0.63	0.63	0.63	0.63	0.63	0.63
Design Live load + DLA	0.00	1240.74	2346.83	4155.05	5424.66	6155.65	6410.65
Shears: Interior beam							
Live load + DLA	692.10	642.46	592.83	493.57	394.30	295.04	195.77
Distribution factor	0.67	0.67	0.67	0.67	0.67	0.67	0.67
Design Live load + DLA	463.71	430.45	397.20	330.69	264.18	197.68	131.17
Shears: Exterior beam							
Live load + DLA	692.10	642.46	592.83	493.57	394.30	295.04	195.77
Distribution factor	0.63	0.63	0.63	0.63	0.63	0.63	0.63
Design Live load + DLA	436.02	404.75	373.48	310.95	248.41	185.88	123.34

After completing constructing table for live load, combination for dead and live load moments and shears by using Strength I, Service I, and Service III state limits should be evaluated. By performing the following equations, summary of factored moment and shear was represented in Table 4.8.

$$Strength\ I = 1.25 * DC + 1.5 * DW + 1.75 * (Design\ Live\ load + DLA)$$

$$Service\ I = 1.00 * (DC + DW + (Design\ Live\ load + DLA))$$

$$Service\ III = 1.00 * (DC + DW) + 0.8 * (Design\ Live\ load + DLA)$$

Table 4. 8 Summary of Facrored Moment(kNm) and Shear (kN)

Summary of Facrored Moment(kNm) and Shear (kN)							
	0L	0.05L	0.1L	0.2L	0.3L	0.4L	0.5L
Location, x=	0	3	6	12	18	24	30
Moments: Interior beam							
Strength I	0.00	5190.81	9829.83	17452.47	22867.90	26076.14	27160.65
Service I	0.00	3774.52	7148.64	12695.67	16641.09	18984.90	19774.80
Service III	0.00	3585.46	6791.03	12062.52	15814.48	18046.90	18797.94
Shears: Interior beam							
Strength I	2052.36	1870.07	1687.79	1323.24	958.67	594.11	229.54
Service I	1456.41	1323.88	1191.36	926.31	661.26	396.22	131.17
Service III	1363.67	1237.79	1111.92	860.17	608.42	356.68	104.93
Moments: Exterior beam							
Strength I	0.00	5823.22	11026.38	19572.54	25638.48	29224.20	30439.26
Service I	0.00	4162.28	7882.37	13996.01	18340.92	20917.09	21787.15
Service III	0.00	3914.13	7413.01	13165.00	17255.99	19685.96	20505.02
Shears: Exterior beam							
Strength I	2044.42	1861.55	1678.70	1312.99	947.27	581.56	215.84
Service I	1461.12	1327.34	1193.56	926.01	658.45	390.90	123.34
Service III	1373.92	1246.39	1118.87	863.82	608.77	353.72	98.67

As it clearly seen from the calculation outputs, the exterior beam controls the design. The following design steps will be shown only for exterior beam. The interior beam calculations would be identical.

4.2.3 Prestressing Force and Area of Strands

To identify the minimum jacking force, P_j and associated area of prestressing strands, A_{ps}, two tensile stress conditions at the bottom fiber of the mid-span of beam at the Service III limit state should be satisfied:

- Case A: No tension under permanent loads

No tension is allowed for components with bonded prestressing tendons or reinforcement, subjected to permanent loads (DC, DW) only. In order to satisfy listed above condition, it is necessary to set the stress at the bottom fiber equal to zero and solve for the required effective prestress force (at service, i.e., after losses), P, to achieve no tension.

The eccentricity of prestressing force at mid-span based on the non-composite section represented by following calculation:

$$e_g = 1.5 - 0.3 = 1.2m$$

$$\frac{P}{A_{nc}} + \frac{Pe}{S_b} - \left(\frac{M_{self-weight} + M_{slab}}{S_b} + \frac{M_{barrier} + M_{DW}}{S_{bf}}\right) = 0$$

$$P = \frac{\frac{M_{self-weight} + M_{slab}}{S_b} + \frac{M_{barrier} + M_{DW}}{S_{bf}}}{\frac{1}{A_{nc}} + \frac{e_g}{S_b}} = 6929.78\ kN$$

- Case B: Tension limited to prevent cracking under total dead and live loads:

Allowable tension for components subjected to the Service III limit state (DC, DW, (0.8) HL-93), related to not worse than moderate corrosion conditions, and located in Environmental Areas I or II

$$\frac{P}{A_{nc}} + \frac{Pe}{S_b} - \left(\frac{M_{self-weight} + M_{slab}}{S_b} + \frac{M_{barrier} + M_{DW} + +0.8 * M_{LL}}{S_{bf}}\right) = -0.19\sqrt{f_c}$$

$$P = \frac{\frac{M_{self-weight} + M_{slab}}{S_b} + \frac{M_{barrier} + M_{DW} + 0.8 * M_{LL}}{S_{bf}} - 0.19\sqrt{f_{c.beam}}}{\frac{1}{A_{nc}} + \frac{e_g}{S_b}}$$
$$= 9275.65kN$$

Comparing both values provided from both conditions, the minimum required effective prestressing force, P, at service level for an exterior girder should be represented as the larger value from Case A and Case B. Therefore, $P = 9275.65kN$.

To estimate the minimum required jacking force, it is necessary to define an prestress losses. Therefore, by assuming total (immediate and longterm) prestress losses of 25% (of the jacking force), the required jacking force (i.e., just before transfer, ignoring minor losses from jacking to detensioning) is represented by:

$$P_j = \frac{9275.65}{0.75} = 12367.53kN$$

The required area of prestressing strands, A_{ps}, jacked to $0.75 f_{pu}$is:

$$A_{ps} = \frac{P_j}{0.75 f_{pu}} = 8988.45\ mm^2$$

Based on the estimation of presstressed area strands, the diameter of strands can be evaluated as 15.24mm. Therefore, required number of strands:

$$N = \frac{A_{ps}}{A_{strand}} = 49.30\ strands$$

As a result, 50 low relaxation strands with 15.24mm diameter should be used in design

Checking results represented by performing following estimations: $A_{ps} = N * A_{strand} = 9116.11mm^2$ and $P_j = 0.75 * A_{ps} * f_{pu} = 12543.18kN$

Figure 4. 3 represents the strands location.

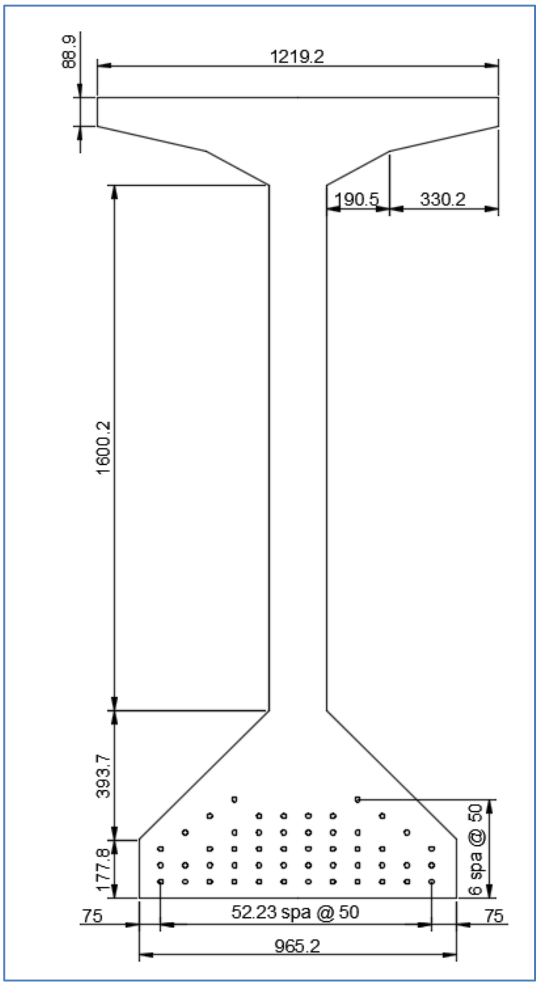


Figure 4. 3 Cross section of I-beam

The centroid of strands from the bottom of the beam can be found by estimation:

$$\text{CGS} = \frac{12*74.25+12*124.25+10*174.25+8*224.25+6*274.25+2*324.25}{50}$$

$$= 164.25\text{mm}$$

According to previous output the actual eccentricity at mid-span for the beam can be evaluated: $e_m = 1.5 - 0.164 = 1.34\text{m}$

4.2.4 Prestress Losses [AASHTO 5.9.5.1-1]

Previous estimations of prestress losses were approximate and it applies only to determine area of strands. Therefore, by applying a trial number of strands and layout, prestress losses can be more accurately evaluated.

According to LRFD Specifications, total prestress losses in prestressing strand stress are assumed as the sum of immediate and long-term losses. More detailed, immediate losses for strands in a PC girder represents by elastic shortering, while long term losses related to concrete creep, shrinkage and steel relaxation.

Loss of prestress for post-tensioned member represented as the sum of the instantaneous loss and time-dependent loss. It should be noted, that the instantaneous loss is the losses occurred due to elastic shortening.

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$$

- *Elastic Shortening [LRDF C5.9.5.2.3a-1]*

Immediate elastic shortening losses are easily determined for PC girders using a closed form solution based on LRFD:

$$\Delta f_{pES} = \frac{\mathrm{A}_{ps} f_{pu}(\mathrm{I}_{nc} + e_m^2 \mathrm{A}_{nc}) - \mathrm{e}_m \mathrm{M}_{self-weight} A_{nc}}{\mathrm{A}_{ps}(\mathrm{I}_{nc} + e_m^2 \mathrm{A}_{nc}) + \frac{\mathrm{A}_{nc}\mathrm{I}_{nc}\mathrm{E}_{c.beam}}{\mathrm{E}_p}} = 130.48 MPa$$

Thus, the initial prestressing stress immediately after transfer = $0.75 f_{pu} - \Delta f_{pES} = 1265.71 MPa$

- *Long Term Losses – Approximate Method [LRFD 5.9.5.3]*

AASHTO LRFD provides three methods to estimate time-dependent losses:

  - a refined estimate
  - the approximate lump sum estimate
  - the background necessary to perform a rigorous time-step analysis

Due to the fact that the designing girder is a post-tensioned one, a approximate method should be used. The refined estimate method calculates time-dependent losses to creep, shrinkage, and relaxation in time between transportation and deck placement.

a. Correction factor for relative humidity of ambient air: $\gamma_h = 1.7 - 0.01H = 0.95$
b. Average annual humidity of Florida (H) is 75.
c. Correction factor for specified concrete strength time at of prestress transfer to concrete member:

$$\gamma_{st} = \frac{5}{1 + f'_{c.beam}} = 0.862$$

- Estimation of relaxation loss is equal to $\Delta f_{pR} = 16.55MPa$ for low relaxation strands.

Therefore, time-dependent losses:

$$\Delta f_{pLT} = 10\frac{0.75 f_{pu} A_{ps}}{A_{nc}} \gamma_h \gamma_{st} + 12\gamma_h \gamma_{st} + \Delta f_{pR} = 156.72MPa$$

Total loss of prestress is equal to:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 287.20MPa$$

$$\Delta f_{pT} = \frac{287.20}{0.75 f_{pu}} * 100\% = 20.57\%$$

Effective stress in prestressing strands (service level): $f_{pe} = 0.75 f_{pu} - \Delta f_{pT} = 1108.99MPa$

According to specifications listed in AASHTO 5.9.3-1, total loss of prestress should be less or equal than 0.8 times yield strength of reinforcing steel for prestressing stress limit at service limit state:

$$0.8 * f_y \geq \Delta f_{pT}$$

$$0.8 * 413.69 = 330.95MPA > 287.20MPa\ (OK)$$

4.2.5 Stress Limits (Compression=+, Tension=-)

4.2.5.1 Initial Stresses

Initial Stresses specifications listed in [SDG 4.3] evaluated according following estimations:

- Limit of tension in top of beam at release:

Outer 15 percent of design beam: $f_{top.outer15} = -6.41MPa$

Outer 70 percent of design beam:

$$f_{top.center70} = 1.38\ MPa$$

- Limit of compressive concrete strength at release: $0.6f_{ci.beam} = 24.65MPa$
- Total jacking force of strands: $P_j = 12543.18kN$
- The actual stress in strand after losses at transfer have occurred: $f_{pe} = 0.75f_{pu} = 1396.19\,{}^{kN}/_{m^2}$
- Total force of strands: $F_{pe} = f_{pe}A_{ps} = 12543.18kN$
- Stress at top beam at center 70%:

$$\sigma_{top.70} = \frac{M_{mid-span}}{S_t} + \left(\frac{F_{pe}}{A_{nc.tr}} - \frac{F_{pe}(yb_{nc.tr} - CGS)}{S_t}\right) = 3.47MPa < 0.6f_{ci.beam} = 24.65MPa\ (OK)$$

- Stress at bottom of beam at center 70%:

$$\sigma_{bottom.70} = -\frac{M_{mid-span}}{S_t} + \left(\frac{F_{pe}}{A_{nc.tr}} + \frac{F_{pe}(yb_{nc.tr} - CGS)}{S_t}\right) = 19.49MPa < 0.6f_{ci.beam} = 24.65MPa\ (OK)$$

4.2.5.2 Final Stresses

Final Stresses specification parameters listed in [LRFD Table 5.9.4.2.1-1 & 5.9.4.2.2-1] evaluated according following estimations:

- The stress due to prestress at the top and bottom of beam at midspan:

$$F_{pe} = \left(0.75f_{pu} - \Delta f_{pT}\right) * A_{ps} = 10.11kN$$

- Stress at top of beam: $\sigma_{pe.topbeam} = \frac{F_{pe}}{A_{nc.tr}} - \frac{F_{pe}(yb_{nc.tr} - CGS)}{S_t} = -5.86MPa$
- Stress at bottom of beam: $\sigma_{pe.bottombeam} = \frac{F_{pe}}{A_{nc.tr}} - \frac{F_{pe}(yb_{nc.tr} - CGS)}{S_t} = 26.54MPa$

4.2.5.3 Service I Limit States

The compressive stresses in the top of the beam will be checked for the following conditions:

- Sum of effective prestress and permanent loads
- Sum of effective prestress and permanent loads and transient loads

Sum of effective prestress and permanent loads:

Firstly, allowable limits should be calculated.

- Limit of compression in slab: $f_{allow1TopSlab} = 0.45 * f_{c.slab} = 13.96MPa$
- Limit of compression in top of beam: $f_{allow1TopBeam} = 0.45 * f_{c.beam} = 26.37MPa$

The stress due to permanent loads can be calculated as follows:

- Stress in top of slab: $\sigma_{1top.Slab} = \frac{M_{barrier+}M_{DW}}{S_t} = 0.063MPa < f_{allow1TopSlab} = 13.96MPa\ (OK)$
- Stress in top of beam:

$$\sigma_{1top.Beam} = \frac{M_{beam} + M_{Slab} + M_{Forms}}{S_{tb}} + \frac{M_{barrier} + M_{DW}}{S_t} + \sigma_{pe.top.beam}$$
$$= 14.37MPa < f_{allow1TopBeam}\ (OK)$$

4.2.5.4 Sum of effective prestress, permanent loads and transient loads

Following estimation should be estimated:

- Limit of compression in slab: $f_{allow2TopSlab} = 0.6 * f_{c.slab} = 18.61MPa$
- Limit of compression in top of beam: $f_{allow2TopBeam} = 0.6 * f_{c.beam} = 35.16MPa$
- Stress at the top of slab:
  $\sigma_{2top.Slab} = \sigma_{1top.Slab} + \frac{M_{LLI}}{S_t} = 0.16MPa < f_{allow2TopSlab} = 18.61MPa\ (OK)$
- Stress at top of beam: $\sigma_{2top.Beam} = \sigma_{1top.Beam} + \frac{M_{LLI}}{S_t} = 14.47MPa < f_{allow2TopBeam} = 35.16MPa\ (OK)$

4.2.5.5 Service III Limit State total stresses

Tension at bottom of beam only

- Limit of tension in bottom of beam: $f_{allowBotBeam} = -0.19\sqrt{f_{c.beam}} = -3.79MPa$

$$\sigma_{Bot.Beam} = \frac{-M_{beam} - M_{Slab} - M_{Forms}}{S_{tb}} + \frac{-M_{barrier} - M_{DW}}{S_b} + \sigma_{pe.bottom.beam}$$

$$+ 0.8\frac{M_{LLI}}{S_b} = -3.45MPa > f_{allowBotBeam} = -3.79MPa\ (OK)$$

4.2.6 Strength I Limit State moment capacity [LRFD 5.7.3]

Following estimation were performed in order to define Strenght I Limit State moment capacity:

- Strength I Limit State design moment: $M_r = 30439.26\text{kNm}$

In order to determine the average stress in the prestressing steel to be used for moment capacity, a factor "k" needs to be computed.

- Value for "k": $k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$
- Stress block factor: $\beta_1 = \min\left(\max\left(0.85 - 0.05 * \frac{f_{c.beam} - 27.6MPa}{6.90MPa}, 0.65\right), 0.85\right) = 0.65$
- Distance from the compression fiber to cg of prestress: $d_p = h - CGS = 1.34m$
- Area of reinforcing mild steel: $A_s = 0m^2$
- Distance from compression fiber to reinforcing mild steel: $d_s = 0m$
- Assumption: $f_s = f_y = 413.67MPa$ [LRFD 5.7.2.1]
- Distance between the neutral axis and compressive face: $c = \frac{A_{ps}f_{pu} + A_s f_s}{0.85 f_{c.beam}\beta_1 b_{tr.exterior} + kA_{ps}\frac{f_{pu}}{d_p}} = 0.35m$
- Depth of equivalent stress block: $a = \beta_1 c = 0.23m$
- Average stress in prestressing steel: $f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) = 1731.27MPa$
- Resistance factor for tension and flexure of prestressed members [LRFD 5.5.4.2]: $\phi' = 1.00$
- Moment capacity provided: $M_{r.prov} = \phi' * \left(A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right)\right) = 37421.56kNm$

Moment capacity provided is larger than Strength I Limit State design moment ($M_r < M_{r.prov}$) (OK)

The assumption that $f_s = f_y$ is true because area of reinforcing mild steel is equal to zero.

4.2.7 Minimum Reinforcement

The minimum reinforcement requirements ensure the factored moment capacity provided is at least equal to the lesser of the cracking moment and 1.33 times the factored moment required by the applicable strength load combinations.

- Modulus of Rupture: $f_r = -0.24\sqrt{f_{c.beam}} = -4.83MPa$ [SDG 1.4.1.B]
- Total unfactored dead load moment on noncomposite section:

$$M_{d.nc} = M_{beam} + M_{slab} + M_{forms} = 15295.5\ kN * m$$

- Flexural cracking variability factor (1.2 for precast segmental structures, 1.6 otherwise):

$$\gamma_1 = 1.6$$

- Prestress variability factor: $\gamma_2 = 1.1$

Ratio of specified minimum yield strength to ultimate tensile strength of reinforcement (0.67 for A615, Grade 60 reinforcement, 0.75 for A706, Grade 60 reinforcement, 1.00 for prestressed concrete structures: $\gamma_3 = 1.00$

- Cracking moment:

$$M_{cr} = \gamma_3((\gamma_1 * f_r + \gamma_2 * \sigma_{pe.bottom.beam}) * S_{bot.tr}) - M_{d.nc} * \left(\frac{S_{bot.tr}}{M_{botnc.tr}} - 1\right)$$
$$= 12751.47\ kN * m$$

- Required flexural resistance: $M_{r.reqd} = \min(M_{cr}, 133\% * M_r) = 12751.47\ kN * m$

According to LRFD 5.7.3.3.2, minimum reinforcement for positive moment is satisfied as $M_{r.prov} \geq M_{r.reqd}$.

4.2.8 Shear Design

4.2.8.1 Nominal Shear Resistance [LRFD 5.8.3.3]

The nominal shear resistance, V_n, shall be determined as the lesser of:

$$V_n = V_c + V_s$$

$$V_n = 0.25 f_c b_v d_v$$

The shear resistance of a concrete member may be separated into a component, V_c , that relies on tensile stresses in the concrete, a component, V_s , that relies on tensile stresses in the transverse reinforcement.

- Nominal shear resistance of concrete section: $V_c = 0.0316\beta\sqrt{f_c} b_v d_v$
- Nominal shear resistance of shear reinforcement section: $V_s = \frac{A_v d_v f_y \cot(\theta)}{s}$
- Effective shear depth: $d_v = \max\left(d_p - \frac{a}{2}, 0.9d_p, 0.72h\right) = 1.97m\ (use\ 0.1L\ for\ shear)$
- Effective width: $b_v = b_w = 177.8mm$
- Factored shear force at the critical section:

$$V_u = 1.25V_{DC} + 1.5V_{DW} + 1.75V_{LL} = 1678.70\ kN$$

- Factored moment on section: $M_u = \max(M(at\ 0.1L), V_u d_v) = 7882.37\ kN * m$

4.2.8.2 β and θ Parameters Method 2 [LRFD 5.8.3.4.2]

Following estimation were performed for evaluation:

- The strain in nonprestressed long tension reinforcement:

$$\varepsilon_s = max\left(-0.0004, \frac{\frac{|M_u|}{d_v} + V_u - A_{ps} * 0.7 * f_{pu}}{E_{c.beam}A_c + E_s A_s + E_p A_{ps}}\right) = -0.0004$$

- Angle of inclination of compression stresses: $\beta = \frac{4.8}{1+750*|\varepsilon_s|} = 3.69$
- Factor relating to longitudinal strain on the shear capacity of concrete: $\theta = 29 + 3500\varepsilon_s = 27.6$
- Nominal shear resistance of concrete section: $V_c = 0.0316\beta\sqrt{f_c} b_v d_v = 312.65kN$

4.2.8.3 Stirrups

Following calculations were performed for defining strirrups

- Size of stirrup bar (AASHTO provides three size of stirrups: “4”,”5”,”6”): take “5”.
- Area of shear reinforcement of stirrups: $A_v = 400\ mm^2$
- Diameter of shear reinforcement: dia = 15.88 mm
- Nominal shear strength provided by shear reinforcement:

$$V_n = \min(\frac{V_u}{\phi_v}, 0.25 f_c b_v d_v) = 2473.50\ kN$$

Therefore, $V_s = V_n - V_c = 2460.85\ kN$

4.2.8.4 Spacing of stirrups

Following calculations were performed for evaluation of spacing of stirrups:

- Minimum transverse reinforcement: $s_{min} = \frac{A_v f_y}{0.0316\beta\sqrt{f_c} b_v} = 1.04\ m$
- Transverse reinforcement required: $s_{req} = \frac{A_v f_y d_v \cot(\theta)}{V_s} = 0.25m$
- Minimum transverse reinforcement required: $spacing = \min\left(s_{min}, s_{req}\right) = 0.25m$

4.2.8.5 Longitudinal Reinforcement [LRFD 5.8.3.5]

For sections not subjected to torsion, longitudinal reinforcement shall be proportioned so that at each section the tensile capacity of the reinforcement on the flexural tension side of the member, taking into account any lack of full development of that reinforcement, shall be proportioned to satisfy:

$$V_s = \min\left(\frac{A_v f_y d_v \cot(\theta)}{spacing}, \frac{V_u}{\phi_v}\right) = 1865.22\ kN$$

General equation for force in longitudinal reinforcement:

$$T = \frac{|M_u|}{d_v} + \left(\frac{V_u}{\phi_v} - 0.5V_s\right) * \cot(\theta) = 5785.13\ kN$$

Equivalent force provided by this steel: $T_{ps} = A_{ps} f_{pe} = 10109.67\ kN$

So, $T_{ps} \geq T\ (OK)$ (LRFD 5.8.3.5)

4.2.9 Traditional Deck Design

Traditional Deck Design was performed using specifications listed in *Approximate Methods of Analysis - Decks [LRFD 4.6.2]*

4.2.9.1 Width of Equivalent Interior Strips [LRFD 4.6.2.1.3]

The deck is designed using equivalent strips of deck width. The equivalent strips account for the longitudinal distribution of LRFD wheel loads and are not subject to width limitations. The width in the transverse direction is calculated for both positive and negative moments. The overhangs will not be addressed in this section.

- Width of equivalent strip for positive moment: $E_{pos} = \left(26 + 6.6\frac{S}{ft}\right) in = 65.6\ in = 1.67\ meter$
- Width of equivalent strip for negative moment: $E_{neg} = \left(48 + 3\frac{S}{ft}\right) in = 66\ in = 1.68\ meter$

The equivalent strips can be modeled as continuous beams on rigid supports, since typical Florida-I beam bridges do not have any transverse beams.

4.2.9.2 Live Loads for Equivalent Strips

All HL-93 wheel loads shall be applied to the equivalent strip of deck width, since the spacing of supporting components in the secondary direction (longitudinal to beams) exceeds 1.5 times the spacing in the primary direction (transverse to beams). [LRFD 4.6.2.1.5]

- HL-93 wheel load: $P = 71.17\ kN$
- HL-93 wheel load for negative moment: $P_{neg} = \frac{P}{E_{neg}}(IM) = 56.34\ kN/m$
- HL-93 wheel load for positive moment: $P_{pos} = \frac{P}{E_{pos}}(IM) = 56.68\ kN/m$
- Location of Negative Live Load Design Moment, the negative live load design moment is taken at a distance from the supports: $Loc_{negative} = \min\left(\frac{1}{3}b_{tf}, 0.38\ meter\right) = 0.40\ meter$
- HL-93 Live Load Design Moments

Table A4-1 in Appendix A4 [LRFD 4.6.2.1.5] may be used to determine the live load design moments. The following conditions should be considered when using these moments:

- The moments are calculated by applying the equivalent strip method to concrete slabs supported on parallel beams.
- Multiple presence factors and dynamic load allowance are included.
- The values are calculated according to the location of the design section for negative moments in the deck (LRFD 4.6.2.1.6). For distances between the listed values, interpolation may be used.
- The moments are applicable for decks supported by at least three beams with a width between the centerlines of the exterior beams of not less than 4.27 meter.
- The values represent the upper bound for moments in the interior regions of the slab.
- A minimum and maximum total overhang width from the center of the exterior girder are evaluated. The minimum is 0.53m and the maximum is the smaller of 0.625*BeamSpacing (1.14 m) and 1.83 m.
- A railing barrier width of 0.53 m is used to determine the clear overhang width. Florida utilizes a railing width of 0.47 m. The difference in moments from the different railing width is expected to be within acceptable limits for practical design.
- The moments do not apply to deck overhangs, which may be detailed in accordance with the provisions of SDG 4.2.4.B without further analysis (see section Deck Overhang Design of these calculations).

S	Positive Moment	NEGATIVE MOMENT						
		Distance from CL of Girder to Design Section for Negative Moment						
		0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4' -0"	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4' -3"	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4' -6"	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4' -9"	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5' -0"	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5' -3"	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5' -6"	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5' -9"	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6' -0"	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6' -3"	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6' -6"	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6' -9"	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7' -0"	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7' -3"	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7' -6"	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7' -9"	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8' -0"	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8' -3"	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8' -6"	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8' -9"	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9' -0"	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9' -3"	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9' -6"	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9' -9"	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10' -0"	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10' -3"	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10' -6"	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10' -9"	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11' -0"	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11' -3"	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11' -6"	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11' -9"	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12' -0"	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12' -3"	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12' -6"	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12' -9"	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13' -0"	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13' -3"	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13' -6"	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13' -9"	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14' -0"	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45
14' -3"	9.14	12.46	11.59	10.72	9.85	8.99	7.38	6.58
14' -6"	9.25	12.67	11.81	10.94	10.08	9.21	7.57	6.72
14' -9"	9.36	12.88	12.02	11.16	10.30	9.44	7.76	6.86

Figure 4. 4 Calculation of Negative and Positive Moments by beam spacing

As our design has 1.83 meter beam spacing, positive live load design moment is

$$M_{LL.pos} = 4.43\ ft * kip = 6.00\ kN * m$$

In addition, $Loc_{negative} = 0.40\ meter = 15.75\ in$ using interpolation method:

$$M_{LL.neg} = (15.75in - 12in) * \left(\frac{4.09ft * kip - 4.41\ ft * kip}{18in - 12in}\right) + 4.41ft * kip$$

$$= 4.21\ ft * kip = 5.71\ kN * m$$

4.2.9.3 Dead Load Design Moments

Following estimations were performed for defining dead load moments

- Design width of deck slab: $b_{slab} = 0.30\ meter$

"DC" loads include the dead load of structural components and non-structural attachments.

- Self-weight of deck slab: $w_{Slab} = (t_{slab} + t_{mill}) * b_{slab} * \gamma_{conc} = 1.55\ {kN}/{m}$
- Weight of traffic barriers: $P_{barrier} = w_{barrier} b_{slab} = 1.87\ kN$
- Weight of median barrier: $P_{median.barrier} = w_{median.barrier} b_{slab} = 2.16\ kN$

4.2.9.4 Analysis Model for Dead Loads

Any plane frame program can be utilized to develop the moments induced by the dead loads. For this design, RISA was used to determine the dead load design moments.

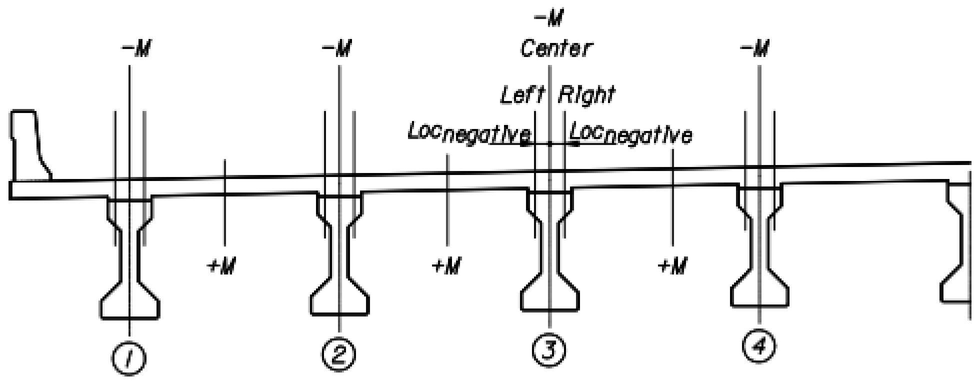


Figure 4. 5 Negative and Positive Moment Locations

Summarized outputs from the software RISA presented in Table 3.9.

Table 4. 9 Summary of Factored Moment (kNm) and Shear (kN)

Beam	Design Moments for DC loads			
	Positive Moment (kN*m)	Negative moment (kN*m)		
		Center	Left	Right
1	0,00	-4,07	-2,77	-2,83
2	0,91	-0,43	-0,09	0,23
3	0,53	-1,41	-0,50	-0,58
4	0,61	-1,14	-0,39	-0,37

At beam 1, the governing negative design moment exists. The main reason of this existence can be represented by overhang, which mostly has more negative moment steel requirements than the interior regions of the deck. Due to the fact, that overhang designed as single independent part, the overhang moments are not focused in this section. For the interior regions, the positive moment in Span 2 and the negative moment to the right of beam 3 govern. Therefore, following estimations are presented:

- Positive moment: $M_{DC.pos} = 0.91\ kN * m$
- Negative moment: $M_{DC.neg} = 0.58\ kN * m$

Moments due to dead load of wearing surface and utilities are zero because weight of future wearing surface is taken as zero in previous sections: $M_{DW.neg} = M_{DW.pos} = 0\ kN * m$

4.2.9.5 Limit State Moments

The service and strength limit states are used to design the section

- Positive Service I Moment: $M_{serviceIpos} = M_{DC.pos} + M_{DW.pos} + M_{LL.pos} = 6.91\ kN * m$
- Negative Service I Moment: $M_{serviceIneg} = M_{DC.neg} + M_{DW.neg} + M_{LL.neg} = 6.29\ kN * m$
- Positive Strength I Moment:

$$M_{strengthIpos} = 1.25M_{DC.pos} + 1.50M_{DW.pos} + 1.75M_{LL.pos} = 11.64\ kN * m$$

- Negative Strength I Moment:

$$M_{strengthIneg} = 1.25M_{DC.neg} + 1.50M_{DW.neg} + 1.75M_{LL.neg} = 10.72\ kN * m$$

4.2.9.6 Moment Design

Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Initial assumption for area of steel required:

- Size of bar: bar=”5”
- Proposed bar spacing: $spacing_{pos} = 0.15\ meters$
- Bar area: $A_{bar} = 200\ mm^2$
- Bar diameter: d=15.88 mm

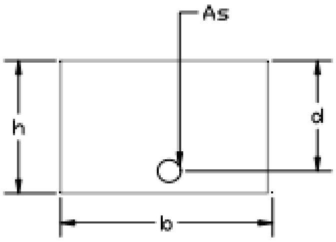


Figure 4. 6 Model for Positive Moment

- Area of steel provided
- fiber to centroid of reinforcing steel:

$$d_{s.pos} = t_{slab} + t_{mill} - cover_{deck} - \frac{dia}{2} = 0.14\, meters$$

In order to check initial assumptions, area of reinforcement steel required must be defined by following formulae:

$$M_{strengthI.pos} = \phi A_{s.reqd.pos} f_y (d_{s.pos} - \frac{1}{2}\left(\frac{A_{s.reqd.pos} f_y}{0.85 f_{c.slab} b}\right))$$

- After per meter of slab: $A_{s.pos} = \frac{A_{bar} * 1\, meter}{spacing_{pos}} = 1333.33\, mm^2$

Distance from extreme compressive calculations, reinforcing steel required ($A_{s.reqd.pos}$) is 1161.29 mm^2

The area of steel provided, $A_{s.pos} = 1333.33\, mm^2$, should be larger than the area of steel required, $A_{s.reqd.pos} = 1161.29\, mm^2$. Once $A_{s.pos}$ is greater than $A_{s.reqd.pos}$, the proposed reinforcing is satisfied for the design moments.

4.12.6.2 Negative Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Initial assumption for area of steel required:

- Size of bar: bar="5"
- Proposed bar spacing: $spacing_{pos} = 0.15\, meter$
- Bar area: $A_{bar} = 200\, mm^2$

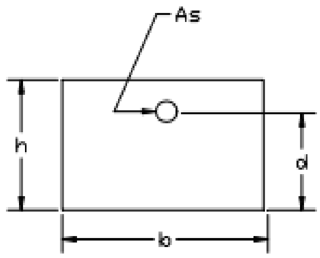


Figure 3. 4 Model for Negative Moment

- Bar diameter: dia=15.88 mm
- Area of steel provided per meter of slab: $A_{s.neg} = \frac{A_{bar} * 1\, meter}{spacing_{pos}} = 1333.33\, mm^2$
- Distance from extreme compressive fiber to centroid of reinforcing steel:

$$d_{s.neg} = t_{slab} + t_{mill} - cover_{deck} - \frac{dia}{2} = 0.14\, meters$$

In order to check initial assumptions, area of reinforcement steel required must be defined by following formulae:

$$M_{strengthI.neg} = \phi A_{s.reqd.neg} f_y (d_{s.neg} - \frac{1}{2}\left(\frac{A_{s.reqd.neg} f_y}{0.85 f_{c.slab} b}\right))$$

After performing estimations, reinforcing steel required ($A_{s.reqd.neg}$) is 696.77 mm^2.

The area of steel provided,$A_{s.neg} = 1333.33\, mm^2$, should be greater than the area of steel required, $A_{s.reqd.neg} = 696.77\, mm^2$. Once $A_{s.neg}$ is greater than $A_{s.reqd.neg}$, the proposed reinforcing is satisfied for the design moments.

Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is a material that can be subjected to cracking. Therefore, several limitations should be taken into account in order to minimize danger. First one, limiting the width of expected cracks under service conditions increases the longevity of the structure. Secondly, potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

The maximum spacing of the mild steel reinforcement for control of cracking at the service limit state shall satisfy: $s \leq \frac{700\gamma_e}{\beta_s f_{c.actual}} - 2d_c$, where $\beta_s = 1 + \frac{d_c}{0.7(t_{slab} - d_c)}$

Exposure factor for Class 1 exposure condition [SDG 3.10]: $\gamma_e = 1.00$

Positive Moment

- Distance from extreme tension fiber to center of closest bar:

$$d_c = cover_{deck} - t_{mill} + \frac{dia}{2} = 0.06\, meters$$

$$\beta_s = 1 + \frac{d_c}{0.7(t_{slab} - d_c)} = 1.581$$

- The neutral axis of the section calculated from following quadratic equation:

$$\frac{1}{2} b x^2 = \frac{E_s}{E_{c.slab}} A_{s.pos}\, (d_{s.pos} - x)$$

$$x = x_{pos} = 46.33\ mm$$

- Tensile force in the reinforcing steel due to service limit state moment:

$$T_s = \frac{M_{serviceIpos}}{d_{s.pos} - \frac{x_{pos}}{3}} = 55.48\ kN$$

- Actual stress in the reinforcing steel due to service limit state moment:

$$f_{s.actual} = \frac{T_s}{A_{s.pos}} = 41.61\ MPa$$

- Required reinforcement spacing: $s_{required} = \frac{700\gamma_e}{\beta_s f_{c.actual}} - 2d_c = 0.27\ meters$
- Provided reinforcement spacing: $spacing_{pos} = 0.15\ meters$

The required spacing of mild steel reinforcement in the layer closest to the tension face shall not be less than the reinforcement spacing provided due to the service limit state moment [LRFD 5.7.3.4]. Therefore, $s_{required} = 0.27 > spacing_{pos} = 0.15\ \ (OK)$

Negative Moment

- Distance from extreme tension fiber to center of closest bar:

$$d_c = cover_{deck} - t_{mill} + \frac{dia}{2} = 0.06\ meters$$

$$\beta_s = 1 + \frac{d_c}{0.7(t_{slab} - d_c)} = 1.581$$

- The neutral axis of the section calculated from following quadratic equation:

$$\frac{1}{2} bx^2 = \frac{E_s}{E_{c.slab}} A_{s.neg}\ (d_{s.neg} - x)$$

$$x = x_{neg} = 46.33\ mm$$

- Tensile force in the reinforcing steel due to service limit state moment:

$$T_s = \frac{M_{serviceIneg}}{d_{s.neg} - \frac{x_{neg}}{3}} = 50.50\ kN$$

- Actual stress in the reinforcing steel due to service limit state moment:

$$f_{s.actual} = \frac{T_s}{A_{s.neg}} = 37.87\ MPa$$

- Required reinforcement spacing: $s_{required} = \frac{700\gamma_e}{\beta_s f_{c.actual}} - 2d_c = 0.48\ meters$
- Provided reinforcement spacing: $spacing_{neg} = 0.15\ meters$

The required spacing of mild steel reinforcement in the layer closest to the tension face shall not be less than the reinforcement spacing provided due to the service limit state moment [LRFD 5.7.3.4]. Thus, $s_{required} = 0.48 > spacing_{neg} = 0.15\ \ (OK)$

Minimum Reinforcement [5.7.3.3.2]

- Area of steel provide: $A_s = \max(A_{s.pos}, A_{s.neg}) = 1333.33\ mm^2$
- Area of steel required for bending: $A_{s.reqd} = \min(A_{s.reqd.pos}, A_{s.reqd.neg}) = 696.77\ mm^2$

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

- Modulus of Rupture: $f_r = 0.24\sqrt{f_{c.slab}} = 3.51\ MPa$
- Distance from the extreme tensile fiber to the neutral axis of the composite section:

$$y = \frac{t_{slab}}{2} = 0.10\ meters$$

- Moment of inertia for the section: $I_{slab} = \frac{1}{12} b t_{slab}^3 = 2.13 * 10^{-4} m^4$
- Section modulus: $S = \frac{I_{slab}}{y} = 2.13 * 10^{-3} m^3$
- Flexural cracking variability factor: $\gamma_1 = 1.6$
- Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement: $\gamma_3 = 0.67$
- Cracking moment: $M_{cr} = \gamma_3 \gamma_1 f_r S = 0.80 * 10^{-3}\ MPa$
- Minimum distance to reinforcing steel: $d_s = \min\left(d_{s.pos}, d_{s.neg}\right) = 0.14\ meters$
- Minimum reinforcement required: $A_{min} = \frac{\frac{M_{cr}}{\phi}}{f_y\left(d_s - 0.5\frac{A_s f_y}{0.85 f_{c.slab} b}\right)} = 516.13\ mm^2$

Required area of steel for minimum reinforcement should not be less than $A_{s.reqd} *$ $133\%\ or\ A_{min}$: $A_{s.req} = \min(A_{s.reqd} * 133\%, A_{min}) = 516.13\ mm^2$

According to LRFD 5.7.3.3.2, design area of steel should be more than required area of steel:

$$A_s > A_{s.req}\ (OK)$$

4.2.10 Deck Overhang Design

4.2.10.1 Deck Slab Design [SDG 4.2.4]

The deck overhang was designed using the traditional design method. According to specifications, the minimum transverse top slab reinforcing may be provided without further analysis where the indicated minimum slab depths are provided in [SDG 4.2.4B] and the total deck overhang is 6 feet or less. Deck slab designed by the traditional design method:

$$A_{sTL} = 1720.43\ mm^2$$

4.2.10.2 Deck Overhang Reinforcement

4.13.2.1 B1. Negative Moment Region - Reinforcement Requirements [SDG 4.2.4B]
Initial assumption for area of steel required:

- Size of bar: bar=”5”
- Proposed bar spacing: $spacing_{pos} = 0.10\ meter$
- Bar area: $A_{bar} = 200\ mm^2$

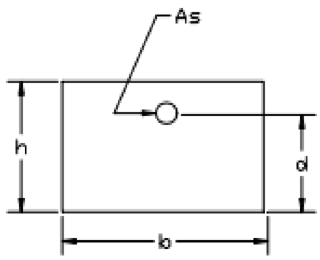


Figure 4. 7 Model for Negative Moment for Deck Overhang

- Bar diameter: d=15.88 mm
- Area of steel provided per meter of slab: $A_{s.overhang} = \frac{A_{bar} * 1\ meter}{spacing_{pos}} = 2000\ mm^2$

Limits for Reinforcement [LRFD 5.7.3.3]
Following estimations were done for completing limits for reinforcement

- Area of steel provide: $A_s = A_{s.overhang} = 2000\ mm^2$

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

- Modulus of Rupture: $f_r = 0.24\sqrt{f_{c.slab}} = 3.51\ MPa$
- Distance from the extreme tensile fiber to the neutral axis of the composite section:

$$y = \frac{t_{slab}}{2} = 0.10\ meters$$

- Moment of inertia for the section: $I_{slab} = \frac{1}{12} b t_{slab}^3 = 2.13 * 10^{-4} m^4$
- Section modulus: $S = \frac{I_{slab}}{y} = 2.13 * 10^{-3} m^3$
- Flexural cracking variability factor: $\gamma_1 = 1.6$
- Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement: $\gamma_3 = 0.62$
- Cracking moment: $M_{cr} = \gamma_3 \gamma_1 f_r S = 0.74 * 10^{-3}\ MPa$
- Minimum distance to reinforcing steel: $d_s = \min\left(d_{s.pos}, d_{s.neg}\right) = 0.14\ meters$
- Minimum reinforcement required: $A_{min} = \frac{\frac{M_{cr}}{\phi}}{f_y(d_s - 0.5\frac{A_s f_y}{0.85 f_{c.slab} b})} = 477.13\ mm^2$

Required area of steel for minimum reinforcement should not be less than $A_{a.TL} * 133\%\ or\ A_{min}$: $A_{s.req} = \min(A_{s.LT} * 133\%, A_{min}) = 477.13\ mm^2$

According to LRFD 5.7.3.3.2, design area of steel should be more than required area of steel:

$$A_s > A_{s.req}\ (OK)$$

4.2.11 Expansion Joint Design

Input Variables to design expansion joint

4.2.11.1 Temperature Movement [SDG 6.3]

Temperature Range by Superstructure Material				
Superstructure Material	Temperature Range (Degrees Fahrenheit)			
	Mean	High	Low	Range
Concrete Only	70	105	35	70
Concrete Deck on Steel Girder	70	110	30	80
Steel Only	70	120	30	90

Figure 4. 8 Temperature range

The temperature values for "Concrete Only" in the preceding table should be used in our design.

- Temperature mean: $t_{mean} = 70°F = 21.11°C$
- Temperature high: $t_{high} = 105°F = 40.56°C$
- Temperature low: $t_{low} = 35°F = 1.67°C$
- Temperature rise: $\Delta t_{rise} = t_{high} - t_{mean} = 19.45°C$
- Temperature fall: $\Delta t_{fall} = t_{mean} - t_{low} = 19.45°C$
- Coefficient of thermal expansion [LRFD 5.4.2.2] for normal weight concrete: $\alpha_t = 6 * 10^{-6} \frac{1}{°F}$
- Temperature Load Factor: $\gamma_{TU} = 1.2$

4.2.11.2 Expansion Joints [SDG 6.4]

Expansion Joint Type	Maximum Open Width "W" (measured in the direction of travel at deck surface)
Hot Poured or Poured Joint without Backer Rod	3/4-inch
Poured Joint with Backer Rod	3-inches
Armored Elastomeric Strip Seal (Single gap)	Per LRFD [14.5.3.2]
Modular Joint (Multiple modular gaps)	Per LRFD [14.5.3.2]
Finger Joint	Per LRFD [14.5.3.2]

Figure 4. 9 Expression Joint Type

A typical joint for most prestressed beam bridges is the poured joint with backer rod.

- Proposed joint width at 70o F (per FDOT Instructions for Design Standards Index 21110): $W = 50.8\ mm$
- Maximum joint width: $W_{max} = 76.2\ mm$
- Minimum joint width: $W_{min} = 0.5W = 25.4mm$

4.2.11.3 Movement [SDG 6.4.2]

Temperature The movement along the beam due to temperature should be resolved along the axis of the expansion joint or skew.

Displacements normal to skew at top of bents

- Temperature rise: $\Delta z_{TempR} = \alpha_t \Delta t_{rise} \cos(Skew)\, L_{span} = 12.6mm$
- Temperature fall: $\Delta z_{TempF} = \alpha_t \Delta t_{fall} \cos(Skew)\, L_{span} = 12.6mm$

Displacements parallel to skew at top of bents

- Temperature rise: $\Delta z_{TempR} = \alpha_t \Delta t_{rise} \sin(Skew)\, L_{span} = 0mm$
- Temperature fall: $\Delta z_{TempF} = \alpha_t \Delta t_{fall} \sin(Skew)\, L_{span} = 0mm$

For poured joint with backer rod, displacements parallel to the skew can be assumed as negligible parts in most joint designs. Therefore, these displacements are ignored.

4.2.11.4 Creep and Shrinkage

The following assumptions are used in the design:

• Creep and Shrinkage prior to day 120 (casting of deck) is neglected for the expansion joint design.

• Creep [LRFD 5.4.2.3] is not considered at this time. After day 120, all beams are assumed to creep towards their centers. The slab will offer some restraint to this movement of the beam. The beam and slab interaction, combined with forces not being applied to the center of gravity for the composite section, is likely to produce longitudinal movements and rotations. For most prestressed beams designed as simple spans for dead and live load, these joint movements due to creep are ignored.

Shrinkage after day 120 is calculated using LRFD 5.4.2.3.

- Creep strain: $\varepsilon_{CR} = 0$
- Shrinkage strain: $\varepsilon_{SH} = 0.00008$
- Strain due to creep and shrinkage: $\varepsilon_{CS} = \varepsilon_{CR} + \varepsilon_{SH} = 0.00008$

The movement along the beam due to creep and shrinkage should be resolved along the axis of the expansion joint or skew.

- Displacements normal to skew at top of bents: $\Delta z_{CS} = \varepsilon_{CS} \cos(Skew)\, L_{span} = 4.8mm$
- Displacements parallel to skew at top of bents: $\Delta x_{CS} = \varepsilon_{CS} \sin(Skew)\, L_{span} = 0mm$

For poured joint with backer rod, displacements parallel to the skew are not significant in most joint designs. For our design, these displacements are ignored.

4.2.11.5 Expansion Joint Design

For prestressed concrete structures, the movement is based on the greater of two cases:

- Movement from the combination of temperature fall, creep, and shrinkage
- Movement from factored effects of temperature

Movement from Creep, Shrinkage and Temperature (SDG 6.4.2)

The combination of creep, shrinkage, and temperature fall tends to "open" the expansion joint.

- Movement from the combination of temperature fall, creep, and shrinkage:

$$\Delta_{CST} = \Delta z_{CS} + \Delta z_{TempF} = 17.4mm$$

- Joint width from opening caused by creep, shrinkage, and temperature:

$$W_{CSTopen} = W + \Delta_{CST} = 68.2\ mm$$

- The joint width from opening should not exceed the maximum joint width:

$$W_{max} = 76.2\ mm > W_{CSTopen} = 68.2\ mm\ (OK)$$

Movement from Temperature (SDG 6.4.2)

- Joint width from opening caused by factored temperature fall:

$$W_{Topen} = W + \gamma_{TU} \Delta \mathrm{z}_{TempF} = 65.9\ mm$$

- Joint width from closing caused by factored temperature rise:

$$W_{Tclose} = W - \gamma_{TU} \Delta \mathrm{z}_{TempR} = 35.7\ mm$$

- The joint width from opening should not exceed the maximum joint width:

$$W_{max} = 76.2\ mm > W_{Topen} = 65.9\ mm\ (OK)$$

- The joint width from closing should not be less than the minimum joint width:

$$W_{Tclose} = 35.7\ mm > W_{min} = 25.4\ mm\ (OK)$$

Temperature Adjustment for Field Placement of Joint

For field temperatures other than 21.11 °C, a temperature adjustment is provided. The adjustment is used during construction to obtain the desired joint width:

$$T_{Adj} = \frac{\Delta z_{TempR}}{\Delta t_{rise}} = 0.65\ \frac{mm}{°C}$$

4.2.11.6 Design Summary

Following outputs generated for design:

- Joint width at 21.11 °C: $W = 50.8\ mm$
- Joint width from opening caused by creep, shrinkage, and temperature: $W_{CSTopen} = 68.2\ mm$
- Joint width from opening caused by factored temperature: $W_{Topen} = 65.9\ mm$
- Joint width from closing caused by factored temperature: $W_{Tclose} = 35.7\ mm$
- Adjustment for field temperatures other than 21.11 °C: $T_{Adj} = 0.65\ \frac{mm}{°C}$

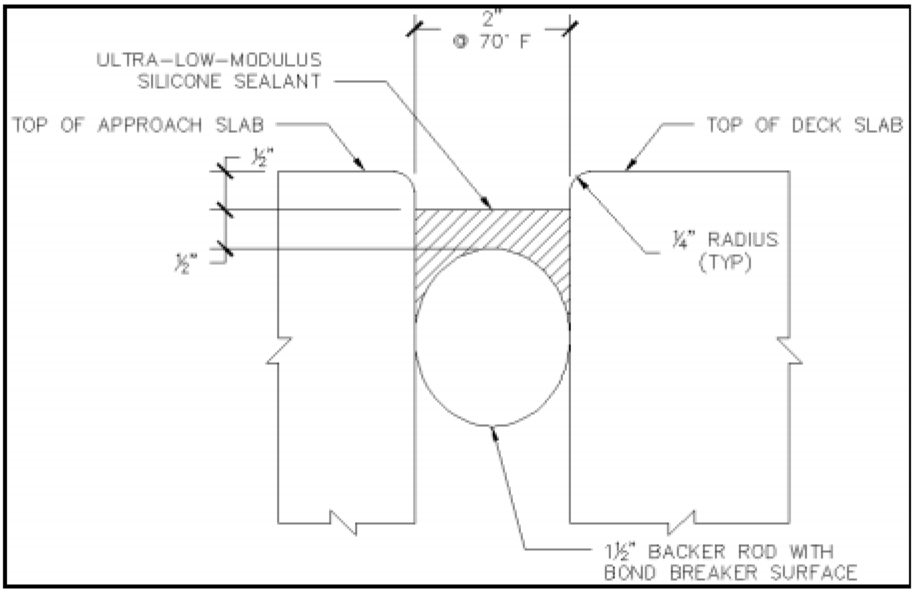


Figure 4. 10 Expression Joint Type

4.3 Substructure Design

4.3.1 Obtain Design Criteria

The first design step is to identify the appropriate design criteria. This includes, but is not limited to, defining material properties, identifying relevant superstructure information, and determining the required pier geometry.

Unit weight of concrete: $w_c = 23.57\ kN/m^3$

Concrete 28-day compressive strength: $f_c = 24.13\ MPa$

Reinforcement strength: $f_y = 413.69\ MPa$

Bearing Height (Fixed, Type A): $H_{brng} = 0.16\ meters$

Bearing width: $W_{brng} = 0.46\ meters$

Bearing Length: $L_{brng} = 0.66\ meters$

All cover dimensions listed below are in accordance with LRFD [Table 5.12.3-1]:

Pier cap: $Cover_{cp} = 0.0635\ m$

Pier column: $Cover_{co} = 0.0635\ m$

Footing top cover: $Cover_{ft} = 0.0508\ m$

Footing bottom cover: $Cover_{fb} = 0.1524\ m$

Select Preliminary Pier Dimensions

Selecting the most optimal pier type depends on site conditions, cost considerations, superstructure geometry, and aesthetics. For this design example, a single column (hammerhead) pier was chosen.

Since the Specifications do not have standards regarding maximum or minimum dimensions for a pier cap, column, or footing, the designer should base the preliminary pier dimensions on FDOT specific standards, previous designs, and past experience. The pier cap, however, must be wide enough to accommodate the bearing.

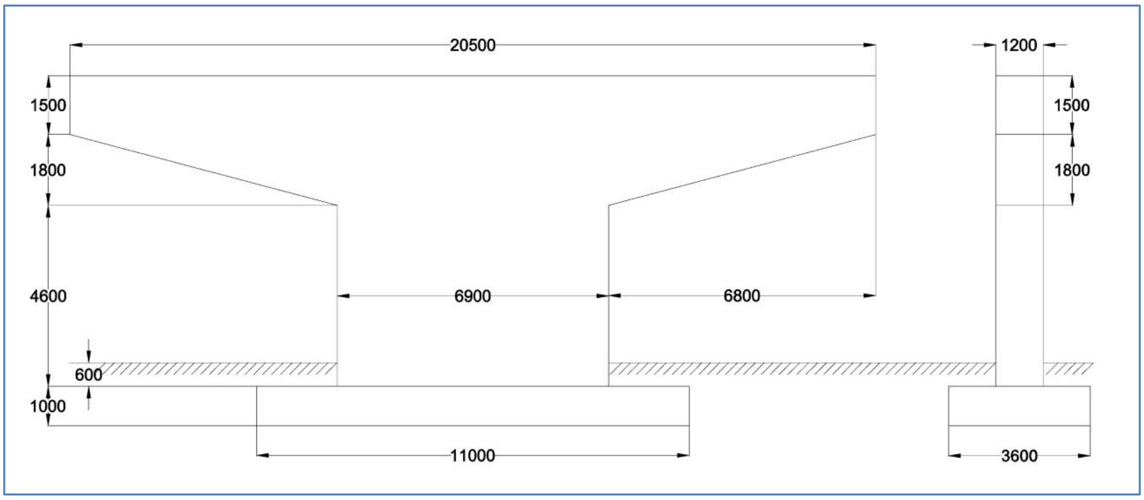


Figure 4. 11 Pier Dimensions

Pier Geometry Dimensions:

$L_{cap} = 20.5\ m$ $\quad L_{col} = 6.9\ m$ $\quad L_{ftg} = 11\ m$ $\quad D_{soil} = 0.6\ m$

$W_{cap} = 1.2\ m$ $\quad W_{col} = 1.2\ m$ $\quad W_{ftg} = 3.6\ m$ $\quad \gamma_{soil} = 18.85\ kN/m^3$

$H_{cap} = 3.3\ m$ $\quad H_{col} = 4.6\ m$ $\quad H_{ftg} = 1\ m$

$H_{cap\_end} = 1.5\ m$

$L_{oh} = 6.8\ m$

	Load Factors							
	Strength I		Strength III		Strength V		Service I	
Load	γ max	γ min	γ max	γ min	γ max	γ min	γ max	γ min
DC	1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.00
DW	1.50	0.65	1.50	0.65	1.50	0.65	1.00	1.00
LL	1.75	1.75	---	---	1.35	1.35	1.00	1.00
BR	1.75	1.75	---	---	1.35	1.35	1.00	1.00
TU	1.20	0.50	1.20	0.50	1.20	0.50	1.20	1.00
WS	---	---	1.40	1.40	0.40	0.40	0.30	0.30
WL	---	---	---	---	1.00	1.00	1.00	1.00
EV	1.35	1.00	1.35	1.00	1.35	1.00	1.00	1.00

Figure 4. 12 Load Factors and Appreciable Pier Limit States

4.3.2 Dead Load Effects

Once the preliminary pier dimensions are selected, the corresponding dead loads can be computed in accordance with LRFD [3.5.1]. The pier dead loads must then be combined with the superstructure dead loads.

Exterior girder dead load reactions at supports: $DL_{ext.beam} = 1025.10\ kN$

Interior girder dead load reactions at supports: $DL_{int.beam} = 992.70\ kN$

Pier cap dead load:

$$DL_{cap} = w_c W_{cap}\left(2 * \left(\frac{H_{cap_{end}} + H_{cap}}{2}\right) * L_{oh} + H_{cap} L_{col}\right) = 1567.22\ kN$$

Pier column dead load: $DL_{col} = w_c W_{col} H_{col} L_{col} = 897.73\ kN$

Pier footing dead load: $DL_{ftg} = w_c W_{ftg} H_{ftg} L_{ftg} = 933.37\ kN$

In addition to the above dead loads, the weight of the soil on top of the footing must be computed. 0.6 meters height of soil above the footing were previously defined. Assuming a unit weight of soil at $18.85\ kN/m^3$ in accordance with LRFD [Table 3.5.1-1]:

$$EV_{ftg} = \gamma_{soil} D_{soil}\left(W_{ftg} L_{ftg} - W_{col} L_{col}\right) = 354.23\ kN$$

4.3.3 Live Load Effects

Exterior girder live load reactions at supports: $LL_{ext.beam} = 692.1\ kN$

Interior girder live load reactions at supports: $LL_{int.beam} = 692.1\ kN$

4.3.4 Other Load Effects

Other load effects that will be considered for this pier design include braking force, wind loads, and temperature loads. For simplicity, buoyancy, stream pressure, ice loads and earthquake loads are not included in this design

4.3.4.1 Braking Force

Since expansion bearings exist at the abutments, the entire longitudinal braking force is resisted by the pier.

In accordance with LRFD [3.6.4], the braking force per lane is the greater of:

- 25 percent of the axle weights of the design truck or tandem
- 5 percent of the axle weights of the design truck plus lane load
- 5 percent of the axle weights of the design tandem plus lane load

The total braking force is computed based on the number of design lanes in the same direction. Also, braking forces are not increased for dynamic load allowance in accordance with LRFD [3.6.2.1]. The calculation of the braking force for a single traffic lane follows:

25 percent of the design truck: $BRK_{trk} = 0.25(142.34 + 142.34 + 35.59) = 80.07\ kN$

25 percent of the design tandem: $BRK_{tan} = 0.25(111.21 + 111.21) = 55.61\ kN$

5 percent of the axle weights of the design truck plus lane load:

$$BRK_{trk\_lan} = 0.05\big((142.34 + 142.34 + 35.59) + (0.64 * 2 * L)\big) = 50.71\ kN$$

5 percent of the axle weights of the design tandem plus lane load:

$$BRK_{tan\_lan} = 0.05\big((111.21 + 111.21) + (0.64 * 2 * L)\big) = 43.71\ kN$$

Use: $BRK = \max\big(BRK_{trk}, BRK_{tan}, BRK_{trk_{lan}}, BRK_{\tan\_lan}\big) = 80.07\ kN$

LRFD [3.6.4] states that the braking force is applied along the longitudinal axis of the bridge at a distance of 1.83 m above the roadway surface. However, since the skew angle is zero for this design example and the bearings are assumed incapable of transmitting longitudinal moment, the braking force will be applied at the top of bearing elevation. For bridges with skews, the component of the braking force in the transverse direction would be applied 1.83 m above the roadway surface.

This force may be applied in either horizontal direction (back or ahead station) to cause the maximum force effects. Additionally, the total braking force is typically assumed equally distributed among the bearings:

$$BRK_{brg} = \frac{BRK}{11} = 7.28\ kN$$

The moment arm about the base of column is: $H_{BRK} = H_{col} + H_{cap} + \frac{H_{brng}}{12} = 7.91\ meters$

4.3.4.2 Wind Load on Superstructure

Prior to calculating the wind load on the superstructure, the structure must be checked for aero elastic instability, LRFD [3.8.3]. If the span length to width or depth ratio is greater than 30, the structure is considered wind-sensitive and design wind loads should be based on wind tunnel studies.

$$\frac{L_{span}}{Width} = 2.93\ (OK) \qquad \frac{L_{span}}{Depth} = 18.69\ (OK)$$

Since the span length to width and depth ratios are both less than 30, the structure does not need to be investigated for aero elastic instability.

To compute the wind load on the superstructure, the area of the superstructure exposed to the wind must be defined. The exposed area is the total superstructure depth multiplied by length tributary to the pier. Due to expansion bearings at the abutment, the transverse length tributary to the pier is not the same as the longitudinal length.

The superstructure depth includes the total depth from the top of the barrier to the bottom of the girder. Included in this depth is any haunch and/or depth due to the deck cross-slope. Once the total depth is known, the wind area can be calculated and the wind pressure applied.

Total depth of superstructure: $H_{super} = 3.21\ meters$

The tributary length for wind load on the pier in the transverse direction is equal to span length: $L_{windT} = L_{span} = 60\ meters$

In the longitudinal direction, the tributary length is the entire bridge length due to the expansion bearings at the abutments: $L_{windL} = L_{bridge} = 170\ meters$

The transverse wind area is: $A_{wsuperT} = H_{super}L_{windT} = 192.6\ m^2$

The longitudinal wind area is: $A_{wsuperL} = H_{super}L_{windL} = 545.7\ m^2$

According to FDOT Specifications, wind pressure in Florida region should be taken as follows:

$$Psup_{trans} = 2.39\ kN/m^2$$

$$Psup_{longit} = 0.91\ kN/m^2$$

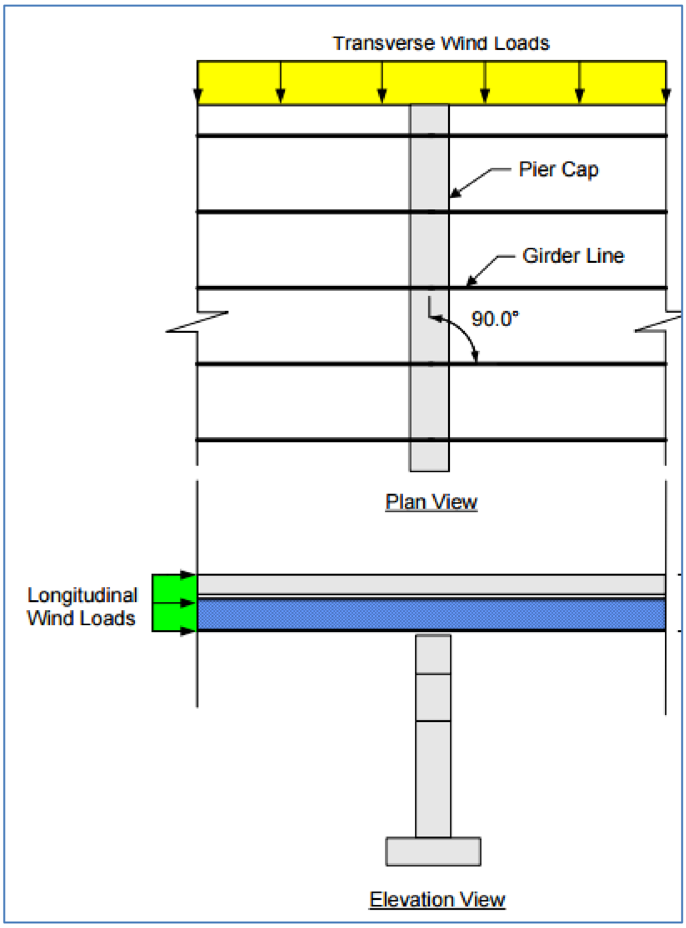


Figure 4. 13 Transverse and Longitudinal Wind Load Effect on Superstructure

The superstructure wind loads acting on the pier (girders) are:

$$WS_{suptrns} = A_{wsuperT} Psup_{trans} = 460.31\ kN$$

$$WS_{suplng} = A_{wsuperL} Psup_{longit} = 496.59\ kN$$

The total longitudinal wind load shown above is assumed to be divided equally among the bearings. In addition, the load at each bearing is assumed to be applied at the top of the bearing. These assumptions are consistent with those used in determining the bearing forces due to the longitudinal braking force.

The transverse wind loads shown above are also assumed to be equally divided among the bearings but are applied at the mid-height of the superstructure.

For calculating the resulting moment effect on the column, the moment arm about the base of the column is:

$$H_{WSlong} = H_{col} + H_{cap} + \frac{H_{brng}}{12} = 7.91\, meters$$

$$H_{WStrsn} = H_{col} + H_{cap} + \frac{H_{brng}}{12} + \frac{H_{super}}{2} = 9.52\, meters$$

However, the transverse load also applies a moment to the pier cap. This moment, which acts about the centerline of the pier cap, induces vertical loads at the bearings as illustrated in Figure E13-1.6-2. The computations for these vertical forces are presented below.

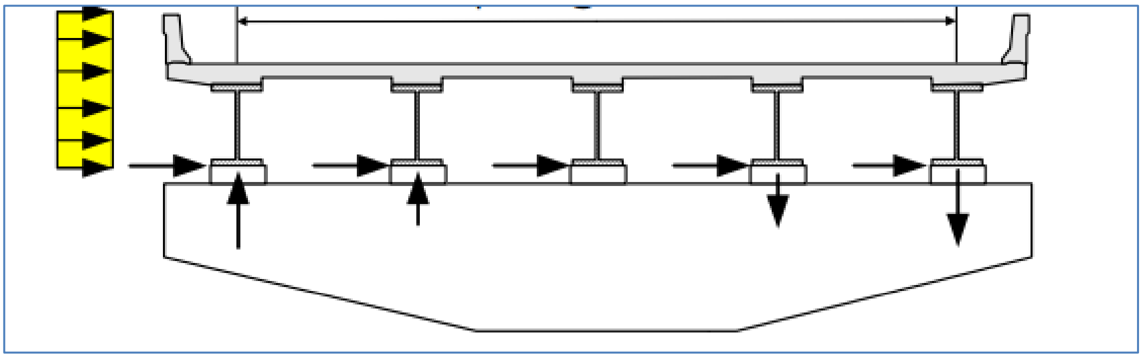


Figure 4. 14 Transverse Wind Loads at Pier Bearings from Wind on Superstructure

$$M_{trns} = WS_{suptrns}\frac{H_{super}}{2} = 738.80\, kN * m$$

Moment of Inertia for the Girder Group:

$$I = \sum Ay^2$$

$$I_{girders} = 2(S^2 + (2S)^2 + (3S)^2 + (4S)^2 + (5S)^2) = 368.38\, m^2$$

$$Reaction = \frac{Moment * y}{I}$$

Using the formula above, the table 4.10 of reactions for each bearing was constructed.

Table 4. 10 Reactions for Each Bearing Due to Transverse Wind Loads

# of Bearing	1	2	3	4	5	6	7	8	9	10	11
Reaction (kN)	18.35	14.68	11.01	7.34	3.67	0.00	-3.67	-7.34	-11.01	-14.68	-18.35

Vertical Wind Load

The vertical (upward) wind load is calculated by multiplying a $0.96\ kN/m^2$vertical wind pressure by the out-to-out bridge deck width. It is applied at the windward quarter-point of the deck only for limit states that do not include wind on live load.

The total vertical wind load is then: $WS_{vert} = 0.96 w_{deck} L_{windT} = 1180.8\ kN$

This load causes a moment about the pier centerline. The value of this moment is:

$$M_{WSvert} = WS_{vert} \frac{w_{deck}}{4} = 6051.6\ kN * m$$

The loads at the bearings are computed as follows: $RWS_{vert} = -\frac{WS_{vert}}{11} + \frac{M_{WSvert} * y}{I_{girders}}$

Table 4. 11 Reactions for Each Bearing Due to Vertical Wind Loads

# of Bearing	1	2	3	4	5	6	7	8	9	10	11
Reaction (kN)	42.97	12.90	-17.16	-47.22	-77.28	-107.35	-137.41	-167.47	-197.53	-227.60	-257.66

Here, a negative value indicates a vertical upward load.

4.3.4.3 Wind Load on Substructure

The Specifications state that the wind loads acting directly on substructure units shall be calculated from a base wind pressure of$P_{sub} = 1.92\ kN/m^2$. These loads are applied simultaneously in the transverse and longitudinal directions of the pier and act simultaneously with the superstructure wind loads

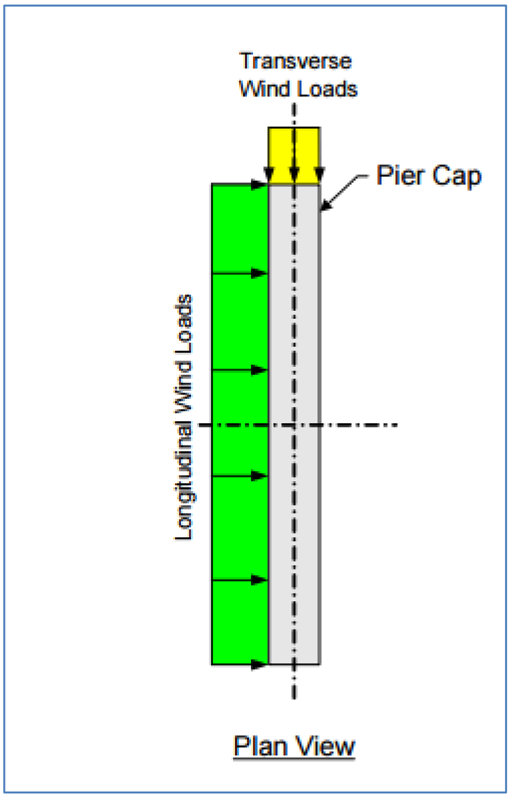


Figure 4. 15 Wind Pressure on Pier

Component areas of the pier cap:

$$A_{capLong} = L_{cap}H_{cap} = 67.65\ m^2$$

$$A_{capTrans} = W_{cap}H_{cap} = 3.96\ m^2$$

Component areas of the pier column:

$$A_{colLong} = L_{col}(H_{col} - D_{soil}) = 27.6\ m^2$$

$$A_{colTrans} = W_{col}(H_{col} - D_{soil}) = 4.8\ m^2$$

The transverse and longitudinal force components are:

$$WS_{subL} = P_{sub}\left(A_{capLong} + A_{colLong}\right) = 182.88\ kN$$

$$WS_{subT} = P_{sub}\left(A_{capTrans} + A_{colTrans}\right) = 16.82\ kN$$

The point of application of these loads will be the centroid of the loaded area of each face, respectively.

$$H_{WSsubL} = \frac{A_{capLong}\left(H_{col} + \frac{H_{cap}}{2}\right) + A_{colLong}(\frac{H_{col} - D_{soil}}{2} + D_{soil})}{A_{capLong} + A_{colLong}} = 5.19\ m$$

$$H_{WSsubT} = \frac{A_{capTrans}\left(H_{col} + \frac{H_{cap}}{2}\right) + A_{colTrans}(\frac{H_{col} - D_{soil}}{2} + D_{soil})}{A_{capTrans} + A_{colTrans}} = 4.25\ m$$

In the AASHTO LRFD design philosophy, the applied loads are factored by statistically calibrated load factors. In addition to these factors, one must be aware of two additional sets of factors which may further modify the applied loads. The first set of additional factors applies to all force effects and are represented by the Greek letter h (eta) in the Specifications, LRFD [1.3.2.1]. These factors are related to the ductility, redundancy, and operational importance of the structure. A single, combined eta is required for every structure. In accordance with FDOT policy, all eta factors are taken equal to one.

4.3.5 Loads Combinations for Substructure Components

4.3.5.1 Pier Cap Force Effects

For pier cap dead load for each bearings, the weight of the cap will be calculated by dividing to sections by midway of spacing.

$$Cap_{DC} = Area_{section} W_{cap} w_c$$

Using formula above, the table 4.12 was constructed.

Table 4. 12 Reactions for Each Bearing Due to Dead Loads

# of Bearing	1	2	3	4	5	6	7	8	9	10	11
Cap_{DC}, kN	100.69	117.66	142.83	169.42	170.84	170.84	170.84	169.42	142.83	117.66	100.69

Calculate the Strength 1 Combined Girder Reactions. Calculation is shown for the only one girder. Similar calculations are performed for the remaining girders.

$$R_u = \gamma_{DCmax}(R_{DC} + Cap_{DC}) + \gamma_{LL} R_{LL}$$

Using formula above, the table 4.13 was constructed.

Table 4. 13 Reactions for Each Bearing Due to Combination of Loads

# of Bearing	1	2	3	4	5	6	7	8	9	10	11
R_u, kN	2618.41	2599.13	2630.59	2663.83	2665.6	2665.6	2665.6	2663.83	2630.59	2599.13	2618.41

4.3.5.2 Pier Column Force Effects

The controlling limit states for the design of the pier column are Strength V (for biaxial bending with axial load). The critical design location is where the column meets the footing, or at the column base. The governing force effects for Strength V are achieved by minimizing the axial effects while maximizing the transverse and longitudinal moments. For Strength V, the factored vertical forces and corresponding moments at the critical section are shown below.

Strength V Axial Force:

$$Ax_{colStrV} = \gamma_{DCminStrV}(2Rext_{DC} + 9Rint_{DC} + DL_{cap} + DL_{col}) + \gamma_{LLStrV} 11R_{LL} - \gamma_{WSSStrV} WS_{vert} = 21910.19\ kN$$

Strength V Transverse moment:

$$MuT_{colStrV} = \gamma_{LLstrV} \sum R_{LL} y + \gamma_{WSSStrV}\left(M_{WSvert} + WS_{suptrns} H_{WStrns} + WS_{subT} H_{WSsubT}\right) = 61800.23\ kN * m$$

Strength V Longitudinal moment:

$$MuL_{colStrV} = \gamma_{BRStrV}(11 * BRK_{brg} H_{BRH} 2) + \gamma_{WSSStrV}\left(WS_{suplong} H_{WSlong} + WS_{subL} H_{WSsubL}\right) = 6587.44\ kN * m$$

For Strength III, the factored transverse shear in the column is:

$$VuT_{col} = \gamma_{WSStrIII}\left(WS_{suptrns} + WS_{subT}\right) = 667.98\ kN$$

For Strength V, the factored longitudinal shear in the column is:

$$VuL_{col} = \gamma_{WSStrV}\left(WS_{suplong} + WS_{subL}\right) + \gamma_{BRSStrV} 11BRK_{brg} = 379.90\ kN$$

4.3.5.3 Pier Pile Force Effects

The foundation system for the pier is a reinforced concrete footing on steel H-piles. The force effects in the piles cannot be determined without a pile layout. The pile layout depends upon the pile capacity and affects the footing design.

Strength 1 Load for Maximum Pile Reaction:

$$Pu_{pile\_Str1} = \gamma_{DCmax}(2Rext_{DC} + 9Rint_{DC} + DL_{cap} + DL_{col} + DL_{ftg}) + \gamma_{LL}11R_{LL} + \gamma_{EVmax}EV_{ftg} = 31779.66\ kN$$

$$MuT_{pile\_Str1} = \gamma_{LL}\sum R_{LL}y = 18998.15\ kN*m$$

$$MuL_{pile\_Str1} = MuL_{colStr1} = 6587.44\ kN*m$$

Minimum Load on Piles Strength V:

$$Pu_{pile\_StrV} = \gamma_{DCminStrV}(2Rext_{DC} + 9Rint_{DC} + DL_{cap} + DL_{col} + DL_{ftg}) + \gamma_{EVminStrV}EV_{ftg} + \gamma_{LLStrV}11R_{LL} - \gamma_{WSSStrV}WS_{vert} = 23104.45\ kN$$

$$MuL_{pile\_StrV} = \gamma_{BRStrV}(11*BRK_{brg}H_{BRH}2) + \gamma_{WSSStrV}\left(WS_{suplong}H_{WSlong} + WS_{subL}H_{WSsubL}\right) = 6587.44\ kN*m$$

$$MuT_{pile\_StrV} = \gamma_{LLstrV}\sum R_{LL}y + \gamma_{WSSStrV}\left(M_{WSvert} + WS_{suptrns}H_{WStrns} + WS_{subT}H_{WSsubT}\right) = 61800.23\ kN*m$$

For Strength III, the factored transverse shear in the footing is equal to the transverse force at the base of the column.

$$HuT_{pileStrIII} = VuT_{col} = 667.98\ kN$$

For Strength V, the factored longitudinal shear in the column is equal to the longitudinal force at the base of the column.

$$HuL_{pileStrV} = VuL_{col} = 379.90\ kN$$

4.3.5.4 Pier Footing Force Effects

The controlling limit states for the design of the pier footing are Strength I (for flexure, punching shear at the column, and punching shear at the maximum loaded pile, and for one-way shear). The footings do not require the crack control by distribution check in LRFD [5.7.3.4]. As a result, the Service I Limit State is not required. There is not a single critical design location in the footing where all of the force effects just mentioned is checked. Rather, the force effects act at different locations in the footing and must be checked at their respective locations. For example, the punching shear checks are carried out using critical perimeters around the column and maximum loaded pile, while the flexure and one-way shear checks are carried out on a vertical face of the footing either parallel or perpendicular to the bridge longitudinal axis. Also note that impact is not included for members that are below ground. The weight of the footing concrete and the soil above the footing are not included in these loads as they counteract the pile reactions.

The resulting Transverse moment applied to the piles is:

$$MuT_{LL\_T} = \gamma_{LL} \sum R_{LL} y = 18998.15\ kN * m$$

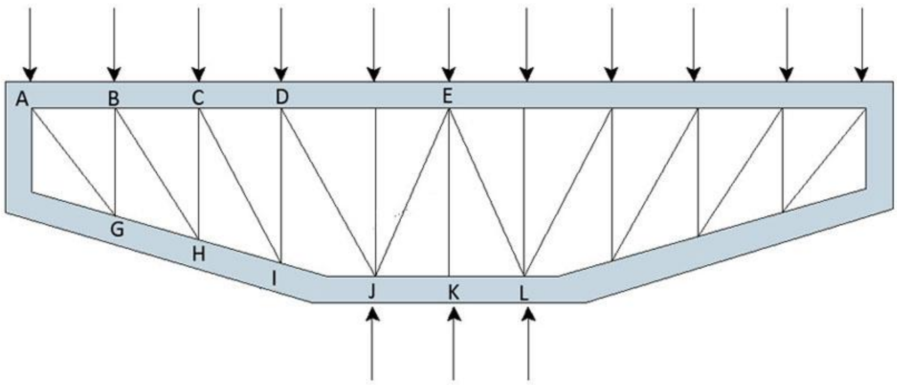
$$MuL_{pile\_StrV} = \gamma_{BR}(11 * BRK_{brg} H_{BRH} 2) = 633.44\ kN * m$$

$$\begin{aligned} Pu_{ftgStr1} &= \gamma_{DCmax}(2Rext_{DC} + 9Rint_{DC} + DL_{cap} + DL_{col}) + \gamma_{LL} 11 R_{LL} \\ &= 30134.74\ kN \end{aligned}$$

4.3.6 Design Pier Cap - Strut and Tie Model (STM)

Prior to carrying out the actual design of the pier cap, a brief discussion is in order regarding the design philosophy that will be used for the design of the structural components of this pier. When a structural member meets the definition of a deep component, the Specifications recommends, although does not mandate, that a strut-and-tie model be used to determine force effects and required reinforcing. LRFD [C5.6.3.1] indicates that a strut-and-tie model properly accounts for nonlinear strain distribution, nonuniform shear distribution, and the mechanical interaction of Vu, Tu and Mu. Use of strut-and-tie models for the design of reinforced concrete members is new to the LRFD Specification.

Loads in pier cups are calculated using strut and tie model, which assumes that loads distribute as trusses inside model. The loads in those "trusses" were calculated using simple joint method.



AG=3357.51 (C)	BC = 5192.12 (T)	IJ = 8376.02 (C)
AB = 2101.62 (T)	HI = 5371.36 (C)	ID = 4867.08 (T)
GH = 2174.04 (C)	CH = 3842.52 (T)	EJ = 6979.45 (C)
BG = 2061.96 (T)	CI = 6473.11 (C)	IL = 9166.77 (C)
BH = 5592.58 (C)	CD = 8375.70 (T)	

Figure 4. 16 Pier Cup Load Calculation

4.3.6.1 Calculate the Tension Tie Reinforcement

For the top reinforcement over the column, the required area of tension tie reinforcement, Ast, in Tie DE (controlling tie) is calculated as follows:

$$Pu_{DE} = 12550.15\ kN$$

$$Ast_{DE} = \frac{Pu_{DE}}{\phi f_y} = 0.03\ m^2$$

Therefore use one row of 9 No.18 bars and one row of 9 No. 11 bars spaced at 0.127 meters for the top reinforcement.

$$As_{No18} = 0.0026\ m^2$$

$$As_{No11} = 0.0010\ m^2$$

$$As_{DE} = 9As_{No18} + 9As_{No11} = 0.032\ m^2\ > 0.03\ m^2\ (OK)$$

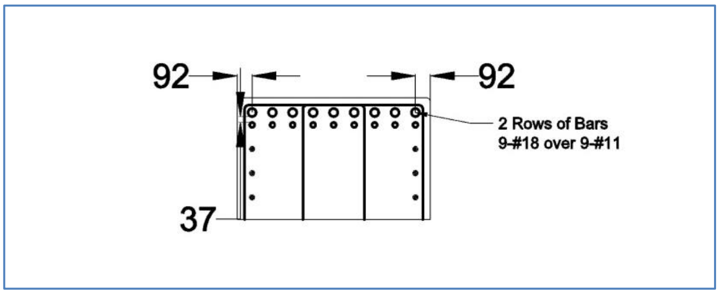


Figure 4. 17 Cap Reinforcement at Tension Tie CD

For the top reinforcement past the first interior girder, the required area of tension tie reinforcement, Ast, in Tie AB is calculated as follows:

$$Pu_{AB} = 2101.62\ kN$$

$$Ast_{AB} = \frac{Pu_{AB}}{\phi f_y} = 0.006\ m^2$$

Therefore use one row of 9 No.8 bars and one row of 5 No. 7 bars spaced at 0.127 meters for the top reinforcement.

$$As_{No18} = 0.0026\ m^2$$

$$As_{No11} = 0.0010\ m^2$$

$$As_{DE} = 4As_{No18} + 9As_{No11} = 0.019\ m^2 \ > 0.006\ m^2\ (OK)$$

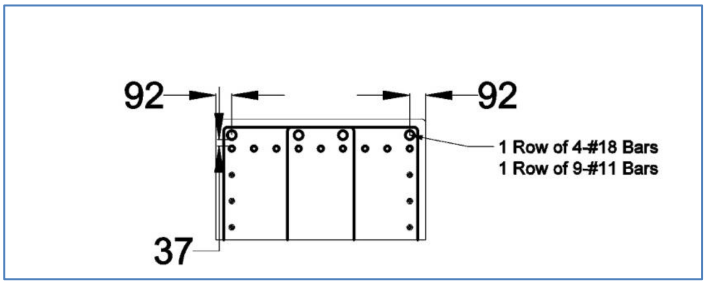


Figure 4. 18 Cap Reinforcement at Tension Tie DE

4.3.6.2 Calculate the Stirrup Reinforcement

The vertical tension ties DI must resist a factored tension of force as shown below. This tension force will be resisted by stirrups with in the specified length of the pier cap. Note that any tension ties located directly over the column do not require stirrup design.

$$Pu_{DI} = 4867.08\ kN$$

$$n = \frac{P_u}{\phi Ast f_y}$$

Try number 18 bars, with four legs.

$$As_{No18} = 0.0026\ m^2$$

$$Ast = 4 * As_{No18} = 0.01\ m^2$$

$$n_{DI} = \frac{Pu_{DI}}{\phi Ast f_y} = 9.86 = 10\ bara$$

The length over which the stirrup shall be distributed is from the face of the column to half way between girders 10 and 11.

$$L_{DI} = 4.5Spacing - \frac{L_{col}}{2} = 1.34\ m$$

$$s_{stirrup} = \frac{L_{DI}}{n_{DI}} = 0.134\ m = 0.13\ m$$

Crack control in disturbed regions:

$$s_{cc} = \frac{Ast}{0.03W_{cap}} = 0.2\ m$$

$$s_{stir} = \min(s_{stirrup}, s_{cc}) = 0.13\ m$$

$$As_{DI} = \frac{L_{DI}Ast}{s_{stir}} = 0.10\ m^2$$

Therefore use No. 18 double-legged stirrups at 0.13 meters spacing in the pier cap.

4.3.6.3 Compression Strut Capacity

After the tension tie reinforcement has been designed, the next step is to check the capacity of the compressive struts in the pier cap versus the limiting compressive stress. Strut JL carries the highest bottom compressive force.

$$Pu_{JL} = 9166.77\ kN$$

The limiting compressive stress, f_{cu}, in the strut can also be computed LRFD [5.6.3.3.3]:

$$f_{cu} = 0.85f_c' = 29.55\ MPa$$

The nominal resistance of Strut IJ is computed based on the limiting stress, f_{cu}, and the strut dimensions. The centroid of the strut was assumed to be at $centroid_{bot}$= 0.11 meters vertically from the bottom face. Therefore, the thickness of the strut perpendicular to the sloping bottom face is:

$$t_{JL} = 2centroid_{bot} = 0.22\ m$$

$$Acs_{JL} = t_{JL}W_{cap} = 0.464\ m^2$$

$$Pr_{JL} = 0.7f_{cu}Acs_{JL} = 9597.84\ kN$$

$$Pr_{JL} = 9597.84\ kN \geq Pu_{JL} = 9166.77\ kN\ (OK)$$

4.3.6.4 Crack Control Reinforcement

In the disturbed regions, the minimum ratio of reinforcement to the gross concrete area is 0.003 in each direction, and the spacing of the bars in these grids must not exceed 0.3 meters, LRFD [5.6.3.6]. Therefore the required crack control reinforcement by assuming 0.15 meters vertical spacing:

$$As_{crack} = 0.003 * 0.15 * W_{cap} = 540\ mm^2$$

Use 4 - No. 7 horizontal bars at 0.15 meters spacing in the vertical direction

$$As_{No7} = 388\ mm^2$$

$$2As_{No7} = 776\ mm^2 \geq As_{crack} = 540\ mm^2 (OK)$$

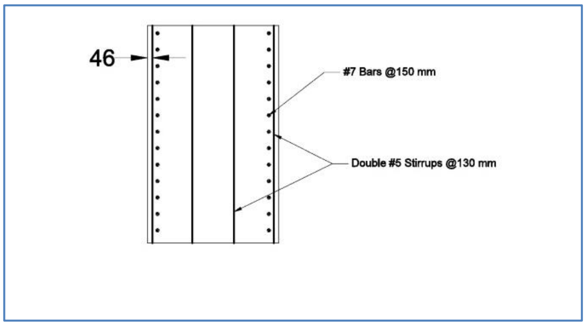


Figure 4. 19 Crack Control Reinforcement

Summary of Cap Reinforcement

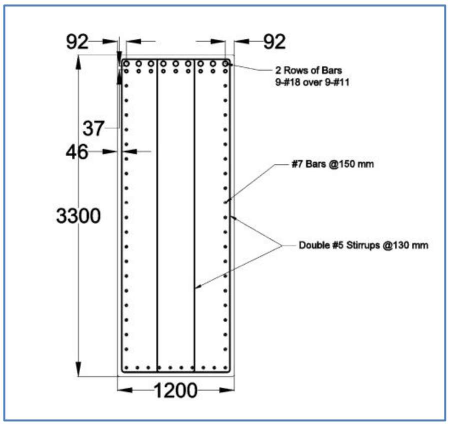


Figure 4. 20 Pier Cap Design Summary

4.3.7 Design Pier Column

A preliminary estimate of the required section size and reinforcement is shown.

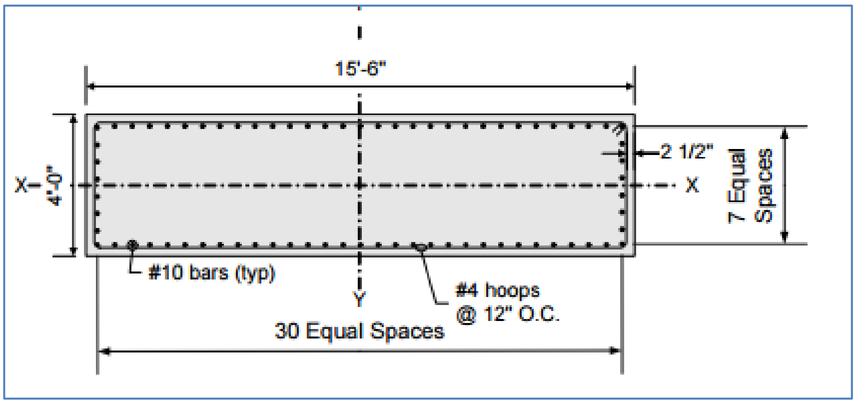


Figure 4. 21 Preliminary Pier Column Design

4.3.7.1 Design for Axial Load and Biaxial Bending (Strength V)

The preliminary column reinforcing is show in figure 4.21 and corresponds to #18 bars equally spaced around the column perimeter. LRFD [5.7.4.2] prescribes limits (both maximum and minimum) on the amount of reinforcing steel in a column. These checks are performed on the preliminary column as follows: Number of bars = 104

$$As_{No18} = 0.0026\ m^2$$

$$As_{col} = As_{No18} * Num_{bars} = 0.27m^2$$

$$A_{gcol} = W_{col}L_{col} = 8.28m^2$$

$$\frac{As_{col}}{A_{gcol}} = 0.03 < 0.08 \text{ (max} \, reinf.check) \, OK$$

$$\frac{As_{col}}{A_{gcol}} = 0.03 > \frac{0.135 f_c}{f_y} = 0.008 \text{ (min} \, reinf.check) \, OK$$

The assessment of the resistance of a compression member with biaxial flexure for strength limit states is dependent upon the magnitude of the factored axial load. This value determines which of two equations provided by the Specification are used. If the factored axial load is less than fifteen percent of the gross concrete strength multiplied by the phi-factor for compression members ($\phi_{\text{axial}} = 0.75$), then the Specifications require that a linear interaction equation for only the moments is satisfied (LRFD [Equation 5.7.4.5-3]). Otherwise, an axial load resistance (Prxy) is computed based on the reciprocal load method (LRFD [Equation 5.7.4.5-1]).

$$Ax_{colStrV} = 21910.19 \, kN < 0.15 \phi_{\text{axial}} f_c A_{gcol} = 22477.10 \, kN \ \ (OK)$$

4.3.7.2 Design for Shear (Strength III and Strength V)

These maximum shear forces do not act concurrently. Although a factored longitudinal shear force is present in Strength III and a factored transverse shear force is present in Strength V, they both are small relative to their concurrent factored shear. Therefore, separate shear designs can be carried out for the longitudinal and transverse directions using only the maximum shear force in that direction.

For the pier column, the maximum factored shear in either direction is less than one-half of the factored resistance of the concrete. Therefore, shear reinforcement is not required. This is demonstrated for the longitudinal direction as follows:

$$\mathrm{b_v = L_{col} = 6.9 \ m} \qquad\qquad \mathrm{h = W_{col} = 1.2m}$$

Conservatively, dv may be calculated as shown below, LRFD [5.8.2.9].

$$\mathrm{d_v = 0.72h = 0.864 \ m} \qquad \beta = 2.0 \qquad \theta = 45 \ \mathrm{deg}$$

The nominal concrete shear strength is:

$$\mathrm{V_c = 0.0316 \beta \sqrt{f_c} b_v d_v = 4937.53 \ kN}$$

The factored shear resistance is: $\phi_v = 0.9$

$$V_r = \phi_v V_c = 4443.78\ \mathrm{kN}$$

$$\frac{V_r}{2} = 2221.89\ \mathrm{kN} > VuL_{col} = 379.90\ \mathrm{kN}\ (OK)$$

It has just been demonstrated that transverse steel is not required to resist the applied factored shear forces. However, transverse confinement steel in the form of hoops, ties or spirals is required for compression members. In general, the transverse steel requirements for shear and confinement must both be satisfied per the Specifications.

4.3.7.3 Transfer of Force at Base of Column

The provisions for the transfer of forces and moments from the column to the footing are new to the AASHTO LRFD Specifications. In general, standard engineering practice for bridge piers automatically satisfies most, if not all, of these requirements.

In this design example, and consistent with standard engineering practice, all steel reinforcing bars in the column extend into, and are developed, in the footing.

This automatically satisfies the following requirements for reinforcement across the interface of the column and footing: A minimum reinforcement area of 0.5 percent of the gross area of the supported member, a minimum of four bars, and any tensile force must be resisted by the reinforcement. Additionally, with all of the column reinforcement extended into the footing, along with the fact that the column and footing have the same compressive strength, a bearing check at the base of the column and the top of the footing is not applicable.

In addition to the above, the Specifications require that the transfer of lateral forces from the pier to the footing be in accordance with the shear-transfer provisions of LRFD [5.8.4]. With the standard detailing practices for bridge piers previously mentioned (i.e., all column reinforcement extended and developed in the footing), along with identical design compressive strengths for the column and footing, this requirement is generally satisfied. However, for the sake of completeness, this check will be carried out as follows:

Area of concrete engaged in shear transfer: $A_{cv} = A_{gcol} = 8.28\mathrm{m}^2$

Area of shear reinforcement crossing the shear plane: $A_{vf} = As_{col} = 0.27\mathrm{m}^2$

For concrete placed against a clean concrete surface, not intentionally roughened, the following values are obtained from LRFD [5.8.4.3].

Cohesion factor: $c_{cv} = 0.52\ MPa$

Friction factor: $\mu = 0.60 \qquad K_1 = 0.2 \qquad K_2 = 0.8$

The nominal shear-friction capacity is the smallest of the following three equations:

$$V_{nsf1} = c_{cv}A_{cv} + \mu A_{vf}f_y = 71297.38\ kN$$

$$V_{nsf2} = K_1 f_c A_{cv} = 39959.28\ kN$$

$$V_{nsf} = K_2 A_{cv} = 10267.2\ kN$$

Define the nominal shear-friction capacity as follows:

$$V_{nsf} = \min\left(V_{nsf1}, V_{nsf2}, V_{nsf3}\right) = 10267.2\ kN$$

The factored shear resistance is: $\phi_v = 0.9$

$$\phi_v V_c = 9240.48\ kN > VuL_{col} = 379.90\ kN\ (OK)$$

This check has a high rate of variation, because the column is usually over designed. If the reinforcement is extended into the footing, the horizontal forces can be eliminated. Since bridge dead load generates shear-key, the shear-friction capacity of the column or footing increases.

4.3.8 Design Pier Piles

Minimum Sizes [SDG 3.5.1]: use 18" (0.46 m) square piling, except for extremely aggressive salt water environments.

Resistance Factors [SDG 3.5.6]: $\phi_{pile} = 0.85$

Pile Driving Resistance [SDG 3.5.12]. The Required Driving Resistance for an 18" (0.46 m) square concrete pile must not exceed [SDG 3.5.12-1]: $R_{n.FDOT.max} = 300\ tons$

4.3.8.1 Foundation Layout

Size of the square concrete piles: $\mathrm{Pile_{size}} = 0.46\ \mathrm{m}$

Number of piles: $n_{piles} = 20$

Foundation layout:

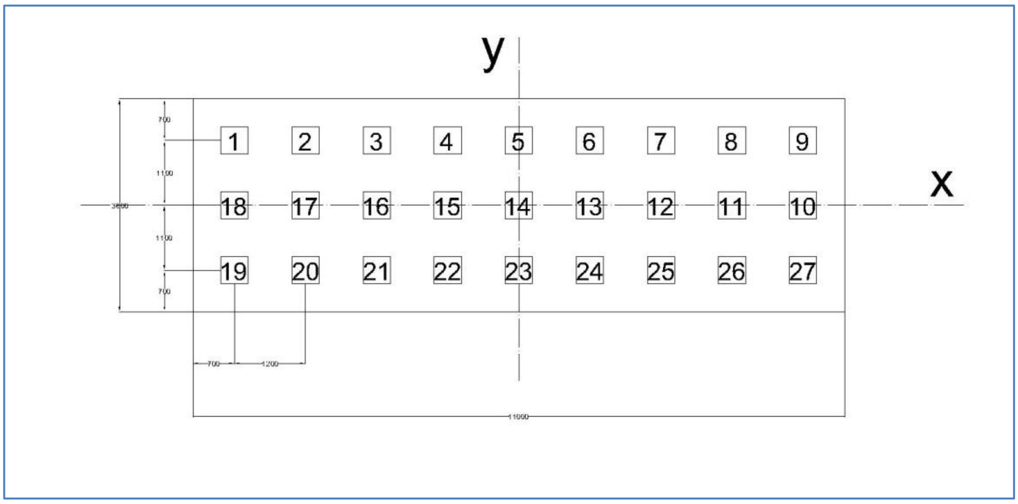


Figure 4. 22 Foundation layout

Axial Load on Pile: $Q_u = \frac{Pu_{pile.StrV}+1.25DL_{ftg}}{n_{pile}} + \frac{MuL_{pileStrV}*Y_{pile}}{\sum_{y}^{n_{pile}} Y^2_{pile_y}} + \frac{MuT_{pileStrV}*X_{pile}}{\sum_{x=1}^{n_{pile}} X^2_{pile_x}}$

Table 4. 14 The coordinates and calculated axial load of each piles

.	X	Y	Q (kN)
1	-4,8	1,1	396,61
2	-3,6	1,1	604,95
3	-2,4	1,1	813,28
4	-1,2	1,1	1021,61
5	0,0	1,1	1229,95
6	1,2	1,1	1438,28
7	2,4	1,1	1646,61
8	3,6	1,1	1854,95
9	4,8	1,1	2063,28
10	4,8	0,0	1806,34
11	3,6	0,0	1598,01
12	2,4	0,0	1389,67
13	1,2	0,0	1181,34
14	0,0	0,0	973,01
15	-1,2	0,0	764,67
16	-2,4	0,0	556,34
17	-3,6	0,0	348,01
18	-4,8	0,0	139,67
19	-4,8	-1,1	91,06
20	-3,6	-1,1	117,27
21	-2,4	-1,1	299,40
22	-1,2	-1,1	507,73
23	0,0	-1,1	716,06
24	1,2	-1,1	924,40
25	2,4	-1,1	1132,73
26	3,6	-1,1	1341,06
27	4,8	-1,1	1549,40

4.3.8.2 Pile Length Calculation

Maximum axial load on pile: $Q_{max} = \max(Q_u) = 2016.28\ kN$

Minimum axial load on pile: $Q_{min} = \min(Q_u) = 91.06\ kN$

Converting these values to tons equivalent: $Ru_{max} = \frac{Q_{max}\ (in\ kips)}{2} = 227\ tons$

Required driving resistance: $R_n = \frac{Ru_{max}}{\phi_{pile}} = 267\ tons < R_{n.FDOT.max} = 300\ tons\ (OK)$

The FB-Deep Program, Version 2.02, was utilized to determine the pile capacity. Using boring data, the program can analyze concrete piles, H-piles, pipe piles, and drilled shafts. It is available from BSI.

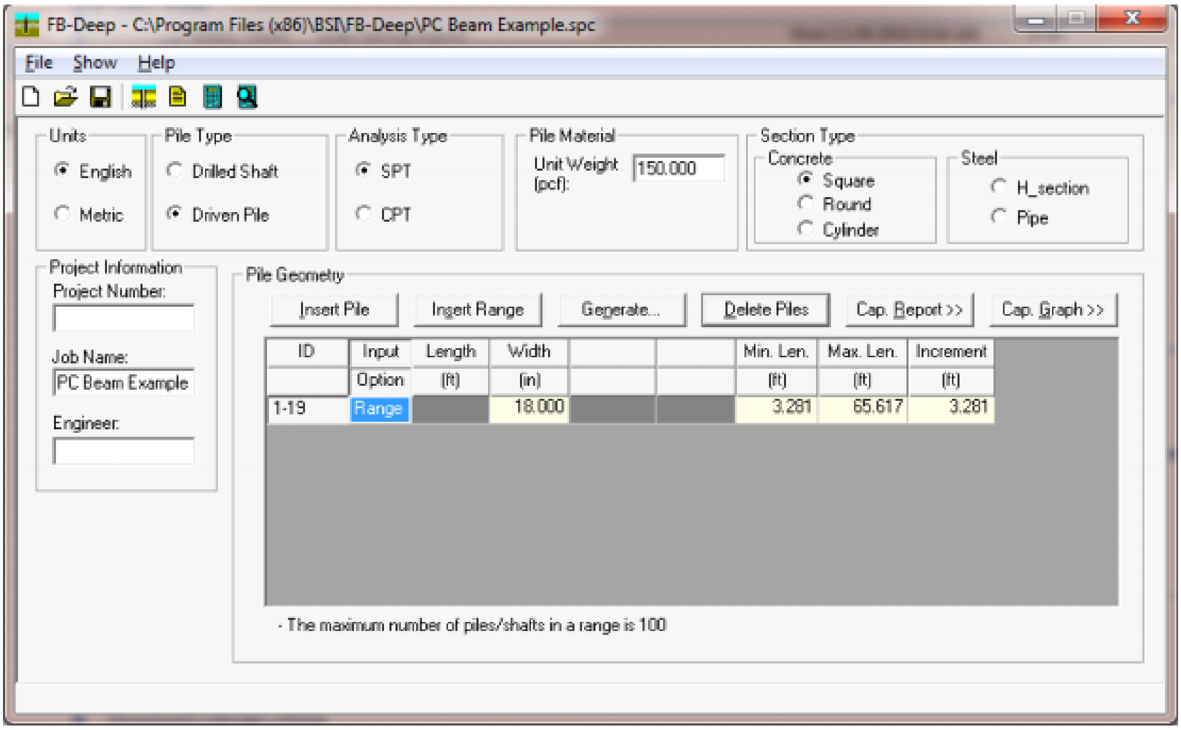
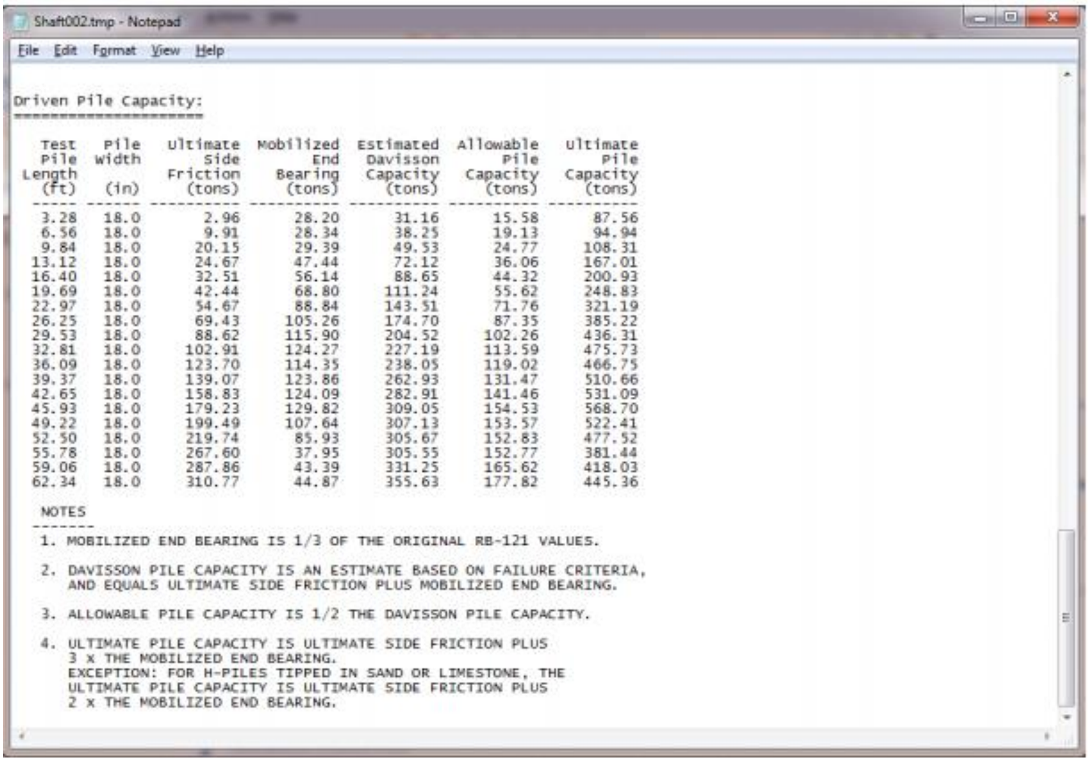


Figure 4. 23Simulation on the computer



Shaft002.tmp - Notepad

File Edit Format View Help

Driven Pile Capacity:

Test Pile Length (ft)	Pile Width (in)	Ultimate Side Friction (tons)	Mobilized End Bearing (tons)	Estimated Davisson Capacity (tons)	Allowable Pile Capacity (tons)	Ultimate Pile Capacity (tons)
3.28	18.0	2.96	28.20	31.16	15.58	87.56
6.56	18.0	9.91	28.34	38.25	19.13	94.94
9.84	18.0	20.15	29.39	49.53	24.77	108.31
13.12	18.0	24.67	47.44	72.12	36.06	167.01
16.40	18.0	32.51	56.14	88.65	44.32	200.93
19.69	18.0	42.44	68.80	111.24	55.62	248.83
22.97	18.0	54.67	88.84	143.51	71.76	321.19
26.25	18.0	69.43	105.26	174.70	87.35	385.22
29.53	18.0	88.62	115.90	204.52	102.26	436.31
32.81	18.0	102.91	124.27	227.19	113.59	475.73
36.09	18.0	123.70	114.35	238.05	119.02	466.75
39.37	18.0	139.07	123.86	262.93	131.47	510.66
42.65	18.0	158.83	124.09	282.91	141.46	531.09
45.93	18.0	179.23	129.82	309.05	154.53	568.70
49.22	18.0	199.49	107.64	307.13	153.57	522.41
52.50	18.0	219.74	85.93	305.67	152.83	477.52
55.78	18.0	267.60	37.95	305.55	152.77	381.44
59.06	18.0	287.86	43.39	331.25	165.62	418.03
62.34	18.0	310.77	44.87	355.63	177.82	445.36

NOTES

1. MOBILIZED END BEARING IS 1/3 OF THE ORIGINAL RB-121 VALUES.
2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA, AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.
3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.
4. ULTIMATE PILE CAPACITY IS ULTIMATE SIDE FRICTION PLUS 3 x THE MOBILIZED END BEARING. EXCEPTION: FOR H-PILES TIPPED IN SAND OR LIMESTONE, THE ULTIMATE PILE CAPACITY IS ULTIMATE SIDE FRICTION PLUS 2 x THE MOBILIZED END BEARING.

Figure 4. 24 Pile Length Calculation

$$pile_{length} = (R_n - 262.93) * \frac{42.65\,ft - 39.37ft}{282.91 - 262.93} + 39.37\,ft = 40.04\,ft = 12.20\,m$$

4.3.9 Pier Footing Design

Distance from centerline of piles to edge of footing: $pile_{edge} = 0.7\ m$

Distance from x-critical section (face of column) to edge of footing along the x-axis:

$$x_{edge} = \frac{L_{ftg} - L_{col}}{2} = 2.05\ m$$

Distance from x-critical section to centerline of piles along the x-axis:

$$x_{crit} = x_{edge} - pile_{edge} = 1.35\ m$$

Distance from y-critical section (face column) to edge of footing along the y-axis:

$$y_{edge} = \frac{W_{ftg} - W_{col}}{2} = 1.2\ m$$

Distance from y-critical section to centerline of piles along the y-axis:

$$y_{crit} = y_{edge} - pile_{edge} = 0.5\ m$$

4.3.9.1 Moments - X Critical Section

Pile loads contributing to longitudinal moment:

$$P = \max(Q_1 + Q_{18} + Q_{19}, Q_9 + Q_{10} + Q_{27}) = 5419.03\ kN$$

Moments at critical section due to pile loads: $My_{pile} = Px_{crit} = 7315.69\ kN * m$

Moment at critical section due to footing weight: $My_{ftg} = \left(L_{ftg}h_{ftg}w_c\right)\frac{x_{edge}^2}{2} = 544.79\ kN * m$

Moments at critical section due to soil: $My_{soil} = \left(L_{ftg}D_{soil}\gamma_{soil}\right)\frac{x_{edge}^2}{2} = 23.77\ kN * m$

4.3.9.2 Moments - Y Critical Section

Pile loads contributing to transverse moment:

$$P = \max(Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6 + Q_7 + Q_8 + Q_9, Q_{19} + Q_{20} + Q_{21} + Q_{22} + Q_{23} + Q_{24} + Q_{25} + Q_{26} + 27) = 11069.53\ kN$$

Moments at critical section due to pile loads: $Mx_{pile} = Py_{crit} = 5534.77\ kN * m$

Moment at critical section due to footing weight: $Mx_{ftg} = (W_{ftg}h_{ftg}w_c)\frac{y_{edge}^2}{2} = 61.09\ kN * m$

Moments at critical section due to soil: $Mx_{soil} = (W_{ftg}D_{soil}\gamma_{soil})\frac{y_{edge}^2}{2} = 8.14\ kN * m$

4.3.9.3 Design Moments

Transverse Footing Design (Mx moments) - Strength I

$$Mx_{Strength} = Mx_{pile} - 0.9Mx_{ftg} - 0.9Mx_{soil} = 5471.69\ kN * m$$

Transverse Footing Design (Mx moments) - Service I

$$Mx_{Service} = Mx_{pile} - Mx_{ftg} - Mx_{soil} = 5464.77\ kN * m$$

Longitudinal Footing Design (My moments) - Strength I

$$My_{Strength} = My_{pile} - 0.9My_{ftg} - 0.9My_{soil} = 6803.30\ kN * m$$

Longitudinal Footing Design (My moments) - Service I

$$My_{Service} = My_{pile} - My_{ftg} - My_{soil} = 6746.44\ kN * m$$

4.3.9.4 Transverse Flexural Design [LRFD 5.7.3.2]

The design procedure consists of calculating the reinforcement required to satisfy the design moment. The procedure is the same for both the transverse and longitudinal moment designs.

Factored resistance: $M_r = Mx_{Strength} = 5471.69\ kN * m$

Initial assumption for area of steel required

Size of bar: ybar=”9”

Bar diameter: $ybar_{dia} = 0.029\ m$

Bar area: $A_{bar} = 645.16\ mm^2$

Distance from extreme compressive fiber to centroid of reinforcing steel:

$$d_s = h_{ftg} - cover_{fb} - \frac{ybar_{dia}}{2} = 0.83\ m$$

To find required area of reinforcement, the following quadratic equation should be solved:

$$M_r = \phi A_s f_y \left(d_s - \frac{1}{2} \left(\frac{A_s f_y}{0.85 f_c L_{ftg}} \right) \right)$$

$$A_s = 0.009\ m^2$$

Use $A_s = 0.01\ m^2$, therefore number of bars: $n_{bar} = \frac{A_s}{A_{bar}} = 15.5 = 16\ bars$

Area of steel provided: $A_s = n_{bar} A_{bar} = 0.01\ m^2$

Moment capacity provided:

$$M_{r.tran} = \phi A_s f_y \left(d_s - \frac{1}{2} \left(\frac{A_s f_y}{0.85 f_c L_{ftg}} \right) \right) = 6095.66\ kN * m$$

Limits for Reinforcement [LRFD 5.7.3.3]

Minimum Reinforcement

The minimum reinforcement requirements ensure the factored moment capacity provided is at least equal to the lesser of the cracking moment and 1.33 times the factored moment required by the applicable strength load combinations.

Modulus of Rupture: $f_r = -0.24\sqrt{f_c} = -3.10 MPa$ [SDG 1.4.1.B]

Section modulus of the footing above the piles: $S = \frac{L_{ftg} h_{ftg}^2}{6} = 1.83\ m^2$

Flexural cracking variability factor (1.2 for precast segmental structures, 1.6 otherwise):

$$\gamma_1 = 1.6$$

Ratio of specified minimum yield strength to ultimate tensile strength of reinforcement (0.67 for A615, Grade 60 reinforcement by SDG 1.4.1 C): $\gamma_3 = 0.67$

Cracking moment:

$$M_{cr} = f_r S \gamma_1 \gamma_3 = 6051.1\ kN * m$$

Check that the capacity provided, $M_{r.tran} = 6095.66\ kN * m$, exceeds minimum requirements, $M_{cr} = 6051.1\ kN * m$. (OK)

Transverse Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions increases the longevity of the structure. Potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

The maximum spacing of the mild steel reinforcement for control of cracking at the service limit state shall satisfy: $s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$, where $\beta_s = 1 + \frac{d_c}{0.7(h_{ftg} - d_c)}$

Exposure factor for Class 1 exposure condition [SDG 3.10]: $\gamma_e = 1.00$

Distance from extreme tension fiber to center of closest bar:

$$d_c = cover_{fb} + \frac{ybar_{dia}}{2} = 0.17\ meters$$

$$\beta_s = 1 + \frac{d_c}{0.7(h_{ftg} - d_c)} = 1.29$$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. It can be calculated from following quadratic equation:

$\frac{1}{2} L_{ftg} x^2 = \frac{E_s}{E_{c.sub}} A_s\ (d_s - x)$

$$x = x_{pos} = 253.44\ mm$$

Tensile force in the reinforcing steel due to service limits state moment:

$$T_s = \frac{Mx_{Service}}{d_s - \frac{x_{pos}}{3}} = 2675.59\ kN$$

Actual stress in the reinforcing steel due to service limits state moment:

$$f_{s.actual} = \frac{T_s}{A_s} = 267.56\ MPa$$

Required reinforcement spacing: $s_{required} = \frac{700\gamma_e}{\beta_s f_{s.actual}} - 2d_c = 0.54\,meters$

Take provided reinforcement spacing: $spacing_{pos} = 0.6\,meters$

4.3.9.5 Longitudinal Flexural Design [LRFD 5.7.3.2]

The design procedure consists of calculating the reinforcement required to satisfy the design moment.

Factored resistance: $M_r = My_{Strength} = 6803.30\,kN * m$

Initial assumption for area of steel required

Size of bar: xbar=”18”

Bar diameter: $xbar_{dia} = 0.058\,m$

Bar area: $A_{bar} = 2580.646\,mm^2$

Distance from extreme compressive fiber to centroid of reinforcing steel:

$$d_s = h_{ftg} - cover_{fb} - \frac{ybar_{dia}}{2} = 0.81\,m$$

To find required area of reinforcement, the following quadratic equation should be solved:

$$M_r = \phi A_s f_y \left(d_s - \frac{1}{2}\left(\frac{A_s f_y}{0.85 f_c W_{ftg}} \right) \right)$$

$$A_s = 0.01\,m^2$$

Use $A_s = 0.01\,m^2$, therefore number of bars: $n_{bar} = \frac{A_s}{A_{bar}} = 3.9 = 4\,bars$

Area of steel provided: $A_s = n_{bar} A_{bar} = 0.01\,m^2$

Moment capacity provided:

$$M_{r.tran} = \phi A_s f_y \left(d_s - \frac{1}{2}\left(\frac{A_s f_y}{0.85 f_c L_{ftg}} \right) \right) = 6894.46\,kN * m$$

Limits for Reinforcement [LRFD 5.7.3.3]

Minimum Reinforcement

The minimum reinforcement requirements ensure the factored moment capacity provided is at least equal to the lesser of the cracking moment and 1.33 times the factored moment required by the applicable strength load combinations.

Modulus of Rupture: $f_r = -0.24\sqrt{f_c} = -3.10MPa$ [SDG 1.4.1.B]

Section modulus of the footing above the piles: $S = \frac{W_{ftg} h_{ftg}^2}{6} = 0.6\ m^2$

Flexural cracking variability factor (1.2 for precast segmental structures, 1.6 otherwise):

$$\gamma_1 = 1.6$$

Ratio of specified minimum yield strength to ultimate tensile strength of reinforcement (0.67 for A615, Grade 60 reinforcement by SDG 1.4.1 C): $\gamma_3 = 0.67$

Cracking moment:

$$M_{cr} = f_r S \gamma_1 \gamma_3 = 1993.92\ kN * m$$

Check that the capacity provided,$M_{r.tran} = 6095.66\ kN * m$, exceeds minimum requirements,$M_{cr} = 1993.92\ kN * m$. (OK)

Transverse Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions increases the longevity of the structure. Potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

The maximum spacing of the mild steel reinforcement for control of cracking at the service limit state shall satisfy: $s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$, where $\beta_s = 1 + \frac{d_c}{0.7(h_{ftg} - d_c)}$

Exposure factor for Class 1 exposure condition [SDG 3.10]: $\gamma_e = 1.00$

Distance from extreme tension fiber to center of closest bar:

$$d_c = cover_{fb} + \frac{ybar_{dia}}{2} = 0.18\, meters$$

$$\beta_s = 1 + \frac{d_c}{0.7(h_{ftg} - d_c)} = 1.28$$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. It can be calculated from following quadratic equation:

$\frac{1}{2} W_{ftg} x^2 = \frac{E_s}{E_{c.sub}} A_s\, (d_s - x)$

$$x = x_{pos} = 126.44\, mm$$

Tensile force in the reinforcing steel due to service limits state moment:

$$T_s = \frac{My_{Service}}{d_s - \frac{x_{pos}}{3}} = 1855.59\, kN$$

Actual stress in the reinforcing steel due to service limits state moment:

$$f_{s.actual} = \frac{T_s}{A_s} = 185.56\, MPa$$

Required reinforcement spacing: $s_{required} = \frac{700\gamma_e}{\beta_s f_{s.actual}} - 2d_c = 0.67\, meters$

Take provided reinforcement spacing: $spacing_{pos} = 0.7\, meters$

4.3.9.6 Shear Design Parameters [LRFD 5.13.3.6]

Distance from extreme compression fiber to centroid of tension steel:

$$d_e = h_{ftg} - cover_{fb} - ybar_{dia} = 0.79\, m$$

Effective shear depth [LRFD 5.8.2.9]: $d_v = \max(0.9d_e, 0.72h_{ftg}) = 0.72\, m$

Shear Resistance

According to LRFD 5.8.3.4.1, values of $\beta = 2.0$ and θ = 45 deg may be assumed for concrete footings in which the distance from point of zero shear to the face of the column, pier or wall is less than 3dv ($y_{crit} < 3d_v$).

Shear - Y Critical Section

Factored pile loads contributing to transverse shear:

$$\mathrm{Vu_T} = \max(\mathrm{Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6 + Q_7 + Q_8 + Q_9, Q_{19} + Q_{20} + Q_{21} + Q_{22} + Q_{23} + Q_{24} + Q_{25} + Q_{26} + 27}) = 11069.53\ \mathrm{kN}$$

Distance between face of column and face of pile:

$$\mathrm{dy_{face}} = \frac{\mathrm{W_{ftg} - W_{col}}}{2} - \left(\frac{\mathrm{Pile_{size}}}{2} + \mathrm{pile_{edge}}\right) = 0.27\mathrm{m}$$

The location of the piles relative to the critical shear plane determines the amount of shear design. According to LRFD 5.13.3.6.1, if a portion of the pile lies inside the critical section, the pile load shall be uniformly distributed over the pile width, and the portion of the load outside the critical section shall be included in shear calculations for the critical section.

$$\mathrm{Transverse_{shear}} = \begin{cases} \text{Full shear, piles are outside of the y} - \text{critical shear plane, if } \mathrm{dy_{face}} \geq \mathrm{d_v} \\ \text{No shear, Y} - \text{critical shear plane is outside pile dimension, if } \mathrm{d_v} \geq \mathrm{dy_{face}} + \mathrm{Pile_{size}} \\ \text{Partial shear, piles intersect y} - \text{critical shear plane, if } \mathrm{dy_{face}} < \mathrm{d_V} \text{ and } \mathrm{d_v} < \mathrm{dy_{face}} + \mathrm{Pile_{size}} \end{cases}$$

$$\mathrm{Transverse_{shear}} = \text{Partial shear, piles intersect y} - \text{critical shear plane}$$

$$\psi_\mathrm{y} = \begin{cases} 1 \text{ if Full shear} \\ 0 \text{ if No shear} \\ \left|1 - \frac{\mathrm{d_v - dy_{face}}}{\mathrm{Pile_{size}}}\right| \end{cases} = 0.02$$

Factored shear along transverse y-critical section: $\mathrm{Vu_T} = \psi_\mathrm{y}\mathrm{Vu_T} = 221.39\ \mathrm{kN}$

The nominal shear resistance for footings: $\mathrm{V_n} = 0.0316\beta\sqrt{\mathrm{f_c}}\mathrm{L_{ftg}d_v} = 8451.62\ \mathrm{kN}$

Check the section has adequate shear capacity: $\mathrm{Vu_T} < \mathrm{V_n}$ (OK)

Shear - X Critical Section

Factored pile loads contributing to longitudinal shear:

$$Vu_L = \max(Q_1 + Q_{18} + Q_{19}, Q_9 + Q_{10} + Q_{27}) = 5419.03\ kN$$

Distance between face of column and face of pile:

$$dx_{face} = \frac{L_{ftg} - L_{col}}{2} - (\frac{Pile_{size}}{2} + pile_{edge}) = 1.12m$$

As $dx_{face} > d_v$, piles are outside of the y-critical shear plane. Therefore, $\psi_y = 1$.

Factored shear along transverse x-critical section: $Vu_L = \psi_y Vu_T = 5419.03\ kN$

The nominal shear resistance for footings: $V_n = 0.0316\beta\sqrt{f_c}W_{ftg}d_v = 6585.99\ kN$

Check the section has adequate shear capacity: $Vu_L < V_n\ (OK)$

CHAPTER 5: GEOTECHNICAL ANALYSIS AND DESIGN

5.1 Mechanically Stabilized Wall

5.1.1 Field exploration

To explore the project site 7 boreholes were drilled at different places. The depth of boreholes logs is 21.3m. Five of them were drilled at bridge location, namely 1-W-1, 1-W-2, 1-W-3, 1-W-4, 1-W-5, and the other two, 2-W-1 and 2-W-2 at the places where sheet pile walls to be installed.

5.1.2 Boring Data

The full schematic data of bridge borehole logs are illustrated in the appendix E. The logs include the type of soil, its consistency, and classification based on the Unified Soil Classification System. The boring data of borehole logs is used to define subsurface profile for Bridge, MSE walls, and Sheet pile walls design. Table 5.1 illustrates subsurface profile for bridge location based on the data obtained from borehole logs 1-W-1, 1-W-2, 1-W-3, 1-W-4, 1-W-5.

Table 5. 1 Subsurface profile at bridge and MSE walls location.

Depth (m)	Soil Type	Soil Description
0 - 0.3	SM-OL	Dark brown organic silty sand
0.3 - 1.2	GM	Dense light brown fine sand with some limestone lenses
1.2 - 4	SP	Loose light brown fine sand with traces of limestone lenses
4 - 4.9	SM	Very loose to loose brown sand
4.9 - 9.1	SP	Medium dense to loose light brown sand
9.1 - 14.3		Light brown porous sandy limestone and calcareous fine sand
14.3 - 21		Light gray porous sandstone and some fine sand

Table 5. 2Summary soil-rock parameters for 1-W-1, 1-W-2, 1-W-3, 1-W-4, 1-W-5

Range of Elevation, m.		**Material Type**	**ø (degrees)**	**Cohesion, C (MPa)**	**Young's Modules, E (MPa)**	**Unsaturated unit weight γ_{unsat} (kN/m3)**	**Saturated Unit weight, γ_{sat} (kN/m3)**
From	To						
-0.3	-9.1	Sand	30	0	19	18	20
-9.1	-14.3	Limestone	38	20	25	25	27
-14.3	-21	Sandstone	40	6.7	20	22	24

5.1.3 MSE wall design

According to the proposed design MSE wall system will be used in 5 places as it can be seen from figure 5.3. Two of them will serve as approach embankment from either side of the bridges, two will serve as a base for elevated roadways between bridges and one will be used to extend the existing road width. In total there are 15 MSE walls that will serve as an abutment for bridges as well as support for pavement and traffic loads. Walls are numerated from 1 to 15 as shown in figure 5.2 Approach embankments on both sides are the same size and have similar ground conditions and elevation. Therefore, MSE wall 1, 2 and 3 designs can be applied to MSE wall 12, 13, 14 as well. MSE walls will be designed using LRFD methodology.

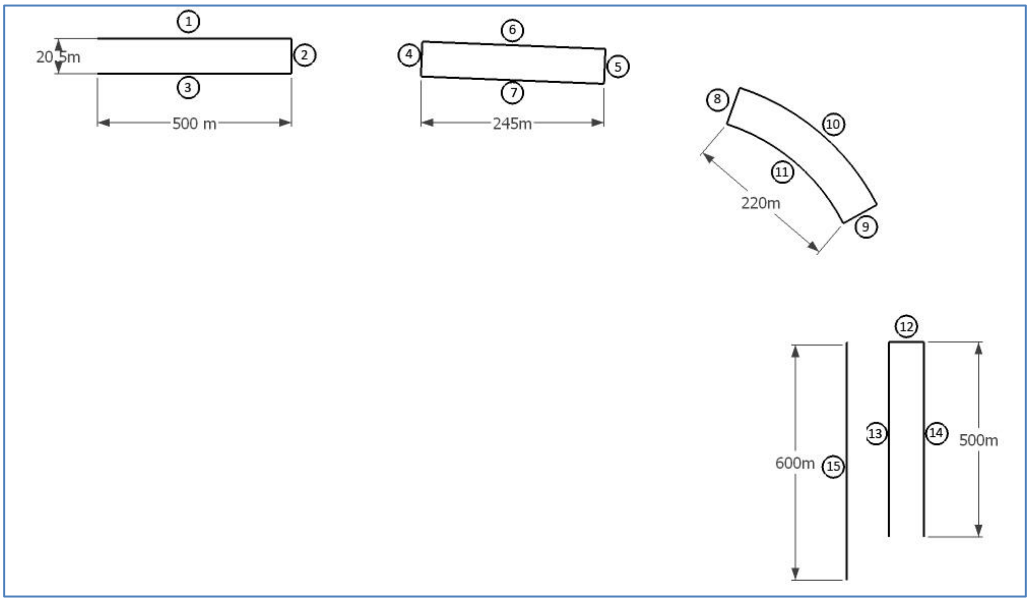


Figure 5. 1Lengths and alignments of 15 MSE walls (not in scale)

The design of walls will be performed in 7 steps based in LRFD methodology.

Step 1. Establishing Project requirements

Geometry

- Wall lengths: lengths and layout of all 15 MSE walls are defined in figure 4.2.
- Wall heights: all MSE walls are same since the elevation is almost at the same level at the bridge location throughout its length. It is designed to be 8 m. The heights of MSE walls are 1, 3, 13, 14 at approaching embankments gradually decreases to 0. The height of the MSE Wall 15 is 5m.
- Wall slope: the wall is vertical i.e. slope is 0 degree from vertical.

Step 2. Establishing project parameters

- Ground conditions of the site
  - Properties of foundation soil are given in subsurface profile in Table XX.
  - Groundwater conditions: the groundwater table is at 0.5m from the ground surface. This needs geotextile placement at the base of MSE walls.
- Reinforced wall fill: filling material needs to be select granular material with no cohesion. Maximum angle of friction of reinforced wall fill material is not greater than 34° (AASHTO, 2007).

Step 3. Estimate Wall Embedment Depth and Reinforcement Length

According to borehole logs in appendix E upper 0.3m layer of the soil is dark brown organic fine sand. To ensure the stability the embedment will be about 0.3m. In total design height of wall is 8 m.

The design of reinforcement length will be 0.7H, where H is the full height of the MSE wall according to AASHTO LRFD (2007). MSE wall height at approach embankments gradually changes from 0 to maximum height (Figure 5.3). In this case we divide the wall into three sections and use appropriate H for reinforcement length determination as in Figure 5.4 instead of gradually changing the reinforcement length, which is not viable from construction perspective. As it can be seen from figure 4.3 three different lengths for reinforcement will be used. For the third section calculated height is 2.2m. However, according to AASHTO LRFD (2007) the minimum of 2.5m will be kept in order to properly spread and compact the reinforcement fill. The reinforcement length in vertical direction does not change throughout the entire height.

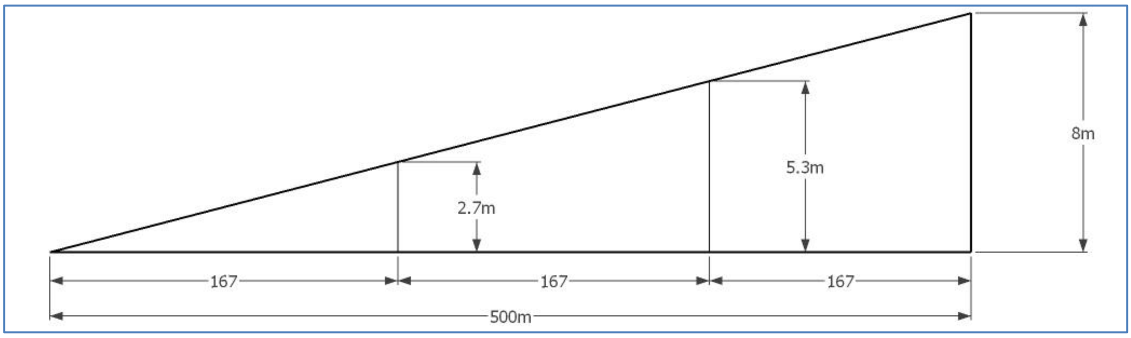


Figure 5. 2 Side view of MSE Walls 1, 3, 13, 14 (not in scale)

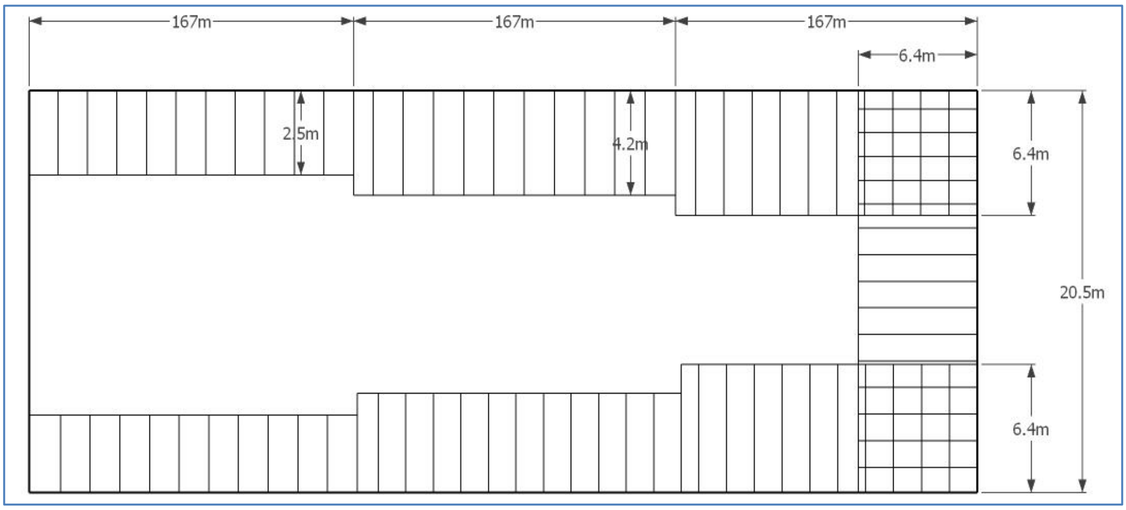


Figure 5. 3 Horizontal alignment of reinforcements for MSE Walls 1, 2, 3, 12, 13, 14 (not in scale)

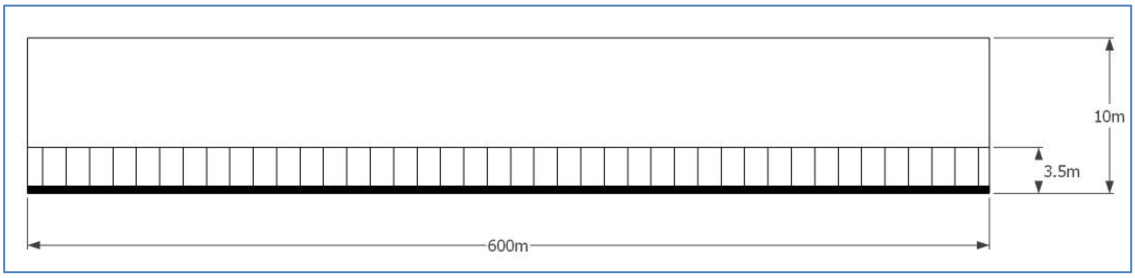


Figure 5. 4 Horizontal alignment of reinforcements for MSE Wall 15 (not in scale)

<u>Step 4</u>. Define Nominal Loads

Primary loads on MSE walls are vertical (EV) and horizontal earth (EH) pressure from retained fill, and live surcharge loading (LS) from traffic. The pressure envelope for MSE wall with traffic surcharge is represented in figure 5.6.

<u>Horizontal backslope loadings</u>: the active earth pressure coefficient for vertical walls with horizontal backlslope is calculated by the following formula:

$$\mathrm{K_{ab}} = \tan^2(45 - \frac{\emptyset'_{\mathrm{b}}}{2}) \qquad (5.1)$$

Here $\emptyset'_b$ is the friction angle of retained backfill. As it was mentioned previously, for MSE walls select granular soil with $\emptyset'_b = 34°$ will be used as backfill. Therefore the active pressure coefficient is $\mathrm{K_{ab}} = 0.28$.

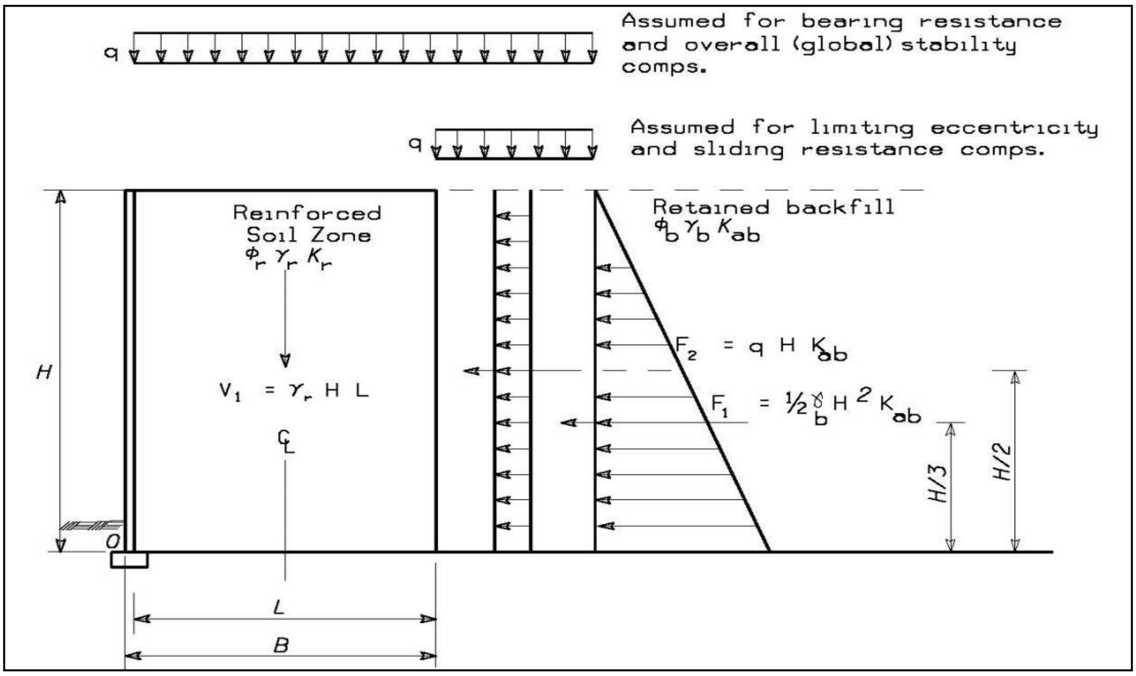


Figure 5. 5 External analysis: nominal earth pressures; pressure envelope for MSE with traffic surcharge (AASHTO, 2007)

Traffic loads: according to Article 11.10.10.2, AASHTO (2007) traffic loading is assumed to be uniform live surcharge equal to or not less than 2ft (0.6m) of earth. For external and internal stability analysis the traffic loads should be treated as an equivalent height of soil, h_{eq}=0.6m.

Usually select granular material used in Florida has unit weight of 18 kN/m^3 (FDOT, 2004). Multiplying the unit weight by equivalent height we will obtain live surcharge loading from traffic exerting on MSE walls.

$$q = \frac{18kN}{m^3} * 0.6m = 10.8kN/m^2$$

Step 5. Evaluate external stability

- **Sliding stability**

For the sliding stability the driving forces are checked against resisting force. Resistance factor provided by AASHTO 2007 (table 5.2 and table 5.3) should be applied for obtained results.

Table 5. 3 External stability resistance factors for MSE wall (Table 11.5.6-1, AASHTO 2007)

Stability mode	**Condition**	**Resistance factor**
Bearing resistance		0.65
Sliding	Soil on soil	1.0
Overall (global) stability	Where geotechnical parameters are well defined, and the slope does not support a structural element	0.75
	Where geotechnical parameters are based on limited information, and the slope support a structural element	0.65

Table 5. 4Table 4. 15 Typical MSE wall load factors (Table 3.4.1-2, AASHTO (2007))

Types of loads	**Load factor**	
	Maximum	Minimum
EH: Horizontal earth pressure • Active	 1.5	 0.90
EV: Vertical Earth pressure • overall stability • retaining walls and abutments	 1.00 1.35	 N/A 1.00
ES: Earth surcharge	1.5	0.75

1. Nominal trust acting on unit width:

   Resultant for retained backfill:

$$F_1 = \frac{1}{2} K_{ab} \gamma_b H^2 \tag{5.2}$$

   Resultant for uniform surcharge:

$$F_2 = K_{ab} q H \tag{5.3}$$

Here,

K_{ab} – Active earth pressure coefficient

γ_b – Moist unit weight of retained backfill soil

H – Height of the wall

q – Uniform surcharge (=$\gamma_r h_{eq}$)

2. Nominal and factored horizontal driving forces

$$\sum \mathrm{F} = \mathrm{F}_1 + \mathrm{F}_2 \tag{5.4}$$

$$\mathrm{P_d} = \gamma_{\mathrm{EH}}\mathrm{F}_1 + \gamma_{\mathrm{LS}}\mathrm{F}_2 \tag{5.5}$$

Maximum EH load factor should be used since it creates higher driving force.

3. Determine the frictional coefficient, μ at the base

Minimum of:

– friction along foundation soil (c'_f + tan$ø'_f$) and

– friction in reinforcement soil (tan$ø'_r$)

4. Calculate the factored resisting force per unit length of wall

The passive resistance of embedment is ignored for sliding stability calculations since it can be removed by further manmade or natural processes later.

$$\mathrm{R_R} = \emptyset_\tau \mathrm{R}_\tau \tag{5.6}$$

Here,

$\emptyset_\tau$ – resistance factor for shear resistance between soil and foundation

R_τ – nominal sliding resistance

From table 4.2 $\emptyset_\tau$ is 1.0 for soil on soil condition, therefore we can take $R_R = R_\tau$

$$\mathrm{R_r} = \gamma_{\mathrm{EV}}\mathrm{V}_1\mu \tag{5.7}$$

Here,

μ – the minimum friction coefficient

V – vertical loads.

5. Check for factored resistance force and factored driving force, and make sure that resistance forces are greater than driving forces. Check for capacity demand ratio to make sure CDR≥1.0:

$$\mathrm{CDR} = \mathrm{R_r}/\mathrm{R_d} \tag{5.8}$$

- **Eccentricity limit check**

Eccentricity, *e* is the distance between resultant forces and the center of reinforced zone as illustrated in figure 5.8. It is calculated by the following formula:

$$\mathrm{e} = \frac{\sum \mathrm{M_R} + \sum \mathrm{M_D}}{\sum \mathrm{V}} \tag{5.9}$$

Nominal retained backfill and surcharge resultant force is calculated from and used to estimate eccentricity with appropriate load factors by following formula (Berg, Christopher and Samtani, 2009):

$$\mathrm{e} = \frac{\gamma_{\mathrm{EH-MAX}} \mathrm{F_1} \left(\mathrm{H}/3\right) + \gamma_{\mathrm{LS}} \mathrm{F_{q-LS}} \left(\mathrm{H}/2\right)}{\gamma_{\mathrm{EV-MIN}} \mathrm{V_1}} \tag{5.10}$$

After finding the ecentricity it should be checked against overturning of the MSE wall. According to Berg, Christopher and Samtani the location of resultant vertical forces is acceptible if it is not greater than the fourth of the reinforcement length, e_{max} = L/4 (2009).

- **Evaluate bearing on foundation**

Bearing failure occurs when the vertical factored pressure exerted by the base of MSE, calculated by Meyerhof-type distribution exceeds the resistance of the foundation soil. To avoid this, the bearing capacity of the soil must be checked against vertical stress.

$$\mathrm{q_r} = \mathrm{q_{uniform}} \tag{5.11}$$

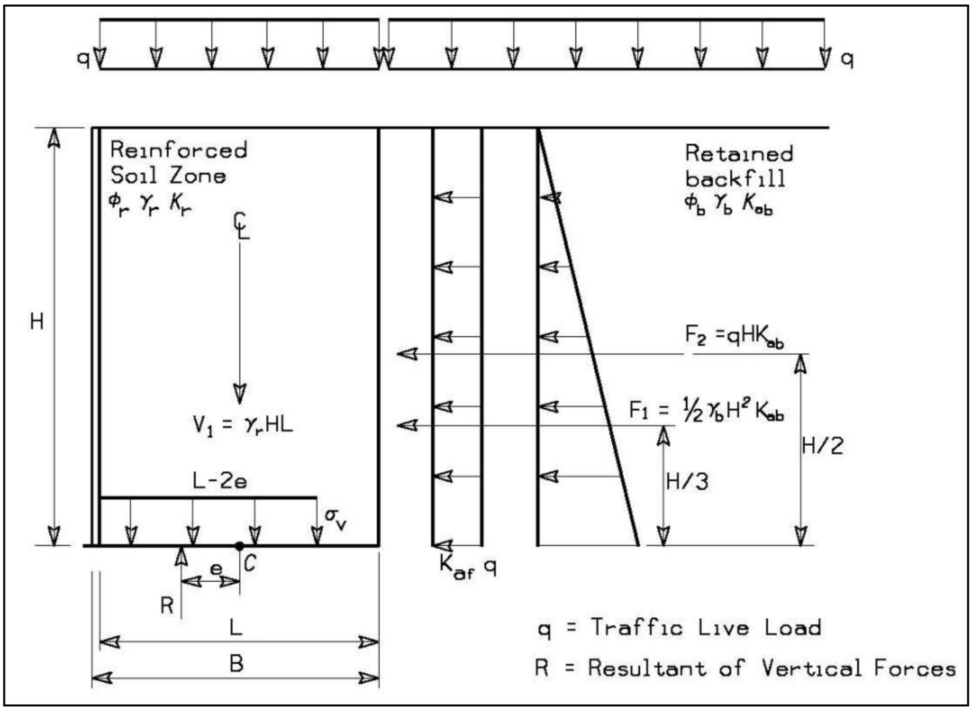


Figure 5. 6 Calculation of eccentricity and vertical stress for bearing and eccentricity check (Berg, Christopher and Samtani, 2009)

To calculate the factored vertical stress, the eccentricity and vertical forces need to be defined.

1. The eccentricity is found in similar way as in eccentricity limit check formula but with maximum load factors. In addition, the traffic live load is added as a vertical stress on reinforced zone, while it was neglected in former one.

$$e_b = \frac{\gamma_{EH-MAX}F_1(H/3)+\gamma_{LS}F_{q-LS}(H/2)}{\gamma_{EV-MAX}V_1+\gamma_{LS}qL} \tag{5.12}$$

2. Factored vertical stress at the base of a wall is estimated by the following formula:

$$\sigma_{V-F} = \frac{\gamma_{EV-MAX}V_1+\gamma_{LS}qL}{L-2e_B} \tag{5.13}$$

   Here, *L* is the length of reinforcement, and *L-2e* is the reduced area of uniform stress at the wall base due to eccentricity.

3. Determine nominal bearing resistance of foundation soil with equation provided by AASHTO 2007.

$$q_n = c_f N_c + 0.5L'\gamma_f N_\gamma \tag{5.14}$$

Here,

c_f – the cohesion of the foundation soil

γ_f – the unit weight of the foundation soil

N_c and N_γ – dimensionless bearing capacity coefficients

L' - effective foundation width, equal to $L\text{-}2e_B$

Dimensionless bearing capacities can be taken from Table 10.6.3.1.2a-1 of AASHTO (2007).

4. Check whether factored bearing resistance is greater than factored vertical bearing stress:

$$q_R \geq q_{V-F} \tag{5.15}$$

Here, $q_R = \emptyset q_n$. Factor $\emptyset$ is the resistance factor and for MSE wall its value is 0.65 (AASHTO 2007).

- **Overall stability check**

Overall stability of the wall is determined in order to examine potential failure planes that may occur under or near reinforced zone. For overall or global stability analysis the MSE wall treated as a whole rigid block. According to Article 11.6.2.3 AASHTO (2007) failure planes is examined by the use of stability computer programs such as ReSSA with appropriate resistance factors. As shown in table 5.3 the shear resistance factor for site with limited geotechnical parameters wall supporting structural element can be conservatively taken as ø= 0.65 (Berg, Christopher and Samtani, 2009) . Appropriate safety factor from shear resistance factor is derived from shear resistance factor:

$$\frac{1}{0.65} \approx 1.5 = \text{FS} \tag{5.16}$$

For the general design of MSE wall it can be assumed that the site has appropriate conditions for stable MSE wall, relying on the stability of near existing MSE walls at the site.

Calculations for the external stability

- **Sliding stability**

1. Nominal trust acting on unit width

   Resultant for retained backfill (Eq. 5.2):

   $$F_1 = \frac{1}{2}K_{ab}\gamma_b H^2 = \frac{1}{2} * 0.28 * 18 * 8^2 = 161kN$$

   Resultant for uniform surcharge (Eq. 5.3):

   $$F_2 = K_{ab}qH = 0.28*10.8*8 = 24.2kN$$

2. Nominal and factored horizontal driving forces (Eq. 5.4, 5.5)

   $$\sum F = F_1 + F_2 = 185.2\ kN$$

   $$P_d = \gamma_{EH}F_1 + \gamma_{LS}F_2 = 1.5 * 161 + 0.75 * 24.2 = 259.7\ kN$$

3. Frictional coefficient

   Foundation soil is loose brown fine sand with friction angle of 30º according to USCS.

   $\mu = c'_f + \tan ø'_f = 0+\tan(30) = 0.58$

   $\mu = \tan ø'_r = \tan(34) = 0.67$

   Friction along foundation soil is the lowest, therefore it will be used for resisting force calculations.

4. Factored resisting force per unit length of wall (Eq. 5.6)

   $$R_r = \gamma_{EV}V_1\mu = 1.0 * 18 * 8 * 6.4 * 0.58 = 524.5\ kN$$

5. Check for the capacity demand ratio

   $$CDR = R_r/P_d = \frac{534.5}{259.7} = 2.06 \geq 1.0$$

   The MSE wall is totally safe against sliding. Parts of MSE wall with lower height due to the slope are also safe against sliding since driving forces and resisting forces will decrease almost proportionally proportionally.

- **Eccentricity limit check**

$$e = \frac{\gamma_{EH-MAX} F_1 \left({}^{H}\!/_{3}\right) + \gamma_{LS} F_{q-LS} \left({}^{H}\!/_{2}\right)}{\gamma_{EV-MIN} V_1} = \frac{1.5 * 161 * \left(\frac{8}{3}\right) + 1.5 * 24.2 * \left(\frac{8}{2}\right)}{1.0 * 921.6}$$

$$= 0.86\ m$$

e = 6.4/2 – 0.86 = 1.24 (from the center of the wall width)

e_{max} = L/4 = 6.4 / 4 = 1.6 $\geq$ 1.24

Eccentricity of MSE wall is acceptable since it is lower than the maximum eccentricity stated by AASHTO 2007.

- **Bearing on foundation**

1. Eccentricity. Note, it is different than in previous check

$$e_b = \frac{\gamma_{EH-MAX} F_1 \left({}^{H}\!/_{3}\right) + \gamma_{LS} F_{q-LS} \left({}^{H}\!/_{2}\right)}{\gamma_{EV-MAX} V_1 + \gamma_{LS} q L} = \frac{1.5 * 16.6 * \left(\frac{8}{3}\right) + 1.5 * 19.9 * \left(\frac{8}{2}\right)}{1.32 * 94.7 + 1.5 * 11.1 * 6.4}$$

$$= 0.8\ m$$

2. Factored vertical stress

$$\sigma_{V-F} = \frac{\gamma_{EV-MAX} V_1 + \gamma_{LS} q L}{L - 2e_B} = \frac{1.5 * 94.7 + 1.5 * 11.1 * 6.4}{6.4 - 2 * 0.8} = 44.39\ kN$$

3. Nominal bearing resistance of foundation soil

$$q_n = c_f N_c + 0.5 L' \gamma_f N_\gamma = 0 + 0.5 * (6.4 - 2 * 0.8) * 1900 * 22.4 = 102.1\ kN$$

Unit weight of loose fine sand is 1900kg/m3 and $N_\gamma = 22.4$ (Table 10.6.3.1.2a-1, AASHTO 2007).

4. Check

$$q_R = \emptyset q_n = 0.65 * 102.1 = 66.3\ kN$$

$$q_R \geq q_{V-F}$$

Factored bearing resistance is greater than factored vertical bearing stress.

Step 6. Evaluate internal stability

- Select type of soil reinforcement
  - Extensible reinforcement
  - Inextensible reinforcement

Inextensible reinforcement provides less flexibility compared with extensible reinforcement. Since the wall will support structural units, i.e. bridge and pavement the inextensible reinforcement will be used.

- Establish vertical layout of soil reinforcement
  In order to provide a coherent reinforced soil zone, vertical spacing of reinforcement should not exceed 800 mm by LRFD standards. For the design 600mm spacing was selected to make it more conservative and compatible with typical panel height.
- Calculate factored tensile forces in the reinforcement layers
  - Horizontal stresses are defined with the following formula:

$$\sigma_H = K_r[\sigma_V] + \Delta\sigma_H \tag{5.17}$$

Here,

K_r – coefficient of lateral earth pressure

σ_V – factored vertical pressure

$\Delta\sigma_H$ – factored horizontal stress

Stress is calculated at the bottom of the wall were it is the most critical. As the height increases the horizontal reinforcement spacing can be increases.

- Maximum tension, T_{MAX}, in each layer per unit length of wall by using horizontal stress:

$$T_{MAX} = \sigma_H S_V \tag{5.18}$$

Here,

S_V – vertical reinforcement spacing

- For discrete reinforcement tension per unit width can be changed to per panel width:

$$P_{TMAX-D} = \frac{\sigma_H S_V W_P}{N_P} \tag{5.19}$$

Here,

P_{TMAX-D} – load in discrete reinforcement element

W_P – width of panel

N_P – number of discrete reinforements per panel width

- Select number of soil reinforcement elements at each level

  Designed panel will have four connections, two for each level as shown in figure 5.9. The width of a panel is 1.2m. Therefore, number of reinforcement elements will be at every 0.6m.
- Lateral and vertical movements

  Maximum allowable movements stated by FDOT is 7cm vertically and 12cm horizontally (2016).

Calculations for the internal stability

- Horizontal stresses

$$\sigma_H = K_r[\sigma_V] + \Delta\sigma_H = 0.28 * (18 * 8 + 10.8) = 43.3kN/m^2$$

- Maximum tension, T_{MAX}, in each layer per unit length of wall

$$T_{MAX} = \sigma_H S_V = 43.3 * 0.6 = 26kN/m$$

- Maximum tension P_{TMAX-D} per panel width.

$$P_{TMAX-D} = \frac{\sigma_H S_V W_P}{N_P} = \frac{26 * 1.2}{2} = 15.6kN$$

Table 5. 5 Maximum tensile forces in each layer

Layer	**Depth H, m**	**Ϭh,kN/m2**	**T_{max}, kN/m**	**P_{Tmax-d}, kN**
1	8	43,3	26,0	15,6
2	7,4	40,3	24,2	14,5
3	6,8	37,3	22,4	13,4
4	6,2	34,3	20,6	12,3
5	5,6	31,2	18,7	11,2
6	5	28,2	16,9	10,2
7	4,4	25,2	15,1	9,1
8	3,8	22,2	13,3	8,0
9	3,2	19,2	11,5	6,9
10	2,6	16,1	9,7	5,8
11	2	13,1	7,9	4,7
12	1,4	10,1	6,0	3,6
13	0,8	7,1	4,2	2,5

Step 7. Tie strip (connection element), reinforcing strip and their connection

The metallic reinforcements for MSE systems are connected with segmental precast panels by either bolting the reinforcement to a tie strip cast in the panel or connected with a bar connector.

For the design bolted connection was selected since it is the most common type and allows quick construction process (figure 5.7). The design load at the connection is equal to the maximum load on calculated in internal stability section. Metal connection hardware that is cast into the panel and extends out the back face of the panel for attachment to the soil reinforcement should not be placed in direct contact with the concrete steel reinforcement. This type of placement could accelerate corrosion of metal soil reinforcement (Cruciform Panels Manual, 2014).

Figure 5. 7 Panel to strip bolted connection

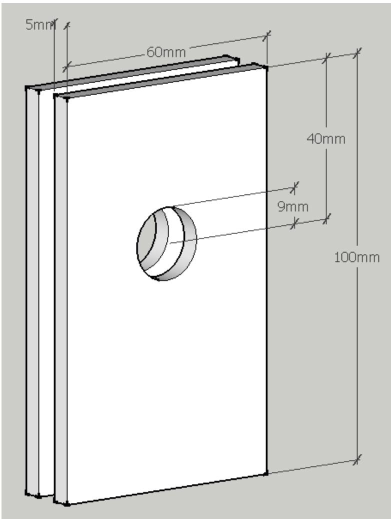


Figure 5. 8 Metallic tie strip design embedded into panel

Reinforcing strip is connected to the embedded panel tie strip by inserting the end of the reinforcing strip into the gap between the two exposed ends of the tie strip as shown in figure 5.9. Strips are bolted through the holes from below, placing a washer on top (see figure 5.10).

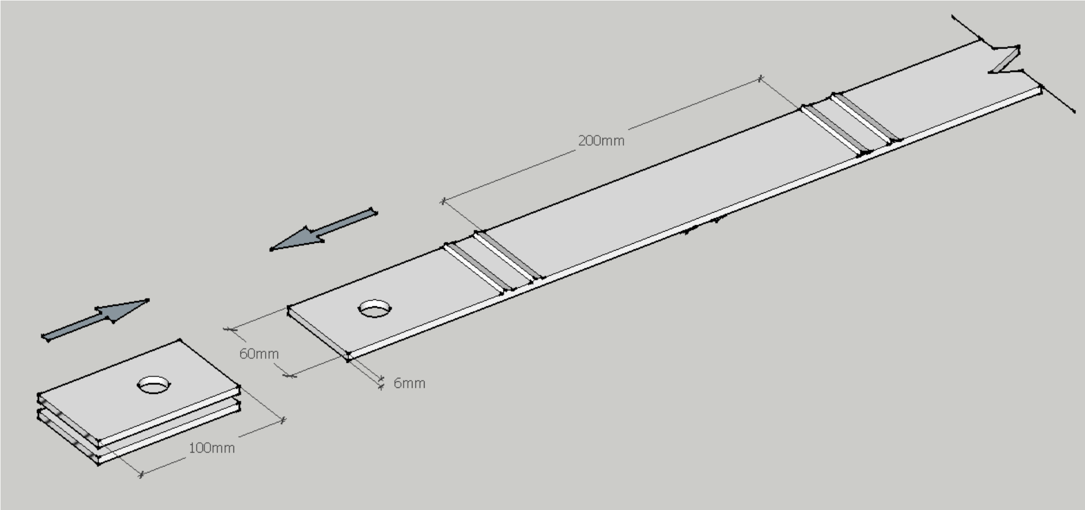


Figure 5. 9 Connection between tie strip and reinforcing strip

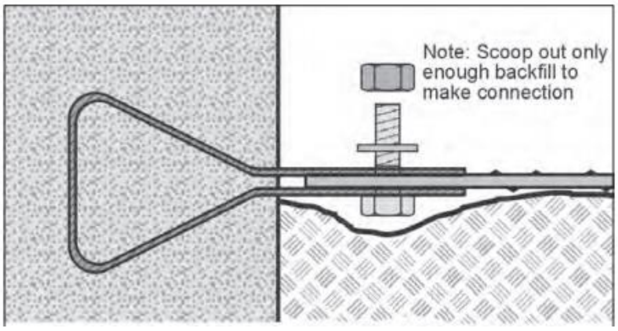


Figure 5. 10 Bolting of tie strip to reinforcing strip

Step 8. Facing elements

Precast concrete panels are produced as segmental panels or full height panel for specific projects. However, vertical alignment of the design is not at one level which requires more estimation for full height panels design. That is why segmental panels are better to use for the project.

Precast segmental face panels are usually produced in square, rectangular and cruciform shapes that may have different pattern on exposed face. Typical size of the panel is in the range of 125 to 200 mm thick, 1-2m high and 1-3m wide (CDOT, 2014). Again, due to the near existing MSE wall designs panels with cruciform shape will be selected for the project as to keep the harmony of the aesthetic view of the site. To make equal spacing the cruciform panels have dimensions of 1350mm in width and 1200mm in height (figure 5.11). Connection elements for reinforcements are placed in equal distances according to the design load calculated in internal stability section.

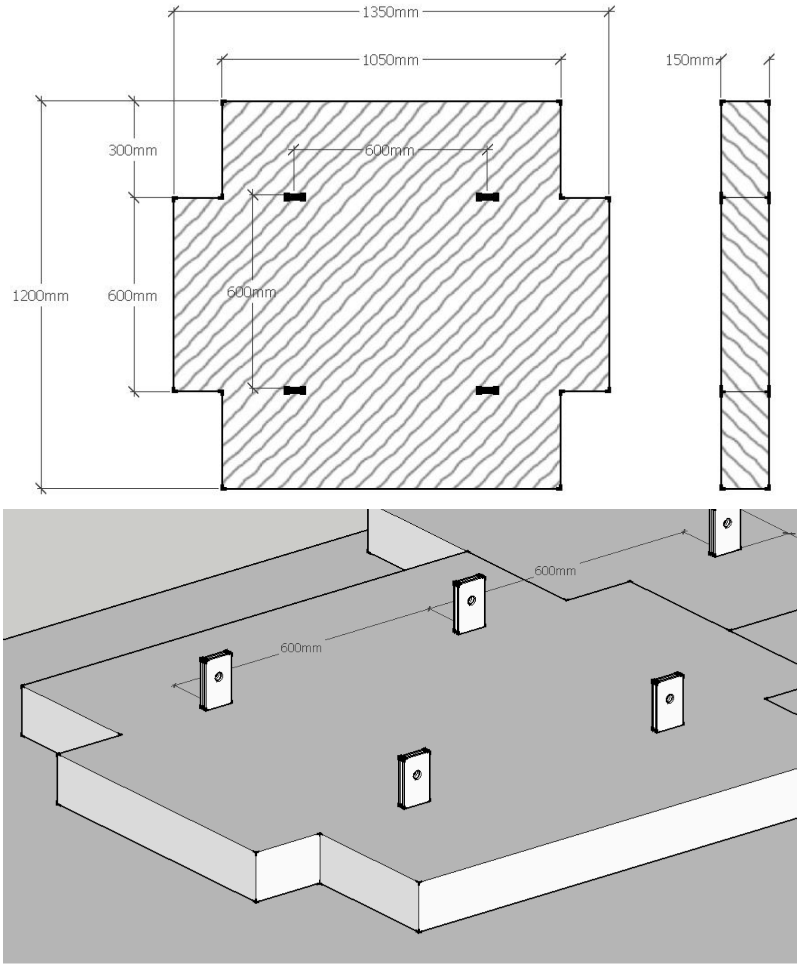


Figure 5. 11 Cruciform precast concrete panel shape and dimensions.

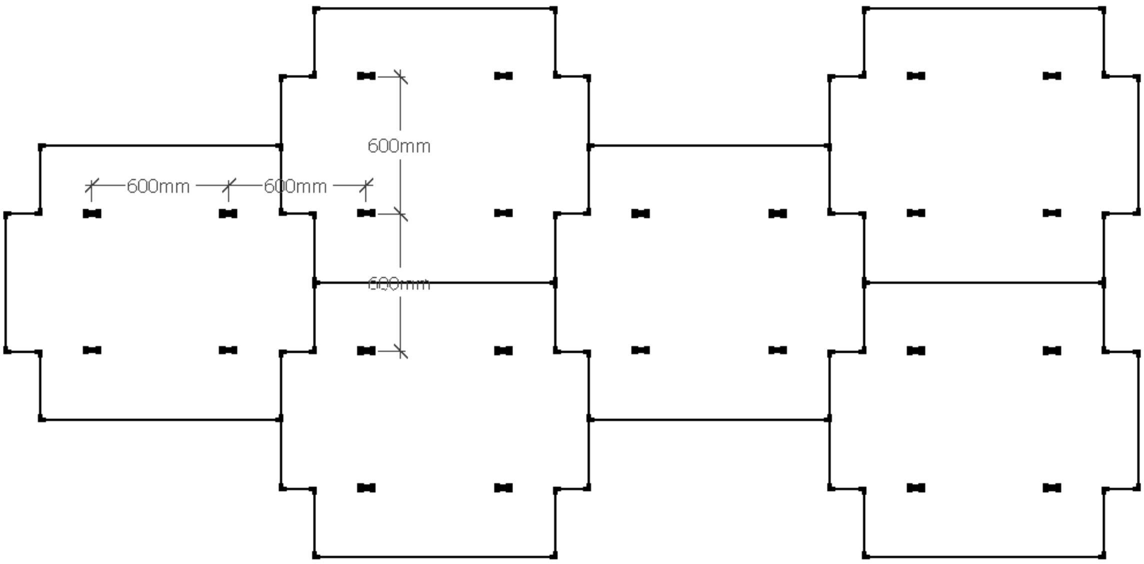


Figure 5. 12 Precast concrete panels alignment.

5.1.4 MSE wall stability check in Plaxis 3D software simulation

MSE walls 1, 2, 3 were simulated in Plaxis 3D as a whole block. Other blocks are similar to this block; therefore results will be applicable to all other MSE walls. The load of the wall block is calculated as distributed load over 600m:

$$q = 10.8 + 18*8 = 154.8 \text{ kN/m2}$$

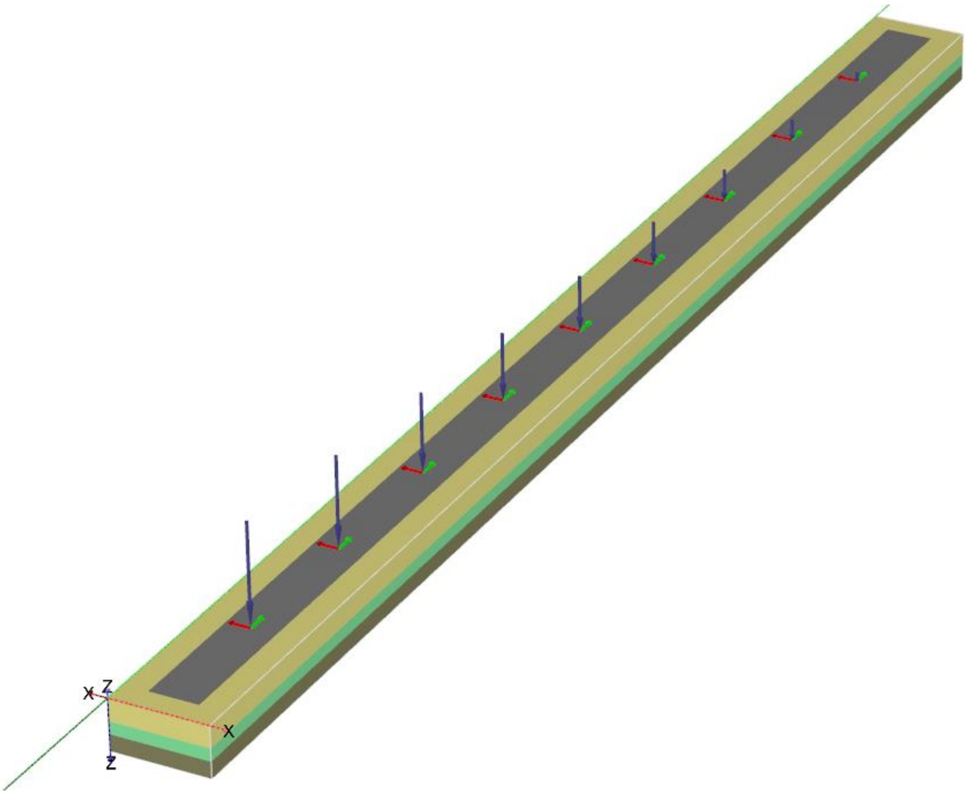


Figure 5. 13 Distributed load onto the soil of MSE wall block

Soil parameters from table 5.2 according to the borehole data. The simulation results are shown in figure 5.13 and 5.14.

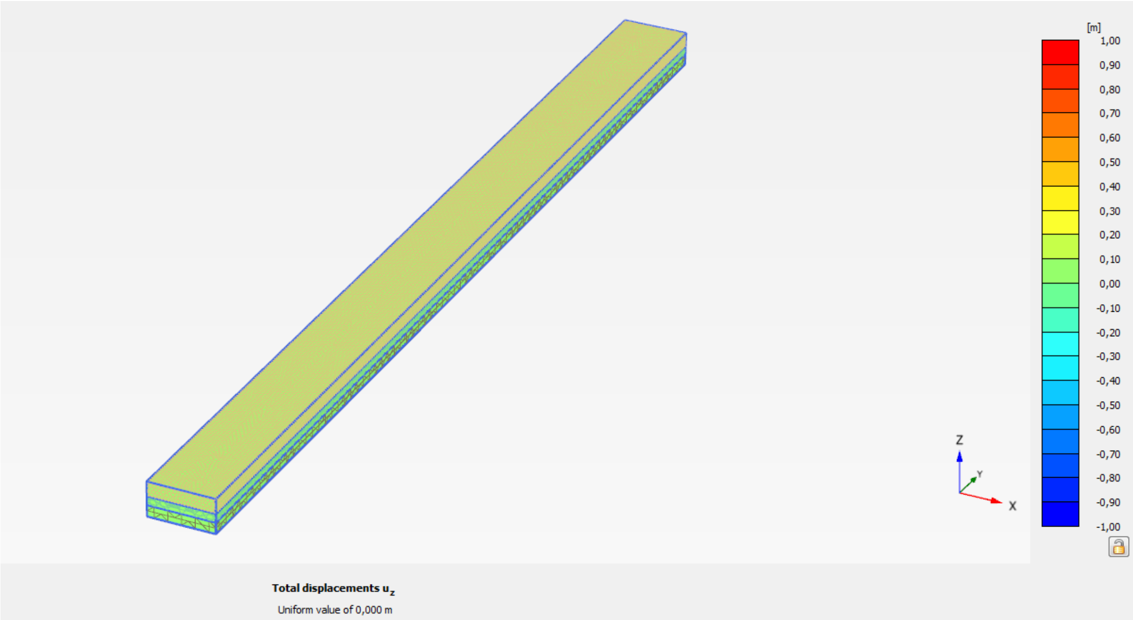


Figure 5. 14Foundation soil before MSE wall construction

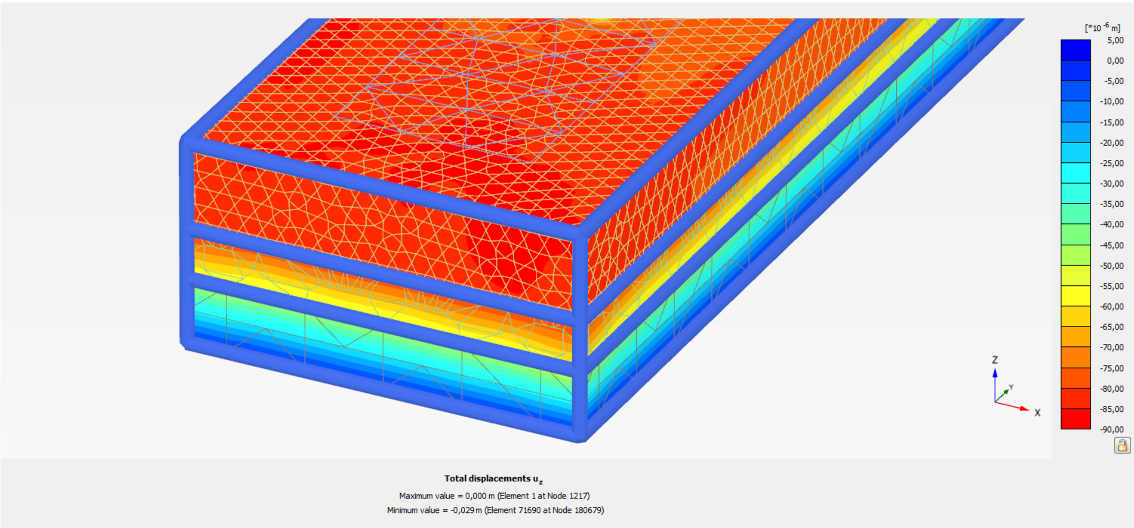


Figure 5. 15 Deformed foundation soil after MSE wall construction

According to simulation results maximum deformation under the wall is 0.03m as it can be seen in figure 5.15. The wall is stable in overall since the foundation soil does not undergo a lot deformation. The results of external stability calculations are confirmed by computer simulation.

5.2 Design of Flexible Walls of Canal

5.2.1 Feasibility analysis

According to the outputs of Capstone 1, **anchored steel sheet pile** option was chosen for design considerations. The main reasons for applying that option are following:

1) **Method of construction**: Steel sheet pile can easily be driven into soil and do not require water detention.
2) **Operational feature**: Steel sheet piles provide longer service life than concrete. The difference can be found in changing bearing capacity of a soil that cause tilting of the concrete wall. On the other perspective, corrosion of steel can be easily solved by increasing the thickness of wall or by covering
3) **Time factor:** Sheet piles require less site preparation than other options
4) **Economic perspective:** Anchored steel sheet piles sustainability can be taken into account, since usually they are made from recycled steel and appropriate quantity of steel would be required. In addition, they can be easily removed for maintenance works while cantilever steel sheet pile option requires more quantity of steel for construction

5.2.2 Conditions and Limitations for Design

Generally, anchored bulkheads are constructed of flexible sheeting, which is restrained by penetration of sheeting below dredge line and by tieback. Following Figure illustrates three common penetration conditions: free earth method, penetration in compact coarse-grained stratum and penetration to top of hard unyielding stratum for design methodology. More detailed information about procedure for computing each of those conditions can be found in Appendix E.

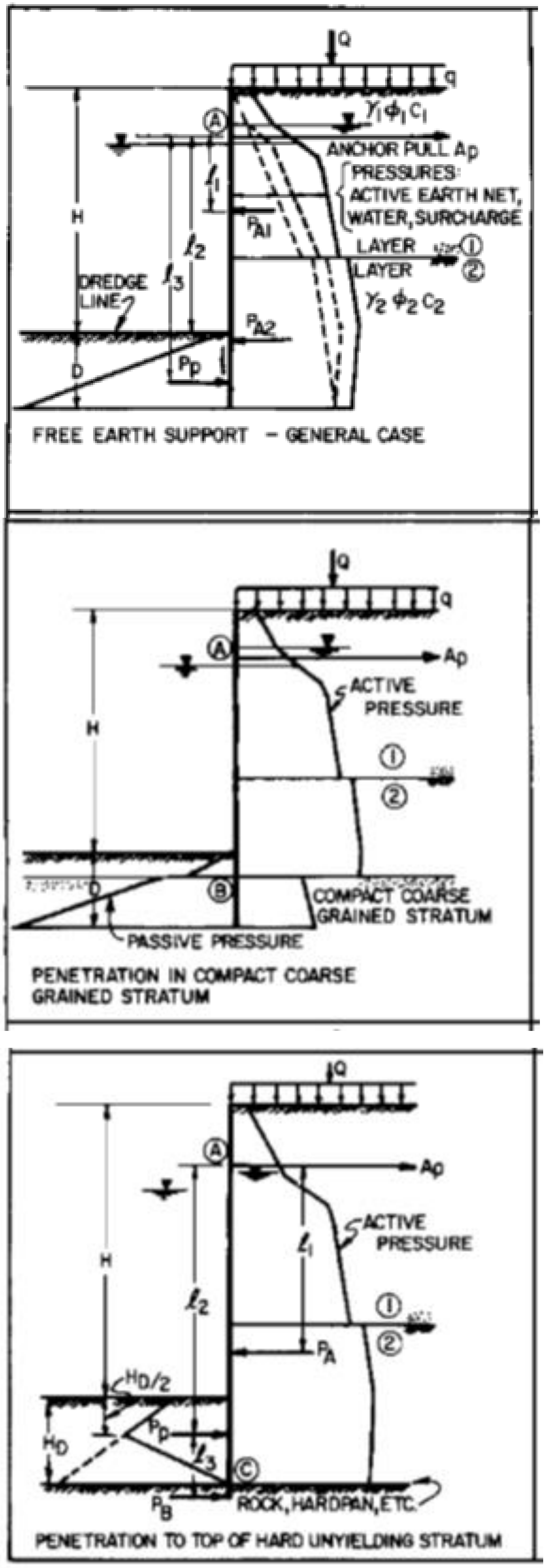


Figure 5. 16Three common conditions for design methodology

Soil surface generated from analyzing boreholes taken from location of placing anchored steel sheet piles shows that free earth method general case should be applied due to absence of soil layers of compact graded stratum and rock, hardpan (necessary for specific cases of penetration).

In addition, following assumptions were considered during the design process:

- The piling should drive for deep enough to produce stability, assuming mobilization of the maximum possible passive resistance.
- The sheet piling should be inflexible and no pivot point placed below the dredge line. In other words, no passive resistance should be developed on the backside of the piling.
- Earth Pressures should be computed based on Coulomb or log-spiral method.
- Horizontal Backfill developed by soil surface
- Uniform Subcharge caused by MSE Wall pressure
- Soil is homogenous as illustrated in soil profile
- Ground water table is horizontal caused by ocean
- Wall friction can be neglected

5.2.3 Methodology

5.2.4.1 Wall Pressures

Compute active and passive pressures using the appropriate formulae. Determine required depth of penetration of sheeting and anchor pull from these pressures. Following Figure illustrates main formulas for computation of general active pressures.

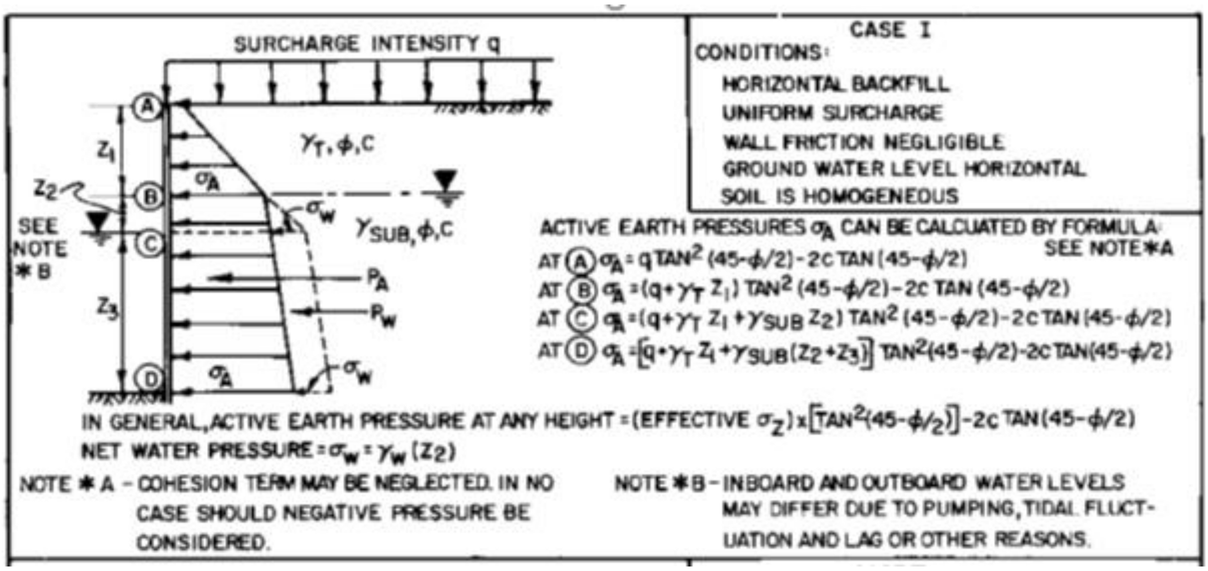


Figure 5. 17 Computation of active pressures

Following equations should be applied in order to define pressures applied into the wall:

$$\mathrm{Pb} = \gamma * \mathrm{H1} * \mathrm{Ka} \tag{5.20}$$

$$\mathrm{Pc1} = \mathrm{Pb} + \gamma' * \mathrm{Hw} * \mathrm{Ka} \tag{5.21}$$

$$\mathrm{Pc2} = (\gamma * \mathrm{H1} + \gamma' * \mathrm{Hw}) * \mathrm{Ka}' \tag{5.22}$$

$$\mathrm{Pe} = \gamma'(\mathrm{Kp}' - \mathrm{Ka}') * \mathrm{D1} \tag{5.23}$$

Following equations are applied to estimate pressure caused by surcharge:

$$\mathrm{Psur} = \mathrm{Qsur} * \mathrm{Ka} \quad \text{for above dredge line} \tag{5.24}$$

$$\mathrm{Psur} = \mathrm{Qsur} * \mathrm{Ka}' \quad \text{for below dredge line} \tag{5.25}$$

Locate point of zero pressure given by formulae:

$$y = \frac{\gamma' * H * Ka'}{(Pp - Pa)} \tag{5.26}$$

Calculate Pa, which is the resultant force of the earth pressure above A point and L explained as the difference below the tie rod level.

In order to reach equilibrium, the wall should be deep enough in order to balance moment created by resultant force Pa by moment caused by net passive pressure about tie rod level. Following equation illustrates formulae for estimation thickness D1:

$$\sum \mathrm{Mt} = \mathrm{L} * \mathrm{Pa} - \frac{(\mathrm{Pp} - \mathrm{Pa}) * \mathrm{D1}}{2} * \left(\mathrm{Ht} + \mathrm{y} + \frac{2}{3}\mathrm{D1}\right) \tag{5.27}$$

Define D1 by using trial and error method due to the existence of 3^{rd} power.

Compute the tie rod by given formulae:

$$\mathrm{T} = \mathrm{Pa} - (4\mathrm{c} - \gamma\mathrm{e} * \mathrm{H}) * \mathrm{D}' \tag{5.28}$$

5.2.4.2 Wall Movements

As it mainly behaves in real life, active pressures are randomly distributed on the wall by deflection, and therefore moves away from the position of maximum moment.

Therefore, it is necessary reduce the computed maximum moment to allow for flexibility of sheeting. According to the figure xx, Moment reduction is a function of the wall flexibility number. Then, select sheeting size by successive approximations so that sheeting stiffness is compatible with reduced design moment.

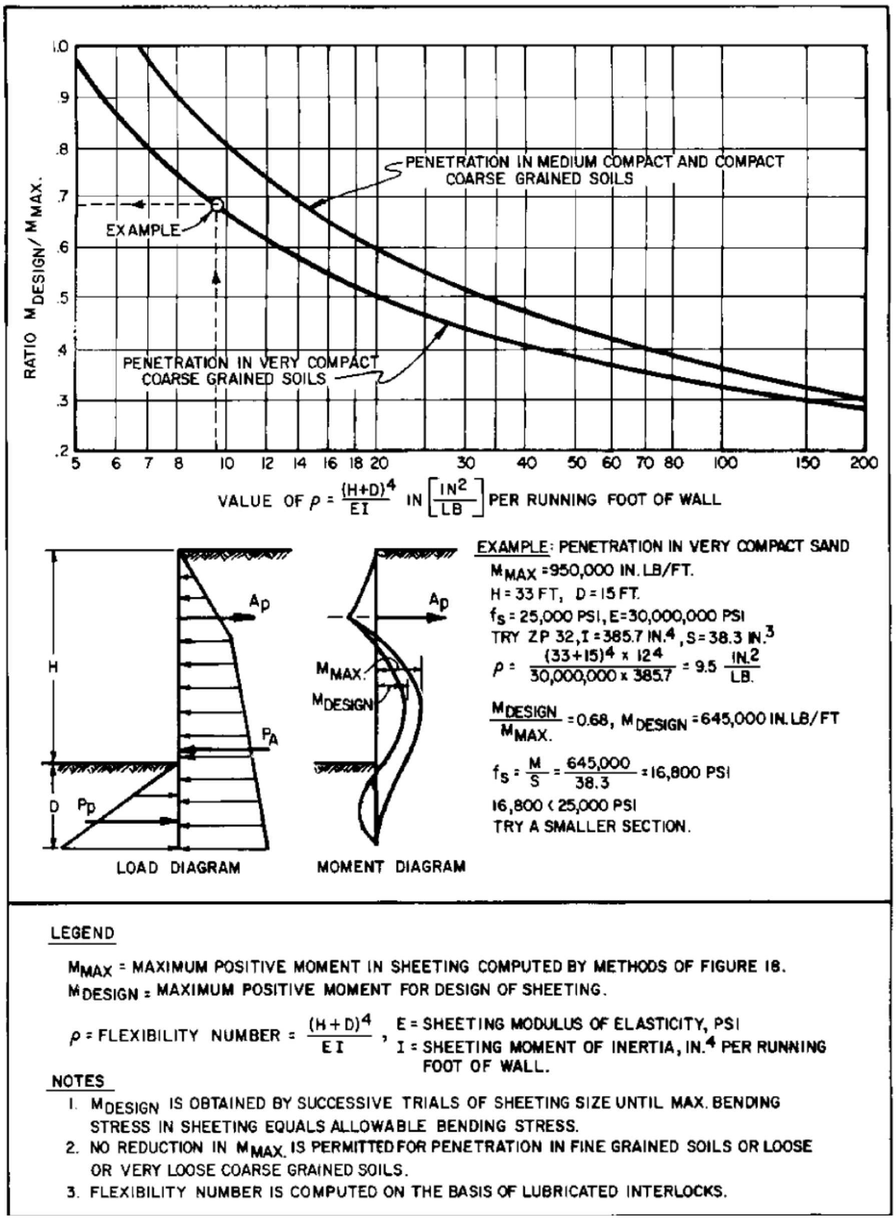


Figure 5. 18 Reduction in Bending Moment in Anchored Bulkhead from wall flexibility

5.2.4.3 Anchorage System

Basically, the most problematic part of anchored bulkheads are created with their anchorage. The main criteria for design of deadman anchorage presented below. It should be noted, that if the deadman must be positioned close to a wall, anchorage resistance is decreased and an additional passive reaction is required for stability at the wall base. Mostly, tie rods can be protected by wrapping, encasement or other ways to resist corrosion.

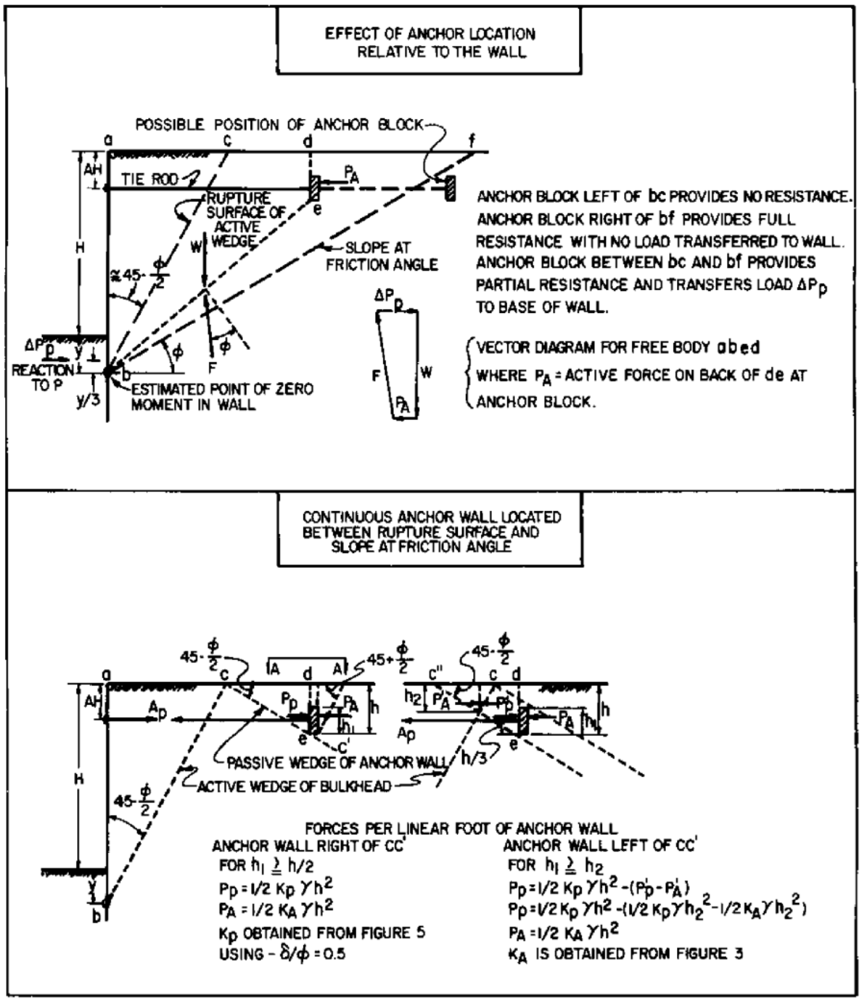


FIGURE 20
Design Criteria for Deadman Anchorage

Figure 5. 19 Design Criteria for Deadman Anchorage

5.2.4.4 Construction Precautions

Precautions during construction are as follows:

1. Removal of some material or placement of fill in the “passive” zone should precede the driving of sheet piles.
2. Deposit backfill by working away from the wall rather than toward it to avoid trapping, a some material adjacent to sheeting.
3. Before anchorage is placed, sheeting is loaded as a cantilever wall, and safety during construction stages should be checked.

5.2.4 Calculations

Input Parameters

Elevation profile of the soil presented in the Table 4.6. As it clearly seen from the Table, soil of the ground is homogenous and therefore case I and Free Earth Method can be applied for estimation.

Table 5. 6 Elevation Profile

Range of Elevation, m.		**Material Type**
From	**To**	
--	0	Fill
0	-8.22	Sand
-8.22	-13	Limestone

Following tables 4.7 and 4.8 illustrates the main properties of the soil presented in the ground in project location. According to soil profile, following soil parameters will be used for design.

Table 5. 7Sand Back Fill Parameters

Sand Back Fill	
Soil's unit weight (γ), kN/m^2	20
Effective soil's unit weight (γ'), kN/m^2	18
Cohesion (c), kN/m^2	0
Angle of internal friction (φ), degree	34

Table 5. 8Limestone Parameters

Limestone	
Soil's unit weight (γ), kN/m^2	27
Effective soil's unit weight (γ'), kN/m^2	25
Cohesion (c), kN/m^2	0
Angle of internal friction (φ), degree	40

5.2.4.1 Wall Pressures

Following Table 4.9 illustrates all pressures applied into the wall including active earth net, water and surcharge pressures.

Table 5. 9Pressure applied into the wall

Pressure applied	Value
Active earth Pressure, $P_{Bx,}$ (kN/m^2)`	14,747
Active earth Pressure, P_{c1} (kN/m^2)	35,670
Active earth Pressure , $P_{c2,}$ (kN/m^2)	33,133
Active earth Pressure P_e, (kN/m^2)	19,822 D_1
Surcharge Pressure,P_{sur} above C, (kN/m^2)	4,021
Surcharge Pressure P_{sur} below C, (kN/m^2)	3,734

Line Load Pressure, P_h kN/m	1.49

Table 5. 10 Resultant of Pressures

Resultant	Numerical Value (N)
P_1	6985.3
P_2	36323.6
P_3	25768.5
P_4	2603.6
P_5	13716.6
P_{sur} above C	353.8*D1+589.7
P_{sur} below C	4989.5

Table 5. 11Calculation of Moments

	Force (N)	Arm (m)	Moment (N*m)
1	6985.3	-0.71	-4959.6
2	36323.6	4.27	155101.8
3	25768.5	5.59	144045.9
4	2603.6	8.10	21089.2
Surcharge +	13716.6	2.74	37583.5
Surcharge -	353.8*D1+589.7	D_1/2+4.37	(35.38*D1+58.97)* (D_1/2+4.37)
Line Load	4989.5	1.65	8232.7
Passive	938.9* D_1^2	-(4.37+2* D_1/3)	-938.9* D_1^2*(4.37+2* D_1/3)

Solve for D_1 for $\sum M$, by trial method and error D_1=1.98 meters

Outputs can be expressed as in table 4.12.

Table 5. 12 Resulting Outputs

Output	Value
Total Penetration D, m	2.43
Designed Depth, D_{des}, m	3.35
Maximum Moment, M, kN*m	151,38

5.2.4.2 Wall Movements

Ratio of Flexibility using equation 4.30:

$$\rho = \frac{(H+D)^4}{EI} = \frac{(36+11)^4}{30*10^6*I} = \frac{3373}{I}$$

According to the ratio of flexibility and using figure listed in appendix, estimation of M design can be found. Following table 5.13 illustrates pile sections that can sustain that M design

Table 5. 13 Pile Sections

	PZ-38	PZ-32
S(per foot)	46.8 in^3	
I (per foot)	280.8 in^4	220.4 in^4
$P=\frac{3373}{I}$	12	15.3
M_{design}(in*K)	980	900
Stress= $\frac{Md}{S}$	21.0	23.5

Based on Rowe's Theory of Moment Reduction the following sheet piles should be used:

PZ-38: Regular Carbon Grade Steel

PZ-32: Regular Carbon Grade Steel

5.2.4.3 Anchorage System

Computation for $h1 \geq h/2$

Input Parameters of Deadman presented as follows:

Table 5. 14Deadman parameters for $h1 \geq h/2$

Deadman:	Value	Place found
Height of deadman (h_1), m	1.52	
Depth to the base of deadman from top of bulkhead, m	1.68	Assumption
Eff. depth to the base of deadman used for design (h), m	1.68	Assumption
Resistance factor	0.23	*Default:0.75*
Load factor	0.46	*Default: 1.5*
Coefficient of passive earth pressure (k_p)	1.99	*From Fig.5, NAVFAC DM-7.2 for -* δ/φ=0.5
Coefficient of active earth pressure (k_a)	0.09	*From Fig.3, NAVFAC DM-7.2*
Coefficient of earth pressure at rest (k_0)	0.30	General case
Distance b/w deadman and bulkhead, m	6.10	Assumption
Width of individual deadman (b), m	--	
C/C spacing of Anchor rods (d), m	--	
Clear spacing b/w deadman anchorages	--	

(L), m		
Bulkhead:		
Depth to "zero" moment from top of bulkhead, m	7.32	Assumption

Following calculation were performed in order to check Deadman efficiency

Table 5. 15Calculation performed for h1 ≥ h/2

Calculation	Value
Minimum distance for deadman to be outside of active failure wedge of bulkhead, m	3.8
Minimum distance for deadman to be outside of friction angle slope of bulkhead, m	10.6
Min.distance to avoid overlap of active wedge of bulkhead and passive wedge of deadman, m	7.0

Comparing values from Tables 5.14 and 5.15, it can be stated that deadman is outside the active failure wedge of Bulkhead wall and overlap between active wedge of bulkhead and passive wedge of deadman can be seen.

In order to define Anchor resistance, computation for pressure applied for deadman is defined. Following table illustrates results

Table 5. 16Pressure calculation for deadman

h_2	0.5	m
Passive force (P_p)	432.3025	kN/m
Factored P_p	324.2269	kN/m
Active force (P_a)	19.03212	kN/m
Factored P_a	28.54818	kN/m

Outputs can be found as follows:

1) For continuous wall

Anchor resistance/linear meter (Apc/d)	295	kN/m

2) For individual anchors

Anchor Resistance (A_p)	720	kip

Computation for h1 < h/2

Input Parameters of Deadman presented as follows:

Table 5. 17Deadman Parameters for h1 < h/2

Deadman:	Value	Place found
Height of deadman (h_1)	1.2	*Assumption*
Depth to base of deadman from top of bulkhead, m	0.8	*Assumption*
Effective depth to the base of deadman (h), m	6.1	Available depth 26.25 m
Distance b/w deadman and bulkhead, m	7.6	*Assumption*
Depth to "zero" moment from top of bulkhead, m	8.5	*Assumption*
Resistance factor (bearing resistance)	0.45	*Ref: AASHTO LRFD 10.5.5.2.2*
Red. Friction angle	0.0	*See Fig.20, NAVFAC DM-7.2*
N_c	6.3	*See AASHTO LRFD, Table 10.6.3.1.2a-1*
N_q	7.8	
N_γ	7.1	
S_c	1	*for strip footing*
S_q	1	
S_γ	1	
d_q	1	*See AASHTO LRFD, Table 10.6.3.1.2a-4*
Water table correction factor, C_{wq}	1	*See AASHTO LRFD, Table 10.6.3.1.2a-2*
Water table correction factor, $C_{w\gamma}$	0.5	
Depth to the center of Deadman, D_f, m	5.5	Assumption

Following calculation were performed in order to check Deadman efficiency

Table 5. 18 Calculation performed for h1< h/2

Minimum distance reqd. b/w bulkhead and deadman		4.7	m
Check	OK..Deadman is outside the active failure wedge of Bulkhead wall		
N_{cm}		6.3	
N_{qm}		7.8	
$N_{\gamma m}$		7.1	
Nominal bearing resistance, q_u, kN/m^2		812	
Factored bearing resistance, q_R, kN/m^2		366	

Outputs can be found as follows:

3) For continuous wall

Anchor resistance/linear foot (Apc/d)	445.5	kN/m

5.2.5 Retaining wall stability check in Plaxis 2D software simulation

Designed anchored bulkhead was simulated in Plaxis 2D software in order to check stability. Following Figure 4.18 illustrate results placing anchored bulkhead without applying load.

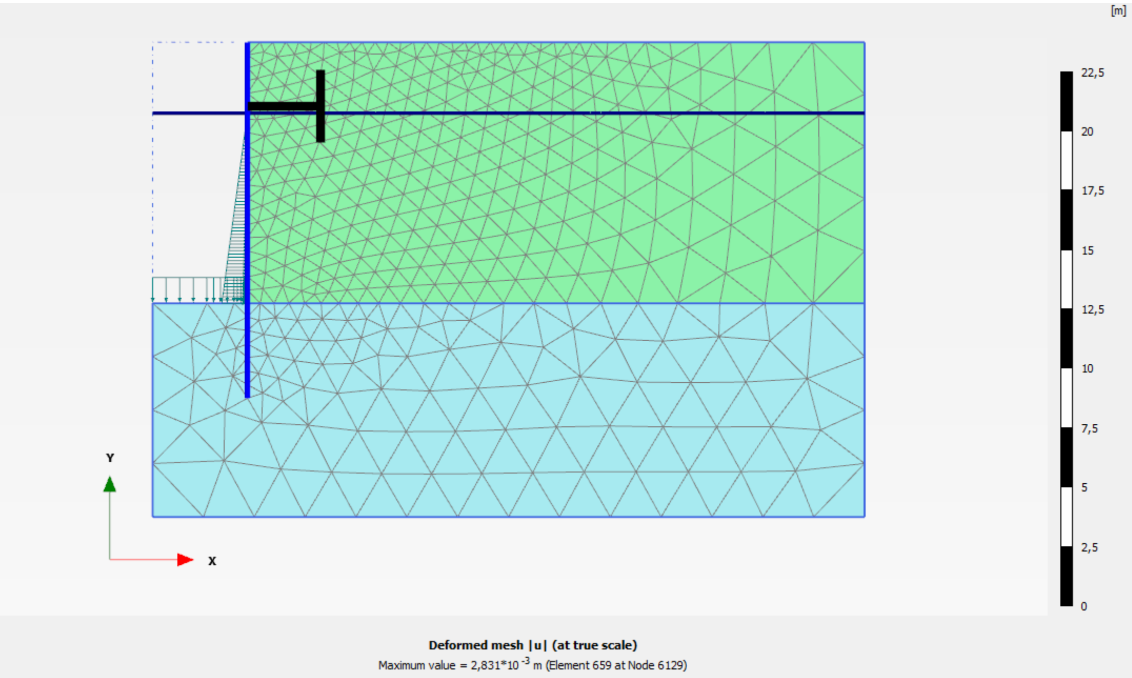


Figure 5. 20 Placement of Anchored Bulkhead

As it clearly seen, deformed displacement of the anchored bulkhead is equal to 3 mm which is less than maximum allowable requirement: 6 cm. More detailed information about behavior of anchored bulkhead can be seen from Figure 4.19, which is scaled 100 times to the original one.

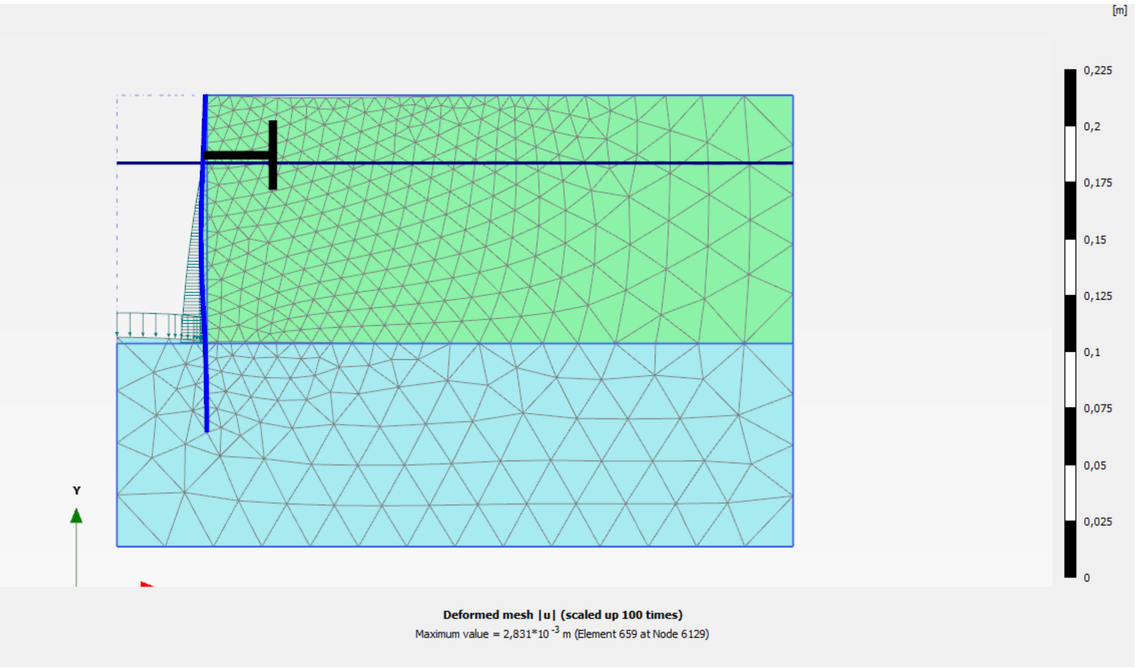


Figure 5. 21 Behavior of an Anchored bulkhead without dead load

Finally, applying dead load created by construction of MSE wall and road pavement, final deformed displacement of the anchored bulkhead can be found.

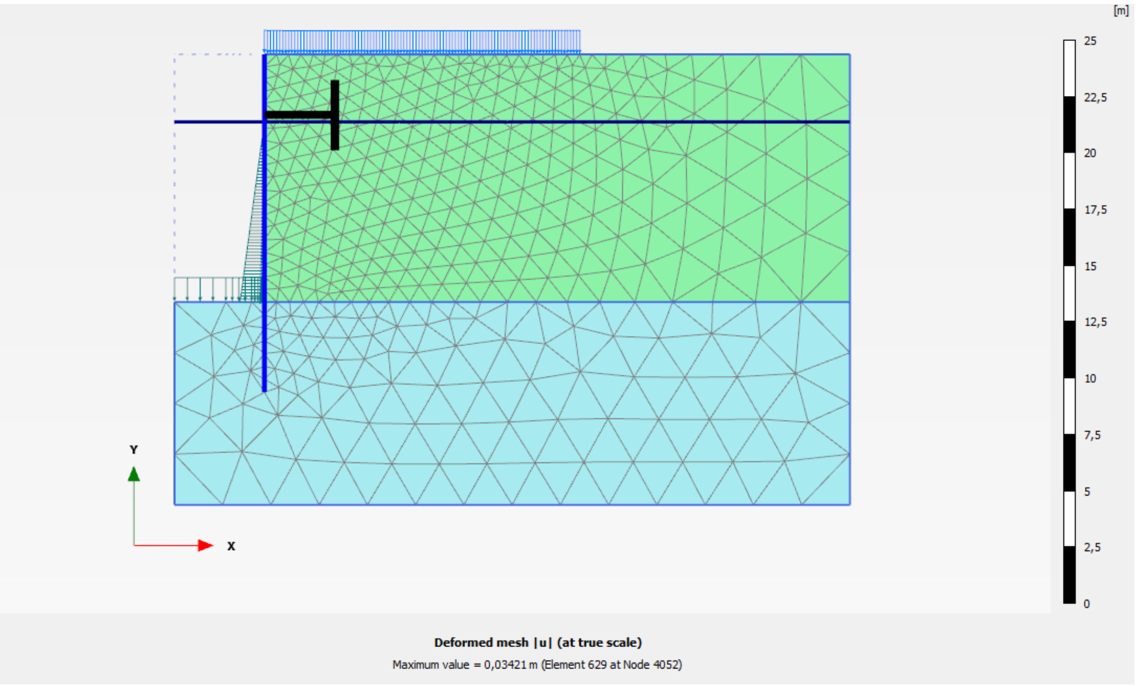


Figure 5. 22 Displacement of the anchored bulkhead under dead load (true scale)

As it clearly seen from the Figure 4.20, deformed displacement under dead load is on applicable range (maximum allowable 6 cm) which shows 3cm of the displacement.

More detailed information regarding to behavior of the anchored bulkhead can be seen from following Figure xx scaled up to 20 times to the original one.

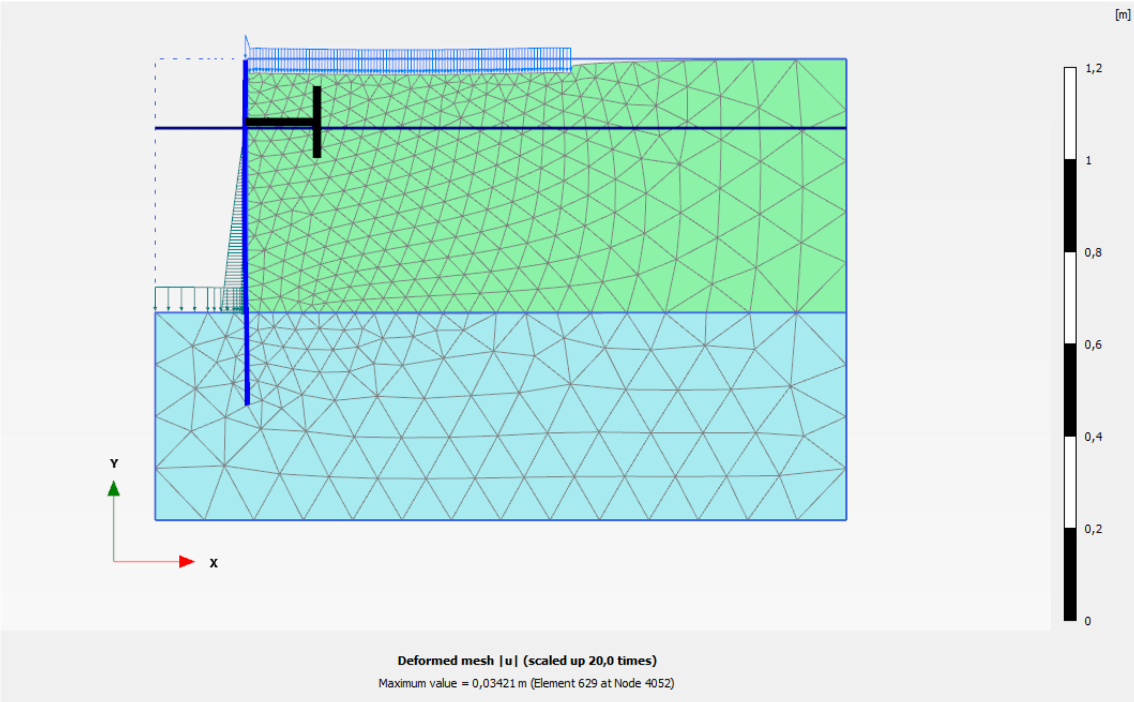


Figure 5. 23 Deformed displacement behavior under dead load of anchored bulkhead (scale 20)

In order to define failure point of the anchored bulkheads, plastic points of the computed design was analysed. Following Figure 4.22 represents outputs of the analysis conducted

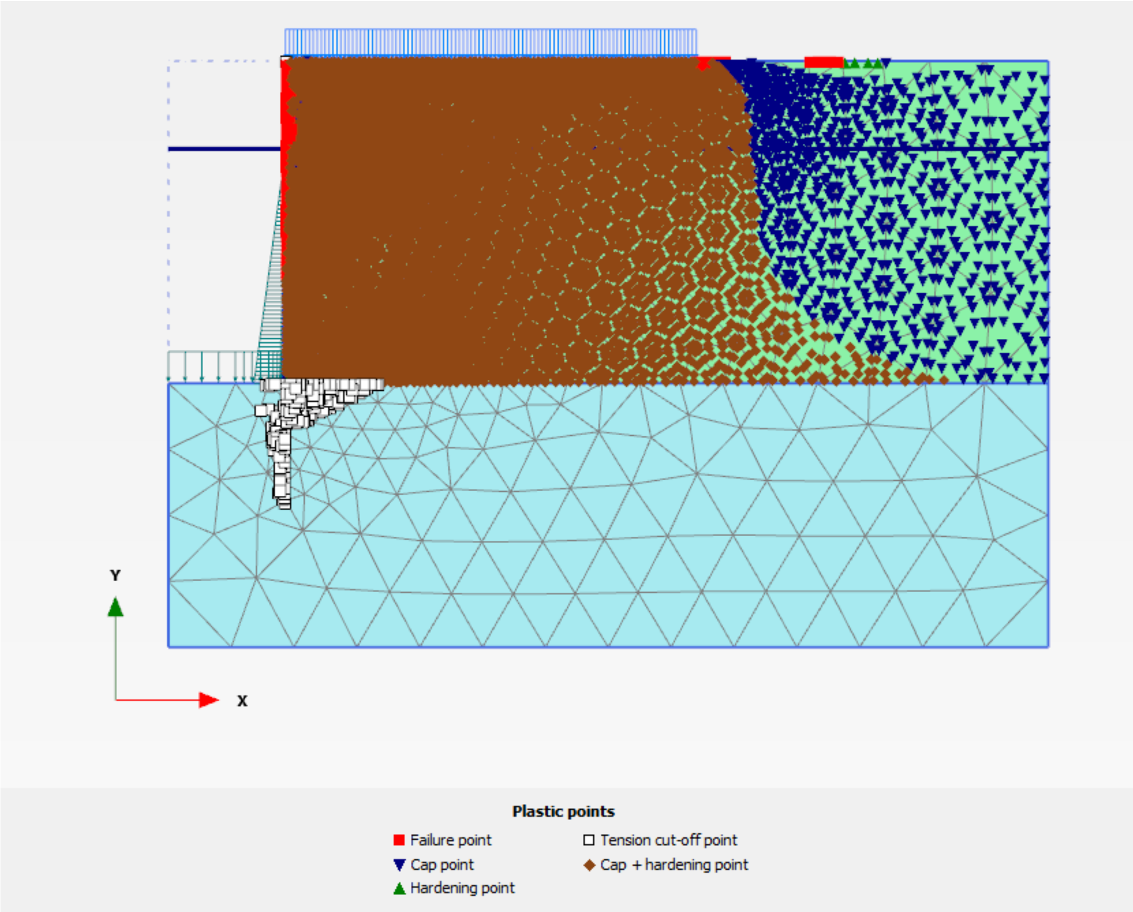
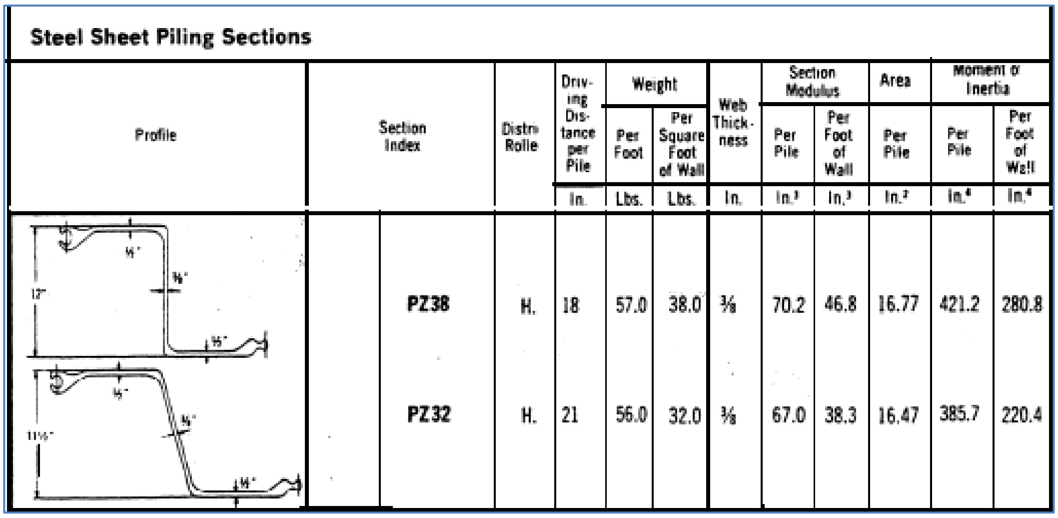


Figure 5. 24 Failure points of the anchored bulkhead.

Therefore, in order to strengthen designed anchored bulkhead, additional reinforcement should be added in failure points. In addition, strength of the deadman anchorage can be added.

5.2.6 Final Flexible Wall Design

According to the calculations and check by Plaxis 2D software following characteristics were developed for construction flexible walls of Peter Pike Canal.



Steel Sheet Piling Sections

Profile	Section Index	Distri Rolle	Driving Distance per Pile	Weight: Per Foot	Weight: Per Square Foot of Wall	Web Thickness	Section Modulus: Per Pile	Section Modulus: Per Foot of Wall	Area: Per Pile	Moment of Inertia: Per Pile	Moment of Inertia: Per Foot of Wall
			In.	Lbs.	Lbs.	In.	In.^{3}	In.^{3}	In.^{2}	In.^{4}	In.^{4}
	PZ38	H.	18	57.0	38.0	⅜	70.2	46.8	16.77	421.2	280.8
	PZ32	H.	21	56.0	32.0	⅜	67.0	38.3	16.47	385.7	220.4

Figure 5. 25Charateristics of PZ 38 and PZ32 Carbon Grade Steel

Table 5. 19 Final Characteristics of sheet pile

Parameter	Value
Height of Sheet Pile	6.5 m
Total length	600 m
Sheet Pile Section	PZ-38 or PZ-32 Regular Carbon Grade Steel
Deadman Anchorage Length	7.6 m
Depth of the Ancrorage Bulkhead	6.1 m

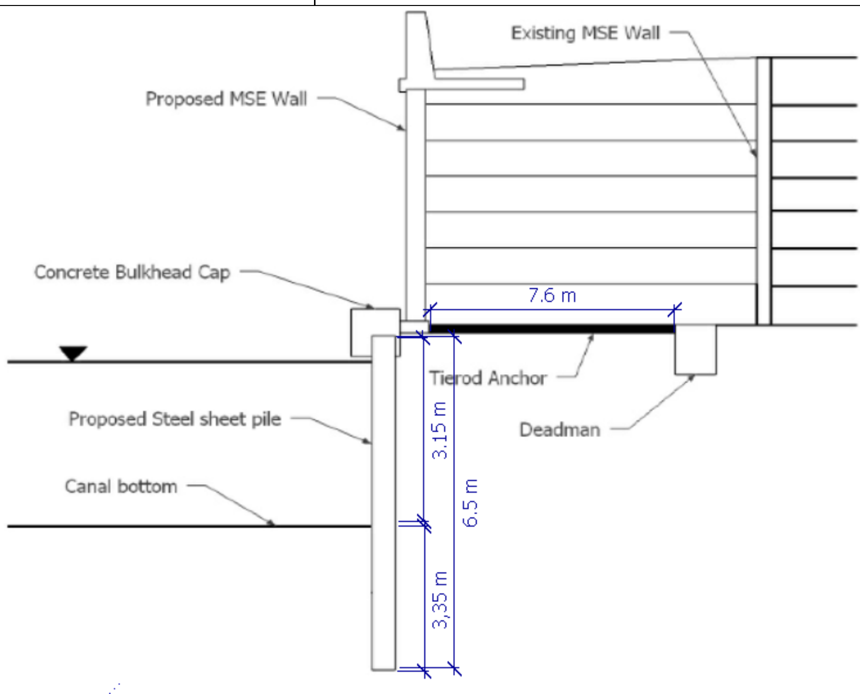


Figure 5. 26Sheet Pile wall design

5.3 Geotechnical analysis and design of Bridge Foundation

5.3.1 Geotechnical analysis of pile foundation

The capacity of pile foundation consists of skin friction on the pile surface and tip resistance at the tip of the pile as shown in the figure 5.27. Ultimate capacity evaluation varies due to the subsurface conditions and soil parameters of the site (see figure 5.28). If there is a rock strata, capacity is equal to tip resistance For weak soils without strong soil layer, tip resistance can be neglected and ultimate capacity of pile will be equal to skin friction resistance. When the pile penetrated into strong soil layer as shown in figure 5.28, both skin friction and tip resistance will used to calculate the ultimate bearing capacity that the pile can carry (Sudheesh 2016).

Ultimate capacity:

$$Q_u = Q_S + Q_P \qquad \text{(XX)}$$

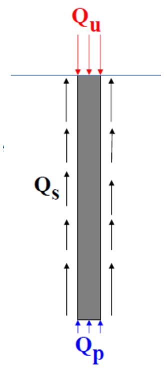


Figure 5. 27Ultimate capacity of concrete pile

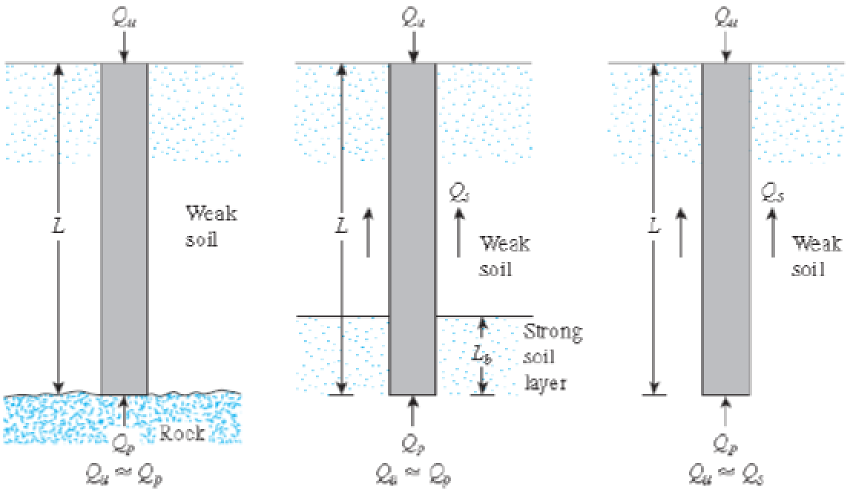


Figure 5. 28Ultimate capacity of concrete pile in different soils

According to the boring data in Appendix D and table 5.2 in Boring data section the site does not have hard rock layer at investigated 21m depth, but from 9.1m the pile penetrates into Limestone. Therefore capacity will be evaluated as compaction pile, where both tip resistance and skin friction resistance is calculated.

Tip resistance or point bearing

Point bearing is calculated by the following formula 5.1.

$$Q_p = A_p q_p = A_p(c'N_c + q'N_q) \tag{5.1}$$

Where,

A_p = area of pile tip

c' = cohesion of the soil supporting the pile tip

q' = effective vertical stress at the level of the pile tip = $\gamma' z$

N_c, N_q= the bearing capacity factors

Skin friction or frictional resistance

Skin resistance is high for driven piles compared to drilled shaft due to the fact that there is pile driving vibration and soil expansion that densify the surrounding soil. The densification may increase up to 2.5 pile diameter (Holtz and Kovacs 2010).

Frictional resistance is calculated by the following formula 5.2.

$$Q_S = \sum p\Delta L f \tag{5.2}$$

Where,

p = perimeter of the pile section

ΔL = incremental pile length over wich p and f are taken to be constant

f = unit friction resistance at any depth z.

Unit friction resistance depends on type of soil. For sand formula 5.3 is used.

$$f = f_{z=L'} = K\sigma'_o \tan\delta' \tag{5.3}$$

Where,

K =effective earth pressure coefficient

σ'_o= effective vertical stress at the depth under consideration, $\sigma'_o = \gamma z$

δ' = soil pile friction angle

L' = Critical depth which is equal to 15 pile diameter

According to Mansur and Hunter report (1970) effective earth pressure coefficient for precast concrete pile is $K=1.5$ and $\delta' = 0.6\emptyset'$.

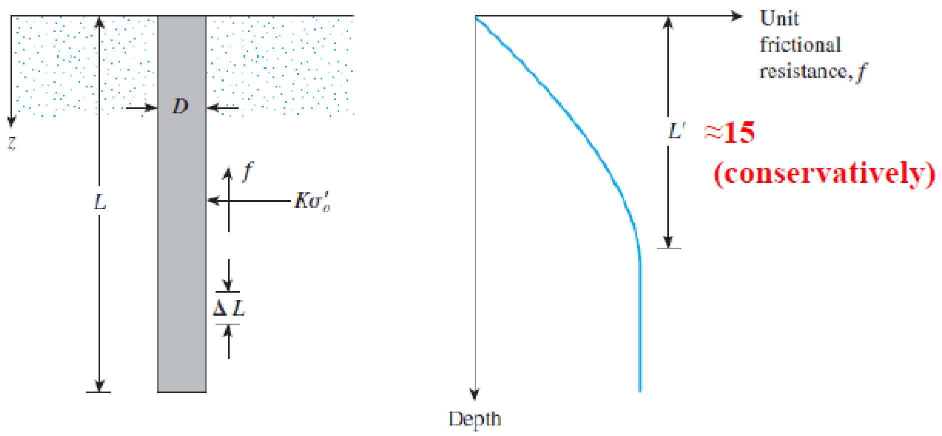


Figure 5. 29 Frictional resistance of pile

Unit frictional resistance for cohesive soil is calculated by the formula 5.4.

$$f = \lambda(\bar{\sigma}'_o + 2c_u) \qquad (5.4)$$

Where,

λ = coefficient (see table 5.20)

$\bar{\sigma}'_o$ = mean effective vertical stress

c_u = mean undrained shear strength

Table 5. 20Coefficient for cohesive soil friction resistance calculation

Embedment length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

Allowable load

Ultimate capacity is not used for the design. The appropriate factor of safety (FS) has to be applies to obtain the allowable loading.

$$Q_{all} = \frac{Q_u}{FS} \tag{5.5}$$

Factor of safety normally ranges from 2.5 to 4.

Negative skin friction

Negative skin friction occurs in two cases. First, when clay soil layer is placed over granular material and the clay gradually consolidates (figure XX, a). Secondly, when granular fill material is placed over clay layer, which results in induced consolidation of clay (figure XX, a). According to borehole data there is no clay layer at site subsurface. Therefore there will be no effect of negative skin friction.

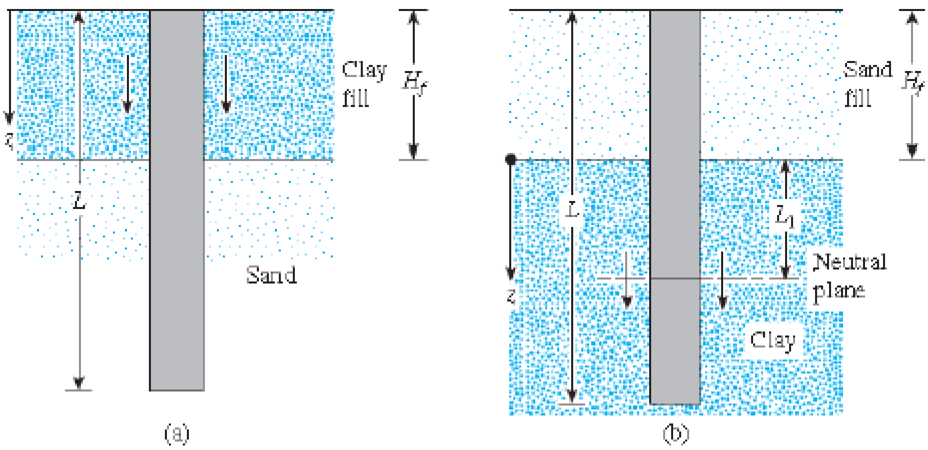


Figure 5. 30 Negative friction on pile due to clay consolidation

Group placement of piles

Piles can be placed in groups of several piles under pile cap. The following requirements have to be followed:

- The distance between piles should be enough to avoid group interaction
- Minimum spacing (d) = 2.5D

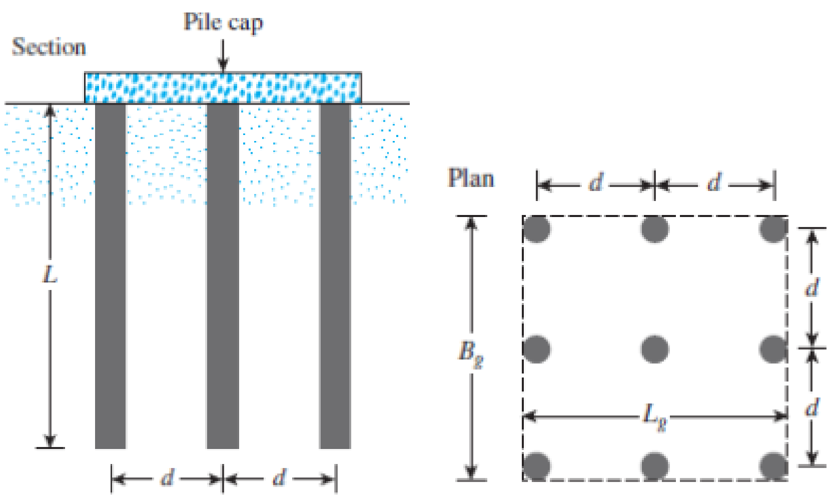


Figure 5. 31Group piles

Group capacity

The ultimate capacity of group piles is evalueted as a block by formula 5.6 using both group skin and tip friction.

$$Q_{g(u)} = Q_{g(S)} + Q_{g(p)} \tag{5.6}$$

Surface frictional resistance for group is estimated by formula 5.6 using group surface:

$$Q_{g(S)} = f\left[2\left(L_g + B_g\right)\right]L \tag{5.6}$$

Tip resistance for group is calculated by formula 5.7 using group footprint area:

$$Q_{g(p)} = (c'N_c + q'N_q)L_g B_g \tag{5.7}$$

Where,

L_g= width of group footprint area

B_g = length of group footprint area

Group efficiency

Group efficiency formula 5.8 is used to check whether piles behave individually or as a block in a group.

$$\eta = \frac{Q_{g(u)}}{\sum Q_u} \leq 1 \tag{5.8}$$

If $Q_{g(u)} > \sum Q_u$, then $\eta = 1$, therefore Piles behave individually

If $Q_{g(u)} < \sum Q_u$, then $\eta < 1$, therefore Piles behave as a block

Elastic settlement of a pile group

To evaluate the elastic settlement of a pile group the following formula 5.9 is used:

$$S_{g(e)} = \frac{qB_g I}{2q_c} \tag{5.9}$$

Where,

q= $Q_g/L_g B_g$

I = influence factor = $1\text{-}L/8 \geq 0.5$

q_c= cone penetration value

5.3.2 Geotechnical design of pile foundation

General notes

Table 5. 21Soil parameters for pile foundation design

Range of Elevation, m.		Material Type	Length, L (m)	ø (degrees)	Cohesion,C (MPa)	Saturated Unit weight, γ_{sat} (kN/m3)
From	To					
-0.3	-9.1	Sand	8.8	30	0	20
-9.1	-12.5	Limestone	2.4	38	20	27

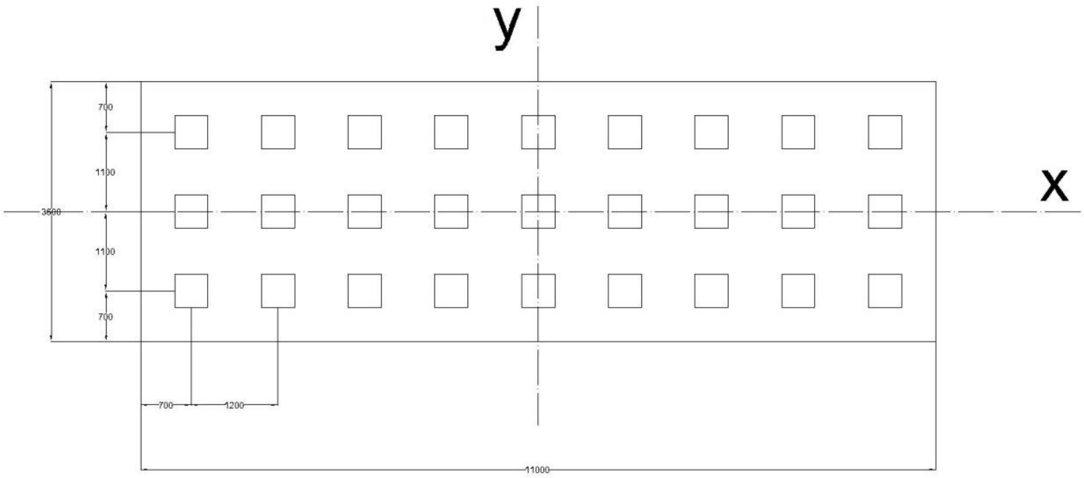


Figure 5. 32 Pile Group horizontal layout

Table 5.22. Concrete pile dimentions

Number of piles	*L, m*	L_g, m	B_g, m	*D, m*
27	12.2	9.6	2.2	0.46

Point bearing for individual pile

From appendix N_c and N_q for 30 degrees friction angle are 77.5 and 64.55, respectively

$$Q_p = A_p q_p = A_p(c'N_c + q'N_q) = 0.46 * 0.46(20000kPa * 77.5 + 27 * 12.2 * 64.55) = 332.5 * 10^3 \text{kN}$$

The concrete pile penetrates into the limestone, which is strong soil layer. This has given significant tip resistance. The obtained resistance is much more greater than the skin friction resistance. Therefore friction resistance can be negleckted and ultimate capacity of the pile can be taken as tip resistance only.

$$Q_u \approx Q_p = 332.5 * 10^3 kN$$

Point bearing for group piles

For bearing capacity of group the same condition applies. There is infignificant friction resistance compared to tip resistance, therefore tip resistance can be taken as ultimate bearing capacity.

$$Q_u \approx Q_{g(p)} = (c'^{N_c} + q'^{N_q})L_g B_g = 2.2 * 9.6(20000kPa * 77.5 + 27 * 12.2 * 64.55) = 33199 * 10^3 \mathrm{kN}$$

Group Efficiency

$$\eta = \frac{Q_{g(u)}}{\sum Q_u} = \frac{33199}{27 * 332.5} = 3.7$$

From the capacity results we have $Q_{g(u)} > \sum Q_u$. Therefore piles in a group behave individually.

Elastic settlement of a pile group

Cone penetration test performend in Florida by University of Florida, Civil Engineering Department gives results of 15Mpa for sand (FDOT Pile shaft design procedures, 2007).The elastic settlement is calculated as follows:

$$q_c = 15Mpa$$

$$q = \frac{Q_g}{L_g B_g} = \frac{33199*10^3}{2.2*9.6} = 1572 * 10^3 kPa$$

$$I = 1\text{-}L/8*B_g = 1\text{-}12.2/8*2.2 = 0.31$$

$$I = 0.5$$

$$S_{g(e)} = \frac{qB_g I}{2q_c} = \frac{1572 * 10^3 * 2.2 * 0.5}{2 * 15000} = 0.05m$$

The pile settles down for for 5cm.

Allowable load

To estimate the allowable load safety factor has to be stated. For the design FS is taken as 3.

$$Q_{all} = \frac{Q_u}{FS} = \frac{33199*10^3}{3} = 11066 * 10^3 kN \tag{5.10}$$

5.4 Geotechnical Analysis and Design of miscellaneous structures

5.4.1 Subsurface Conditions

The subsurface exploration program conducted for the project was particularly for different overhead sign structures (sign and toll gantry) to be placed at various segments within the work zone of the project.

The encountered subsurface soil condition at proposed sign location is generally similar to the findings of the previous subsurface exploration program conducted for retaining walls, MSE walls and bridge foundation. Summarized information can be found in Table 5.22.

Table 5. 22Equivalent Soil Parameter estimation

Soil Type	**Elevation (ft.)**		**Safety N (blows/ft)**	**Corrected N1 (blows/ft)**	**f (deg.)**	**g_{total} (pcf)**	**g_{effective} (pcf)**	**Effective overburden @mid-layer (tsf)**
	From	**to**						
Limerock Fill	5.0	2.0	27.0	49.0	38	115	115.0	0.086
Limerock Fill	2.0	-1.0	27.0	41.8	38	115	52.6	0.212
Limestone	-1.0	-5.0	10.0	13.9	36	120	57.6	0.309
Sand	-5.0	-13.0	7.0	8.9	30	105	42.6	0.452
Sand	-13.0	-18.5	12.0	14.0	32	112	49.6	0.605
Equivalent Soil Parameters				**20.3**	**33.6**		**57.3**	

According to findings, the soil material is limestone ranges in 13-14 blows per foot at the depth between 0.3048 m (1 ft) to 1.55 m (5 ft) . The material underneath of the limestone is represented by sand ranges 8.5 - 15 blows per foot and found in depth between 1.55 m (5 ft) and 3.92 m (13 ft). Based on the review of the boring logs equivalent unit weight was estimated as **917 kg per cubic meter (**57.3 pcf) and degree 33.6.

Input parameters of the drilled shaft of the sign structures and tolling structures can be found in the following Figure XX.

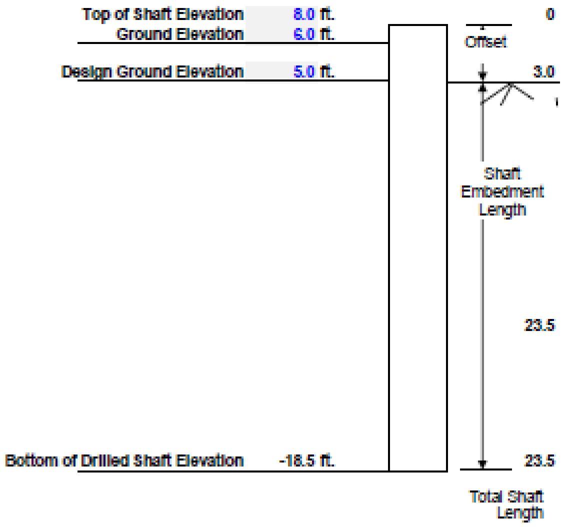


Figure 5. 33Input parameters for the drilled shaft

According to the Figure 5.33, the main inputs of the can be organized in Table 5.23

Table 5. 23 Main input parameters for drilled shaft

Design Ground elevation,	5.0	ft	1.524	m
Water Elevation,	2.0	ft	0.6096	m
Shaft Length Above Groundwater	6.0	ft	1.8288	m
Shaft Length Below Groundwater,	17.5	ft	5.334	m

5.4.2 Drilled Shaft Torsion Check

Following Figure 5.34 Illustrates applied moments and forces to the drilled shaft.

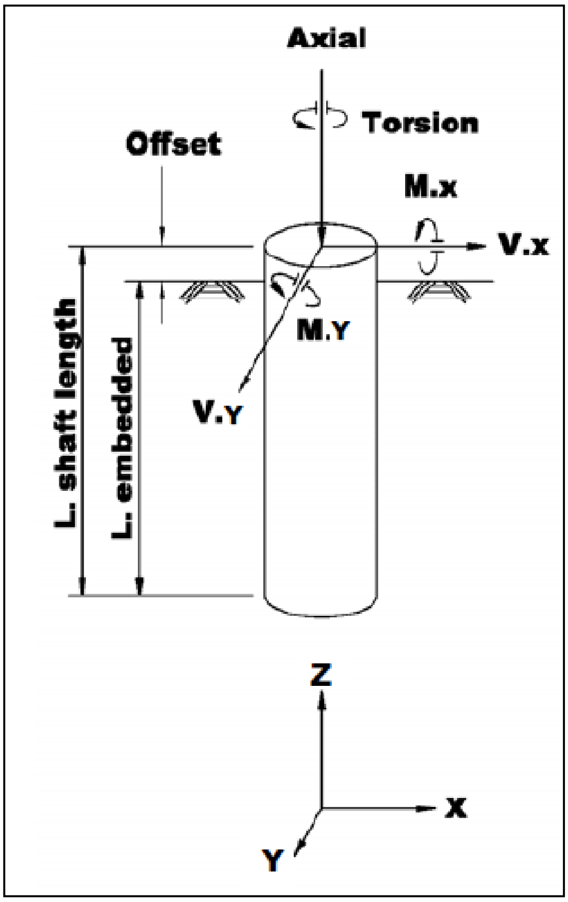


Figure 5. 34 Moments and Forces of the drilled shaft

In order to check torsional resistance of the designed drilled shaft following formulaes should be calculated:

$$Torsion_{Skin} = \pi D L F_s \left(\frac{D}{2}\right) \tag{5.11}$$

$$Torsion_{Tip} = \pi \left(\frac{D}{2}\right)^2 L\gamma_{conc} \left(\frac{D}{3}\right) \mu \tag{5.12}$$

$$Torsion_{Total} = Torsion_{Skin} + Torsion_{Tip} \tag{5.13}$$

Where,

$$F_s = \sigma_v \omega_{fdot} \tag{5.14}$$

$$\sigma_v = \gamma_{soil} \left(\frac{L}{2}\right) \tag{5.15}$$

Following parameters illustrated in Tables were obtained during geotechnical investigation. For Cohesionless soil:

Table 5.24 Soil Parameters

f	33.6	degrees		
m = tan f	0.66			

Diameter (ft)(m)	5	ft	1.55	m
w_{fdot}	1.50			
Gamma)	0.057	kips/ft^3	9.00	kN/m3
Torsional Load	658.75	kip-ft	9613.8	kN/m
Axial Load	12.54	kips	55.77	kN
Guess Length	23.5	ft	7.2	m

Using formulaes given before calculated Safety Factor for Torsion resistance is equal to 1.5 which is satisfactory for the construction (1.3 – minimal requirement value). More detailed calculations can be found in Appendix G.

5.4.3 Lateral Response of Drilled Shafts

There are several methods for analysis of laterally loaded drilled shafts and all of them can be subdivided into three main categories: the elastic theory based approach, the discrete and independent spring based approach, and the finite element based continuum approach. Besides, those methods can also be evaluated on the basis of the ability of the analysis to provide a complete load-deflection solution or only the ultimate capacity solution. Due to the fact, that ultimate capacity estimation necessary for geotechnical investigation, focus was adapted only for methods that apply following characteristic, namely Broms method.

Broms Method

The main utility that uses Broms method is to consider drilled shaft as a beam on an elastic foundation. Therefore, simplification of assumptions can be adopted regarding the ultimate soil reactions along the length of a pile. The Broms method is focused on two boundary conditions: a free pile head and a restrained shaft head. Also, the Broms method can handle not only short drilled shafts (piles), but also long drilled shafts (piles). This method however is only suitable for homogeneous soil, which would be either cohesive soils or cohesionless soils

Calculations

The main formula represented by Broms Method is the estimation of ultimate lateral resistance.

$$P_{ult} = \frac{K_p \gamma D L^3}{2(e+L)} \tag{5.16}$$

Following Table 5.24 represent input parameters used in the calculation

Table 5. 24Input parameters

Offset =	0.0	ft	0	m
Friction Angle =	33.6	degrees		
Moment (x-direction) =	679.5	kip - ft	921.402	kNm
Moment (y-direction) =	256.5	kip - ft	347.814	kNm
Shear (x-direction) =	27.86	kips	123.921	kN
Shear (y-direction) =	5.572	kips	24.7843	kN
Shaft Diameter =	5	feet	1.524	m
Soil Unit Weight =	0.057	kcf	918.002	kg/m3
Pile Embedment Length =	23.5	feet	7.1628	m

Based on those input parameter, several calculations were performed. Following table 5.25 represents the results of those calculations. More detailed information can be found in Appendix G.

Table 5. 25Resultants of calculations

Resultant Moment =	726.3	kip-ft	984.864	**kNm**
Resultant Shear =	28.4	kips	126.375	**kN**
sin (phi) =	0.55			
Kp =	3.47			
Equivalent "e" for applied load =	25.6	ft	7.79172	m
Total "e" = Equivalent "e" + Offset	25.6	ft	7.79172	m
Ultimate Lateral Resistance =	131.7	kips	585.69	kN

Therefore, defined Safety Factor is equal to 4.6 which is within safety zone (minimum Safety Factor required is 2.0)

5.4.4 Axial Capacity

Axial capacity of a single drilled shaft is estimated using the soil strength parameters presented in Table 5.25. Results of the axial pile analysis are included in Appendix G. Based on the results of the analysis, the foundation depth of the designed overhead sign structure is enough to resist the axial demands provided by structural engineer.

5.5 Geotechnical stability analysis of existing structures subsequent to the new construction activities

Construction of the new MSE walls and bridge near existing structures requires analysis of geotechnical stability of existing structures. Along the alignment of the proposed expressway that connects SR826 and I75 there is no vertical buildings. The only structures nearby are existing bridges. To construct the MSE wall embankment new fill will be delivered to the site. Additional fill may cause settlement of soil of the adjacent bridges piers. The settlement of the soil near existing bridge foundation will result in additional loading due to the negative skin friction.

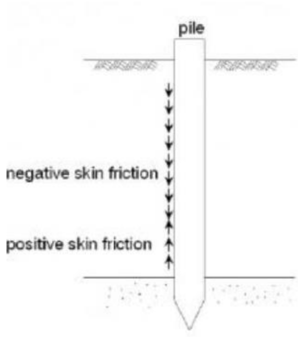


Figure 5. 35 Negative skin friction on pile

5.5.1 Negative Skin Friction

Negative skin friction occurs when the soil is soft, consolidated or there is a settlement of soil due to an external loadings. This creates downward force that exerts an increased loading on the piles as shown in figure 5.35. As a result additional loading decreases the bearing capacity of the piles. Negative skin friction may also occur when the rate of soil settlement is greater than the pile settlement (Tran and Nguyen, 2003).

5.5.2 Existing Bridges Foundation stability analysis

In order to check the stability of existing bridges against negative friction, settlement of soil due to MSE wall construction has to be evaluated. To measure the amount of settlement computer program simulation is used, namely Plaxis 3D.

Simulation was build using soil parameters provided in boring data section. MSE block was implemented for the simulation as uniform load indicated with blue meshed lines at the center of investigated ground part (see Figure 5.36). The figure illustrates Plaxis 3D simulation results for foundation soil condition under new constructed MSE wall block. The regions with the highest settlement are indicated by red color. According to figure the settlement level decreases as the depth increases. Also, from the figure the maximum soil settlement is 0.029 m which is under the MSE walls block.

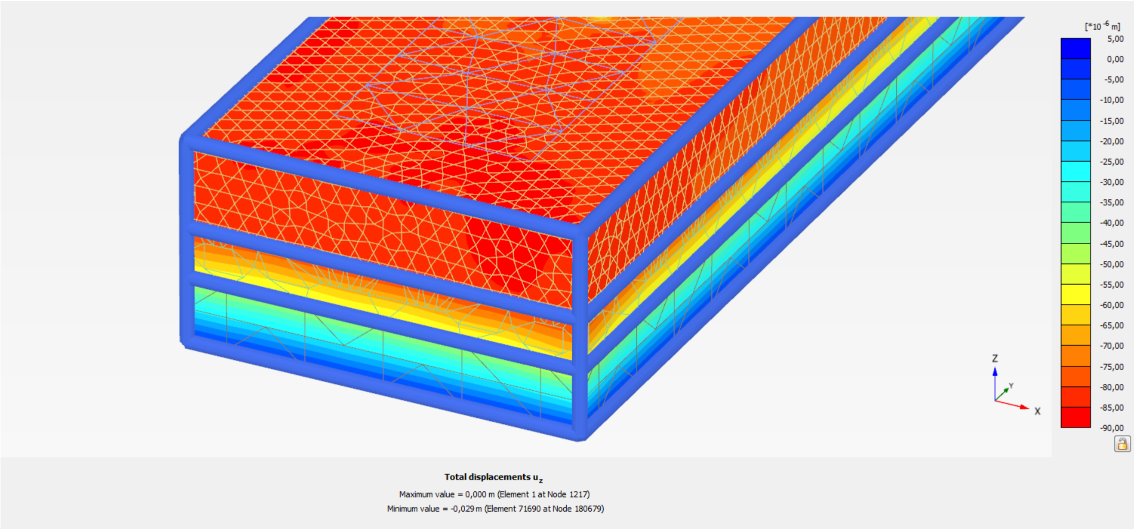


Figure 5. 36 Deformed foundation soil after MSE wall construction

Usually settlement takes place in soft soils, consolidation soil mass. The soil of the project location is mainly sand and limestone, and its condition is quite good according to table 5.2. This was verified by the simulation results. The settlement of 0.029m at critical point is insignificant for the project and lies within allowable range stated by AASHTO LRFD. The settlement decreases as it goes away from MSE walls block. Therefore, it can be concluded that construction of new MSE walls will create almost no affect to nearby existing structures.

CHAPTER 6: ROADWAY PAVEMENT ANALYSIS AND DESIGN

6.1 Roadway Pavement Analysis

The pavement structure is the system of pavement layers that support the traffic loads and transfers them to the roadbed soil. The concept is the “top-down” order as pyramid, starting from stronger to weaker layers as illustrated in figure 6.1.

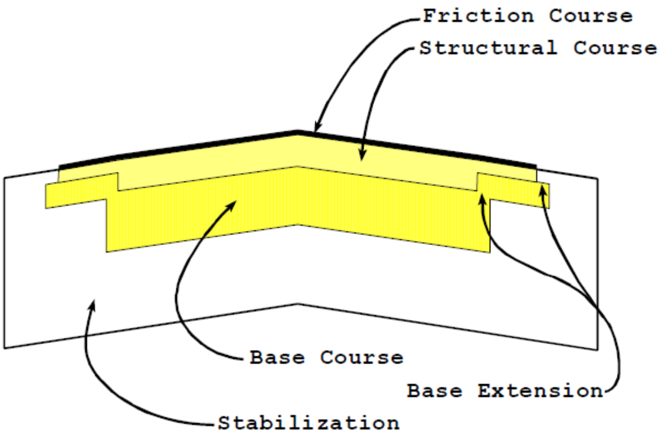


Figure 6. 1Roadway Typical Section

The pavement design procedures are conducted following Florida Flexible Pavement Design Manual 2016 which is based on AASHTO Guide for Design of Pavement Structures 1993 and Federal Highway Administration publications (FHWA).

Friction course

Friction course – is the first pavement layer which is designed to provide skid resistant surface. Florida Department uses the following friction course types:

- **FC-12.5 -** dense graded mix, typical thickness is 38 mm (1.5 inches)
- **FC-9.5 -** dense graded mix, typical thickness is 25 mm (1 inch)
- **FC-5 -** open graded mix, typical thickness is 19 mm (0.75 inches)

FC-12.5 and FC-9.5 friction courses provide smooth rideing surface with medium friction number while FC-5 provides the most effective skid resistance with high friction number. The open graded mixture provides rapid removal of water between tire and pavement and decreases the potential of hydroplaning. No any other layer should be placed onto FC-5.

The friction course should be applied to all projects with design speed of greater than 35 mph (56 km/h) selected based on design speed and number of lanes as shown is table 6.1.

Table 6. 1 Required Friction Courses for design speed of 35 mph (56 km/h) or greater

Design speed	**Two lane**	**Multilane**
35-45 mph (56-72km/h)	FC-12.5 or FC-9.5	FC-12.5 or FC-9.5
50 mph (80km/h) or greater	FC-12.5 or FC-9.5	FC-5

Structural course

Structural course transfers and distributed traffic load to the base course. Florida Department uses the following friction course types:

- Structural Course **Type SP-9.5**
- Structural Course **Type SP-12.5**
- Structural Course **Type SP-19**

AASHTO nominal maximum aggregate sizes and consistent thickness ranges for the structural courses are provided in table 6.2.

Table 6. 2Aggregate sizes and thickness ranges for the structural courses

Type mix	**Aggregate size, mm**	**Thickness range, inch(mm)**	
		Min	**Max**
Type SP-9.5	9.5	1 (25)	1.5 (38)
Type SP-12.5	12.5	1.5 (38)	2.5 (64)
Type SP-19	19	2 (51)	4 (102)

Moreover there are some restrictions for the application of structural courses as follows (Flexible Pavement Design Manual 2016):

SP-9.5 Limited to the top two structural layers, two layers maximum.

SP-9.5 May not be used on Traffic Level D and E applications.

SP-19.0 May not be used in the final (top) structural layer below FC-5 mixtures. Type SP-19.0 mixtures are permissible in the layer directly below FC-9.5 and FC-12.5 mixtures.

Table 6.3 provided by Florida Flexible Pavement Design Manual shows the different combinations of the structural courses to meet the design thickness considering above mentioned restrictions.

Table 6. 3 Layer thickness for asphalt concrete structural courses

Course Thickness (in)	LAYER THICKNESS (inches)																		
	SP-19.0 with SP-12.5 Top Layer			SP-19.0 with SP-9.5 Top Layer			SP-12.5			SP-12.5 with SP-9.5 Top Layer			SP-9.5		SP-19.0 1st Layer with SP-12.5 2nd Layer and Top Layer			SP-12.5 1st Layer with SP-9.5 2nd Layer and Top Layer		
	1	2	3	1	2	3	1	2		1	2	3	1	2	1	2	3	1	2	3
1													1							
1½							1½						1½							
2							2						1	1						
2½							2½			1½	1		1½	1						
3				2	1		1½	1½		2	1		1½	1½						
3½	2	1½		2½	1		2	1½		2	1½									
										2½	1									
4	2½	1½		3	1		2	2		2½	1½									
	2	2		2½	1½		2½	1½												
4½	2½	2		3	1½		2½	2							1½	1½	1½	2	1½	1
	2	2½																		
5	3	2		2	2	1	2½	2½		2	1½	1½			2	1½	1½	2	1½	1½
	2½	2½					2	1½		2	2	1						2½	1½	1
5½	2	2	1½	2½	2	1	2½	1½	1½	2	2	1½			2½	1½	1½	2½	1½	1½
							2	2	1½	2½	2	1			2	2	1½			
6	2½	2	1½	2½	2½	1	2	2	2	2½	2½	1			2½	2	1½			
	2	2	2	3	2	1	2½	2	1½	2½	2	1½			2	2	2			

Base course

Base course is designed to support the structural course layers and distributed the traffic loads to subbase layer. The course may have different material types and different layer thicknesses but with equivalent structural numbers grouped as optional base course. Table 6.4 contains optional base course groups for different structural numbers that are used by Florida Department of Transportation.

Type B-12.5 asphalt base course are commonly used when there is no possibility for the stabilized subgrade layer and the site has bad groundwater level. It can be used either with or without the combination of subbase layer. Nevertheless, they give the same Structural Number.

Table 6. 4Optional Base Groups and Structural Numbers

BASE THICKNESS AND OPTION CODES											
Base Group	Structural Range	Base Group Pay Item Number	Base Options								
			Limerock, LBR 100	Cemented Coquina, LBR 100	Shell Rock, LBR 100	Bank Run Shell, LBR 100	Recycled Concrete Aggregate, LBR 150 **	Graded Aggregate Base, LBR 100	Type B-12.5	B-12.5 And 4" Granular Subbase, LBR 100 *	RAP Base
			Structural Number (Per. in.)								
			(0.18)	(0.18)	(0.18)	(0.18)	(0.18)	(0.15)	(0.30)	(0.30 & 0.15)	(NA)
1	0.65-0.75	701	4"	4"	4"	4"	4"	4½"	△ 4"		□ 5"
2	0.80-0.90	702	5"	5"	5"	5"	5"	5½"	△ 4"		
3	0.95-1.05	703	5½"	5½"	5½"	5½"	5½"	6½"	△ 4"		
4	1.05-1.15	704	6"	6"	6"	6"	6"	7½"	△ 4"		
5	1.25-1.35	705	7"	7"	7"	7"	7"	8½"	4½"		
6	1.35-1.50	706	8"	8"	8"	8"	8"	9"	5"		
7	1.50-1.65	707	8½"	8½"	8½"	8½"	8½"	10"	5½"		
8	1.65-1.75	708	9½"	9½"	9½"	9½"	9½"	11"	5½"		
9	1.75-1.85	709	10"	10"	10"	10"	10"	12"	6"	4"	
10	1.90-2.00	710	11"	11"	11"	11"	11"	Ø 13"	6½"	4½"	
11	2.05-2.15	711	12"	12"	12"	12"	12"	Ø 14"	7"	5"	
12	2.20-2.30	712	12½"	12½"	12½"	12½"	12½"		7½"	5½"	
13	2.35-2.45	713	Ø 13½"	Ø 13½"	Ø 13½"	Ø 13½"	Ø 13½"		8"	6"	
14	2.45-2.55	714	Ø 14"	Ø 14"	Ø 14"	Ø 14"	Ø 14"		8½"	6½"	
15	2.60-2.70	715							9"	7"	

Subbase

Subbase course is the layer with specified thickness and designed to support base course and distribute the traffic loads to roadbed soil. Subbasse course is directly related and limited to base sourse type.

Stabilized Subgrade

Stabilized subbase is the layer that serves as working platform for subbase and base. It usually has the thickness of 12 inch and bid as Type B Stabilization with LBR 40. The advantages of Stabilized Subgrade layer is that it provides additional strength to the pavement system at low cost. However, in some projects stabilized subgrade to technology cannot be used due to several conditions and requires achieving the structural strength with thicker layers of base and structural courses. The conditions are as follows:

- Limited working area
- Urban areas where construction time is critical and due to limit to the adjacent buildings and businesses.

6.2 Roadway Pavement design

The thickness of pavement layers is determined using AASHTO Procedure. In order to define thickness of layers Structural Numbers (SN) are to be calculated. FDOT Flexible Pavement Design Manual provides systematic tables to determine the SN of a pavement based on the AASHTO design equation for the flexible pavement. The tables are in Appendix H. Department uses ΔPSI=1.7 for serviceability limit.

FDOT Flexible Pavement Design Manual provides steps for SN determination with the following input parameters (2016):

- Determine 18-kip Equivalent Single Axle Loads (ESAL's). It can be obtained from Project Traffic Forecasting Handbook (2012). If not provided it is calculated using AADT and other input parameters.
- Determine Resilient Modules (M_R) which is used to describe the strength of the roadbed soil. For low volume roadways, the design Limerock Bearing Ratio (LBR) value can be used to calculate M_R.
- Determine the Reliability (%R) value which characterizes the safety factor. Recommended values ranges between 75% to 99%. For calculation a Standard Deviation (S_o) of 0.45 will be used and Standard Normal Deviate (Z_R) depends on Reliability.

Using this input parameters SN for the layer is defined from the appendix H which is further used to determine the course layer thicknesses.

Step 1. Estimation the design ESAL and Traffic Level identification

$ESAL_D$ for the required pavement can be calculated by formula 6.1.

$$ESAL_D = \sum_{y=1}^{y=x}(AADT * T_{24} * D_F * L_F * E_{18} * 365) \tag{6.1}$$

Here,

$AADT$ – average annual daily traffic;

x – The Design year

T_{24} – percent of heavy truck during 24 hour. Since the heavy trucks are most critical for the factor for the pavement, the estimation for the future truck volume is important. Usually 6 more tired trucks are considered;

D_F – Directional distribution factor. For one way traffic the value is 1, for two way traffic it is 0.5.

L_F – Lane factor. It converts directional trucks to lane trucks, depends on number of lanes and AADT. Table 6.5 gives different values of lane factor according to type of road and AADT.

E_{18} – Equivalency factor. It measures how much damage an average heavy truck can cause in terms of 18 kip ESAL. The equivalency factor can be determined from table 6.6.

Table 6. 5Lane Factors (L_F) for different types of roadways

Total AADT	Number of Lanes In One Direction: Two Lanes L_F	Three Lanes L_F
4 000	0.94	0.82
8 000	0.88	0.76
12 000	0.85	0.72
16 000	0.82	0.70
20 000	0.81	0.68
30 000	0.77	0.65
40 000	0.75	0.63
50 000	0.73	0.61
60 000	0.72	0.59
70 000	0.70	0.58
80 000	0.69	0.57
100 000	0.67	0.55
120 000	0.66	0.53
140 000	-	0.52
160 000	-	0.51
200 000	-	0.49

Table 6. 6 Equivalency Factors (E_{18}) for different types of roadways

	Flexible Pavement	Rigid Pavement
Freeways		
Rural	1.05	1.60
Urban	0.90	1.27
Arterials and Collectors		
Rural	0.96	1.35
Urban	0.89	1.22

First input parameter to be determined is AADT. The data is available from FDOT Florida Traffic Online service (2015). From this official service traffic report about particular roadway can be derived, namely AADT and truck factor, T (figure 6.2).

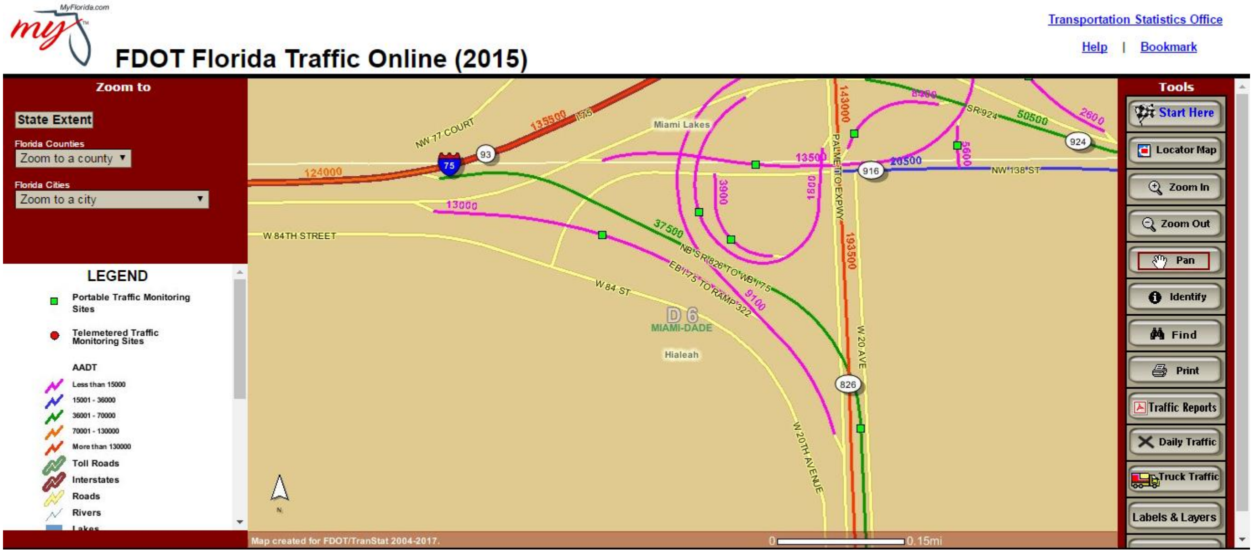


Figure 6. 2 FDOT Florida Traffic Online Service.

For calculation of ESAL design year has to be determined. According to AASHTO design period for the new constructed or fully reconstructed flexible pavement is 20 years (1993). Other guidelines for the design period determination are provided in Table 6.7.

Table 6. 7Design periods for flexible pavements (AASHTO 1993)

New Construction or Reconstruction	20 Years
Pavement Overlay without Milling	8 to 20 Years
Pavement Overlay with Milling	
Limited Access	12 to 20 Years*
Non-Limited Access	14 to 20 Years*
Pavement Overlay of Rigid Pavement	8 to 12 Years

After calculation of ESAL Traffic level is identified based on its value. Table 6.8 provides the relationship between ESAL and Traffic level.

Table 6. 8 Traffic Equivalent for the Design ESAL ranges for asphalt concrete structural courses.

AASHTO DESIGN $ESAL_D$ RANGE (MILLION)	TRAFFIC LEVEL
< 0.3	A
0.3 to < 3	B
3 to < 10	C
10 to < 30	D
>= 30	E

Step 2. Resilient modules estimation

Resilient module, M_R is calculated from Design LBR Values by formula 6.2.

$$M_R(PSI) = 10^{[0.7365*\log(LBR0)]} * 809 \qquad (6.2)$$

If the design LBR and MR values are not available from FDOT District Materials office Pavement Design Engineer may select LBR value not exceeding LBR 40 value (Flexible Pavement Design Manual 2016). According to manual the most common LBR values is around LBR 22 for the district.

Step 3. Reliability Value (%R) determination

Reliability value is determined depending on roadway facility type and its condition. Different reliability values are shown in table 6.9.

Table 6. 9 Reliability Values (%R) for different roadway facilities

Facility	New	Rehabilitation
Limited Access	80 - 95	95 - 99
Urban Arterials	80 - 90	90 - 97
Rural Arterials	75 - 90	90 - 95
Collectors	75 - 85	90 - 95

Step 4. Structural Number S_{NR} determination

With the parameters obtained from step 1-3, Structural Number is determined from design in appendix H.

Step 5. Layer thicknesses calculation

With known SN_R value individual layer thicknesses are calculated by formula 6.3.

$$SN_R = a_1D_1 + a_2D_2 + a_3D_3 + a_4D_4 \qquad (6.3)$$

Where,

a – layer coefficient

D –layer thickness

Layer 1 – friction course

Layer 2– structural course

Layer 3 – base course

Layer 4 – stabilized subgrade

Layer coefficients are used to represent relative strengths of different pavement materials. FDOT provides table 6.10 for coefficients used for each layer depending on the material type.

Firstly, friction layer and stabilized subgrade will be determined based on design speed and pavement type. Every friction course type has its own thickness given in table 6.1. Structural course and base course group combination depending on structural number is determined from table 6.11. Further, detailed thickness values are selected for each of layers from tables 6.3 and 6.4. For the design the thickness values are rounded to the nearest ½ inch.

Table 6. 10 Structural coefficients for different pavement layers, new constructed or reconstructed.

Layer Type	Layer Coeff. per inch	Spec. Sec.
FC-5	0.00	337
FC-12.5, FC-9.5	0.44	337
Superpave Type SP (SP-9.5, SP-12.5, SP-19.0)	0.44	334
Limerock (LBR 100)	0.18	200
Cemented Coquina (LBR 100)	0.18	911
Shell Rock (LBR 100)	0.18	200
Bank Run Shell (LBR 100)	0.18	200
Graded Aggregate (LBR 100)	0.15	204
Recycled Concrete Aggregate (LBR 150)	0.18	911
Type B-12.5	0.30	234
Limerock Stab. (LBR 70)	0.12	230
Shell Stab. (LBR 70)	0.10	
Sand Clay (LBR 75)	0.12	
Soil Cement (500 psi)	0.20	
Soil Cement (300 psi)	0.15	
Type B Stab. (LBR 40)	0.08	
Type B Stab. (LBR 30)	0.06	
Type C Stab.	0.06	
Cement Treated (300 psi)	0.12	
Lime Treated	0.08	

Table 6. 11Combined Structural Numbers for structural course and base course

Optional Base	Structural Course - Inches										
	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
1	1.12	1.38									
2	1.34	1.56									
3	1.43	1.65	1.87								
4	1.52	1.74	1.96	2.18							
5	1.70	1.92	2.14	2.36	2.58						
6	1.88	2.10	2.32	2.54	2.76	2.98					
7		2.16	2.41	2.63	2.85	3.04					
8		2.37	2.59	2.81	3.03	3.25	3.47	3.69			
9				2.90	3.12	3.34	3.56	3.78			
10					3.30	3.52	3.74	3.96	4.18		
11					3.48	3.70	3.92	4.14	4.36	4.58	
12						3.79	4.01	4.23	4.45	4.67	4.89
13							4.19	4.41	4.63	4.85	5.07
14							4.28	4.50	4.72	4.94	5.16
15							4.46	4.68	4.90	5.12	5.34

Step 6. Checking layer thicknesses and base group against required minimums

While selecting the thickness values and base group the minimum requirements for the pavement type provided in table 6.12 must be checked.

Table 6. 12 Required minimum pavement layer thicknesses for new pavement construction or reconstruction

18-kip ESAL's 20 year period	Minimum Structural Course	Minimum Base Group
Limited Access	4"	9
Greater than 3,500,000	3"	9
Ramp less than 3,500,000	2"	9
300,000 to 3,500,000	2"	6
Less than 300,000	$1\,^{1}/_{2}$"	3
Limited Access Shoulder	$1\,^{1}/_{2}$"	1
Residential Streets, Parking Areas, Shoulder Pavement, Bike Paths	1"	1
Shared Use Paths	$1\,^{1}/_{2}$"	1

6.2.1 Pavement Design Calculations

General Pavement Parameters:

Flexible Pavement for the Expressway connecting SR826 and I75

Condition:	New
Type:	Freeway
Access:	Limited access
Number of lanes:	4
Number of directions:	2
Design Speed	70mph (112km/h)

Step 1. Estimation the design ESAL and Traffic Level identification

$$ESAL_D = \sum_{y=1}^{y=x}(AADT * T_{24} * D_F * L_F * E_{18} * 365)$$

AADT and T_{24} are obtained from FDOT Florida Traffic Online. AADT is predicted to increase on average by 500 each year during design period (FDOT Project Traffic Forecasting Handbook, 2014).

Site Information	
Feature	**1**
Road Name	NB SR 826 TO WB I 75
Site	876085
Description	RAMP 87075616 FROM NB SR826 TO NB I-75, 900' N OF SR 826
Section	87075616
Milepoint	0.17
AADT	**37500**
Site Type	Portable
Class Data	No
K Factor	9
D Factor	99.9
T Factor	6.2
TRAFFIC REPORTS (provided in format)	
Miami-Dade County	**Annual Average Daily Traffic**
	Historical AADT Data
	No Synopsis Report Available

Figure 6. 3Traffic report for SR826-I75 roadway

From SR826-I75 roadway traffic report AADT=37500 and T_{24}=6.5% (Figure 6.3).

Design period for the new construction is 20 years according to table 6.7.

For the two Lane in one direction with AADT of 37500 Lane factor L_F is 0.75 (Table 6.5).

Equivalency factor for Urban Freeway with flexible pavement is 0.9 (Table 6.6).

Table 6. 13 ESAL calculation for design period of 20 years with predicted AADT increase.

Year	**AADT**	**Annual ESAL**	**Accumulated ESAL**
1	37500	286411	286411
2	38000	290230	576641
3	38500	294049	870689
4	39000	297867	1168557
5	39500	301686	1470243
6	40000	305505	1775748
7	40500	309324	2085072
8	41000	313143	2398214
9	41500	316961	2715176
10	42000	320780	3035956
11	42500	324599	3360555
12	43000	328418	3688973
13	43500	332237	4021210
14	44000	336056	4357265
15	44500	339874	4697139
16	45000	343693	5040833
17	45500	347512	5388344
18	46000	351331	5739675
19	46500	355150	6094825
20	47000	358968	6453793

From Table 6. $ESAL_{20}$ is 6453793. This value is rounded to ESAL=6500000 for simplicity.

ESAL=6500000 corresponds to Traffic Level C according to table 6.8.

Step 2. Resilient modules, M_R estimation

Resilient module M_R with Design LBR 22 is calculated as follows by formula 6.4:

$$M_R(PSI) = 10^{[0.7365*\log(22)]} * 809 = 7882\, psi$$

Obtained value is rounded up to M_R= 8000 psi (55160 kPa)

Step 3. Reliability Value (%R) determination

From table the reliability value, %R is within the range of 80-95% for New Limited Access roadway. Reliability of 90% is selected for the pavement design.

Step 4. Structural Number SN_R determination

Input parameters:

%R=90%

M_R= 8000 psi

$ESAL_{20}$=6500000

With this parameters the SN_R for the pavement is 4.58 from Appendix H.

Step 5. Layer thicknesses calculation

$$SN_R = a_1D_1 + a_2D_2 + a_3D_3 + a_4D_4 \qquad (6.5)$$

According to table 6.1, for design speed of 70mph (112km/h) and multilane roadway, FC-5 friction course must be used. The thickness of FC-5 is 0.75inch (38mm) but it has no structural strength according to table 6.2. Usually with FC-5 stabilized subgrade (layer 4) is practiced. However, due to limitations during construction stabilized subgrade will be eliminated. Reasons are described in section 4.3. Using the formula the following is obtained:

$$4.58 = 0 * 0.75 + a_2D_2 + a_3D_3$$

$$4.58 = a_2D_2 + a_3D_3$$

Structural course and base course has a SN value of 4.58. With this value the thickness of structural course and optional base group are 5.5inch (140mm) and group 1, respectively (table 6.4).

For the detail design of Structural course we refer to the table 6.3. There are several options with different combinations of structural course types. For 5.5 inch SP-19, SP-19 and top SP-12.5 layers combination was selected, with thicknesses of 2 inch, 2 inch and 1.5 inch, respectively.

For the base course base courses other than TypeB-12.5, which is asphalt base, cannot be used due to stabilized subgrade elimination. From table 6.3, 5 inch (127mm) Type B-12.5 was selected with combination of 4 inch Granular Subbase LBR 100, which corresponds to the base group 11.

Step 6. Checking layer thicknesses and base group against required minimums

According to table 6.12 minimum structural course and minimum base group for the limited access new pavement construction are 4inch and group 9, respectively. From this it can be stated that proposed design pavement thicknesses satisfies the required minimums.

6.2.2 Finalized pavement design

Table 6. 14 Flexible pavement layer thicknesses

	Course name	**Thickness**	
		inches	**mm**
Friction course	FC-5	0.75	20
Structural course	SP-12.5	1.5	40
	SP-19	2	50
	SP-19	2	50
Base course	Type B-12.5	5	130
Subbase course	Granular Subbase LBR 100	4	100

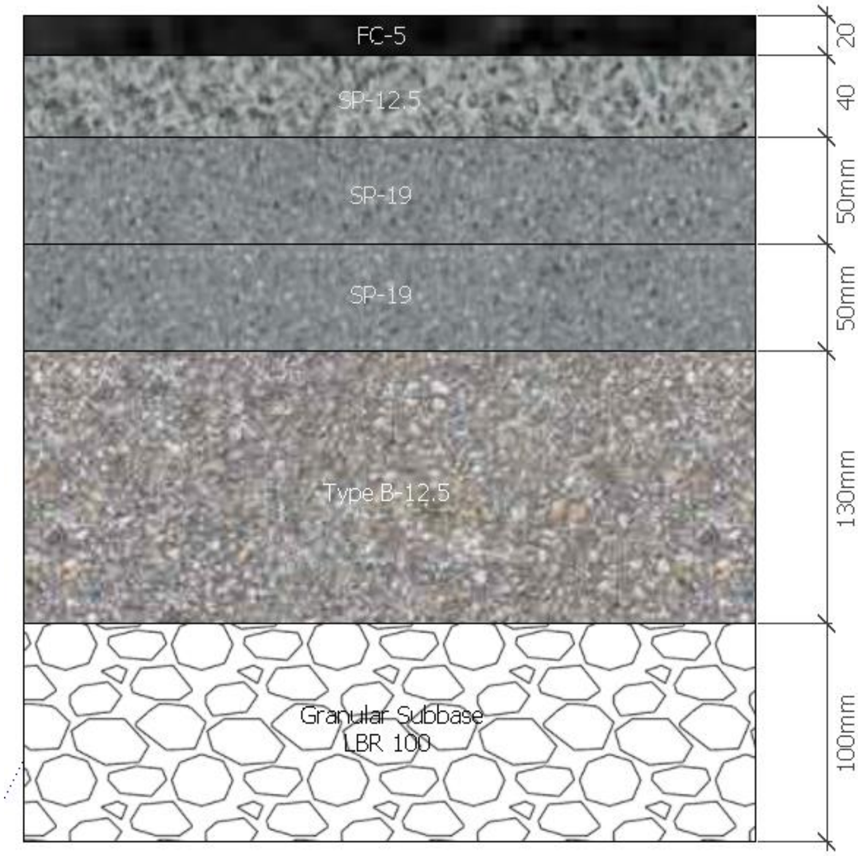


Figure 6. 4 Roadway Pavement layer thicknesses

6.3 Bridge Pavement Analysis

Pavement on the bridge deck has to resist to aging and deformation, meet skid resistance requirements, absorb and transfer traffic loads to the supporting structures. It functions as an anti-skid resistant, even surface for traffic and at the same time as a protection for underlying supporting structures from weather conditions. Bridge deck will deform due to temperature and traffic loads. Therefore flexible pavement will be more applicable. To fulfill all its functions bridge deck pavement consists of following layers.

- Sealing layer
- Waterproofing system
- Drainage layer
- Protecting course
- Wearing course / Surface layer

Sealing consists of surface preparation and bonding. Bonding is required to provide strong adhesion of waterproofing layer to concrete deck and fill the voids. Moreover this layer provided resistant to sheer forces. Before sealing the surface must be clean, dry and sound (European Asphalt Pavement Association, 2013).

Waterproofing layer directly affects the bridge durability. Penetration of salt water and de-icing fluids, as well as the CO2 mitigation deteriorates the bridge structural elements. Application of waterproofing layer will protect the bridge from environmental effects. During hot asphalt mixture placing the layer protects bridge deck elements from high temperature. There are 3 types of waterproofing system: sheet, liquid and mastic layer system. Sheet is the most common type and usually made of bituminous polymeric or elastomeric materials. They are preformed and create continuous membrane.

Drainage course is used to relief the waterproofing layer from any water pressure in case of seepage of water through asphalt layers. No water will penetrate through waterproofing but will be drained along drainage layer and directed to drain channels. The layer is usually made of open graded asphalt concrete with high air void ratio.

Protecting course is usually served as second waterproofing layer and consists of nonporous mastic asphalt.

Surface asphalt layer should be flat and have good skid resistance in order to provide comfortable drive. To meet durability requirements a layer should have followings:

- sufficient resistance against deterioration
- protection of the deck plate and the waterproofing layer
- resistance to fatigue
- resistant to permanent deformation
- possibility to spread the loads

Moreover, tack coat is required to provide good adhesion. Generally the asphalt mixture types used on the bridges are Dense Asphalt Concrete, Mastic Asphalt and Stone Mastic Asphalt (SMA).

6.4 Bridge Pavement Design

- Sealing layer:

Sandblasted concrete surface is coated with primer coat. Sand dusted epoxy primer can be used as primer.

- Waterproofing layer:

Two layers of bituminous polymeric sheets with the thickness of 4.5mm will be used. The sheets are normally 10m long and 1 m wide. The overlapping must be at least 10cm (Wegan, 2000).

- Drainage layer

For drainage layer open graded asphalt concrete with thickness of 20 mm will be used. The asphalt concrete should have large content of air voids. Drain channels and drip pipes should be installed parallel to the edge beams as illustrated in figure XX to remove the water from drainage layer.

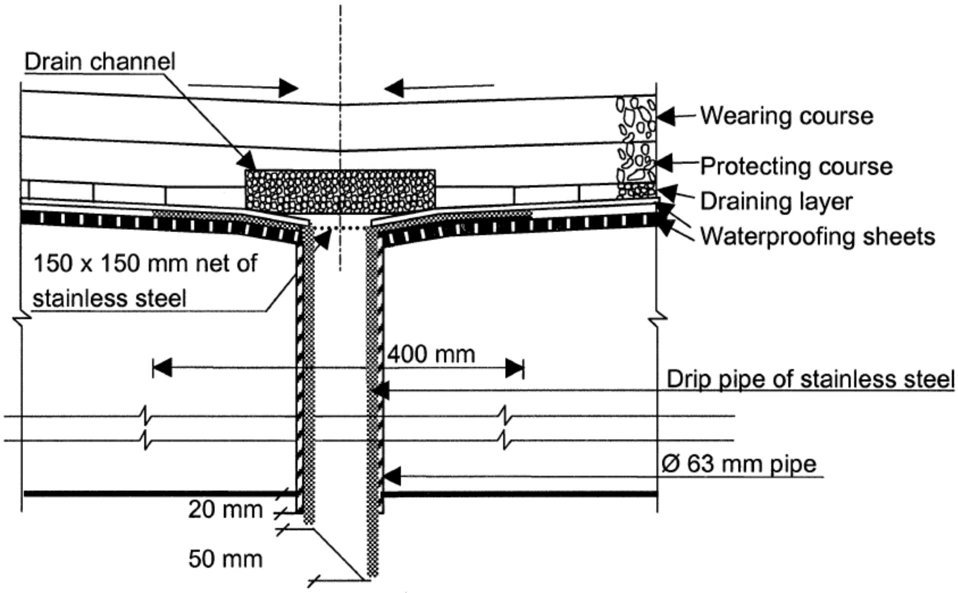


Figure 6. 5Cross section of drain channel and drip pipe

- Protecting course

Protecting layer is made of low air void content asphalt concrete with hard bitumen grade, and high binder content. The maximum thickness for the layer is 50mm. For the design 40mm thickness is selected.

- Wearing course

Wearing course is designed of stone mastic asphalt with the thickness of 30 mm. After Stone Mastic Asphalt Friction course FC-5 will be applied with similar thickness of 20mm as for other parts of roadway pavement.

Table 6. 15 Bridge pavement layer thicknesses

	Course name	Thickness	
		inches	mm
Friction course	FC-5	0.75	20
Wearing course	Stone Mastic Asphalt	1.13	30
Protective course	Dense Asphalt concrete	1.5	40
Drainage layer	Open Graded Asphalt concrete	0.75	20

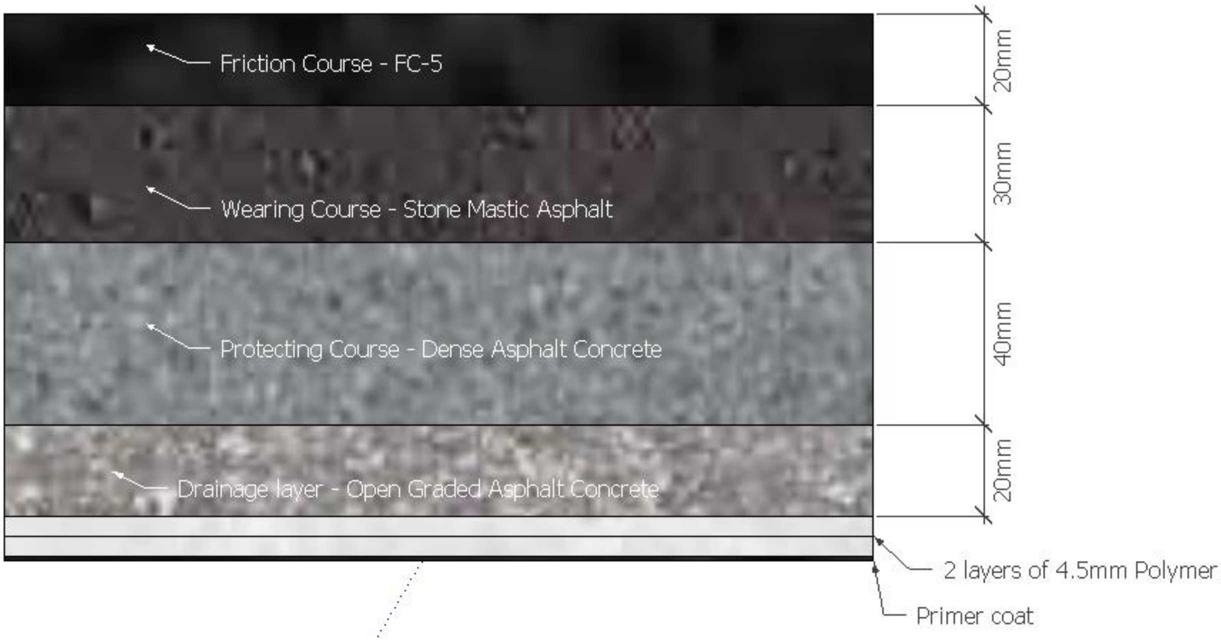


Figure 6. 6Bridge pavement layers thicknesses design

CHAPTER 7: CONSTRUCTION MANAGEMENT

7.1 Construction Sequence and Methods

7.1.1 Site Planning

Effective site planning has significant importance in the successful completion of the construction project. Since the site of this project is located at the intersection of two congested highways, a number of constraints are imposed for the construction activities. The issues to be considered under 'Site Planning' include:

- Site Constraints
- Site Layout
- Site Services

5.1.1.1 Site Constraints

The construction works should not significantly interfere with existing traffic flow. Well-planned temporary traffic management schemes are needed.

5.1.1.2 Site Layout

A well planned site layout is important in locating plant and equipment, site offices and work areas within site boundaries and existing contour levels. This is critical in completing a construction project on schedule in a safe and efficient manner. Site layout plan and locations of site offices are shown in this section.

- Layout Plan (figure or map)
- Site offices (store room and offices for workers)

7.1.2 Construction Planning

5.1.2.1 Traffic Schemes

In order to maintain the traffic during the construction and interfering existing traffic flow, it is planned to construct in three phases. The Palmetto Hospital located on the north of the I-75 Expressway will always be accessible during the construction.

Phase 1

During the first phase, zone A and B of SR 826 (figure 5. 1) will be widened towards inside by isolating the middle lanes during uncongested hours, and temporary barrier walls will be constructed for protection during construction of overhead structures such as video management system (VMS), toll framework, and other structures. After these installations temporary barriers will be removed.

The west part of the W 20th Avenue (figure 7.2) will be constructing, while the traffic of the north bridge will stay on existing road, and the south bridge traffic will be shifted to the W 22nd Avenue. The Peter Pike's Canal narrowing will be started, and north flyover bridge substructure construction will start.

I-75 traffic will be transferred towards the middle lanes, while widening will take place. In addition, noise walls are planned to be built.

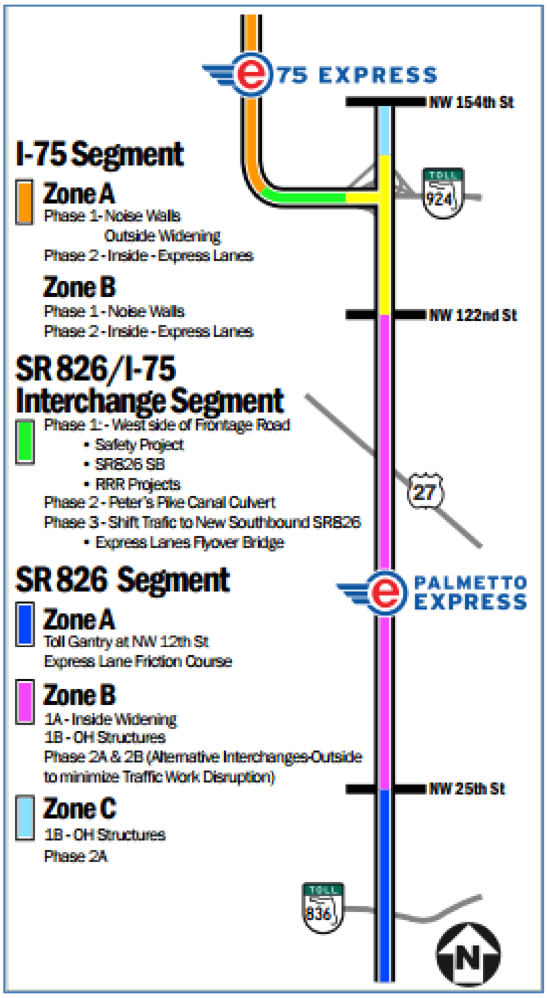


Figure 7. 1Construction Phasing and Traffic Management Plan Minimizes Disruption to Public

Phase 2

During the second phase the alternative outside lanes for SR 826 will be constructed. At the same time, the north bridge of the flyover will be started to build. After finishing alternative roads on the east of the SR 826, the traffic will be transferred there, and construction of central flyover bridge will start.

All of the improvements of I-75 will be finished, and the traffic will be shifted outside.

Phase 3

During this phase construction of the south bridge and approaches will be done. The traffic will be transferred to the new wider SR 826.

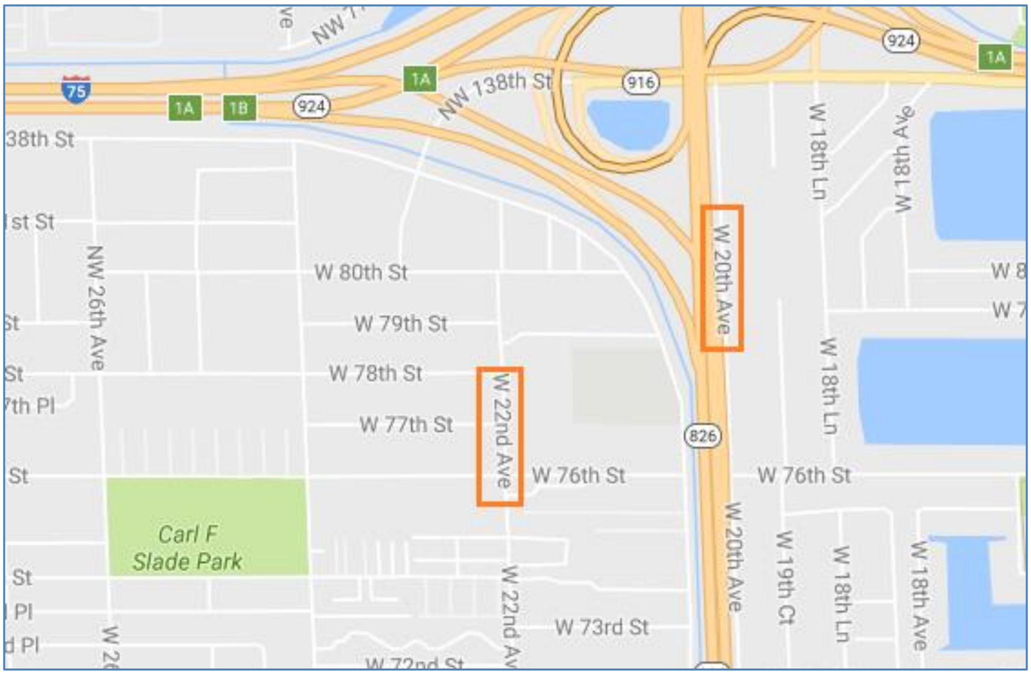


Figure 7. 2 Location of the W 20th and W 22nd Avenue

7.1.3 Construction of Bridge Structures

7.1.3.1 Highway Bridge

All of the 3 reinforced concrete highway bridges were constructed using a typical construction method and sequence:

1. Bridge foundation
2. Pile cap
3. Bridge column
4. Bridge bearing
5. Bridge deck
6. Ancillary works, e.g. vehicle parapets and bitumen laying on bridge deck

7.1.3.2 Driven Pile

Impact hammers (Figure 7. 3) are usually used to drive piles in such type of foundation. The main part of an impact hammer is a heavy ram weight that is lifted hydraulically or mechanically to certain height that is termed "stroke" and dropped onto the pile head.

The kinetic energy generated by the falling ram is transmitted to the pile, causing the pile to penetrate the ground.

Many different pile driving hammers are commercially available, and the major distinction between hammers is how the ram is raised and how it impacts the pile. The size of the hammer is characterized by its maximum potential energy, referred to as the "rated energy." The rated energy can be expressed as the product of the hammer weight and the maximum stroke. However, the actual energy transferred to the pile is much less a result of energy losses within the driving system and pile. The average transferred energies range from 25 percent for a diesel hammer on a concrete pile to 50 percent for an air hammer on a steel pile.

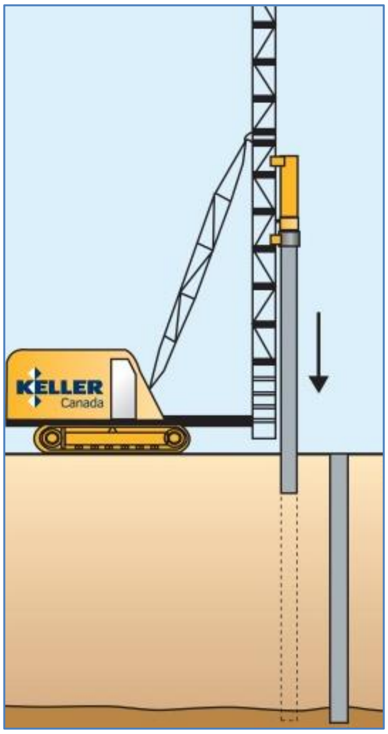


Figure 7. 3 Impact Hammer

7.1.3.3 Pile Cap

Firstly, the formwork is built with the use of plywood. Next step is the insertion of the reinforcement. After that concrete is poured, vibrated, and cured.

7.1.3.4 Pier and Pier Cap

Initially, the reinforcement is placed, after that the formwork is placed with the use of plywood. Finally, concrete is poured, vibrated, and cured.

7.1.3.5 Abutment

For our design we have chosen integral abutment, where the piles are located in one row, but they are driven together with all other piles. Thus, the first step after it is pile capping, also described above, and later placing reinforcement, constructing formwork and pouring the concrete.

7.1.3.6 Beam

The beam is erected with the help of the tall tower crane holding the I-beam from two ends, and located distinctly on bearing by the builders.

7.1.3.7 Deck

Firstly, the formwork is built with the use of plywood. Next step is the insertion of the reinforcement. After that concrete is poured, vibrated, and cured

7.1.3.8 Retaining Walls

- MSE walls
- Sheet Piles

7.1.4 Costruction Methods

The key features of the construction works in our project are:

- Widening main roads and using existing slip lanes to reduce traffic impacts when permanent works begin
- completing works on multiple fronts to reduce program time
- completing intersections in less time
- carrying out core construction activities during off-peak hours (including material delivery times)

In order to develop an approach that provides the most efficient and best value solution for the construction of the project it is necessary to look into the extensive local and international experience in similar type projects, especially the I-95 Express Project. One of the most pleasant solution is to divide the project into 3 separate construction segments illustrated in Figure 7.4 in order to minimize work overlapping, streamlines planning and coordination. More detailed description of construction process can be found in Appendix F. Following construction process summarizes and completes construction management plan:

The SR 826 Segment

Firstly, in a work zone mostly construction methods includes widening, milling, overbuilding and resurfacing of the existing roadway SR 826.Also, bridge widening occur in this segment, including the widening over the railroad. This work zone includes work along Peter's Pike Canal, starting at NW 106th St. to NW 122nd St. Due to the early start for procurement of sheet piles, early start will be allow to this significant portion of the project. Finally, tolling facilities will also be developed within this zone.

I-75/SR 826 Interchange Segment.

Construction works in this particular segment namely interchange between SR 826 and I-75 consist of a lot of independent precast concrete I-girder bridges. Those regulations applied in order to minimize the amount of structures while improving maintainability. In addition, in the defined zone several additional construction works are applied including the express lanes approach to the flyover, widening and reconstruction of the SB lanes, bridge widening and milling, overbuild and resurfacing of the mainline. The installation of the necessary retaining walls at Peter's Pike Canal in this zone is an important element in the completion of the interchange modifications.

I-75 Segment.

Most of the construction works in this particular work zone based on widening of the existing lanes to the outside to provide for construction of the express lanes in the median and noise walls on the outside and involves the removal of unsuitable soils from the median and construction of new managed lanes and noise walls.

7.1.5 Intelligent Transportation System (Its)

Intelligent transportation system aimed on minimizing the travel time of all travelers and merchandise while ensuring safety through the fair distribution of available resources, especially under scenario of increasing travel speeds, a significantly large number of travelers, and a high demand for precise and timely information by travelers. In order to achieve the aim, ITS should apply a seamless and natural integration of the different models of transportation, including vehicular traffic through distributed control and coordination algorithms subject to social norms, policies and guidelines. After the providing safety integration of ITS system, traveler will:

1. Gain access to accurate status information of any transportation mode from any point of the system
2. Compute the most efficient route or reroute across all different transportation modes based on available data
3. Be permitted to effect reservations, dynamically, even on the road

In our project, ITS will be used for traffic monitoring and management and motorists will be kept informed of any change in traffic patterns. It is necessary to explore the placement of supplemental ITS work zone technologies to proactively identify and respond to incidents as they occur. Construction management includes furnishing, installation, interconnection , integration and maintenance of ITS a and tolling ITS devices. In addition, installation or modification of the inductive loop detectors should be required in advance at existing traffic count stations. Existing devices which must be relocated or removed or cannot be immediately placed in their new location will be safely stored for future use. In such cases, a temporary/portable device will be utilized to facilitate system operations.

As it clearly from the Figure7.4, all ITS system requires standalone, subsystem, and system level testing. All of the procedures necessary for ITS system will be performing in early construction phase.

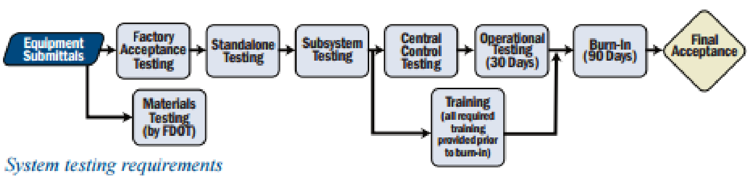


Figure 7. 4 Construction Process of ITS System

7.1.6 Schedule

Our final schedule has 917 days of construction followed by 168 days (6 months) of Toll Equipment Installation and Testing by FTE, resulting in a total Proposed Contract Time (PCT) of 1282 Days (Calendar Days).

The FCC/GLF/KHA team schedule has careful incorporated the following assumptions and restrictions within the Proposed Project Schedule:

1. Acceleration of the ITS and Tolling infrastructure.
2. A Detailed Traffic Management Plan
3. Field Investigation Time for additional survey, verification, and geotechnical investigations.

4. Required Department review time for submittals and release for Construction.
5. Utility Coordination, Design and Relocation.
6. ITS Testing and Integration including coordination and integration with existing segments and burn-in period.
7. FTE Toll Equipment Installation and Testing.

In order to accelerate the project completion, we have phased in an "Early Works" design plans package. This package will include the accelerated components of the Safety Project, Noise Walls as well as early embankment and drainage activities. With early submittals and review by the Department along with accommodation for RFC, the Early Works Plan Package will be ready for construction in early 2017 as soon as environmental permit modifications are received.

Following Figures illustrates critical path and construction management of project. More detailed information can be found in Appendix F

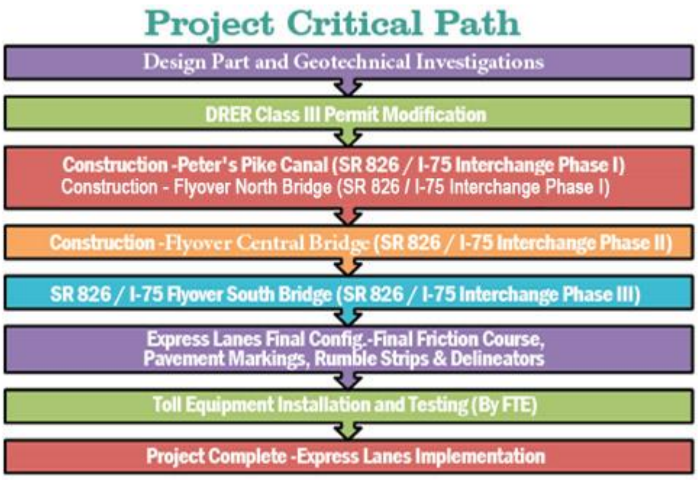
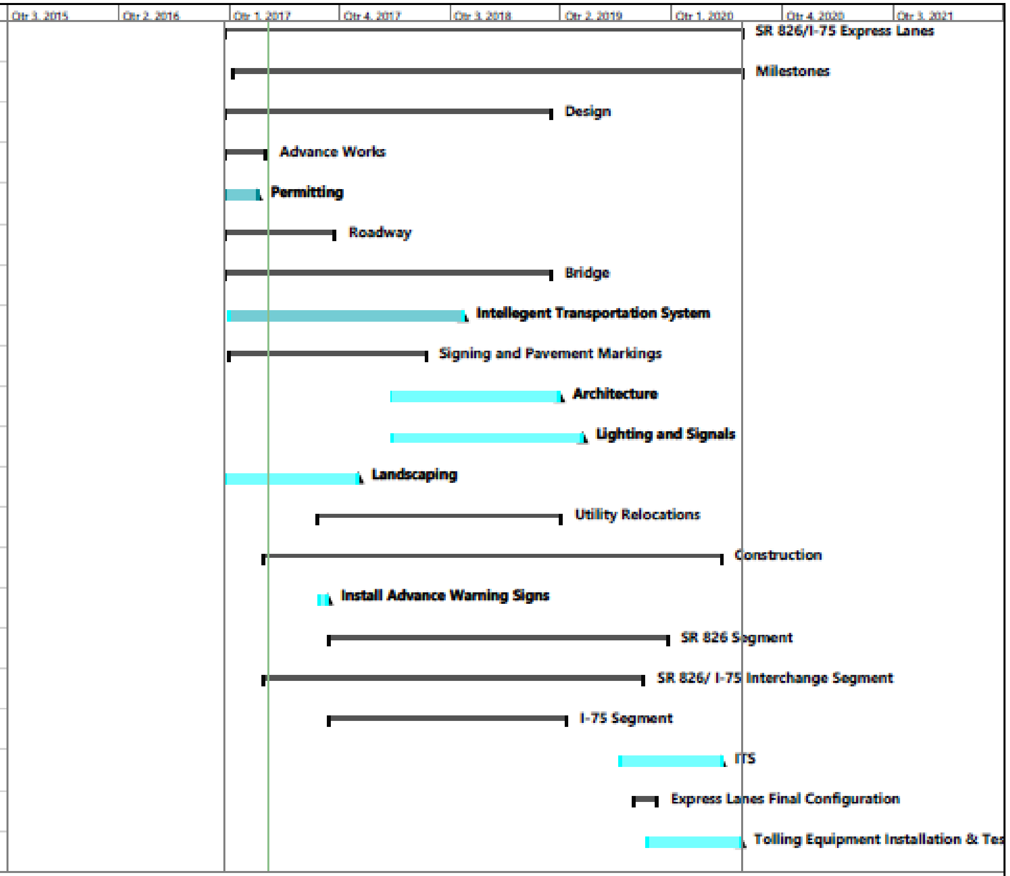


Figure 7. 5 Project Critical Path

ID	Task Name	Duration	Start	Finish
1	**SR 826/I-75 Express Lanes**	**917 days?**	**Mon 19/12/16**	**Tue 23/06/20**
2	**Milestones**	**900 days?**	**Wed 11/01/17**	**Tue 23/06/20**
12	**Design**	**579 days**	**Mon 19/12/16**	**Thu 07/03/19**
13	**Advance Works**	**75 days**	**Mon 19/12/16**	**Fri 31/03/17**
22	Permitting	59 days	Mon 19/12/16	Thu 09/03/17
23	**Roadway**	**196 days**	**Tue 20/12/16**	**Tue 19/09/17**
28	**Bridge**	**575 days**	**Fri 23/12/16**	**Thu 07/03/19**
41	Intellegent Transportation System	414 days	Fri 30/12/16	Wed 01/08/18
42	**Signing and Pavement Markings**	**348 days**	**Fri 30/12/16**	**Tue 01/05/18**
46	Architecture	299 days	Mon 05/02/18	Thu 28/03/19
47	Lighting and Signals	339 days	Tue 06/02/18	Fri 24/05/19
48	Landscaping	234 days	Fri 23/12/16	Wed 15/11/17
49	**Utility Relocations**	**431 days**	**Tue 08/08/17**	**Tue 02/04/19**
57	**Construction**	**809 days**	**Tue 28/03/17**	**Fri 01/05/20**
58	Install Advance Warning Signs	15 days	Fri 11/08/17	Thu 31/08/17
59	**SR 826 Segment**	**600 days**	**Mon 04/09/17**	**Fri 20/12/19**
86	**SR 826/ I-75 Interchange Segment**	**672 days**	**Tue 28/03/17**	**Wed 23/10/19**
133	**I-75 Segment**	**421 days**	**Mon 04/09/17**	**Mon 15/04/19**
156	ITS	180 days	Mon 26/08/19	Fri 01/05/20
157	**Express Lanes Final Configuration**	**41 days**	**Mon 30/09/19**	**Mon 25/11/19**
160	Tolling Equipment Installation & Testing	168 days	Wed 30/10/19	Fri 19/06/20



Project: Capstone Construction
Date: Sat 08/04/17

Task · Split · Milestone · Summary · Project Summary · Inactive Task · Inactive Milestone · Inactive Summary · Manual Task · Duration-only · Manual Summary Rollup · Manual Summary · Start-only · Finish-only · External Tasks · External Milestone · Deadline · Progress · Manual Progress

Figure 7. 6 Schedule of the Project

7.2 Risk Assessment

All of the construction project should consider high risks which are associated with any system within the project. In order to overcome these risks, risk management can be used as approach to identify, understand, analyze and eliminate these risks in order to succeed and complete project objectives. In addition, risk management allows distinguish project success significantly faster as a result of allocating resources more efficiently. According to Banaitiene and Banaitis 2012, construction field has always works with uncertainties and risks that can lead to severe consequences. Thus, Risk management tool significantly decrease this factor and therefore, tries to minimize negative impacts based on specific criteria including quality, cost and time. In addition, cost risks are relatively high and therefore can have effect for the whole project. Therefore, risk management plays important role during the whole project lifetime.

Basically, one of the main risk management tools that are used for the most of the project is categorizing risks according to the table 7.2. In this Table, namely Risk Severity Matrix, all of the risks can be categorized based on their importance. Table consists of two main parameters including likelihood and severity of the risk explained by vertical and horizontal numbers respectively. Color of the specific parts indicates the danger of the risks regarding to the project. Thus, red zone risks should be in priority and focused in first order. Then focus should be relocated to the orange zone risks. The green zones are playing insignificant role and has almost no effect to the project. However, these risks are also should be considered since they can be easily relocated into orange or even more to the red zone risks (Larson, 2011).

Following structure should be accomplished to the Table:

- Impact level: 1-negligible; 2-minor; 3-moderate; 4-major; 5-severe
- Probability level: 1-rare; 2-unlikely; 3-possible; 4-likely; 5-almost certain

Based on following formulae, risk value for each risk associated within the scope of the project can be evaluated:

$$Risk\ Value = Impact\ Value * Probability\ Value \tag{7.1}$$

Table 7. 1Risk Severity Matrix

Likehood	5	P4	P7	D5	E5	C6
	4	P3	P6	P8	P9	C5
	3	D1	D3	C3	E4	C4
	2	P2	E2	P5	C2	D4
	1	P1	E1	C1	D2	E3
		1	2	3	4	5
		Probability				

As it can be clearly seen from the Table 7.2, 25 risks selected were divided into main categories: Construction, Design, Environmental and Project management spheres. All those risks, their risk value and mitigation plan can be found in table D.1 in Appendix D.

Most significant risk evaluation was described in following way:

- Construction risks:

  This risk value can be proved by investigating most of the existing construction projects where labor and staff are regularly subjected to injuries, the main of which can cause fatal ends.

- Design risks:

  The most valuable risk associated with level of detailed drawings since they make big delays and take a lot of time to achieve the needed quality of drawings. In addition, those delays leads to losing money and therefore impact of this risk numbered as 5.
- Environmental issues:

  Bad weather conditions and seismic activity in Miami play important role in creating big delays and cost associated with those delays. In addition, seismic activity can damage existing structures that should necessary to do again.
- Project management risks:

  The main risks that should be caused in this type based on poor quality concerns since they create delays, additional costs and further go to disapproval.

Efficient Risk management can negotiate and minimize unexpected losses, increase productivity, improve schedule performance if it would be monitored and controlled during whole project timeline.

7.3 COST ANALYSIS

This section of the project contains the preliminary estimation for the initial and maintenance costs for the project, and payback period of the project. Bridge and roadway construction costs are evaluated by "parameter cost estimate" method. This analysis is made by estimating the cost of each construction unit such as site works, foundation, abutment, and others, estimated as parameter cost. Cost analysis is necessary in order to predict the expenditures, income, and approximate payback period of the project.

7.3.1 Initial Cost

7.3.1.1 Bridge Construction

There is a procedure consisting of three steps that determine bridge cost using FDOT data. During the first step considering the finished preliminary design and the average material costs the estimation is made. The second step uses site adjustment factors in order to regulate the total bridge expenses obtained from the first step. The final step is comparison of the determined cost per square foot and the historical cost for corresponding structural component. This procedure gives reasonably accurate results as long as there is no odd condition, which will influence the bridge cost. This estimation can be made for the project, which has finished preliminary design. Preliminary design includes structural component selection, component sizes, and reinforcement that were developed in previous sections. The scope of this section obtains the construction cost for superstructure and substructure of the whole bridge. The costs of the following items are out of the cost estimation scope: bridge fenders, lightings, deck drainage system, embankment, and approach slab.

Step 1: Based on the preliminary and structural design and analysis, bridge components type and their dimensions were estimated. The general information about bridge dimensions is summarized in table 5.1. Type of chosen bridges is medium span simple bridge with concrete deck and pre-stressed I-beam concrete girders. Three bridges of the current project have different spans. North (first) bridge has four spans, with maximum 60 m span. Middle bridge has one 60 m span, while south (third) bridge has three spans, 60 m each.

Table 7. 2 Description of bridge components

<table>
<tr><th></th><th>Components</th><th>Type</th><th>Dimensions/Amount</th><th>Unit</th></tr>
<tr><td rowspan="3">Substructure</td><td>Piles</td><td>Prestressed Concrete Piling</td><td></td><td></td></tr>
<tr><td>Footing</td><td>Cast-in-place</td><td>11 × 3.6 × 1</td><td>m × m × m</td></tr>
<tr><td>Column</td><td>Cast-in-place</td><td>6.9 × 1.2 × 4.6</td><td>m × m × m</td></tr>
<tr><td rowspan="4">Superstructure</td><td>Bearings</td><td>Neoprene Bearing Pads</td><td>11</td><td>Amount</td></tr>
<tr><td>Girders</td><td>Prestressed Concrete Girders</td><td>11</td><td>Amount per span</td></tr>
<tr><td>Reinforcing Steel</td><td>-</td><td>150</td><td>Amount per span</td></tr>
<tr><td>Barriers</td><td>Concrete, cast-in-place</td><td>8,430</td><td>m</td></tr>
<tr><td rowspan="2">Ret. Str.</td><td>MSE Walls</td><td>Permanent</td><td>550</td><td>m</td></tr>
<tr><td>Sheet piles</td><td>Steel / Permanent, Anchored</td><td>630</td><td>m</td></tr>
<tr><td rowspan="2"></td><td rowspan="2">Bearings</td><td>Neoprene Bearing Pads</td><td>0.25 × 0.80 × 0.08</td><td>m × m × m</td></tr>
<tr><td>Neoprene Bearing Plates</td><td>0.30 × 0.91 × 0.003</td><td>m × m × m</td></tr>
</table>

According to the dimensions and volume of selected elements, amount of related materials for each structural component was calculated. Unit costs for each element are taken from the FDOT Report “SDG 9 BDR Bridge Cost Estimating General” (2007, 2009), FDOT Report “Temporary Design Bulletin” (2009), Bridge Development Report (FDOT, 2016), “Generic Cost Per Mile Models” (2016), FDOT Report “Update on Highway Construction Cost Trends in Florida” (2007), FDOT Report “FLORIDA TRANSPORTATION TRENDS AND CONDITIONS” (2014), and dimensions of bearings from FDOT Report named “COMPOSITE ELASTOMERIC BEARING PADS - PRESTRESSED FLORIDA-I & AASHTO TYPE II BEAM” (2014). By multiplying amount to the unit cost of each component total cost was estimated and assembled into table 5.2. The overall cost was estimated as $48,518,146.

Table 7.3 (Source: *Chapter 9 - BDR Cost Estimating*, 2007)

	Work Type	Unit	Quantity	Unit Cost	Total Cost
General Requirements	Move In	ls	1	Labor $8,112 Equipment $3,204	$ 11,316
	Clean Up	ls	1	Labor $4,248 Equipment $1,240	$ 5,488
				Total this account	$ 16,804
Sitework	Excavation & Backfill	m^3	249,600	$ 10.6	$ 2,645,760
				Total this account	2,645,760
Concrete	Precast - Piles	m	2,305.8	$ 278.9	$ 643,087
	Precast - Beams	m	490	$ 951.44	$ 428,150
	CIP - Abutment	m^3	923	$ 1,307.95	$ 1,206,584
	CIP - Footings	m^3	277.2	$ 1,307.95	$ 362,563.74
	CIP - Columns	m^3	804.68	$ 1,307.95	$ 1,052,481
	CIP - Deck	m^3	3,013.5	$ 1,438.75	$ 4,335,673
	CIP - Barrier	m	1,000	$ 377.30	$ 3,773
				Total this account	4,763,823
Metals	Steel, structural, reinforcing	kg		$ 2.54	
	Bearing Plates	m	Can be disregarded		
				Total this account	
Other	Neoprene Bearing Pads	m^3	0.176	$ 22,954.56	$ 4,040
	Bridge Pavement	m	7,430	$ 4,281.09	$ 31,808,523
	MSE Walls	m^2	25,400	$ 365.97	$ 9,296,000
Total Bridge Construction Cost =$48,518,146					

<u>Step 2:</u> According to Bridge Development Report (FDOT, 2016), for urban constructions construction cost should be increased by 6 percent. Since our project is located at Miami-Dade, total cost was multiplied by 1.06. The total cost is then estimated as $51,429,235.

<u>Step 3:</u> After all calculations, obtained cost is compared with historic data. The cost found from step two should be in the range for medium span concrete pre-stressed simple span bridge construction cost from FDOT “BDR Cost Estimating” (“BDR Cost Estimating”, 2009) data, where costs for such bridges ranges between $85 and $155 per

square foot. Our bridge construction cost satisfies this condition. The process should produce a reasonably accurate cost estimate.

7.3.1.2 Total Project Construction

The total project length is 7.43 km, 490 meters of which corresponds to the bridge length, and rest 7 km belong to roadway construction. This expressway connects two urban arterials. According to FDOT Report "Generic Cost per Mile Models" (2016), widening two-lane urban arterial to four lanes is estimated to cost $3,139 per meter ($5,051,909.23 per mile), and adding additional two lanes to existing two lane arterial costs $2,653.20 per meter ($4,269,907.62 per mile). In order to calculate the total cost for the project costs for widening SR 826, adding two lanes in the middle of I-75, bridge construction, and sheet pile walls construction should be summed. Expenses for each of the mentioned term, total cost, and calculations are written in table 7.3.

Table 7.3 Total Project Cost Compounds

Work Type	Quantity	Unit	Calculations	Total Cost
Widen 2 Lane Urban Arterial to 4 Lane	1,200	m	1,200 m × $3,139.11	3,766,932
Add 2 Lanes to Existing 2 Lane Undivided Arterial	5,230	m	5,230 m × $2,653.20	13,876,236
Bridge Construction	490	m	(Compare with 125 × 1.2) $/ft2 ×	51,429,235
Sheet Pile Construction	3,954	m^2	$387.50 /$m^2$ × 3,954 m^2	$ 1,532,240

The total roadway construction cost equals to $70,604,643.

7.3.2 Maintenance Cost

There are five types of failures that mostly affect highway maintenance: roadway wearing out, which can be potholes or pavement distortions, roadside deterioration, and drainage deformation.

7.3.2.1 Bridge Maintenance

The most significant three components of the bridge deformations are decided to consider during bridge maintenance cost estimation. These are the wearing surface, the decking, and the superstructure. The major portion of maintenance works are made in these areas. Table 7.28 shows repairing costs and their average frequencies. The table was constructed using data from Zayed (2002).

Table 7. 26 Bridge maintenance cost

Repair Type	Area, m^2	Unit price, $	Total cost per unit work, $	Frequency, years	Total cost per year, $
Non- Destructive Asphalt Testing	9,950	2.15	21,392	4	5,348
Repave/milling	9,950	21.5	213,925	15	14,260
Wearing Surface Sealing	9,950	2.15	21,392	4	5,348
Bridge Inspection	-	-	800	2	400
Concrete Patch and Repair	16,700	100	1,670,000	15	111,333
Total					136,689

Thus, total maintenance cost for designed bridges is equal to $136,689.

7.3.2.2 Roadway Maintenance

Cost of HMA pavement maintenance was calculated out of the average spending on HMA in Texas throughout the period of starting from 1969. Data was provided by Texas Department of Transportation (2012) and equal to $5.45 per square meter. As designed pavement area is 143,500 m^2, maintenance cost for roadway is equal to $782,075.

Finally, the preliminary maintenance cost for the whole project is equal to the sum of the bridge maintenance cost and roadway maintenance cost, which equals to $918,764 per year.

7.3.3 Income

7.3.3.1 Direct Income

The expressway expects to serve for 15,000 daily traffic. The charge for using expressway is planned to be $0.62 per vehicle. Thus, expected annual income from charging of vehicles is equal to $3,394,500 per year.

7.3.3.2 Indirect Income

This section investigates saving cost due to reduction of travel time. Saved time can be spent to other activities such as family time, leisure, and more work hours. There are two types of vehicle driver purposes on expressway, for work and non-work related purposes. The value of saved time for two categories is different. The value of non-work related purposes is 50% of average wage rate, while saved time for work purposes is valued as 100% of average wage rate (U.S. Department of Transportation). Bureau of Labor Statistics (2015) provides that the average wage rate in Florida is $20.60 per hour. According to National Household Travel Survey (2009), the vehicles with non-work related purposes are 86.1%, while 13.9% is work related vehicle on expressways. Saved time could be calculated by subtraction of the previous time spend on way to new time spend. Table 7.29 shows estimation of travel time savings is equal to $2,761,955 per year.

Table 7. 27 Estimation of travel time savings

Category	Daily traffic	Saved time per vehicle, hours	Value of time, $ per hour	Daily total savings, $	Annual total saving, $
Work related	2,085	0.043	20.60	1,846.9	674,119
Non-work related	12,915	0.043	10.30	5,720,1	2,087,836
Total					2,761,955

The overall annual income is the sum of the direct income and indirect income, and equal to $6,156,455 per year.

7.3.4 Payback period

Payback period is calculated by Equation 7.1 and equal to 12.75 years.

$$\text{Payback Period} = \frac{\text{Initial construction cost}}{\text{Overall income} - \text{maintenance cost}} = \frac{\$70{,}604{,}643}{(\$6{,}156{,}455 - \$918{,}764)\text{ per year}} = 11.77 \text{ years.} \tag{7.1}$$

CHAPTER 8: BRIDGE MONITORING – POST CONSTRUCTION

8.1 Monitoring Strategy

Structural parameters of the bridge are monitored during the service life of the bridge in order to estimate real behavior of the bridge, and perform reliability analysis in case of any accidents. The service life of the bridge is also estimated during monitoring by comparing theoretical values of the bridge service life and evaluation of bridge deterioration.

Major bridges are usually monitored for obviation of accidents and making quick decisions in order to eliminate high losses they create. Therefore, bridges are permanently monitored during their service life. During the monitoring the certain parameters are measured and compared with designed theoretical values, which predetermine the following action. There are three levels of action for each structural measurement parameter:

a. Level 1: No action required
b. Level 2: Reanalysis by the designer
c. Level 3: Stop traffic and deep analysis of the bridge is performed.

Here, level 2 requires calculation and analysis of the bridge structure again by the designer after certain occurrence. Usually this level ends up with inaction, maintenance implementation, or repairing. Conversely, level 3 demands serious actions, and takes place, if significant accident, such as earthquake or storm, happens.

The monitored parameters can be classified into three types: depending on the environment, structural behavior, and durability. The environmentally dependent parameters usually are wind speed, air temperature, seismic acceleration, and water level. The structural behavior parameters are typically the displacements, the internal temperatures, the strains and vibrations at certain points. The parameters indicating the durability usually are reinforcement corrosion, carbonation depths or chloride content profiles (Branco and de Brito, 2004).

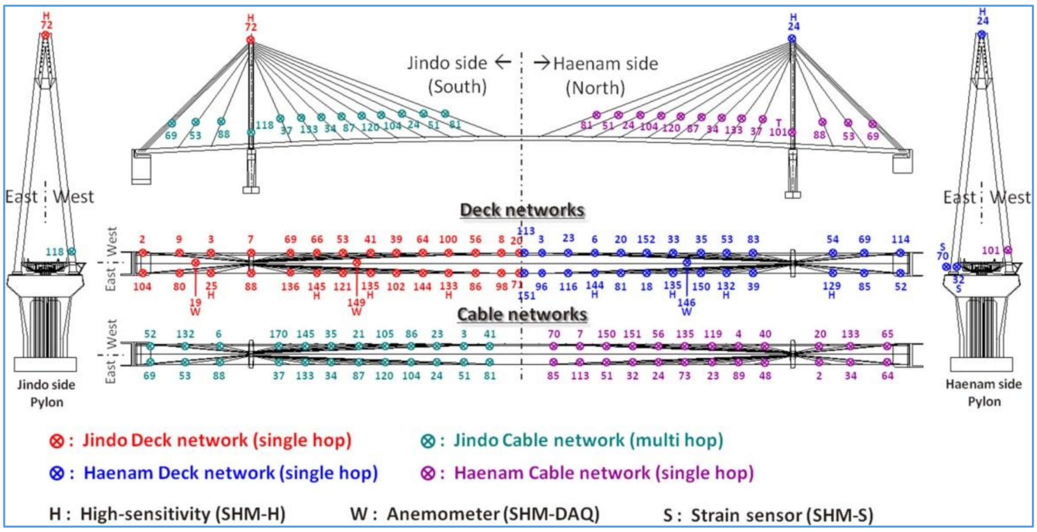


Figure 8. 1Example of bridge monitoring

8.2 Measurement of Structural Parameters

Structural parameters are typically measured in situ, what requires special equipment insertion during the construction. There are two ways of measuring: manually and remotely in the centralized office where all of the indicator results are collected, either way the equipment should have easy access.

8.1.1 Measurement of Displacements

There are three main parts of the bridge, where displacements are important to monitor: main structural elements, joints, and movable bearings.

Displacements in main structural elements are vertical displacements in the middle of the spans and horizontal displacements of the towers and tall columns. These displacements usually are done by surveying with a precision approximately 0.5 mm.

Displacements in joints are measured by the displacement electrical transducers installed between the sides of the joints or mechanically by displacement mechanical gauges.

Displacements in movable bearings are done the similar to the joint displacement estimation.

8.1.2 Measurement of Rotations

Rotations of towers and tall columns can be measured either by electrical inclinometer transducers or mechanical air bubble inclinometers.

8.1.3 Measurement of Strains

Strains in main bridge parts are usually measured with electrical strain gauges located in mid-spans, supports and at the column bases as shown in figure 8.1, and are attached to the reinforcement or wire gauges inside concrete. They are more durable and precise.

8.1.4 Measurement of Temperatures

Temperatures in bridge slab cross sections are obtained by positioning thermocouples at distinct depths (figure 8.2). They give opportunity for estimating temperature difference and average temperatures in the section, and permit online measurements.

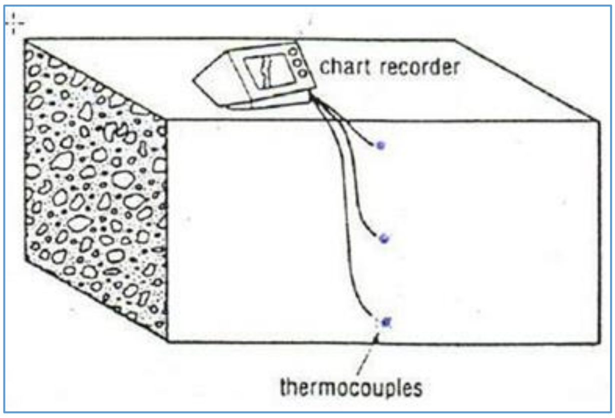


Figure 8. 2Location of thermocouples

8.1.5 Measurement of Forces

Forces in bearings, steel bars, and cables can be estimated with the use of the electrical load cells. In bearings they are located between bearings and the superstructure. The alternative for the steel bars are electrical strain gauges that can be attached to them, while the forces in cables can also be estimated by measuring the vibrations, frequency of which have connection with the tension forces, or by strain or displacement gauges (simple version is shown in figure 8. 3).

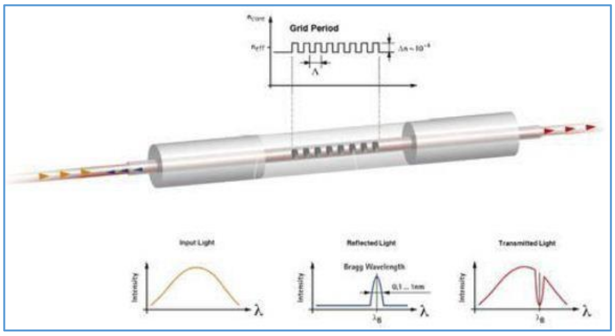


Figure 8. 3 Simple strain sensor

8.1.6 Measurement of Vibrations

The vibrations can be measured by the use of the electrical accelerometers having sensitivity starting from 0.1 Hz. The result can be in accelerations, velocities, or vibration dislocation.

8.1.7 Correlation with Environmental Parameters

Most of the structural parameters are environmentally dependent. Therefore, certain environmental features should be monitored: air temperature for correspondence with displacements, daily humidity for obtaining interdependence with creep and shrinkage, rainfall for monitoring relation with shrinkage, seismic parameters of the earth for correlation with vibrations, and average speed of the wind and windflaw for estimating relation with wind displacement.

8.3 Monitoring Durability of the Bridge

Bridge materials deterioration is a quite slow procedure, which does not require online monitoring. Moreover, it is not always possible to obtain the most deteriorating part of the bridge element. Even if the equipment is installed, the highest damage can occur in any other place. Therefore, conditions of the bridge elements are checked periodically, once or twice per year. On the other hand, equipment can be placed in certain points after establishing the location of the highest deterioration.

The parameters for durability monitoring in situ are the corrosion potential, the depth of carbonation, and depth of the chloride concentration in the cover. Using results of the parameter indicators, the deterioration evolution can be checked, updated, and further actions for maintenance can be identified.

The corrosion evolution models can give different results from the prototype bridge, thus, some reinforcements can be left close to the surface, which can be corrosion evolution model for other reinforcements.

CHAPTER 9: CONCLUSION

To sum up, this project designed expressway connecting I-75 and SR 826/Palmetto Expressway, which should result in the reduction of the congestion up to approximately 30% of current traffic. Considering the construction site conditions, for connecting two highways bridges, MSE walls, and sheet piles are required. Preliminary structural and geotechnical design were provided. Project was designed according AASHTO LRFD 2012 Specification. Carried feasibility and cost analysis show that chosen bridge components have better technical, operational, economic, and time characteristics. The construction cost of the project was estimated as $70,604,643. The overall payback period is 11.77 years.

The major part of the project is bridge design, in total three bridges. Several bridge alternatives were compared on the basis of lower maintenance and life-cycle cost. The most feasible and aesthetically pleasing bridge was designed. It was achieved by choosing cost efficient and simple structure that uses precast pre-tensioned concrete I-beams, CIP concrete deck, and concrete piers integrated with superstructure. CIP concrete deck was chosen, because precast concrete I-beams require cast-in-place structure. Concrete piers integrated with superstructure ensures that bridge resist the loads as the solid structure.

Two retaining structures are introduced for the project. MSE wall was the best choice due to its low cost, ease of construction and flexibility. In total 15 MSE Walls are used to retain bridge approach embankments and base of elevated roadways. Preliminary design was carried out which proposes MSE Walls with cruciform segmental concrete facings and inextensible reinforcement. Part of Palmetto Expressway will be expanded in width to the canal nearby, which requires retaining the road base. The soil at the base of canal is weak with low bearing capacity according to boring data. Therefore, it was decided to use anchored steel sheet pile wall. Report provides load capacity calculations and preliminary design of sheet piles. Driving piles will be used for bridge foundation due to the fact that the site has weak soil strata. Preliminary design suggests using 60cm prestressed concrete piles. For toll station drilled shaft foundation is suggested due to high lateral wind load effect.

Project report includes construction management part, where the organized flow of the activities and work breakdown structure needed for achievement objectives within the scope of the project and evaluate risks associated with the construction process, are provided.

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Appendix A

Derivation of the Minimum Stopping Distance on Crest Vertical Curve (for S>L)

When the Stopping Sight Distance is less than the length of crest, the solution for case shown in figure 2.2 should be performed.

The minimum length of the vertical curves of parabola is calculated as in equation A.1.

$$L_{min} = \frac{AS^2}{200(\sqrt{H_1}+\sqrt{H_2})^2} \quad (\text{for } S < L) \tag{A.1}$$

Instead of H_1 and H_2 1.1 m and 0.6 m were substituted. In result, equation 2.1 was obtained.

Appendix B

Figure B. 1 Join Between Existing Highway and Designed Expressway

Figure B. 2 Conceptual Drawing of Bridge 1

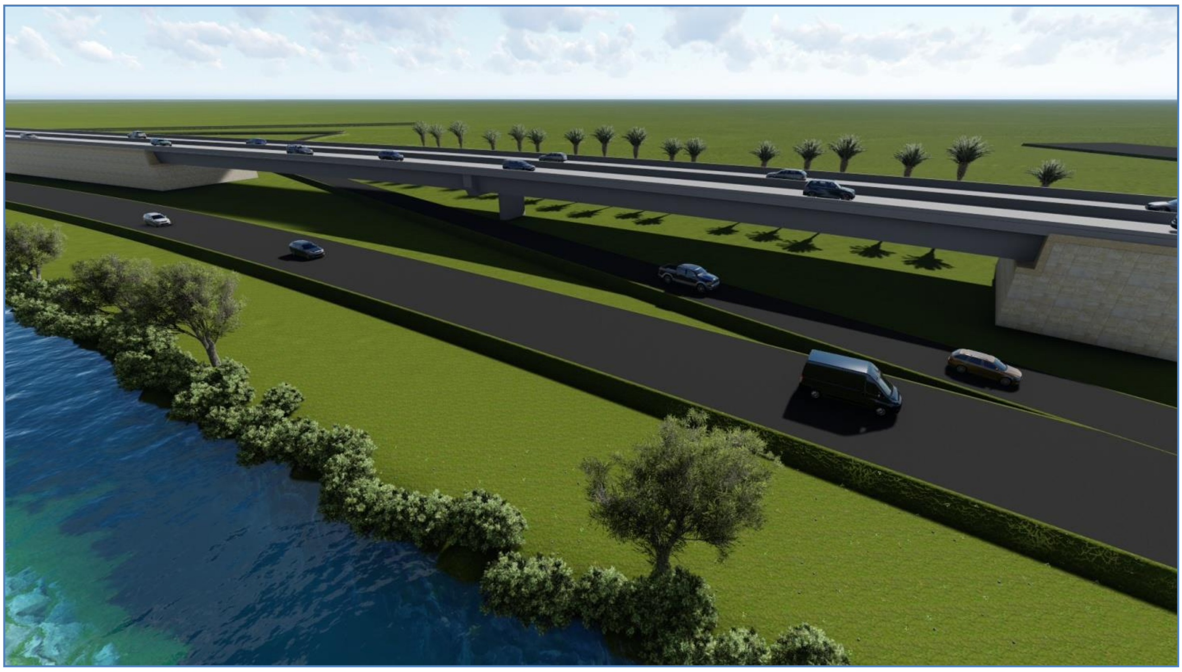


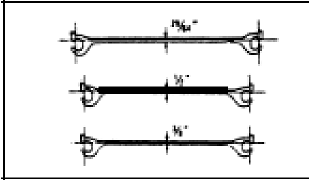
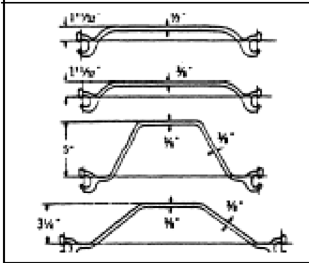
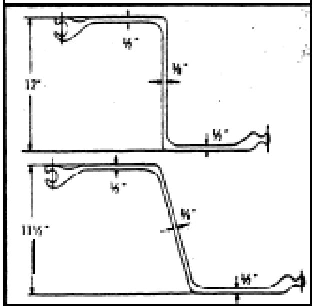
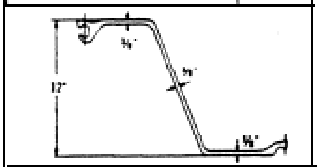
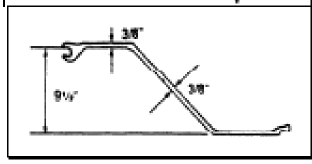
Figure B. 3 Conceptual Drawing of Bridge 2

Figure B. 4 Conceptual Drawing of Bridge 3

Figure B. 5 Conceptual Drawing of Bridge 3

Appendix C.1

Steel Sheet Piling Sections

Profile	Section Index		Distrib. Rolled	Driving Distance per Pile	Weight Per Foot	Weight Per Square Foot of Wall	Web Thickness	Section Modulus Per Pile	Section Modulus Per Foot of Wall	Area Per Pile	Moment of Inertia Per Pile	Moment of Inertia Per Foot of Wall
				In.	Lbs.	Lbs.	In.	In.^{3}	In.^{3}	In.^{2}	In.^{4}	In.^{4}
	Interlock with Each Other	PSX 32	H.	16½	44.0	32.0	29/64	3.3	2.4	12.94	5.1	3.7
		PS32*	H.S.	15	40.0	32.0	½	2.4	1.9	11.77	3.6	2.9
		PS28	H.S.	15	35.0	28.0	⅜	2.4	1.9	10.30	3.5	2.8
	Interlock with Each Other	PSA 28*	H.	16	37.3	28.0	½	3.3	2.5	10.98	6.0	4.5
		PSA 23	H.S.	16	30.7	23.0	⅜	3.2	2.4	8.99	5.5	4.1
		PDA 27	H.S.	16	36.0	27.0	⅜	14.3	10.7	10.59	53.0	39.8
		PMA 22	H.S.	19⅝	36.0	22.0	⅜	8.8	5.4	10.59	22.4	13.7
	Interlock with Each Other and with PSA23 or PSA28	PZ38	H.	18	57.0	38.0	⅜	70.2	46.8	16.77	421.2	280.8
		PZ32	H.	21	56.0	32.0	⅜	67.0	38.3	16.47	385.7	220.4
		PZ27	H.	18	40.5	27.0	⅜	45.3	30.2	11.91	276.3	184.2
		PZ 22	H.	22	0.3	22.0	⅜	34.8	9.0	11.9	167	91.1

*Sections PS32 and PSA28 ate infrequently rolled and we do not advise their use in a design unless an adequate tonnage can be ordered at one time to assure a minimum rolling.

Complete data regarding these sections will be found in a separate publication entitled "USS Steel Sheet Piling?

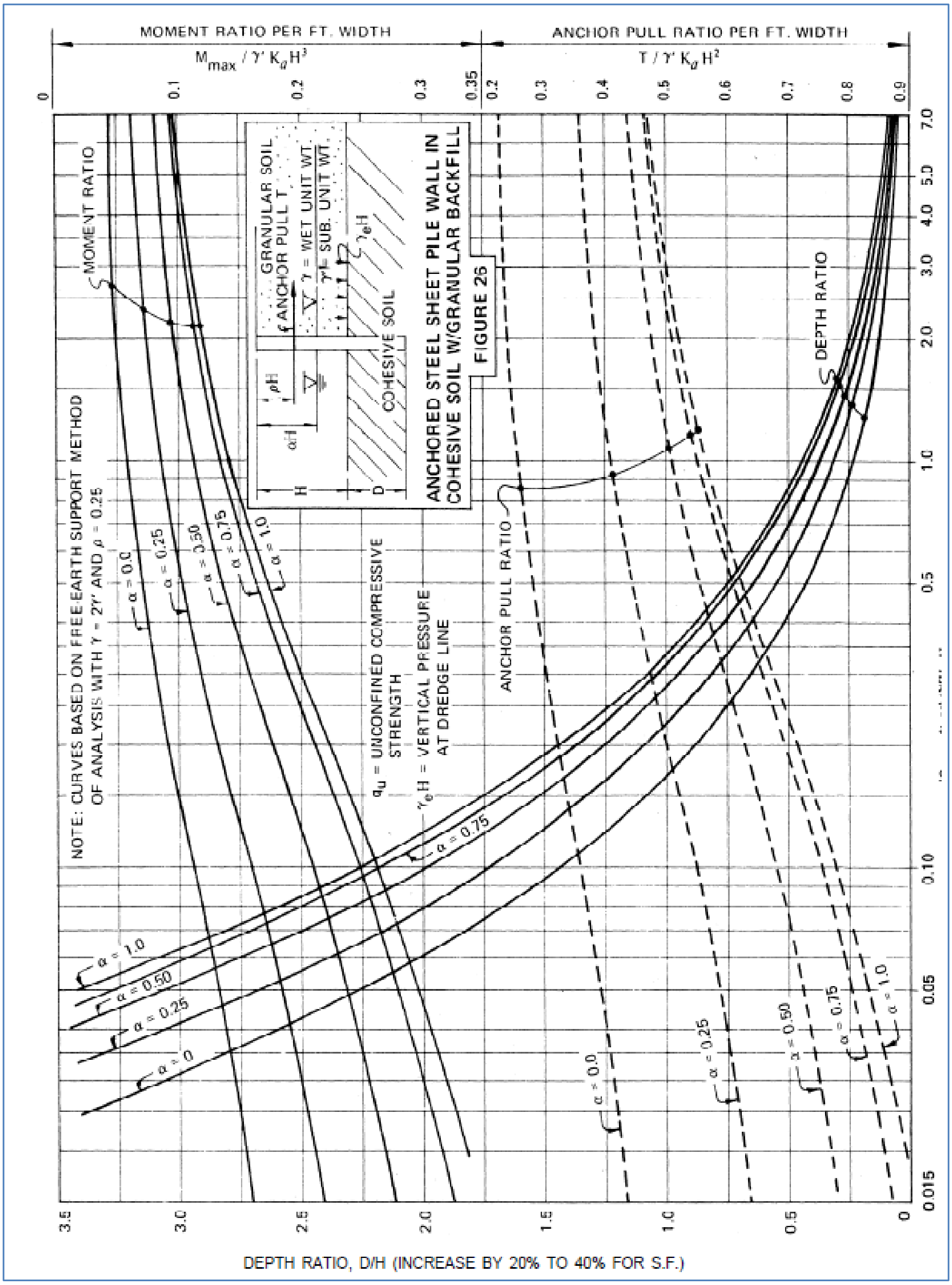
H-Homestead, Pa. (Pittsburgh District)
S-South Chicago (Chicago District)

Suggested Allowable Design Stresses-Sheet Piling

Steel Brand or Grade	Minimum Yield Point, psi	Allowable Design Stress, psi*
USS-EX-TEN 55 (ASTM A572 GR 55)	55,000	35,000
USS EX-TEN 50 (ASTM A572 GR 50)	50,000	32,000
USS MARINER STEEL	50,000	32,000
USS EX-TEN 45 (ASTM A572 GR 45)	45,000	29,000
Regular Carbon Grade (ASTM A 328)	38,500	25,000

*Based on 65% of minimum yield point. Some increase for temporary Overstresses generally permissible.

Appendix C.2



MOMENT RATIO PER FT. WIDTH
$M_{max} / \gamma' K_a H^3$
ANCHOR PULL RATIO PER FT. WIDTH
$T / \gamma' K_a H^2$
ANCHORED STEEL SHEET PILE WALL IN COHESIVE SOIL W/GRANULAR BACKFILL
FIGURE 26
GRANULAR SOIL
ANCHOR PULL T
γ = WET UNIT WT.
γ' = SUB. UNIT WT.
COHESIVE SOIL
NOTE: CURVES BASED ON FREE EARTH SUPPORT METHOD OF ANALYSIS WITH $\gamma = 2\gamma'$ AND $\rho = 0.25$
q_u = UNCONFINED COMPRESSIVE STRENGTH
$\gamma_e H$ = VERTICAL PRESSURE AT DREDGE LINE
MOMENT RATIO
ANCHOR PULL RATIO
DEPTH RATIO
DEPTH RATIO, D/H (INCREASE BY 20% TO 40% FOR S.F.)

Appendix D

Table D.1 Risk Categories and Mitigation Plan

Number	Risk Category	Risk Value	Mitigation Plan
Construction Works			
C1	Wrong choice of materials	3	Conduct an investigation for alternative materials and solutions; in severe cases – redo the activity
C2	Equipment Damaged	8	Repair existing equipment or replace by the new one
C3	Non-certified changes in order	9	Invite additional supervisor to monitor and control the process of construction order
C4	Errors in completion of structural/ geotechnical works	15	Involve to the process qualified engineers; increase checking, quality assurance and number of supervisors
C5	Technological changes	20	Ask to necessary modifications
C6	Human Health and Safety	25	Conduct necessary training including safety regulations knowledge and insurance operations
Design Risks			
D1	Uncertainties	3	Try to be minimize number of uncertainties by increasing communication between departments
D2	Conflict Situations in Standardization	4	Consult with qualified engineers in order to choose appropriate standard
D3	Controversies between architectural/structural/geotechnical designs	6	Try to neglect these problems by increasing interaction on those departments
D4	Design Errors	10	Try to minimize number of errors in design by focusing into details and checking by qualified professional engineers
D5	Non-acceptable level of detailed drawings	15	Notify engineering department that they should provide acceptable level of detailed drawings

			for starting works on-site
Environmental Risks			
E1	Failure to meet green certification standards	2	Try to use other alternative materials that more eco-friendly to the nature
E2	Non-acceptable environmental analysis	4	Investigate more specials into investigation of environmental analysis
E3	Hazardous materials	5	Read maintenance of mentioned materials and appropriate in right way
E4	Availability of resources	12	Find appropriate place for allocation of needed resources in order to avoid time delays
E5	Weather Conditions	20	Delay construction work until necessary weather conditions can be available
Project Management Risks			
P1	Miscommunication between labor/staff	1	Try to neglect any conflicts between mentioned people by improve work conditions
P2	Violation of professional ethics	2	Provide several lectures regarding to engineering ethics code
P3	Inflation and Tax	4	Optimize work by employing less-costly alternative methods and materials. Follow financial plan
P4	Delays of construction materials	5	Improve transportation service; for severe cases – change producer
P5	Quantity of needed materials	6	Involve additional supervisor to control the process of delivering the material; consumption those materials; check list should be checked
P6	Productivity	8	Improve communication between labor and staff; in more severe cases - increase number of labor
P7	Unexpected Schedule Delays	10	Provide additional labor and mechanical instruments for completing work earlier
P8	Actual amount of work	12	Control should be performed in order to

			decrease the gaps between baseline and current activities
P9	Concerns regarding to quality	16	Increase the number of labor answering for quality; increase communication between quality control and assurance department

Appendix E

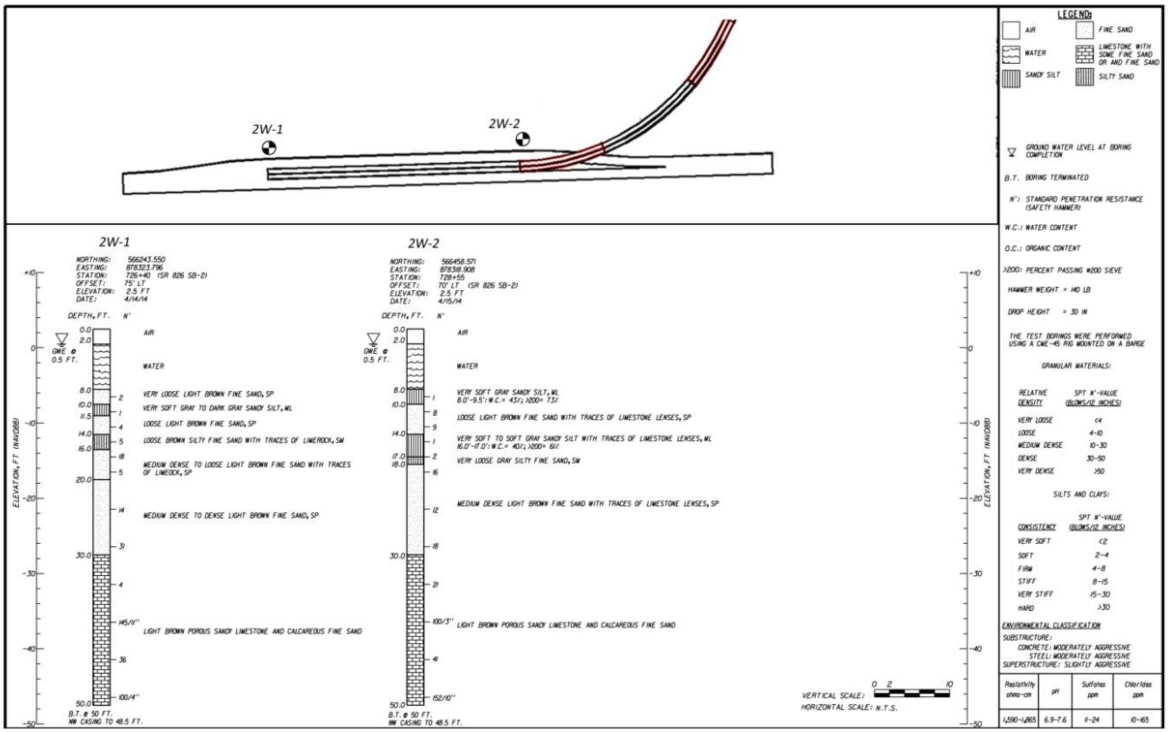


Figure E. 1 Sheet pile borings

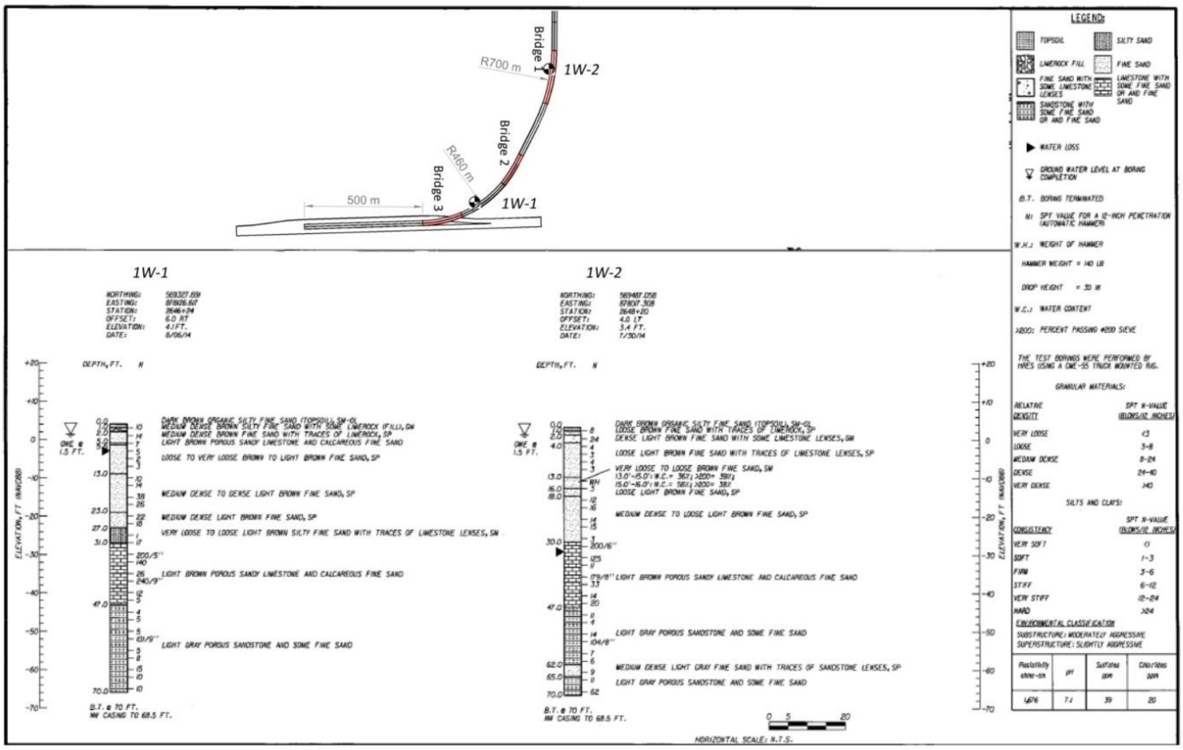


Figure E. 2 Bridge borings

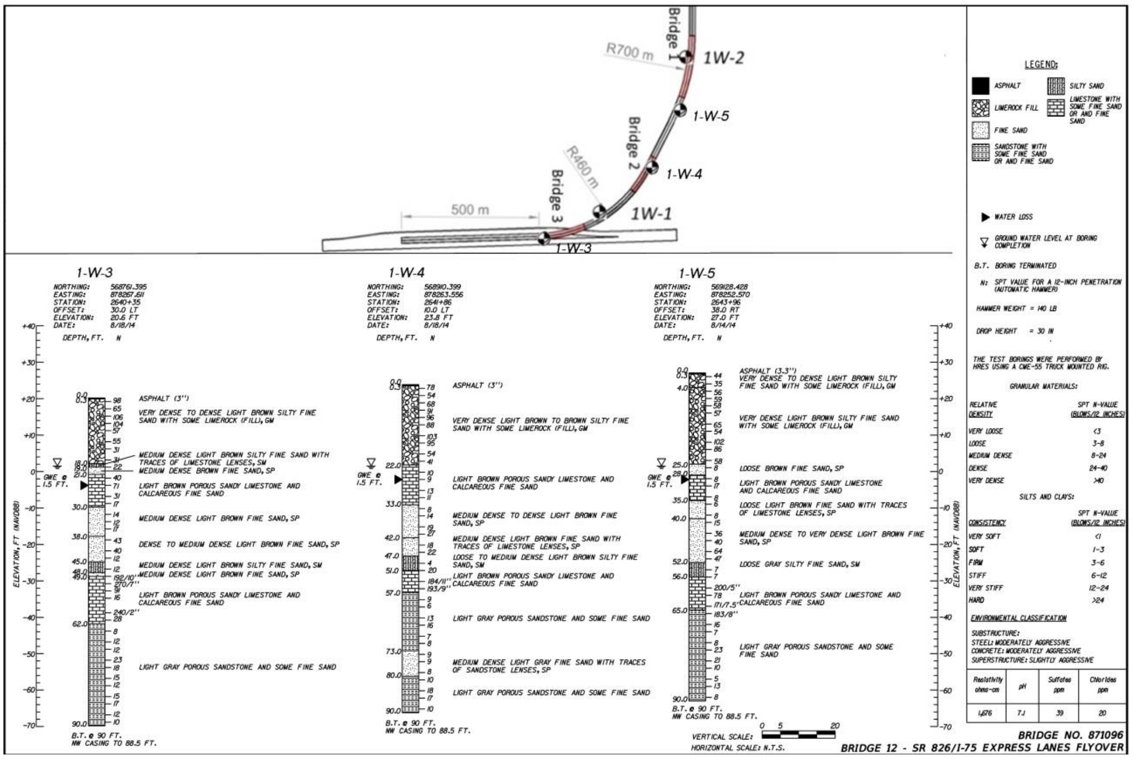
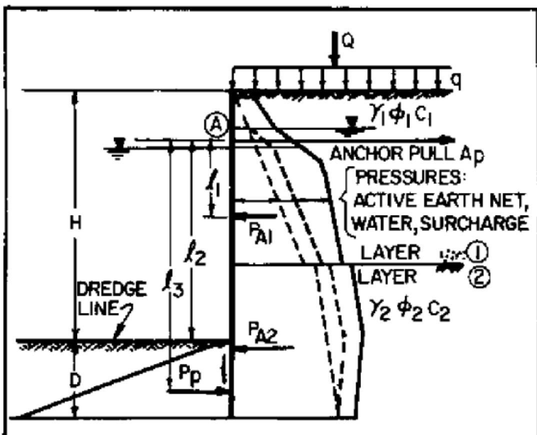
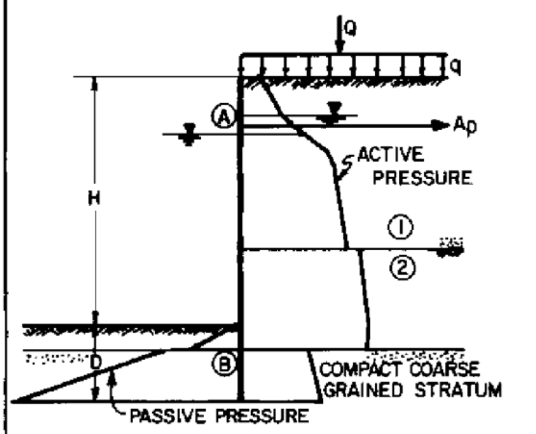


Figure E. 3 Bridge Borings



FREE EARTH SUPPORT – GENERAL CASE

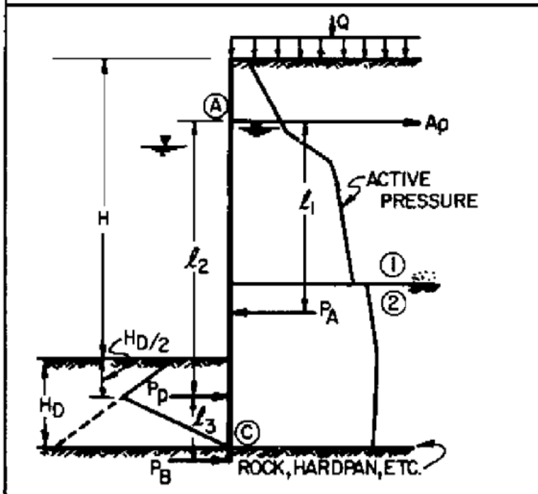
1. COMPUTE PRESSURES BY METHODS OF FIGURES 2 TO 7 PASSIVE PRESSURES FOR CLEAN COARSE GRAIN SOILS INCLUDE WALL FRICTION (δ). TABLE 1. FOR ACTIVE OR PASSIVE PRESSURES IN ALL OTHER SOIL TYPES, IGNORE WALL FRICTION.
2. DEPTH OF PENETRATION REQUIRED: TAKE MOMENTS ABOUT POINT Ⓐ AND SOLVE FOR D: $P_{A1}\ell_1 + P_{A2}\ell_2 = \frac{P_p}{F_S}\ell_3$
   F_S = 2 TO 3 FOR COARSE GRAINED SOILS
   F_S = 1.5 TO 2 FOR FINE GRAINED SOILS
3. ANCHOR PULL: $A_p = \left[P_{A1} + P_{A2} - P_p/F_S\right] d$, d = ANCHOR SPACING
4. MAXIMUM BENDING MOMENT (M_{MAX}) IN SHEETING COMPUTED BY THE FREE EARTH SUPPORT METHOD AND APPLYING $P_{A1}, P_{A2}, P_p/F_S$ AND A_p. FOR SHEETING IN SAND APPLY MOMENT REDUCTION FOR FLEXIBILITY OF FIGURE 19.
5. INCRESE PENETRATION COMPUTED (D) BY 20% TO ALLOW FOR DREDGING, SCOUR, ETC.



PENETRATION IN COMPACT COARSE GRAINED STRATUM

DESIGN STEPS 1, 2, AND 3 SAME AS ABOVE EARTH SUPPORT.

4. COMPUTE MAXIMUM BENDING MOMENT (M_{MAX}) IN SHEETING BY FREE EARTH SUPPORT METHOD APPLYING P_A, P_p/F_S AND A_p.
5. COMPUTE ρ ACCORDING TO FIGURE 19. IF $\rho \geq 20$, M_{DESIGN} IS COMPUTED FOR THE SPAN ⒶⒷ ASSUMING SIMPLE SUPPORT AT POINT Ⓑ
   IF $\rho < 20$ OBTAIN MOMENT REDUCTION FOR FLEXIBILITY FROM FIGURE 19.
6. INCREASE PENETRATION COMPUTED (D) BY 20% TO ALLOW FOR DREDGING, SCOUR, ETC.



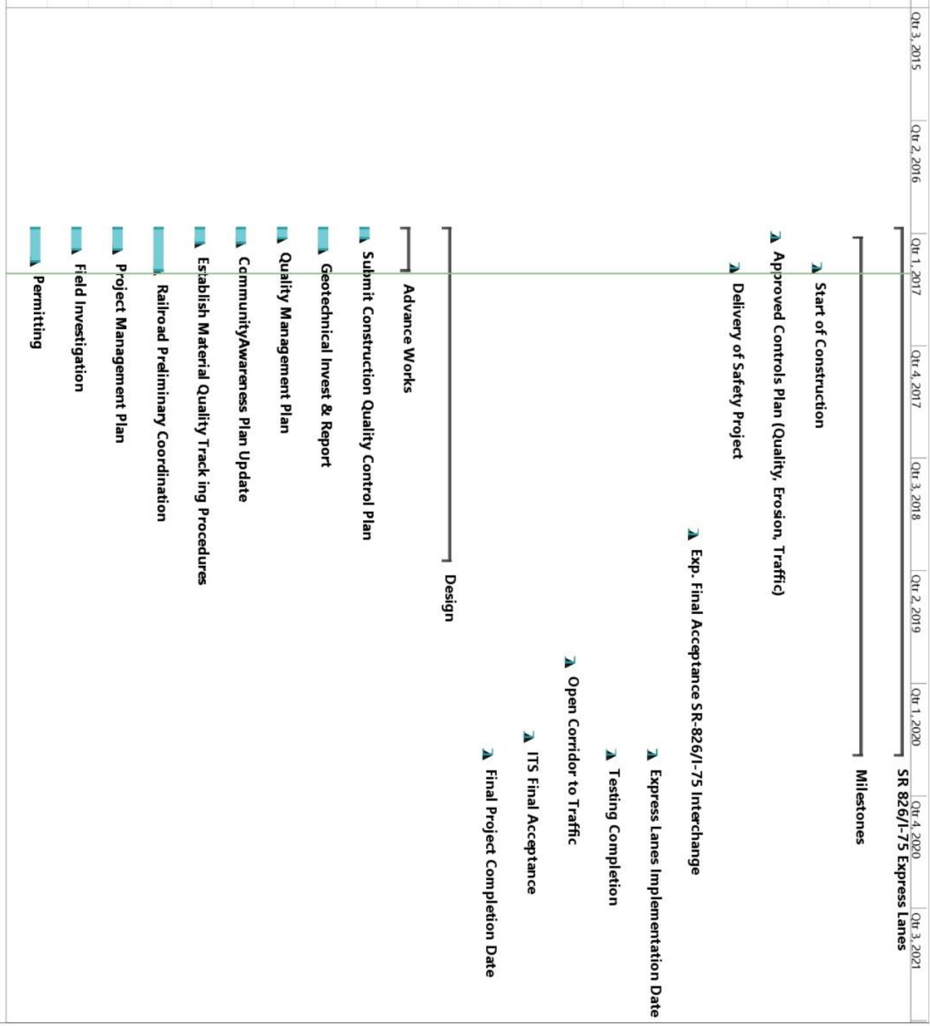
PENETRATION TO TOP OF HARD UNYIELDING STRATUM

1. COMPUTE PRESSURES AS ABOVE. EXCEPT THAT PASSIVE PRESSURE DECREASES TO ZERO AT TOP OF HARD STRATUM.
2. PENETRATION IN HARD STRATUM:
   TAKE MOMENTS ABOUT POINT Ⓐ AND SOLVE FOR P_B:
   $$P_A\ell_1 - \frac{P_p}{F_S}\ell_2 = \frac{P_B}{F_S}(\ell_2 + \ell_3)$$
   ESTIMATE IF REACTION P_B CAN BE PROVIDED BY SHALLOW PENETRATION IN HARD STRATUM.
3. ANCHOR PULL: $A_p = \left[P_A - \frac{P_p}{F_S} - \frac{P_B}{F_S}\right] d$
4. MAXIMUM BENDING MOMENT IN SHEETING COMPUTED BY APPLYING P_A, P_p AND A_p TO SPAN Ⓐ-Ⓒ ASSUMING SIMPLE SUPPORT AT Ⓒ. NO REDUCTION FOR FLEXIBILITY.

Figure E. 4 Design Criteria for Anchored Bulkhead

Appendix F

ID	Task Name	Duration	Start	Finish
1	**SR 826/I-75 Express Lanes**	**917 days?**	**Mon 19/12/16**	**Tue 23/06/20**
2	**Milestones**	**900 days?**	**Wed 11/01/17**	**Tue 23/06/20**
3	Start of Construction			Tue 28/03/17
4	Approved Controls Plan (Quality, Erosion, Traffic)			Wed 11/01/17
5	Delivery of Safety Project			Mon 27/03/17
6	Exp. Final Acceptance SR-826/I-75 Interchange			Fri 04/01/19
7	Express Lanes Implementation Date			Tue 23/06/20
8	Testing Completion			Tue 23/06/20
9	Open Corridor to Traffic			Mon 11/11/19
10	ITS Final Acceptance			Mon 11/05/20
11	Final Project Completion Date			Tue 23/06/20
12	**Design**	**579 days**	**Mon 19/12/16**	**Thu 07/03/19**
13	**Advance Works**	**75 days**	**Mon 19/12/16**	**Fri 31/03/17**
14	Submit Construction Quality Control Plan	18 days	Mon 19/12/16	Wed 11/01/17
15	Geotechnical Invest & Report	38 days	Mon 19/12/16	Wed 08/02/17
16	Quality Management Plan	19 days	Mon 19/12/16	Thu 12/01/17
17	CommunityAwareness Plan Update	26 days	Mon 19/12/16	Mon 23/01/17
18	Establish Material Quality Track ing Procedures	26 days	Mon 19/12/16	Mon 23/01/17
19	Railroad Preliminary Coordination	75 days	Mon 19/12/16	Fri 31/03/17
20	Project Management Plan	38 days	Mon 19/12/16	Wed 08/02/17
21	Field Investigation	38 days	Mon 19/12/16	Wed 08/02/17
22	Permitting	59 days	Mon 19/12/16	Thu 09/03/17

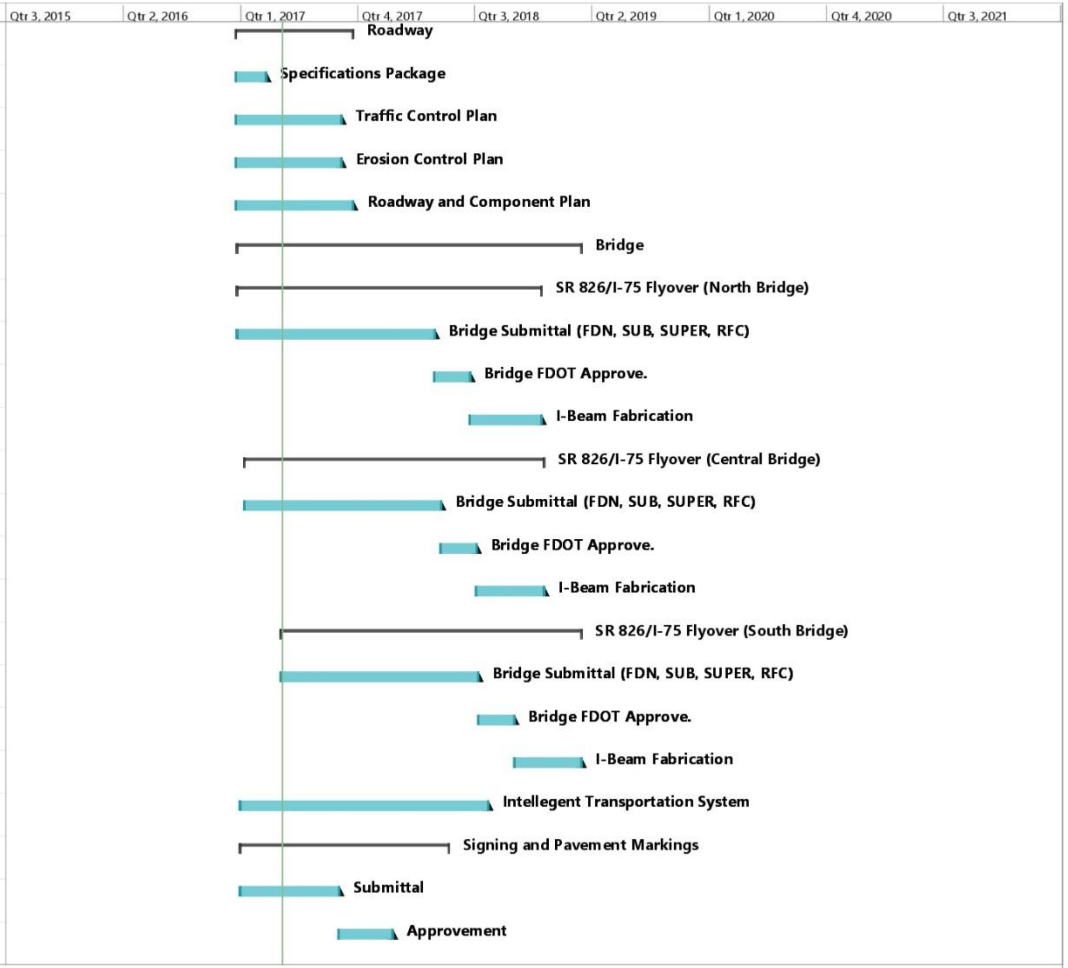


Project: Capstone Construction
Date: Sat 08/04/17

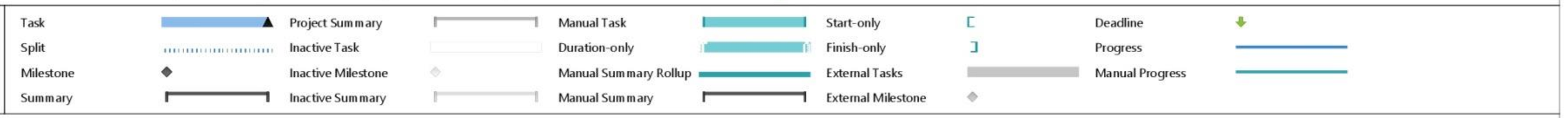
Task	Project Summary	Manual Task	Start-only	Deadline
Split	Inactive Task	Duration-only	Finish-only	Progress
Milestone	Inactive Milestone	Manual Summary Rollup	External Tasks	Manual Progress
Summary	Inactive Summary	Manual Summary	External Milestone	

Page 1

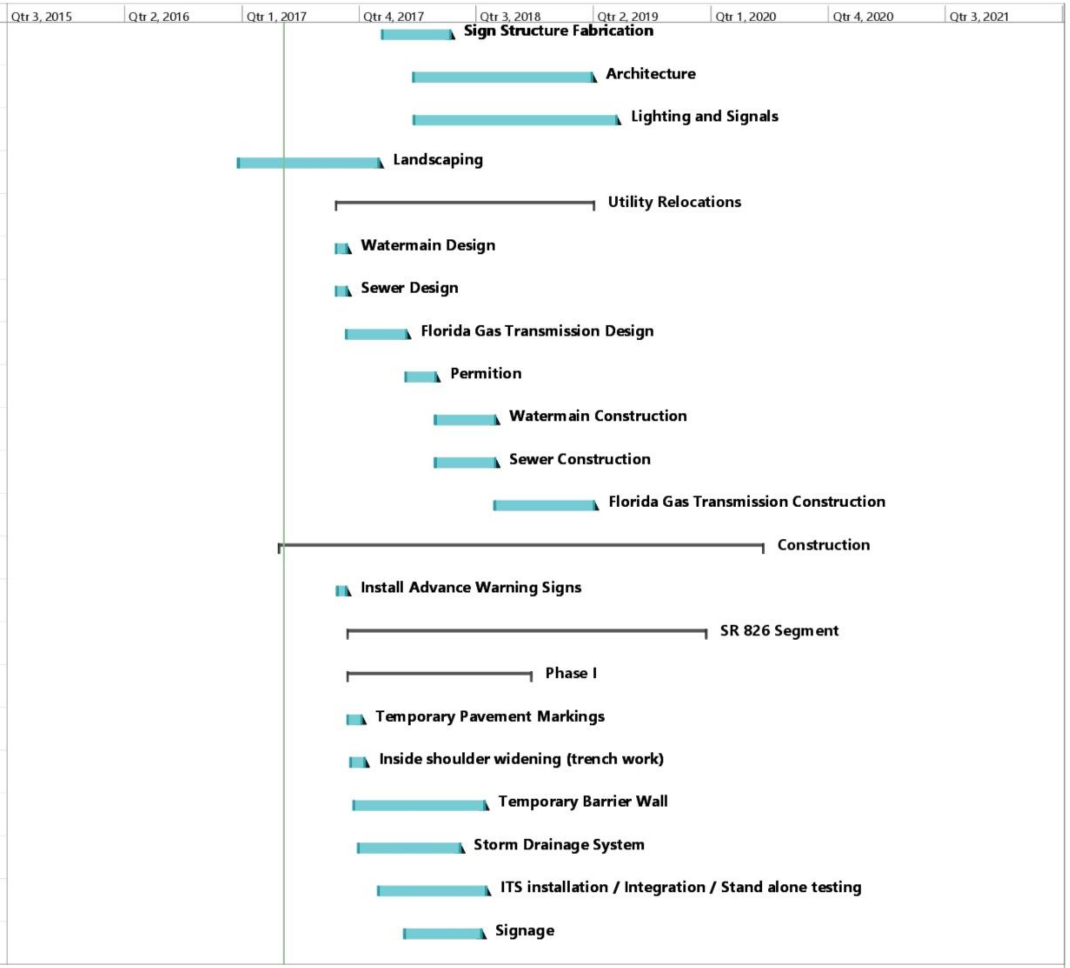
ID	Task Name	Duration	Start	Finish
23	**Roadway**	**196 days**	**Tue 20/12/16**	**Tue 19/09/17**
24	Specifications Package	51 days	Tue 20/12/16	Tue 28/02/17
25	Traffic Control Plan	177 days	Tue 20/12/16	Wed 23/08/17
26	Erosion Control Plan	177 days	Tue 20/12/16	Wed 23/08/17
27	Roadway and Component Plan	196 days	Tue 20/12/16	Tue 19/09/17
28	**Bridge**	**575 days**	**Fri 23/12/16**	**Thu 07/03/19**
29	**SR 826/I-75 Flyover (North Bridge)**	**508 days**	**Fri 23/12/16**	**Tue 04/12/18**
30	Bridge Submittal (FDN, SUB, SUPER, RFC)	330 days	Fri 23/12/16	Thu 29/03/18
31	Bridge FDOT Approve.	60 days	Thu 29/03/18	Wed 20/06/18
32	I-Beam Fabrication	120 days	Wed 20/06/18	Tue 04/12/18
33	**SR 826/I-75 Flyover (Central Bridge)**	**501 days**	**Mon 09/01/17**	**Mon 10/12/18**
34	Bridge Submittal (FDN, SUB, SUPER, RFC)	330 days	Mon 09/01/17	Fri 13/04/18
35	Bridge FDOT Approve.	60 days	Fri 13/04/18	Thu 05/07/18
36	I-Beam Fabrication	113 days	Thu 05/07/18	Mon 10/12/18
37	**SR 826/I-75 Flyover (South Bridge)**	**503 days**	**Tue 04/04/17**	**Thu 07/03/19**
38	Bridge Submittal (FDN, SUB, SUPER, RFC)	330 days	Tue 04/04/17	Mon 09/07/18
39	Bridge FDOT Approve.	60 days	Tue 10/07/18	Mon 01/10/18
40	I-Beam Fabrication	113 days	Tue 02/10/18	Thu 07/03/19
41	Intelligent Transportation System	414 days	Fri 30/12/16	Wed 01/08/18
42	**Signing and Pavement Markings**	**348 days**	**Fri 30/12/16**	**Tue 01/05/18**
43	Submittal	166 days	Fri 30/12/16	Fri 18/08/17
44	Approvement	90 days	Fri 18/08/17	Thu 21/12/17



Project: Capstone Construction
Date: Sat 08/04/17



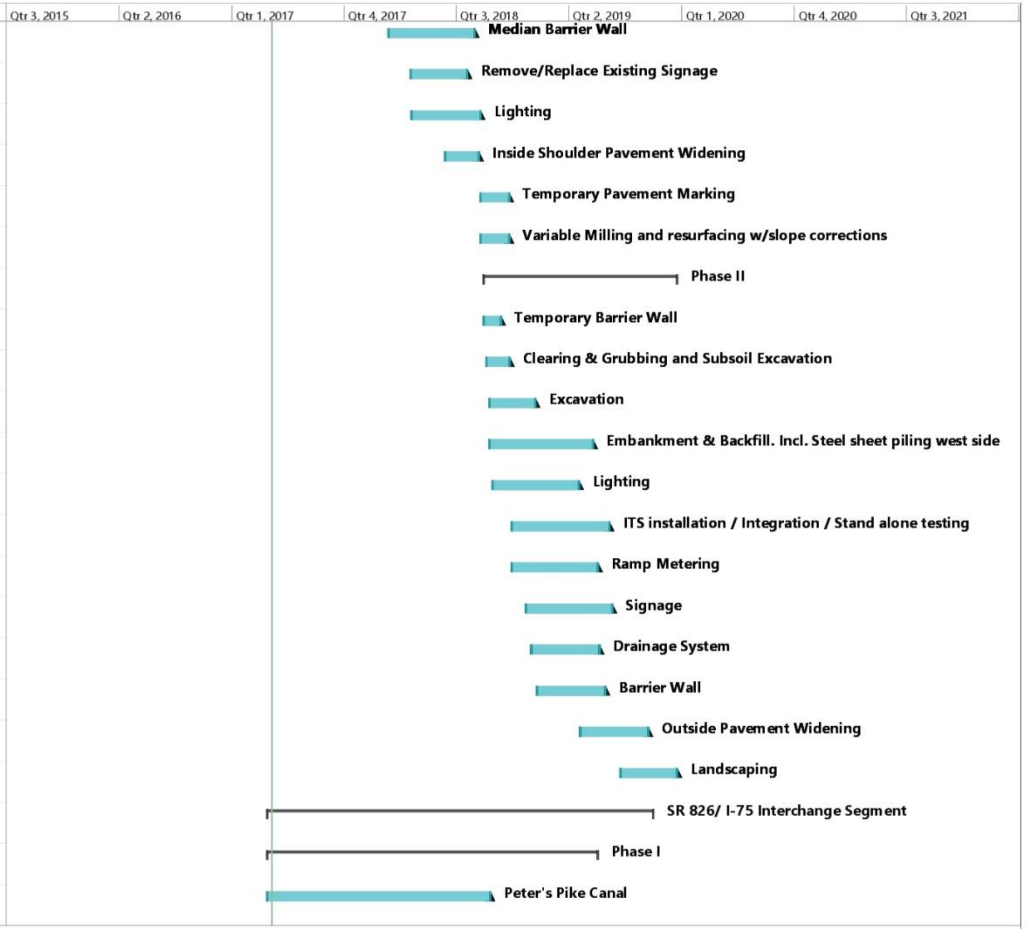
ID	Task Name	Duration	Start	Finish
45	Sign Structure Fabrication	113 days	Fri 24/11/17	Tue 01/05/18
46	Architecture	299 days	Mon 05/02/18	Thu 28/03/19
47	Lighting and Signals	339 days	Tue 06/02/18	Fri 24/05/19
48	Landscaping	234 days	Fri 23/12/16	Wed 15/11/17
49	**Utility Relocations**	**431 days**	**Tue 08/08/17**	**Tue 02/04/19**
50	Watermain Design	19 days	Tue 08/08/17	Fri 01/09/17
51	Sewer Design	19 days	Tue 08/08/17	Fri 01/09/17
52	Florida Gas Transmission Design	100 days	Fri 01/09/17	Thu 18/01/18
53	Permition	50 days	Thu 18/01/18	Wed 28/03/18
54	Watermain Construction	100 days	Wed 28/03/18	Tue 14/08/18
55	Sewer Construction	100 days	Wed 28/03/18	Tue 14/08/18
56	Florida Gas Transmission Construction	166 days	Tue 14/08/18	Tue 02/04/19
57	**Construction**	**809 days**	**Tue 28/03/17**	**Fri 01/05/20**
58	Install Advance Warning Signs	15 days	Fri 11/08/17	Thu 31/08/17
59	**SR 826 Segment**	**600 days**	**Mon 04/09/17**	**Fri 20/12/19**
60	**Phase I**	**307 days**	**Mon 04/09/17**	**Tue 06/11/18**
61	Temporary Pavement Markings	25 days	Mon 04/09/17	Fri 06/10/17
62	Inside shoulder widening (trench work)	25 days	Mon 11/09/17	Fri 13/10/17
63	Temporary Barrier Wall	220 days	Mon 18/09/17	Fri 20/07/18
64	Storm Drainage System	170 days	Fri 29/09/17	Thu 24/05/18
65	ITS installation / Integration / Stand alone testing	180 days	Wed 15/11/17	Tue 24/07/18
66	Signage	130 days	Mon 15/01/18	Fri 13/07/18



Project: Capstone Construction
Date: Sat 08/04/17

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Progress | Manual Progress

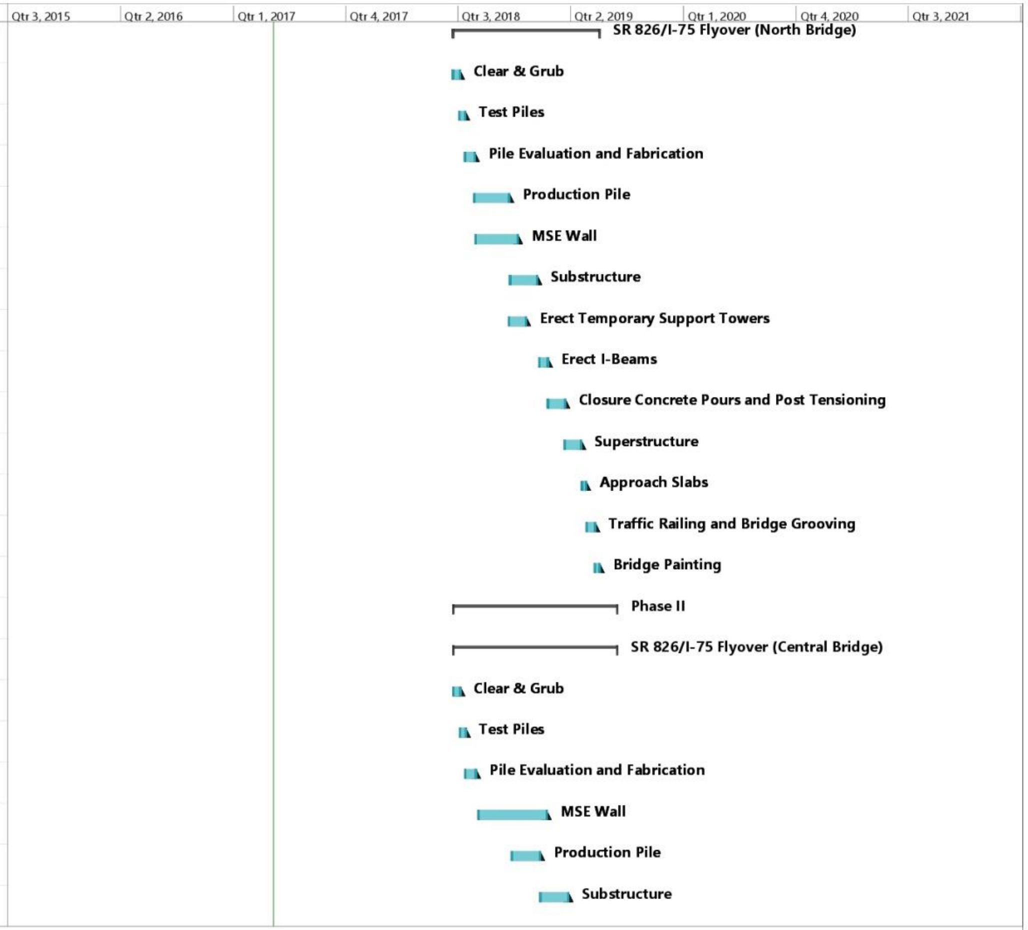
ID	Task Name	Duration	Start	Finish
67	Median Barrier Wall	150 days	Wed 17/01/18	Tue 14/08/18
68	Remove/Replace Existing Signage	100 days	Mon 12/03/18	Fri 27/07/18
69	Lighting	120 days	Wed 14/03/18	Tue 28/08/18
70	Inside Shoulder Pavement Widening	60 days	Mon 04/06/18	Fri 24/08/18
71	Temporary Pavement Marking	50 days	Wed 29/08/18	Tue 06/11/18
72	Variable Milling and resurfacing w/slope corrections	50 days	Wed 29/08/18	Tue 06/11/18
73	**Phase II**	**337 days**	**Thu 06/09/18**	**Fri 20/12/19**
74	Temporary Barrier Wall	30 days	Thu 06/09/18	Wed 17/10/18
75	Clearing & Grubbing and Subsoil Excavation	40 days	Thu 13/09/18	Wed 07/11/18
76	Excavation	80 days	Thu 20/09/18	Wed 09/01/19
77	Embankment & Backfill. Incl. Steel sheet piling west side	180 days	Thu 20/09/18	Wed 29/05/19
78	Lighting	150 days	Fri 28/09/18	Thu 25/04/19
79	ITS installation / Integration / Stand alone testing	170 days	Tue 13/11/18	Mon 08/07/19
80	Ramp Metering	150 days	Tue 13/11/18	Mon 10/06/19
81	Signage	150 days	Tue 18/12/18	Mon 15/07/19
82	Drainage System	120 days	Mon 31/12/18	Fri 14/06/19
83	Barrier Wall	120 days	Mon 14/01/19	Fri 28/06/19
84	Outside Pavement Widening	120 days	Mon 29/04/19	Fri 11/10/19
85	Landscaping	100 days	Mon 05/08/19	Fri 20/12/19
86	**SR 826/ I-75 Interchange Segment**	**672 days**	**Tue 28/03/17**	**Wed 23/10/19**
87	**Phase I**	**575 days**	**Tue 28/03/17**	**Mon 10/06/19**
88	Peter's Pike Canal	389 days	Tue 28/03/17	Fri 21/09/18



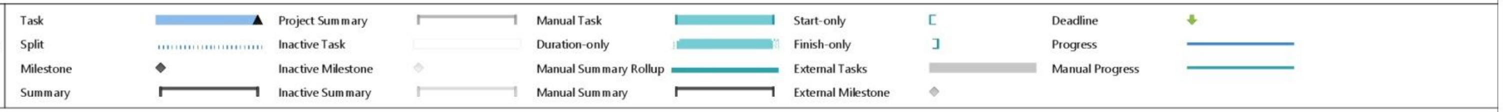
Project: Capstone Construction
Date: Sat 08/04/17

Task | Project Summary | Manual Task | Start-only | Deadline
Split | Inactive Task | Duration-only | Finish-only | Progress
Milestone | Inactive Milestone | Manual Summary Rollup | External Tasks | Manual Progress
Summary | Inactive Summary | Manual Summary | External Milestone

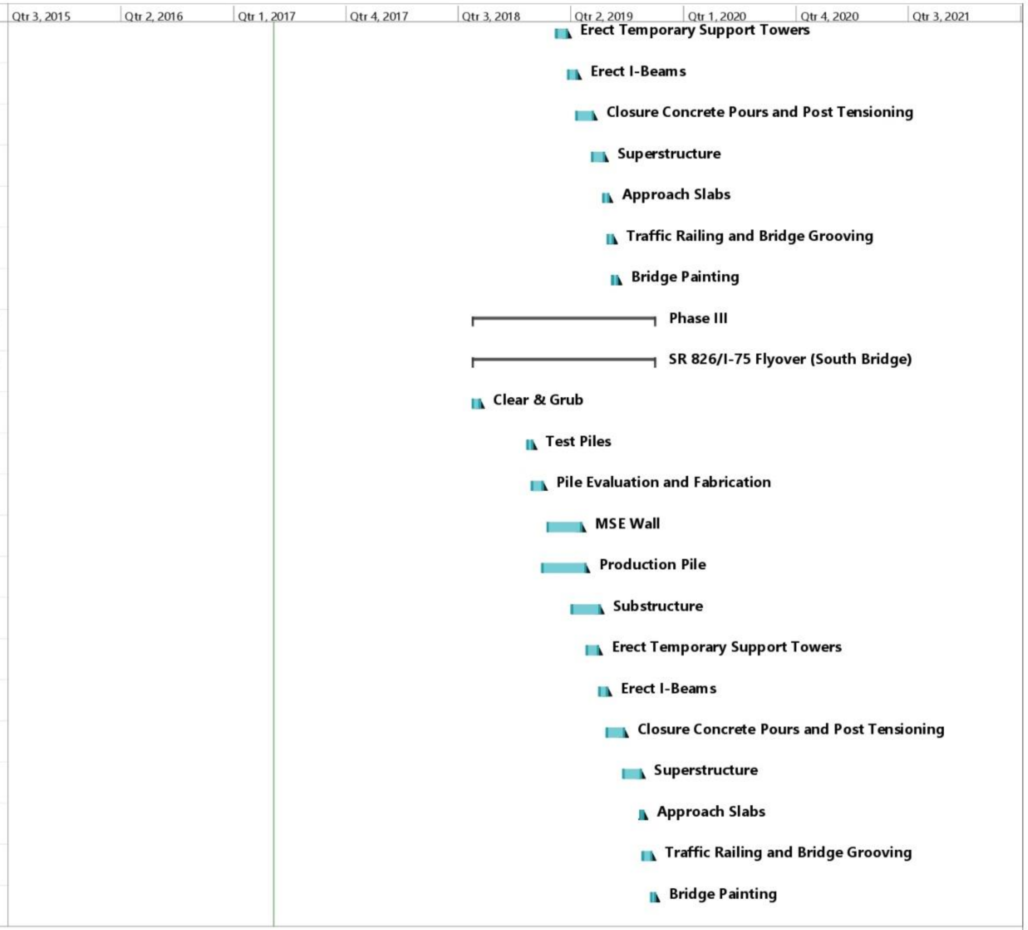
ID	Task Name	Duration	Start	Finish
89	**SR 826/I-75 Flyover (North Bridge)**	**255 days**	**Tue 19/06/18**	**Mon 10/06/19**
90	Clear & Grub	13 days	Tue 19/06/18	Thu 05/07/18
91	Test Piles	10 days	Thu 05/07/18	Wed 18/07/18
92	Pile Evaluation and Fabrication	18 days	Wed 18/07/18	Fri 10/08/18
93	Production Pile	61 days	Fri 10/08/18	Fri 02/11/18
94	MSE Wall	75 days	Mon 13/08/18	Fri 23/11/18
95	Substructure	48 days	Mon 05/11/18	Wed 09/01/19
96	Erect Temporary Support Towers	30 days	Fri 02/11/18	Thu 13/12/18
97	Erect I-Beams	15 days	Tue 15/01/19	Mon 04/02/19
98	Closure Concrete Pours and Post Tensioning	30 days	Tue 05/02/19	Mon 18/03/19
99	Superstructure	30 days	Mon 18/03/19	Fri 26/04/19
100	Approach Slabs	8 days	Mon 29/04/19	Wed 08/05/19
101	Traffic Railing and Bridge Grooving	15 days	Fri 10/05/19	Thu 30/05/19
102	Bridge Painting	8 days	Thu 30/05/19	Mon 10/06/19
103	**Phase II**	**285 days**	**Wed 20/06/18**	**Tue 23/07/19**
104	**SR 826/I-75 Flyover (Central Bridge)**	**285 days**	**Wed 20/06/18**	**Tue 23/07/19**
105	Clear & Grub	13 days	Wed 20/06/18	Fri 06/07/18
106	Test Piles	10 days	Fri 06/07/18	Thu 19/07/18
107	Pile Evaluation and Fabrication	18 days	Thu 19/07/18	Mon 13/08/18
108	MSE Wall	120 days	Mon 20/08/18	Fri 01/02/19
109	Production Pile	49 days	Fri 09/11/18	Wed 16/01/19
110	Substructure	48 days	Fri 18/01/19	Tue 26/03/19



Project: Capstone Construction
Date: Sat 08/04/17



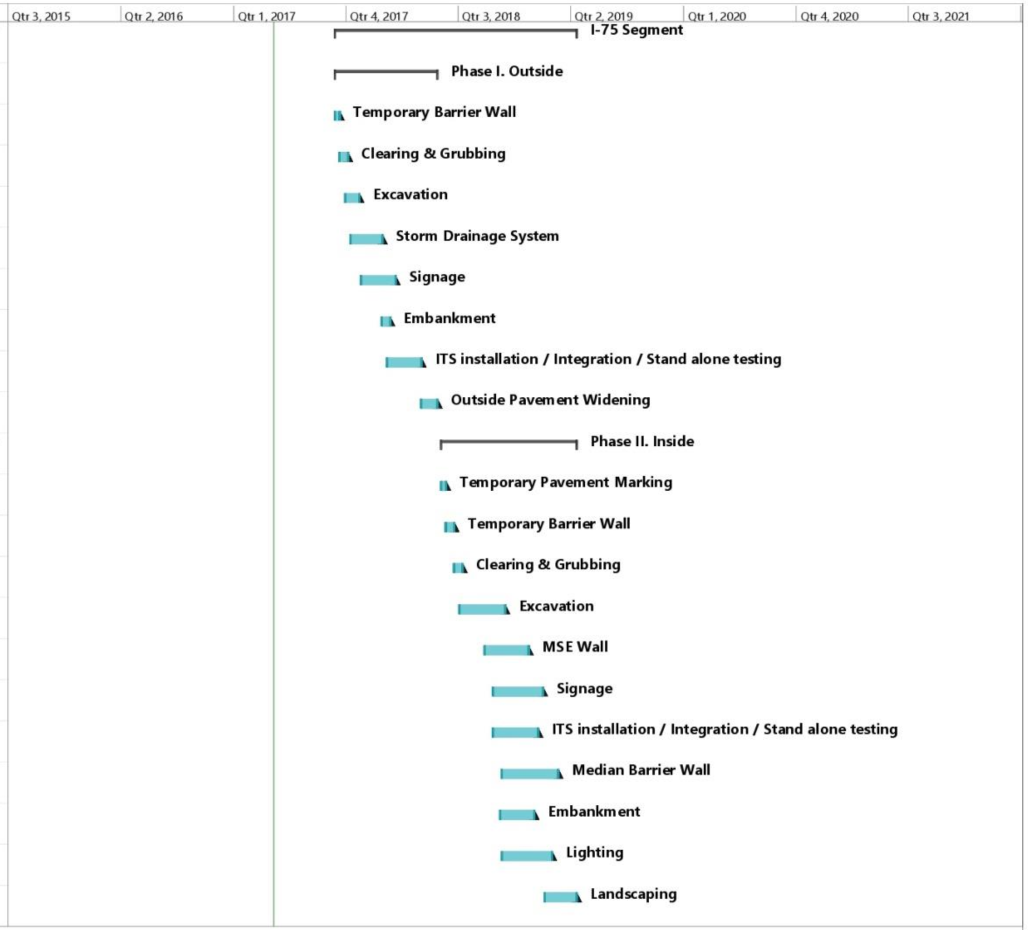
ID	Task Name	Duration	Start	Finish
111	Erect Temporary Support Towers	20 days	Mon 25/02/19	Fri 22/03/19
112	Erect I-Beams	15 days	Tue 26/03/19	Mon 15/04/19
113	Closure Concrete Pours and Post Tensioning	30 days	Mon 15/04/19	Fri 24/05/19
114	Superstructure	20 days	Fri 24/05/19	Thu 20/06/19
115	Approach Slabs	8 days	Thu 20/06/19	Mon 01/07/19
116	Traffic Railing and Bridge Grooving	10 days	Mon 01/07/19	Fri 12/07/19
117	Bridge Painting	8 days	Fri 12/07/19	Tue 23/07/19
118	**Phase III**	**318 days**	**Mon 06/08/18**	**Wed 23/10/19**
119	**SR 826/I-75 Flyover (South Bridge)**	**318 days**	**Mon 06/08/18**	**Wed 23/10/19**
120	Clear & Grub	13 days	Mon 06/08/18	Wed 22/08/18
121	Test Piles	10 days	Mon 17/12/18	Fri 28/12/18
122	Pile Evaluation and Fabrication	18 days	Fri 28/12/18	Tue 22/01/19
123	MSE Wall	60 days	Mon 04/02/19	Fri 26/04/19
124	Production Pile	75 days	Tue 22/01/19	Mon 06/05/19
125	Substructure	48 days	Thu 04/04/19	Mon 10/06/19
126	Erect Temporary Support Towers	20 days	Fri 10/05/19	Thu 06/06/19
127	Erect I-Beams	15 days	Mon 10/06/19	Fri 28/06/19
128	Closure Concrete Pours and Post Tensioning	30 days	Fri 28/06/19	Thu 08/08/19
129	Superstructure	30 days	Thu 08/08/19	Wed 18/09/19
130	Approach Slabs	5 days	Wed 18/09/19	Tue 24/09/19
131	Traffic Railing and Bridge Grooving	15 days	Tue 24/09/19	Mon 14/10/19
132	Bridge Painting	8 days	Mon 14/10/19	Wed 23/10/19



Project: Capstone Construction
Date: Sat 08/04/17

Task	Project Summary	Manual Task	Start-only	Deadline	
Split	Inactive Task	Duration-only	Finish-only	Progress	
Milestone	Inactive Milestone	Manual Summary Rollup	External Tasks	Manual Progress	
Summary	Inactive Summary	Manual Summary	External Milestone		

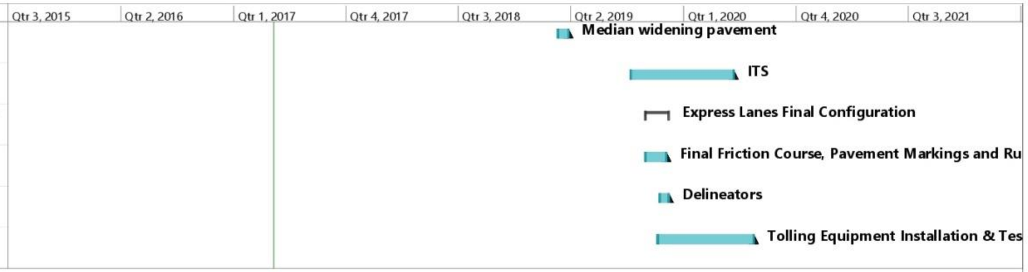
ID	Task Name	Duration	Start	Finish
133	**I-75 Segment**	**421 days**	**Mon 04/09/17**	**Mon 15/04/19**
134	**Phase I. Outside**	**180 days**	**Mon 04/09/17**	**Fri 11/05/18**
135	Temporary Barrier Wall	10 days	Mon 04/09/17	Fri 15/09/17
136	Clearing & Grubbing	15 days	Fri 15/09/17	Thu 05/10/17
137	Excavation	25 days	Fri 29/09/17	Thu 02/11/17
138	Storm Drainage System	56 days	Thu 12/10/17	Thu 28/12/17
139	Signage	60 days	Tue 07/11/17	Mon 29/01/18
140	Embankment	15 days	Wed 27/12/17	Tue 16/01/18
141	ITS installation / Integration / Stand alone testing	60 days	Tue 09/01/18	Mon 02/04/18
142	Outside Pavement Widening	30 days	Mon 02/04/18	Fri 11/05/18
143	**Phase II. Inside**	**236 days**	**Mon 21/05/18**	**Mon 15/04/19**
144	Temporary Pavement Marking	10 days	Mon 21/05/18	Fri 01/06/18
145	Temporary Barrier Wall	15 days	Fri 01/06/18	Thu 21/06/18
146	Clearing & Grubbing	15 days	Thu 21/06/18	Wed 11/07/18
147	Excavation	80 days	Wed 04/07/18	Tue 23/10/18
148	MSE Wall	78 days	Mon 03/09/18	Wed 19/12/18
149	Signage	88 days	Mon 24/09/18	Wed 23/01/19
150	ITS installation / Integration / Stand alone testing	80 days	Mon 24/09/18	Fri 11/01/19
151	Median Barrier Wall	100 days	Mon 15/10/18	Fri 01/03/19
152	Embankment	60 days	Thu 11/10/18	Wed 02/01/19
153	Lighting	88 days	Tue 16/10/18	Thu 14/02/19
154	Landscaping	56 days	Mon 28/01/19	Mon 15/04/19



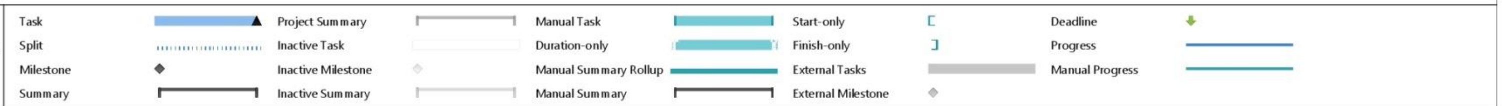
Project: Capstone Construction
Date: Sat 08/04/17

Task, Split, Milestone, Summary, Project Summary, Inactive Task, Inactive Milestone, Inactive Summary, Manual Task, Duration-only, Manual Summary Rollup, Manual Summary, Start-only, Finish-only, External Tasks, External Milestone, Deadline, Progress, Manual Progress

ID	Task Name	Duration	Start	Finish
155	Median widening pavement	18 days	Fri 01/03/19	Tue 26/03/19
156	ITS	180 days	Mon 26/08/19	Fri 01/05/20
157	**Express Lanes Final Configuration**	**41 days**	**Mon 30/09/19**	**Mon 25/11/19**
158	Final Friction Course, Pavement Markings and Rumble Strips	37 days	Mon 30/09/19	Tue 19/11/19
159	Delineators	16 days	Mon 04/11/19	Mon 25/11/19
160	Tolling Equipment Installation & Testing	168 days	Wed 30/10/19	Fri 19/06/20



Project: Capstone Construction
Date: Sat 08/04/17



Appendix G

Expressway Company.

Job Name: SR 826/I-75 EXPRESS LANES **Date: 11/03/2017**

Drilled Shaft Torsion Check (based on FDOT equation, SM Vol-9, 13.6)

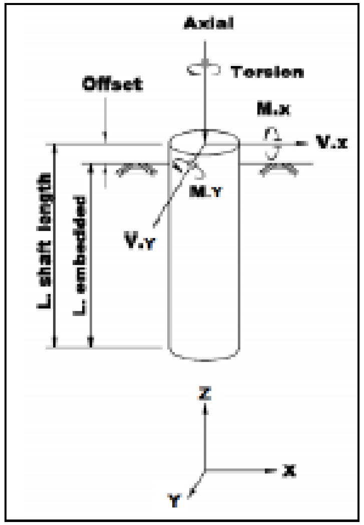
SIGN STRUCTURE No.: 1

EQUIVALENT SOIL PARAMETER ESTIMATION:

Soil Type	Elevation (ft.)		Safety N (blows/ft)	Corrected N1 (blows/ft)	ϕ (deg.)	γ_{total} (pcf)	$\gamma_{effective}$ (pcf)	Effective overburden @mid-layer (tsf)
	From	to						
Limerock Fill	5.0	2.0	27.0	49.0	38	115	115.0	0.086
Limerock Fill	2.0	-1.0	27.0	41.8	38	115	52.6	0.212
Limestone	-1.0	-5.0	10.0	13.9	36	120	57.6	0.309
Sand	-5.0	-13.0	7.0	8.9	30	105	42.6	0.452
Sand	-13.0	-18.5	12.0	14.0	32	112	49.6	0.605
Equivalent Soil Parameters				**20.3**	**33.6**		**57.3**	

Design Ground elevation, ft. (m)	5.0	ft	1.524	m
Water Elevation, ft. (m)	2.0	ft	0.6096	m
Shaft Length Above Groundwater, ft.	6.0	ft	1.8288	m
Shaft Length Below Groundwater, ft. (	17.5	ft	5.334	m

Note: SPT N corrected for overburden pressure (FDOT Soils and Foundation Handbook, P 66)



For Cohesionless Soil,

ϕ (degrees)	33.6
$\mu = \tan\phi$	0.66
Diameter (ft)	5
ω_{fdot}	1.50
Gamma (kips/ft³)	0.057
Torsional Load (kip-ft)	658.75
Axial Load (kips)	12.54
Guess Length (ft)	23.5

Torsional Resistance Components:

Skin contribution	932.2
Tip Contribution	52.8
Total	**985.0**

Torsional resistance (Ref: FDOT Structures Manual)

$$Torsion_{Skin} = \pi D L F_s \left(\frac{D}{2}\right)$$

$$Torsion_{Tip} = \pi \left(\frac{D}{2}\right)^2 L \gamma_{conc} \left(\frac{D}{3}\right) \mu$$

$$Torsion_{Total} = Torsion_{Skin} + Torsion_{Tip}$$

Where, $F_s = \sigma_v \omega_{fdot}$ and $\sigma_v = \gamma_{soil} \left(\frac{L}{2}\right)$

Calculated Safety Factor (Torsion) **1.5** **(1.3 Required)**

Expressway Company.

Job Name: SR 826/I-75 EXPRESS LANES **Date: 11/03/2017**

Drilled Shaft Lateral Load Check - Brom's Method (Short Pile in Cohesionless Material)

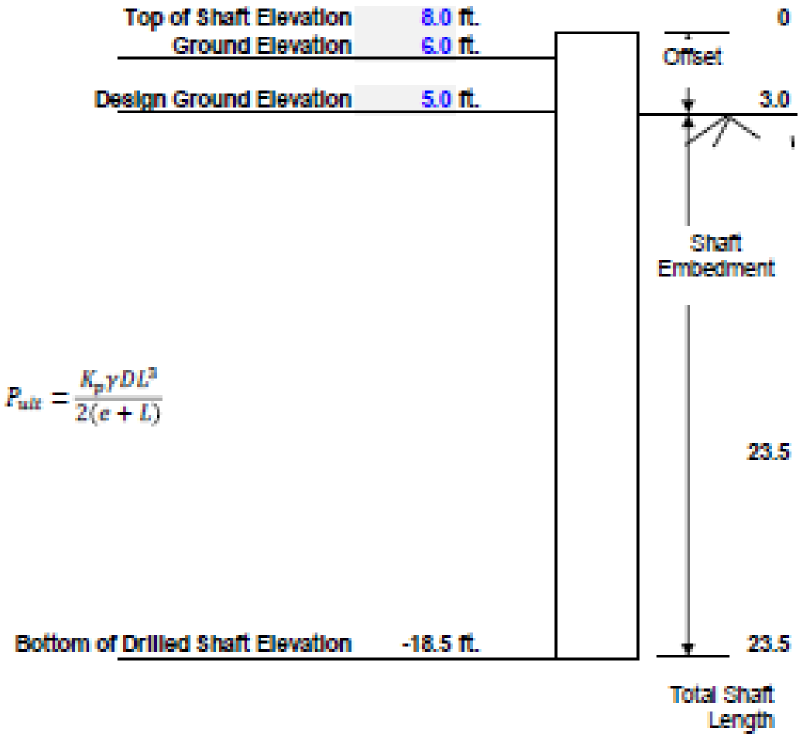
SIGN STRUCTURE No.: 1

Offset	0.0	ft	0	m
Friction Angle	33.6	degrees		
Moment (x-direction)	679.5	kip - ft	921.402	kNm
Moment (y-direction)	256.5	kip - ft	347.814	kNm
Shear (x-direction)	27.86	kips	123.921	kN
Shear (y-direction)	5.572	kips	24.7843	kN
Shaft Diameter	5	feet	1.524	m
Soil Unit Weight	0.057	kcf	918.002	kg/m3
Pile Embedment Length	23.5	feet	7.1628	m

Resultant Moment	726.3	kip-ft	984.864	kNm
Resultant Shear	28.4	kips	126.375	kN
sin (phi)	0.55			
Kp	3.47			
Equivalent "e" for applied load	25.6	ft	7.79172	m
Total "e" = Equivalent "e" + Offset	25.6	ft	7.79172	m
Ultimate Lateral Resistance	131.7	kips	585.69	kN

Safety Factor = 4.6 **Minimum Safety Factor of 2.0 required**

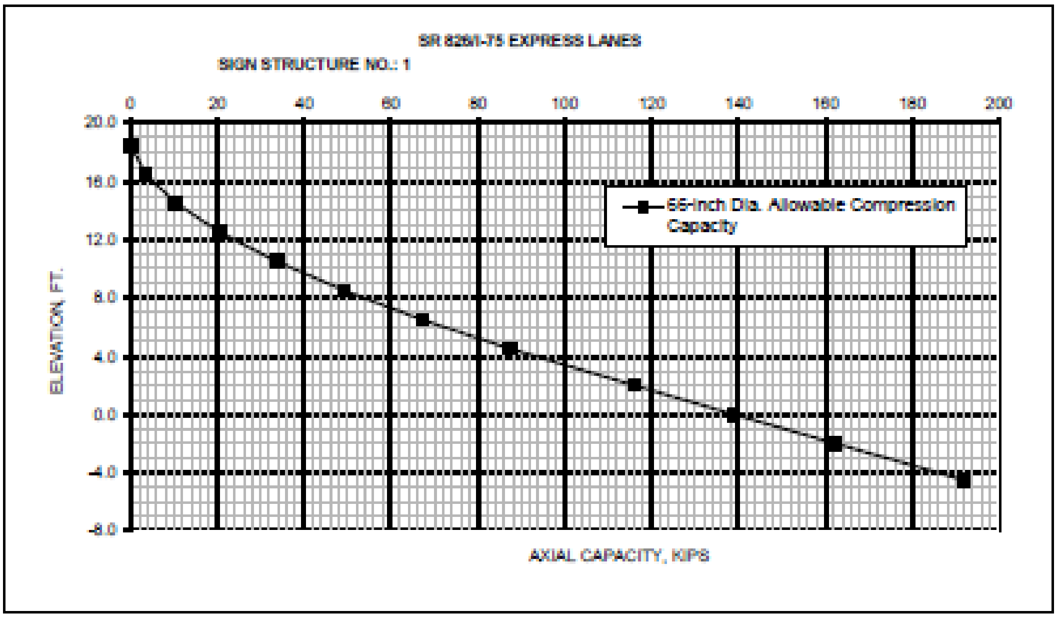
$$P_{ult} = \frac{K_p \gamma D L^3}{2(e+L)}$$



DRILLED SHAFT AXIAL COMPRESSION CAPACITY
SR 826 / I-75 EXPRESS LANES - SIGN STRUCTURES
FLORIDA DEPARTMENT OF TRANSPORTATION
SIGN STRUCTURE NO.: 1
EXPRESSWAY COMPANY

Depth, ft.	Elevation, ft.	Layer Thkns., ft.	Ave. "N" Safety	β	βc	Effective Unit Weight, pcf	Effective Vertical Stress, psf	Ultimate Side Friction, ksf	66-inch Dia. Drilled Shaft Compression Capacity (Allowable), kips
0.0	18.5	0.0	0	1.20	0.00	0.0	0	0.0	0
2.0	16.5	2.0	12	1.20	0.96	105.0	210	0.2	3
4.0	14.5	2.0	12	1.20	0.96	105.0	420	0.4	10
6.0	12.5	2.0	12	1.17	0.94	105.0	630	0.6	21
8.0	10.5	2.0	12	1.12	0.89	105.0	840	0.8	34
10.0	8.5	2.0	12	1.07	0.86	105.0	1050	0.9	49
12.0	6.5	2.0	12	1.03	0.83	105.0	1260	1.0	67
14.0	4.5	2.0	12	0.99	0.80	105.0	1470	1.2	87
16.5	2.0	2.5	12	0.95	0.76	105.0	1733	1.3	116
18.5	0.0	2.0	12	0.92	0.74	42.6	1818	1.3	139
20.5	-2.0	2.0	12	0.89	0.71	42.6	1903	1.4	162
23.0	-4.5	2.5	12	0.85	0.68	42.6	2009	1.4	192

Note: The safety factor applied on the ultimate side friction to arrive at the allowable drilled shaft compression capacity is 2.0



DESIGN OF DEADMAN ANCHORAGE
(Ref: NAVFAC DM-7.2, 1982)
SR 826/I-75 EXPRESS LANES
FLORIDA DEPARTMENT OF TRANSPORTATION
EXPRESSWAY COMPANY

Note: The spreadsheet is only applicable to the case: $h1 \geq h/2$

INPUT

Soil Properties:

Soil's unit weight (γ), pcf	115	
Angle of internal friction (φ), degree	34	
Cohesion (c), psf	0	

Deadman:

Height of deadman (h_1), ft	5	*h1>=h/2, the spreadsheet is applicable*
Depth to the base of deadman from top of bulkhead, ft	5.25	
Eff. depth to the base of deadman used for design (h), ft	9	Available depth 12 ft
Resistance factor	0.75	*Default:0.75*
Load factor	1.5	*Default: 1.5*
Coefficient of passive earth pressure (k_p)	6.54	*From Fig.5, NAVFAC DM-7.2 for - δ/φ=0.5*
Coefficient of active earth pressure (k_a)	0.28	*From Fig.3, NAVFAC DM-7.2*
Coefficient of earth pressure at rest (k_0)	0.44	
Distance b/w deadman and bulkhead, ft	28	
Width of individual deadman (b), ft	5	
C/C spacing of Anchor rods (d), ft	9.33	
Clear spacing b/w deadman anchorages (L), ft	4.33	

Bulkhead:

Depth to "zero" moment from top of bulkhead, ft	24

CALCULATIONS

Minimum distance for deadman to be outside of active failure wedge of bulkhead	12.63	ft	3.84904	m
Minimum distance for deadman to be outside of friction angle slope of bulkhead	35.21	ft	10.7323	m
Min.distance to avoid overlap of active wedge of bulkhead and passive wedge of deadman	31.68	ft	9.65651	m

Check1: OK..Deadman is outside the active failure wedge of Bulkhead wall

Check2: Overlap between active wedge of bulkhead and passive wedge of deadman

h_2	1.526	ft	0.46514	m
Passive force (P_p)	29621.8	lb/ft	42892.4	kg/m
Factored P_p	22216.35	lb/ft	32169.3	kg/m
Active force (P_a)	1304.1	lb/ft	1888.34	kg/m
Factored P_a	1956.15	lb/ft	2832.51	kg/m

Continuous wall:

Anchor resistance/linear foot (A_{pc}/d)	20.26	kip/ft	295.6744	kN/m

Individual Anchors:

Anchor resistance(A_p)	161.74	kip	719.4403	kN

Appendix H

Table H. 1 Required structural number (SN_R), 90%reliability (%R), resilient modulus (mr) range 4,000 psi to 18,000 psi, resilient modulus (M_R), (psi x 1000)

$ESAL_D$	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
100 000	3.02	2.77	2.59	2.44	2.31	2.21	2.12	2.04	1.97	1.91	1.86	1.81	1.76	1.72	1.68
150 000	3.23	2.97	2.77	2.61	2.47	2.36	2.27	2.19	2.11	2.05	1.99	1.94	1.89	1.84	1.80
200 000	3.39	3.11	2.90	2.73	2.60	2.48	2.38	2.30	2.22	2.15	2.09	2.03	1.98	1.94	1.89
250 000	3.52	3.23	3.01	2.84	2.69	2.57	2.47	2.38	2.30	2.23	2.17	2.11	2.06	2.01	1.97
300 000	3.62	3.33	3.10	2.92	2.78	2.65	2.55	2.46	2.37	2.30	2.24	2.18	2.12	2.07	2.03
350 000	3.71	3.41	3.18	3.00	2.85	2.72	2.61	2.52	2.44	2.36	2.30	2.23	2.18	2.13	2.08
400 000	3.79	3.49	3.25	3.07	2.91	2.78	2.67	2.58	2.49	2.42	2.35	2.29	2.23	2.18	2.13
450 000	3.87	3.56	3.32	3.13	2.97	2.84	2.73	2.63	2.54	2.46	2.39	2.33	2.27	2.22	2.17
500 000	3.93	3.62	3.38	3.18	3.02	2.89	2.77	2.67	2.59	2.51	2.44	2.37	2.31	2.26	2.21
600 000	4.05	3.73	3.48	3.28	3.12	2.98	2.86	2.76	2.67	2.58	2.51	2.45	2.39	2.33	2.28
700 000	4.14	3.82	3.57	3.36	3.20	3.05	2.93	2.83	2.73	2.65	2.58	2.51	2.45	2.39	2.34
800 000	4.23	3.90	3.64	3.44	3.27	3.12	3.00	2.89	2.80	2.71	2.63	2.57	2.50	2.44	2.39
900 000	4.31	3.97	3.71	3.51	3.33	3.18	3.06	2.95	2.85	2.76	2.69	2.62	2.55	2.49	2.44
1 000 000	4.38	4.04	3.78	3.57	3.39	3.24	3.11	3.00	2.90	2.81	2.73	2.66	2.60	2.54	2.48
1 500 000	4.65	4.30	4.03	3.81	3.62	3.46	3.33	3.21	3.10	3.01	2.92	2.85	2.78	2.71	2.65
2 000 000	4.85	4.50	4.21	3.99	3.79	3.63	3.49	3.36	3.25	3.16	3.07	2.99	2.91	2.85	2.78
2 500 000	5.01	4.65	4.36	4.13	3.93	3.76	3.62	3.49	3.38	3.27	3.18	3.10	3.02	2.95	2.89
3 000 000	5.14	4.77	4.48	4.25	4.05	3.88	3.73	3.60	3.48	3.37	3.28	3.19	3.12	3.04	2.98
3 500 000	5.25	4.88	4.59	4.35	4.14	3.97	3.82	3.69	3.57	3.46	3.36	3.28	3.20	3.12	3.06
4 000 000	5.35	4.98	4.68	4.44	4.23	4.06	3.90	3.77	3.65	3.54	3.44	3.35	3.27	3.19	3.12
4 500 000	5.44	5.06	4.76	4.52	4.31	4.13	3.98	3.84	3.72	3.61	3.51	3.42	3.33	3.26	3.19
5 000 000	5.52	5.14	4.83	4.59	4.38	4.20	4.04	3.90	3.78	3.67	3.57	3.47	3.39	3.31	3.24
6 000 000	5.66	5.27	4.96	4.71	4.50	4.32	4.16	4.02	3.89	3.78	3.67	3.58	3.49	3.41	3.34
7 000 000	5.78	5.38	5.07	4.82	4.61	4.42	4.26	4.12	3.99	3.87	3.77	3.67	3.58	3.50	3.43
8 000 000	5.88	5.48	5.17	4.91	4.70	4.51	4.35	4.20	4.07	3.95	3.85	3.75	3.66	3.58	3.50
9 000 000	5.97	5.57	5.26	5.00	4.78	4.59	4.43	4.28	4.15	4.03	3.92	3.82	3.73	3.65	3.57
10 000 000	6.06	5.65	5.33	5.07	4.85	4.66	4.50	4.35	4.22	4.10	3.99	3.89	3.79	3.71	3.63
15 000 000	6.39	5.97	5.64	5.37	5.14	4.95	4.77	4.62	4.48	4.36	4.25	4.14	4.05	3.96	3.88
20 000 000	6.63	6.20	5.86	5.59	5.35	5.15	4.98	4.82	4.68	4.55	4.44	4.33	4.23	4.14	4.06
25 000 000	6.82	6.38	6.04	5.76	5.52	5.32	5.14	4.98	4.84	4.71	4.59	4.48	4.38	4.29	4.20
30 000 000	6.98	6.53	6.18	5.90	5.66	5.45	5.27	5.11	4.96	4.83	4.71	4.60	4.50	4.41	4.32
35 000 000	7.12	6.66	6.31	6.02	5.78	5.57	5.38	5.22	5.07	4.94	4.82	4.71	4.61	4.51	4.42
40 000 000	7.24	6.78	6.42	6.13	5.88	5.67	5.48	5.32	5.17	5.04	4.91	4.80	4.70	4.60	4.51
45 000 000	7.34	6.88	6.52	6.22	5.97	5.76	5.57	5.41	5.26	5.12	5.00	4.88	4.78	4.68	4.59
50 000 000	7.44	6.97	6.61	6.31	6.06	5.84	5.65	5.49	5.34	5.20	5.07	4.96	4.85	4.76	4.66
60 000 000	7.61	7.13	6.76	6.46	6.21	5.99	5.79	5.62	5.47	5.33	5.21	5.09	4.98	4.88	4.79
70 000 000	7.76	7.27	6.90	6.59	6.33	6.11	5.91	5.74	5.59	5.45	5.32	5.20	5.09	4.99	4.90
80 000 000	7.88	7.40	7.01	6.70	6.44	6.22	6.02	5.85	5.69	5.55	5.42	5.30	5.19	5.09	4.99
90 000 000	8.00	7.51	7.12	6.80	6.54	6.31	6.11	5.94	5.78	5.64	5.51	5.39	5.28	5.17	5.08
100 000 000	8.10	7.60	7.21	6.90	6.63	6.40	6.20	6.02	5.86	5.72	5.59	5.47	5.35	5.25	5.15