**A Replacement/Rehabilitation of the Old Ivy Creek Bridge using Accelerated Construction Methods**

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# Little Ivy Creek Bridge Replacement using Accelerated Bridge Construction Methods

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## 1.0 Project Problem Statement

The US Route 250 bridge over Little Ivy Creek is currently in poor condition. After inspection, the Virginia Department of Transportation has determined that the bridge is in need of replacement or rehabilitation. Currently, the bridge has an average daily traffic (ADT) of 11,500 vehicles a day. Both rehabilitation or replacement will necessitate traffic be temporarily restricted on US Route 250/Ivy Road between Crozet and Charlottesville. VDOT has been asked to consider multiple delivery and construction methods, with the goal of limiting traffic impacts to a maximum of two weeks. The estimated traffic impact of conventional construction methods would be three months, while keeping one lane open at all times by using a signalized system. If accelerated construction methods are used for the rehabilitation or replacement of this bridge, a maximum traffic impact of two weeks may be feasible. Our team has been asked to determine and present the best solution possible given the above information.

## 2.0 Statement of Project Scope

Our team defined a project scope from analysis of the above problem statement. We defined three Areas of Work (AOW) as follows: geotechnical engineering/design, structural engineering/design, and project controls.

The geotechnical AOW will determine the existing soil conditions, and any changes to the existing conditions that become necessary to provide a safe, suitable foundation as the project design develops. The structural AOW will develop all substructure and superstructure designs and supporting calculations. Cost/benefit analyses of design and construction method alternatives, and preliminary cost estimating and construction scheduling of the final design are grouped under the project controls AOW.

We summarized our project scope with a Project Goal Statement as follows, "To provide a structurally sound replacement / rehabilitation of the Rt. 250 bridge over Little Ivy Creek with minimal time disruption to the travelling public, in a safe and cost-effective manner".

## 3.0 Existing Conditions

The existing bridge was built in 1932 of traditional reinforced concrete. The structure consisted of a single-span deck supported on abutments, with a spread footer foundation, and concrete railing for safety. Wing walls were used to support the roadway embankment on each side of the bridge. Current ADT is 11,500 vehicles per day. The following tables detail the bridge dimensions and creek/river conditions respectively.

**Table 1: Existing Structure Dimensions**

| | |
|---|---|
| Span Length | 35’ |
| Span Width | 35’ |
| Elevation, Start of Bridge | 517.51’ |
| Elevation, End of Bridge | 516.00’ |
| Footing Length | 36’ |
| Footing Width | 3-4’ |
| Footing Depth | 2’ |
| Earth Cover, Footing | 1.5-2.5’ |

**Table 2: Creek Conditions**

| | |
|---|---|
| Normal Water Elevation | 502.27’ |
| High Water Elevation | 509.9’ |
| Drainage Area | 2.257 sq. miles |

Our team was provided with a structural inspection report produced by VDOT in early 2018. This report detailed multiple deficiencies with the existing structures’ condition. The overall condition was rated as poor. The span showed significant honeycombing, cracking, and visible reinforcement steel. Scale was present along the outside edges. The substructure was in slightly better condition, however reinforcing steel was also exposed and efflorescence was observed. We were also provided with several borehole reports from VDOT, showing existing soil conditions in various areas surrounding the existing bridge location.

## 4.0 Design Constraints

Our team made the decision to replace rather than rehabilitate the existing structure. From the supplied inspection reports, the superstructure was determined to be in too poor condition for rehabilitation. While the substructure was a suitable candidate for rehabilitation, this would only extend its lifespan for a short time. The substructure would still need to be replaced in the near future, necessitating more traffic disruption. The team considers preventing these future traffic impacts to be sufficient justification for a full structure replacement.

From aerial footage and an in-person exploration of the site, it was clear that construction and laydown space was extremely limited. To minimize impacts to the surrounding community (namely shops, a country club, and a church), shipping prefabricated and ready-to-place members to the site would be ideal. Availability of suitable precast concrete members in the surrounding markets significantly outweighs the availability of prefabricated steel modules, and therefore concrete will be the material of choice for this structure.

## 5.0 Cost Benefit Analysis

Given the design constraints presented above, our team chose two alternatives for analysis. To minimize impacts to traffic and the nearby community, and with the goal of limiting these impacts to two weeks, accelerated construction methods were the foundation for both alternatives. Alternative 1 consisted of precast concrete modular box culverts. Alternative 2 was a 3-sided bridge similar to the existing, that would be built from precast concrete members (beams, abutments, wingwalls, etc…).

On December 26th, 2020 the team met in person with Braden Chapman, VDOT Assistant Bridge Engineer for the Culpeper district. Mr. Chapman was directly involved with the design and preconstruction process of the actual U.S. 250 bridge replacement performed by VDOT. The purpose of this meeting was to investigate the cost/benefit analysis that VDOT performed when choosing a box culvert design and accelerated construction methods, in order to inform our own choice of design. The following paragraphs summarize the information we received from this meeting.

Initially, VDOT considered building a temporary road beside the project area to serve as a short detour during construction. However, a temporary road would either interfere with the Exxon gas station directly beside the bridge, or require an expensive wall system and temporary bridge. Building a new structure underneath the existing, and then raising it into place over a very short road closure period was also considered. While this would minimize traffic impacts, it was rejected as there was not enough space for construction equipment to work safely while traffic was "live".

VDOT then performed a traffic study on Rt. 250 to determine the traffic impacts caused by a signalized single lane closure for 3 months, which would be required for traditional construction methods. Estimated traffic queues were approximately 1/2 mile in each direction due to the 11,500 vehicle ADT on Rt. 250, as seen in Figure 1. Alternatively, accelerated construction methods were estimated to require a 2-week full road closure, using Rt. 64 as a detour. A public hearing was held on January 10, 2017 to allow Albemarle county residents to provide their input and ask questions. The public was given 10 days to provide feedback on the two options presented; a single lane closure for a period of 3 months, or a full road closure for a period of 2 weeks. 74 out of 85 respondents voted in favor of the two-week closure. As a result, VDOT proceeded with an accelerated construction method.


**Figure 1: Traffic Impact of Single Lane Closure, Rt. 250**

A significant amount of engineering judgement and previous experience was used in determining what type of bridge design would be effective. A 3-sided bridge, similar to the existing structure, was considered. However, 3-sided bridges typically still have form-pour-cure foundations. VDOT engineers knew that form-pour-cure foundations were likely to require too much time to construct given the goal of a 2-week project schedule. Alternatively, precast box culverts were known to be the fastest accelerated construction method available. Therefore, VDOT chose to use precast box culverts and an accelerated construction delivery method.

VDOTs decision making process supported our team's decision to use accelerated construction methods. It also informed our choice of design alternative; specifically, a 3-sided bridge design was rejected due to VDOT's analysis of form-pour-cure foundations. As such, our team proceeded to design a precast concrete box culvert system which would be delivered using accelerated construction methods, a full road closure and detour to Rt. 64, and an expected project timeline of 2 weeks.

## 6.0 Structural Design

In structural design, estimated loads must be calculated in order to design members to resist those loads. Box culvert design guidelines from WisDOT and MnDOT were located online and consulted to inform this analysis. When a design consideration was chosen from one over the other it is specified. Preference of design considerations was primarily an issue of which reference had better information on a particular facet. To simplify the analysis of the box culvert bridge, the bridge was analyzed as a rigid frame. The approximate strip method utilizing a 1-foot wide design strip was used in order to determine the amount of reinforcement required per foot of box culvert in the direction perpendicular to traffic flow.

### 6.1: Dimensions

#### 6.1.1: Outer Dimensions

The first step in the structural analysis was determining the outer dimensions required for the box culvert. Because the existing plans were old and potentially inaccurate, it was decided that records from the USGS National Map 1-meter Digital Elevation Model (DEM) would be used to determine the depth and width of the hollow that needed to be spanned. ArcGIS Pro was used to analyze this data and generate a graph of distance along a section of the road versus elevation. Appropriate figures are included in appendix B. From this information, the bridge needed to span roughly 40 feet and have a height of roughly 11 feet. This allows the culvert to rest roughly a foot below the stream bed to allow for water flow as well as ensuring at least 2 feet of fill above the culvert to distribute the vehicular load. The depth of fill varied from 3.2 ft at one end of the box culvert to 4.6 feet on the other end. Our design consists of two twin-cell precast concrete box culverts side by side and assumed to act monolithically. The cell span, measured from the inside of one wall to the inside of the opposite wall, was thus taken as 9 feet. The cell rise, measured from the inside of the bottom slab to the inside of the top slab, was taken as 10 feet. This allows for an outside diameter roughly equal to the determined requirements; the exact dimensions are dependent upon the wall and slab thicknesses.

#### 6.1.2: Inner Dimensions

For the determination of dead loads due to self-weight and the shear and moment capacities later on, the inner dimensions were required as well. In this analysis, the minimum required thicknesses of the walls and of the top and bottom slabs were used as a starting point. If the shear or moment resistance are later found to be inadequate, the following adjustments will need to be made: 1) the dead loads from self-weight will need to be recalculated for the new thicker slabs or walls, 2) the dimensions of the representative frame will need to be changed appropriately, and 3) the starting and ending magnitudes of various forces will need to be recalculated to account for these changes in dimension. From the MnDOT guidelines, included in the appendix, the minimum top slab thickness is 9 inches and the minimum bottom slab thickness is 10 inches for a box culvert with a span greater than 8 ft. From the WisDOT guidelines, included in the appendix, the minimum wall thickness is 10 inches for a box culvert with a rise greater than or equal to 10 feet and an apron wall height less than 11.75 feet. The haunch size was chosen to be 12 inches vertically and horizontally as per MnDOT guidelines. For our design, the total bridge span thus comes out to 41 feet and the bridge height comes out to be 11 feet 7 inches. A CAD drawing of the culvert with dimensions and one graphically explaining variable names are included in appendix A. The dimensions are summarized below:

**Table 3: Summary of Box Culvert Design Dimensions**

| Dimension | Value |
|---|---|
| Cell Span, S | 9 ft |
| Cell Rise, R | 10 ft |
| Wall thickness, $t_W$ | 10 in |
| Haunch thickness, $t_H$ | 12 in |
| Bottom slab thickness, $t_B$ | 10 in |
| Top slab thickness, $t_T$ | 9 in |
| Outside Bridge Span, $B_C$ | 41 ft |
| Outside Bridge Height, $H_C$ | 11 ft 7 in |
| Fill Height on Top Left Side, $H_{S,TL}$ | 38.33 in |
| Fill Height on Top Right Side, $H_{S,TR}$ | 54.76 in |
| Fill Height on Bottom Left Side, $H_{S, BL}$ | 177.33 in |
| Fill Height on Bottom Right Side, $H_{S,BR}$ | 193.76 in |

### 6.2: Loads

The following load types were taken into account: component dead loads (DC), vertical earth loads (EV), horizontal earth loads (EH), live load surcharge (LS), water loads (WA), dynamic load allowance (IM), and vehicular live loads (LL). Additionally, the bottom slab will be subject to the reaction forces from the soil. This will be calculated by summing the total forces in the vertical direction and applying them equally over the area of the slab. This assumes that loads are equally distributed over the bottom slab, as per the WisDOT guidelines. All loads are calculated based on the 1-foot wide design strip, $W_D$. The loads will be load factored and the members checked for adequacy for both strength and service requirements.

#### 6.2.1: Load Combinations

##### 6.2.1.1: Strength Limit States

As per the MnDOT guidelines, the following load combinations should be considered for strength requirements:

Ia: Maximum vertical load and maximum horizontal load:

$$1.25DC + (1.30)(1.05)EV + 1.75(LL + IM) + (1.35)(1.05)EH_{max} + 1.75LS$$

Ib: Maximum vertical load and minimum horizontal load:

$$1.25DC + (1.30)(1.05)EV + 1.75(LL + IM) + 1.00WA + (0.9/1.05)EH_{min}$$

Ic: Minimum vertical load and maximum horizontal load:

$$0.90DC + (0.9/1.05)EV + (1.35)(1.05)EH_{maxi} + 1.75LS$$

### 6.2.1.2: Service Limit States

The following load combinations should be checked for service requirements:
Ia: Maximum vertical load and maximum horizontal load:

$$1.00DC + 1.00EV + 1.00(LL + IM) + 1.00EH_{max} + 1.00LS$$

Ib: Maximum vertical load and minimum horizontal load:

$$1.00DC + 1.00EV + 1.00(LL + IM) + 1.00WA + 1.00EH_{min}$$

Ic. Minimum vertical load and maximum horizontal load:

$$1.00DC + 1.00EV + 1.00EH_{maxi} + 1.00LS$$

### 6.2.2: Component Dead Loads (DC)

The component dead loads are due to self-weight of the box culvert structure assuming Normal Weight Concrete with a density, $\gamma_C$, of 150 lb/ft$^3$. They include: self-weight of the top slab acting as a distributed load along the length of the top slab, self-weight of the bottom slab acting as a distributed load along the length of the bottom slab, the self-weight of the walls acting as a point load on the base of the wall, and the self-weight of the haunches acting as a point load on the base of the wall.

#### 6.2.2.1: Self-weight of Top Slab

The self-weight of the top slab acts as a uniformly distributed load distributed along the length of the top slab. It is calculated using:

$$\omega_{DC,T} = \gamma_c * t_T * W_D$$

$$\omega_{DC,T} = 150\frac{lb}{ft^3} * \left(9in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{DC,T} = 112.5\frac{lb}{linear\ foot}$$

#### 6.2.2.2: Self-weight of Bottom Slab

The self-weight of the bottom slab acts as a uniformly distributed load distributed along the length of the bottom slab. It is calculated using:

$$\omega_{DC,B} = \gamma_c * t_B * W_D$$

$$\omega_{DC,B} = 150\frac{lb}{ft^3} * \left(10in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{DC,B} = 125\ lb/linear\ foot$$

#### 6.2.2.3: Self-weight of Walls

The self-weight of the walls acts as a point load acting at the base of the walls. It is calculated as:

$$P_{DC,W} = \gamma_c * t_W * Rise * W_D$$

$$P_{DC,W} = 150\frac{lb}{ft^3} * \left(10in * \frac{ft}{12in}\right) * 10ft * 1ft$$

$$P_{DC,W} = 1250\ lb/wall$$

### 6.2.2.4: Self-weight of Haunches

The self-weight of the haunches is assumed to act as a point load acting at the base of the walls. This is a simplification as the force has an eccentricity from the wall associated with it, but this force is small and the eccentricity is assumed to be negligible. It is calculated as:

$$P_{DC,H} = \gamma_c * \frac{1}{2} t_H^2 * W_D$$

$$P_{DC,H} = 150 \frac{lb}{ft^3} * \frac{1}{2}(1ft^2) * 1ft$$

$$P_{DC,H} = 75\ lb/haunch$$

### 6.2.3: Vertical Earth Loads (EV)

The vertical earth loads take into account the weight of the fill from the top of the top slab to the top of the pavement surface and acts as a distributed load over the length of the top slab. Fill density, $\gamma_S$, is taken as 120 lb/ft$^3$. It includes an interaction factor, $F_{VE}$, in this case for embankment conditions. Because the soil height is different on the left and right sides of the culvert, this will be a trapezoidal load. For the left side, the interaction factor is given by:

$$F_{EV} = 1 + 0.20 \frac{H_s}{B_c} \leq 1.15$$

$$F_{EV,L} = 1 + 0.20 * \frac{38.33in}{492in} \leq 1.15$$

$$F_{EV,L} = 1.016 \leq 1.15$$

And the vertical earth load acting on a 1-foot design strip for the left side is given by:

$$\omega_{EV} = F_{EV} * \gamma_S * H_S * W_D$$

$$\omega_{EV,L} = 1.016 * 120 \frac{lb}{ft^3} * \left(38.33in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EV,L} = 389.4\ lb/linear\ foot$$

For the right side, the interaction factor is given by:

$$F_{EV,R} = 1 + 0.20 * \frac{54.76in}{492in} \leq 1.15$$

$$F_{EV,R} = 1.022 \leq 1.15$$

And the vertical earth load acting on a 1-foot design strip for the right side is given by:

$$\omega_{EV,R} = 1.022 * 120 \frac{lb}{ft^3} * \left(54.76in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EV,R} = 559.6\ lb/linear\ foot$$

### 6.2.4: Horizontal Earth Loads (EH)

The horizontal earth loads take into account the soil pressure on the exterior walls. This pressure increases with depth, d, and is found using the equivalent fluid method. For at-rest conditions, the coefficient of at-rest lateral pressure, $K_0$, is taken as 0.5. Because our load cases include a minimum and maximum lateral earth pressure, we take the minimum lateral earth pressure as half that of the maximum lateral earth pressure, or a $K_0$ value of 0.25. The equation for the distributed lateral earth force, found by multiplying the lateral earth pressure by the design strip width of 1 foot is given by:

$$\omega_{EH} = K_0 * \gamma_s * d * W_D$$

At the top left of the culvert, depth = $H_{S,TL}$:

$$\omega_{EH,TL_{max}} = 0.5 * 120 \frac{lb}{ft^3} * \left(38.33in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,TL_{max}} = 192\ lb/linear\ foot$$

$$\omega_{EH,TL_{min}} = 0.25 * 120 \frac{lb}{ft^3} * \left(38.33in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,TL_{min}} = 96\ lb/linear\ foot$$

At the bottom left of the culvert, depth = $H_{S,BL}$:

$$\omega_{EH,BL_{max}} = 0.5 * 120 \frac{lb}{ft^3} * \left(177.33in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,BL_{max}} = 887\ lb/linear\ foot$$

$$\omega_{EH,BL_{min}} = 0.25 * 120 \frac{lb}{ft^3} * \left(177.33in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,BL_{min}} = 443\ lb/linear\ foot$$

At the top right of the culvert, depth = $H_{S,TR}$:

$$\omega_{EH,TR_{max}} = 0.5 * 120 \frac{lb}{ft^3} * \left(54.76in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,TR_{max}} = 274\ lb/linear\ foot$$

$$\omega_{EH,TR_{min}} = 0.25 * 120 \frac{lb}{ft^3} * \left(54.76in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,TR_{min}} = 137\ lb/linear\ foot$$

At the bottom right of the culvert, depth = $H_{S,BR}$:

$$\omega_{EH,BR_{max}} = 0.5 * 120 \frac{lb}{ft^3} * \left(193.76in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,BR_{max}} = 969\ lb/linear\ foot$$

$$\omega_{EH,BR_{min}} = 0.25 * 120 \frac{lb}{ft^3} * \left(193.76in * \frac{ft}{12in}\right) * 1ft$$

$$\omega_{EH,BR_{min}} = 484\ lb/linear\ foot$$

### 6.2.5: Live Load Surcharge (LS)

A live load surcharge is applied laterally on an exterior wall of the box culvert and an equal and opposite equilibrating force is applied to the opposite wall when a vehicular load is expected to act on the surface of the backfill within a distance equal to half the distance from the top of the pavement to the bottom of the box culvert. This is obviously the case for our box culvert as it is used as a bridge. The live load surcharge utilizes an equivalent height corresponding to the actual fill height of soil above the culvert. The equivalent height decreases with actual height according to an AASHTO table found from WisDOT that is included in the appendix. For depths not listed in the table, linear interpolation is used to find the equivalent height. It also includes an active coefficient of lateral earth pressure, $K_a$, of 0.33. Because our load cases include a maximum and minimum horizontal load, we needed to determine whether a vehicle approaching from the left side or from the right side would cause the maximum condition. This, as a consequence of the equivalent height decreasing with depth, is the side with less fill above the top of the culvert – the left side. The minimum condition is when there is no live load surcharge, so the live load surcharge for a vehicle approaching from the right side is not checked.

The live load surcharge is given by:

$$\omega_{LS} = K_a * \gamma_s * h_{eq} * W_D$$

At the top left of the culvert, depth = $H_{S,TL}$:

$$h_{eq,TL} = 4ft_{eq}$$

$$\omega_{LS,TL} = 0.33 * 120\frac{lb}{ft^3} * 4ft * 1ft$$

$$\omega_{LS,TL} = 158\ lb/linear\ foot$$

At the bottom left of the culvert, depth = $H_{S,BL}$:

$$h_{eq,BL} = 3ft_{eq} - \left(\frac{\left[177.33in * \frac{ft}{12in}\right] - 10ft}{20ft - 10ft}\right) * \left(3ft_{eq} - 2ft_{eq}\right)$$

$$h_{eq,BL} = 2.52\ ft_{eq}$$

$$\omega_{LS,BL} = 0.33 * 120\frac{lb}{ft^3} * 2.52ft * 1ft$$

$$\omega_{LS,BL} = 100\ lb/linear\ foot$$

### 6.2.6: Water Load (WA)

Load cases considering situations where the culvert is both full of water and empty must be analyzed. This is done using a hydrostatic distribution. The lateral water pressure for a full culvert is applied on the walls and bottom slab away from the center of the culvert. At the top of the culvert:

$$\omega_{WA,T} = 0\ lb/linear\ foot$$

At the bottom of the culvert:

$$\omega_{WA,B} = \gamma_w * Rise * W_D$$
$$\omega_{WA,B} = 62.4\frac{lb}{ft^3} * 10ft * 1ft$$
$$\omega_{WA,B} = 624\ lb/linear\ foot$$

### 6.2.7: Dynamic Load Allowance (IM)

The dynamic load allowance for culverts is reduced based on the depth of fill over the culvert. It will increase the effect of the vertical vehicular live load. Because it reduces with depth of fill, approach from the left side is considered as a maximum case. The minimum case is when there is no dynamic loading and thus approach from the right side is not checked. For both the strength and service limit states, the dynamic load allowance is given by:

$$IM = 33 * (1.0 - 0.125 * H_S)$$
$$IM = 33 * \left(1.0 - 0.125 * 38.33in * \frac{ft}{12in}\right)$$
$$IM = 19.8\%$$

### 6.2.8: Vehicular Live Loads (LL)

Live loads are assumed to distribute laterally with depth. Designers are permitted to increase the footprint of the load with increasing depth of fill. The load is spread laterally 1.15 times the height of fill in each direction for every foot of fill above the culvert. The intensity of live loads at a given depth is assumed to be uniform over the entire footprint. As per the MnDOT guidelines, the tire contact area for each wheel has a width of 20 inches and a length of 10 inches. A multiple presence factor, MPF, of 1.20 on a single loaded lane is used for both the strength and service limit states. Per AASHTO specifications, a tandem truck axle and a single HL-93 truck axle configuration must be checked. An axle width of 6 feet is used for both the truck and the tandem. A wheel spacing of 4 feet is used for the design tandem. A loaded wheel weight, $P_w$, of 16000 pounds is used for the design truck and a wheel weight of 12500 pounds is used for the design tandem. Additionally, the truck has a wheel weight of 4000 pounds on the front of the cab. Because the footprint is spread out more with increasing depth of fill, the maximum vertical load case is where the fill is shallowest – the left side. Because the minimum vertical load case is when there is no vehicular live load, the right side is not checked. As per MnDOT guidelines, our culvert has a span of less than 15 feet and thus lane loads are not applied.

The weight of a single axle of the HL-93 truck, taking into account the dynamic load allowance, is:

$$F_{Truck} = 2 * P_{w,Truck} * MPF * (1 + IM)$$
$$F_{Truck} = 2 * 16000lb * 1.2 * (1 + 0.198)$$
$$F_{Truck} = 46013\ lb$$

The weight of a single axle of the HL-93 truck cab, taking into account the dynamic load allowance, is:

$$F_{Cab} = 2 * P_{w,Cab} * MPF * (1 + IM)$$
$$F_{Cab} = 2 * 4000lb * 1.2 * (1 + 0.198)$$
$$F_{Cab} = 11503\ lb$$

The weight of a single axle of the design tandem, taking into account the dynamic load allowance, is:

$$F_{Tandem} = 2 * P_{w,Tandem} * MPF * (1 + IM)$$
$$F_{Tandem} = 2 * 12500lb * 1.2 * (1 + 0.198)$$
$$F_{Tandem} = 35947\ lb$$

The footprint widths and lengths allow us to spread the weight of the axles of the design truck and design tandem over a footprint area. The weight is thus spread out over the width of the footprint perpendicular to the roadway and over the length of the footprint parallel to the roadway. The width of the load footprint at the top of the culvert is given by:

$$W = W_{axle} + W_{tire} + 1.15 * H_S$$

The width of the footprint for both the HL-93 truck and the design tandem at the top left of the culvert is thus:

$$W = 6ft + 20in * \frac{ft}{12in} + 1.15 * 38.33in * \frac{ft}{12in}$$
$$W = 11.34\ ft$$

The length of the footprint for an HL-93 truck at the top left of the culvert is given by:

$$L_{Truck} = L_{tire} + 1.15 * H_S$$
$$L_{Truck} = 10in * \frac{ft}{12in} + 1.15 * 38.33in * \frac{ft}{12in}$$
$$L_{Truck} = 4.51\ ft$$

The length of the footprint for the tandem axle at the top left of the culvert is given by:

$$L_{Tandem} = Wheel\ Spacing + L_{tire} + 1.15 * H$$
$$L_{Tandem} = 4ft + 10in * \frac{ft}{12in} + 1.15 * 38.33in * \frac{ft}{12in}$$
$$L_{Tandem} = 8.51\ ft$$

Now that we have the weight of the axles and the length and width of the footprints, we can calculate an area load on the load footprint by dividing the load of the axle by the footprint area of the axle. SAP2000 can model a group of moving point loads across a frame. We are looking at a 1-foot design width which will take its representative fraction of the footprint width's load. We can thus reduce this area load on the load footprint area to a point load moving across the frame and its design width of:

$$P = \frac{F_{Vehicle}}{W} * W_D$$

For the loaded axle of an HL-93 truck this load comes out to be:

$$P_{Truck} = \frac{F_{Truck}}{W} * W_D$$

$$P_{Truck} = \frac{46013lb}{11.34\ feet} * 1ft$$
$$P_{Truck} = 4058\ lb$$
For the cab axle of an HL-93 truck this load comes out to be:
$$P_{Cab} = \frac{F_{Cab}}{W} * W_D$$
$$P_{Cab} = \frac{11503lb}{11.34\ feet} * 1ft$$
$$P_{Cab} = 1015\ lb$$
For the axle of a design tandem this load comes out to be:
$$P_{Tandem} = \frac{F_{Tandem}}{W} * W_D$$
$$P_{Tandem} = \frac{35947lb}{11.34\ feet} * W_D$$
$$P_{Tandem} = 3170\ lb$$
The point loads are moved across the top of the frame in SAP2000 along a path. The design tandem is assumed to have a distance of 4 feet between its axle point loads as per AASHTO code. To take into account the varying lengths of the design truck, a short truck and a long truck were modeled. Both have a distance between the point load from the cab's axle to the first loaded axle of 14 feet. The short truck then has a distance of 14 feet from loaded axle to loaded axle. The long truck has a distance of 26 feet from loaded axle to loaded axle. The AASHTO code specifies that the distance from loaded axle to loaded axle varies from between 14 feet and 30 feet. The 26-foot distance was chosen so that the entire length of the truck would fit on the bridge at the same time, as the model is only 40.17 feet in length. Appendix B contains figures from the AASHTO code and a CAD drawing of the model.

### 6.2.9: Reaction Forces on Bottom Slab

As stated previously, loads on the bottom slab are analyzed as a vertical earth pressure load resisting the box culvert. This assumes that the forces are evenly distributed across the bottom slab. This is a reasonable assumption for new box culverts.
The distributed reaction forces on the bottom of the slab are then given by:
$$\omega_{R,B} = \sum \frac{\mathrm{\omega_i} * \mathrm{L_i} + \mathrm{P_i}}{\mathrm{B_C}}$$
and must be checked for each load combination.

### 6.2.10: Load Summary by Location

**Table 4: Load Summary by Location**

| Applied Load Location | Applied Load Components |
|---|---|
| Top slab length | $\omega_{DC,T} + \omega_{EV} + \omega_{LL+IM}$ |
| Bottom slab length | $\omega_{DC,B} + \omega_{R,B}$ |
| Wall base | $P_{DC,W} + P_{DC,H}$ |
| Left exterior wall length | $\omega_{EH} + \omega_{LS} + \omega_{WA}$ |
| Right exterior wall length | $\omega_{EH} + \omega_{LS} + \omega_{WA}$ |

### 6.3: Modeling in SAP2000

Structural analysis was done in SAP2000. The aforementioned load patterns (location and forces) were defined and load cases were defined as combinations of these load patterns. The dimensions of the model were taken with members modeled as being located in the middle of their respective walls or slabs. The bottom left joint was modeled as a pinned joint and the other bottom joints were modeled as rollers. A CAD drawing of the model with dimensions and the model rendered in SAP2000 with member and joint numbers is included in the appendix.
In order to obtain the equilibrating force on the bottom slab, the model was run once without a force on the bottom slab. The base reactions were then determined from the analysis. The equilibrating force was taken to be the base reaction force in the vertical direction divided by the width of the box culvert.

### 6.3.1: Summary of Structural Analysis Results

The results obtained from SAP2000 were analyzed to determine the maximum axial force, shear force, and moment in each member type for both the strength limit states and the service limit states. The member types are as follows: the top slab, the bottom slab, a single wall, and the double wall where the two twin-celled box culverts meet. The results are summarized below.

#### 6.3.1.1: Summary of Strength Limit State Results

**Table 5: Summary of Strength Limit State Results**

| | Top Slab | Bottom Slab | Single Wall | Double Wall |
|---|---|---|---|---|
| **Maximum Axial Force (kip)** | 0.355 | 1.367 | -2.436 | -5.437 |
| **Minimum Axial Force (kip)** | -5.754 | -7.651 | -20.862 | -21.736 |
| **Maximum Shear Force (kip)** | 13.264 | 6.088 | 6.386 | 0.851 |
| **Minimum Shear Force (kip)** | -13.941 | -6.147 | -7.085 | -1.19 |
| **Maximum Moment (ft*kip)** | 15.941 | 11.9068 | 14.8123 | 11.9558 |
| **Minimum moment (ft*kip)** | -20.3918 | -7.6437 | -15.4969 | -10.0395 |

#### 6.3.1.2: Summary of Service Limit State Results

**Table 6: Summary of Service Limit State Results**

| | Top Slab | Bottom Slab | Single Wall | Double Wall |
|---|---|---|---|---|
| **Maximum Axial Force (kip)** | 0.069 | 0.715 | -1.919 | -5.554 |
| **Minimum Axial Force (kip)** | -3.773 | -5.233 | -13.856 | -14.601 |
| **Maximum Shear Force (kip)** | 8.304 | 4.544 | 4.347 | 0.444 |
| **Minimum Shear Force (kip)** | -8.787 | -4.622 | -4.837 | -0.713 |
| **Maximum Moment (ft*kip)** | 9.9085 | 8.8051 | 9.4068 | 7.0168 |
| **Minimum moment (ft*kip)** | -12.9631 | -5.5323 | -9.9127 | -5.5037 |

### 6.4: Design

#### 6.4.1: Moment Capacity of a Reinforced Concrete Beam

From the structural analysis in SAP2000, both slabs and all walls are subjected to positive and negative moments. Because of this, the slab must be doubly-reinforced with steel in both faces.

Despite this, for ease of calculation, we ignore the compression steel unless the extra bending strength is needed as per the WisDOT design manual. In other words, we treat the slab as a singularly-reinforced concrete beam. For rectangular sections, the nominal moment capacity of a singularly-reinforced concrete beam is a function of the area of steel ($A_S$), the yield strength of steel ($f_y$), the depth of the tension steel centroid from the compression face of the concrete ($d_S$), and the depth of the Whitney Stress Block (a). The nominal moment capacity is given by:

$$M_n = A_S f_y \left(d_S - \frac{a}{2}\right)$$

The depth of the Whitney Stress Block (a) is a function of the area of steel ($A_S$), the yield strength of steel ($f_y$), the 28-day design strength of concrete ($f'_C$), and the width of the concrete beam (b). The depth of the Whitney Stress Block is given by:

$$a = \frac{A_S f_y}{0.85 f'_C b}$$

Therefore, the nominal moment capacity can also be written as:

$$M_n = A_S f_y \left(d_S - \frac{A_S f_y}{1.7 f'_C b}\right)$$

The depth of the Whitney Stress Block is related to the depth of the plastic neutral axis (c) by the relation:

$$a = \beta_1 c$$

Where $\beta_1$ is a function of the 28-day design strength of concrete (psi) given by:

$$\beta_1 = -5 * 10^{-5}/psi * (f'_C - 4000psi) + 0.85$$

In order to resist the load, the following inequality must hold:

$$\phi_M M_n \geq M_u$$

Where the flexural strength reduction factor ($\phi_M$) for tied flexural reinforcement is a function of the strain in the tension steel ($\varepsilon_t$). The strength reduction factor is given by:

$$\phi_M = \begin{cases} 0.75, & \varepsilon_t \leq 0.002 \\ 0.75 + (\varepsilon_t - 0.002)(50), & 0.002 < \varepsilon_t < 0.005 \\ 0.90, & \varepsilon_t \geq 0.005 \end{cases}$$

And strain compatibility is used to relate the concrete crushing strain ($\varepsilon_{cu}$), depth to the tension steel, and depth to the neutral axis to the strain in the tension steel. The strain in the tension steel is given by:

$$\varepsilon_t = \frac{d_S - c}{c} * \varepsilon_{cu}$$

And the concrete crushing strain is:

$$\varepsilon_{cu} = 0.003$$

### 6.4.2: Moment Capacity of Box Culvert Elements

As stated above, nominal moment capacity is a function of the area of tension steel, the yield strength of steel, the depth of the steel from the compression face, the 28-day design strength of concrete, and the width of the concrete beam. In other words, nominal moment capacity is a function of both section properties and material properties. The material properties are assumed to be the following:

$$f_y = 60\ ksi$$
$$f_C' = 4\ ksi$$

And since β1 is a function of the 28-day design strength of concrete:

$$\beta_1 = -5 * \frac{10^{-5}}{psi} * (f_C' - 4000psi) + 0.85$$
$$\beta_1 = -5 * \frac{10^{-5}}{psi} * (4000psi - 4000psi) + 0.85$$
$$\beta_1 = 0.85$$

Additionally, the width of the concrete beam is the 1-foot design width, or:

$$b = 12\ in$$

By examining the equation for nominal moment capacity, we can see that moment capacity increases with the depth of the tension steel from the compression face of the slab. Additionally, in order to mitigate corrosion in the reinforcing steel, there must be a prescribed distance between the outside of the tension steel and the tension face of the concrete. This distance is called clear cover ($h_{Clear}$).

Since concrete clear cover is defined as the measurement from the tension face to the outside of the tension steel and the depth of steel is measured from the compression face to the tension steel centroid, the diameter of the reinforcing bars ($d_b$) must also be taken into account. Therefore, if there is only one row of tension steel, the depth to the tension steel centroid in a wall or slab is given by:

$$d_S = t - h_{Clear} - \frac{1}{2}d_b$$

WisDOT requires 3 inches of clear cover for the bottom steel in the bottom slab, 2½ inches of clear cover for the top steel in the top slab if there is no fill, and 2 inches of clear cover for all other reinforcing steel. MnDOT requires between 1½ inches and 2 inches of clear cover. We will thus assume 2 inches of clear cover in both faces of all walls and the top slab. We will assume 2 inches of clear cover in the top steel in the bottom slab and 3 inches of clear cover for the bottom steel in the bottom slab. Additionally, MnDOT specifies that #6 rebar is the maximum allowable size of reinforcement bars for box culverts. The diameter of #6 rebar ($d_{\#6}$) is 0.75 inches and the area ($A_{\#6}$) is 0.442 square inches.

With known quantities for the yield strength of steel, 28-day design strength of concrete, depth of tension steel, width of the concrete beam, and an assumed value for the flexural strength reduction factor, the only unknown quantity is the area of the steel. We can solve for the area of steel required as follows:

$$\phi_M M_n \geq M_u$$
$$M_n \geq \frac{M_u}{\phi_M}$$

$$A_S f_y \left( d_S - \frac{A_S f_y}{1.7 f_C' b} \right) \geq \frac{M_u}{\phi_M}$$

$$A_S f_y d_S - \frac{A_S^2 f_y^2}{1.7 f_C' b} \geq \frac{M_u}{\phi_M}$$

$$A_S f_y d_S - \frac{A_S^2 f_y^2}{1.7 f_C' b} - \frac{M_u}{\phi_M} \geq 0$$

$$\frac{f_y^2}{1.7 f_C' b} A_S - f_y d_S A_S + \frac{M_u}{\phi_M} \leq 0$$

Solving for the area of steel required using the quadratic formula:

$$A_S = \frac{f_y d_S \pm \sqrt{\left(f_y d_S\right)^2 - 4\left(\frac{f_y^2}{1.7 f_C' b}\right)\left(\frac{M_u}{\phi_M}\right)}}{2\left(\frac{f_y^2}{1.7 f_C' b}\right)}$$

Because we know the diameter of one #6 steel bar, we can solve for the number of bars required:

$$A_S = n_{\#6} A_{\#6}$$

$$n_{\#6} = \frac{A_S}{A_{\#6}}$$

Since we cannot have a fraction of a bar, we must always round up. In other words, the actual number of reinforcing bars is given by:

$$n_{\#6_{Actual}} = ceil(n_{\#6})$$

And the actual area of steel is given by:

$$A_{S_{Actual}} = n_{\#6_{Actual}} * A_{\#6}$$

In order to determine the area of steel required, we have to assume a value for the flexural strength reduction factor. We will assume that the flexural strength reduction factor is 0.90. The flexural strength reduction factor is 0.90 when the strain in the tension steel is greater than or equal to 0.005 – when the section is tension-controlled. From this we can derive an expression for the maximum area of steel allowed for a value of 0.90 for the strength reduction factor:

$$\varepsilon_t \geq 0.005$$

$$\frac{d_S - c}{c} * \varepsilon_{cu} \geq 0.005$$

$$\left(\frac{d_S}{c} - 1\right) * \varepsilon_{cu} \geq 0.005$$

$$\left(\frac{\beta_1 d_S}{a} - 1\right) * \varepsilon_{cu} \geq 0.005$$

$$\left(\frac{0.85 \beta_1 f_C' b d_S}{A_S f_y} - 1\right) * \varepsilon_{cu} \geq 0.005$$

$$A_S \leq \frac{0.85 \beta_1 f_C' b d_S \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

Therefore, in order to use a value of 0.90 for the flexural strength reduction factor the actual area of steel must satisfy:

$$A_{S_{Actual}} \leq \frac{0.85\beta_1 f_C' b d_S \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

Additionally, as per the MnDOT guidelines, the minimum flexural reinforcement ratio (ρ) is 0.002. This gives us another requirement to check for the area of steel:

$$\rho \geq 0.002$$
$$\frac{A_S}{A_g} \geq 0.002$$
$$A_S \geq 0.002 * A_g$$
$$A_S \geq 0.002 * bt$$

Therefore, in addition to the moment capacity requirements for the area of flexural steel reinforcement, the actual area of steel must satisfy:

$$A_{S_{Actual}} \geq 0.002 * bt$$

**6.4.2.1: Moment Capacity of Top Slab**

Our box culvert has fill over the top slab so we take the clear cover to be 2 inches as per WisDOT guidelines. The thickness of the top slab is 9 inches. We are using #6 bars with a diameter of 0.75 inches. Therefore, the depth to the tension steel from either direction is given by:

$$d_{S,T} = t - h_{Clear} - \frac{1}{2} d_b$$
$$d_{S,T} = 9in - 2in - \frac{1}{2} * 0.75in$$
$$d_{S,T} = 6.625\ in$$

**6.4.2.1.1: Area of Steel in Bottom Face of Top Slab to Resist Positive Moment**

In the top slab, a positive moment results in compression in the top face of the slab and tension in the bottom face of the slab. Therefore, the amount of steel in the bottom face of the top slab is dictated by the maximum positive moment. The maximum factored positive moment in the top slab is:

$$M_{u,T}^{+} = 15.941\ ft * kip * \left(\frac{12in}{ft}\right)$$
$$M_{u,T}^{+} = 191.292\ in * kip$$

The area of tension steel needed in the bottom face to resist the positive moment, assuming a value for the flexural strength reduction factor of 0.90 that will have to be checked later, is:

$$A_{S,T}^{+} = \frac{f_y d_{S,T} \pm \sqrt{\left(f_y d_{S,T}\right)^2 - 4\left(\frac{f_y^2}{1.7 f_C' b}\right)\left(\frac{M_{u,T}^{+}}{\phi_M}\right)}}{2\left(\frac{f_y^2}{1.7 f_C' b}\right)}$$

$$A_{S,T}^{+} = \frac{60\frac{kip}{in^2} * 6.625in \pm \sqrt{\left(60\frac{kip}{in^2} * 6.625in\right)^2 - 4\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)\left(\frac{191.29in * kip}{0.90}\right)}}{2\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)}$$

$A_{S,T}^{+} = 0.571\ in^2, 8.439\ in^2$

We take the lower of the two values since both are valid roots. From now on we will only report the lower value. So, the area of steel in the bottom face needed to resist the positive moment is:

$$A_{S,T}^{+} = 0.571\ in^2$$

And the number of #6 bars needed is given by:

$$n_{\#6,T}^{+} = \frac{A_{S,T}^{+}}{A_{\#6}}$$

$$n_{\#6,T}^{+} = \frac{0.571 in^2}{0.442 in^2}$$

$$n_{\#6,T}^{+} = 1.29$$

So, we take the actual number of #6 bars as two:

$$n_{\#6,T_{Actual}}^{+} = 2$$

This makes the actual area of steel:

$$A_{S,T_{Actual}}^{+} = n_{\#6,T_{Actual}}^{+} * A_{\#6}$$

$$A_{S,T_{Actual}}^{+} = 2 * 0.442 in^2$$

$$A_{S,T_{Actual}}^{+} = 0.884\ in^2$$

In order for our assumption of 0.90 for the flexural strength reduction factor to be valid, we must check that the following holds true:

$$A_{S,T_{Actual}}^{+} \leq \frac{0.85 \beta_1 f_C' b d_{S,T} \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

$$A_{S,T_{Actual}}^{+} \leq \frac{0.85 * 0.85 * 4\frac{kip}{in^2} * 12in * 6.625in * 0.003}{(0.005 + 0.003) * 60\frac{kip}{in^2}}$$

$$A_{S,T_{Actual}}^{+} \leq 1.436\ in^2$$

$$0.884\ in^2 \leq 1.436\ in^2\ (✔)$$

Finally, we must check that the minimum flexural reinforcement ratio is satisfied:

$$A^{+}_{S,T_{Actual}} \geq 0.002 * bt$$
$$A^{+}_{S,T_{Actual}} \geq 0.002 * 12in * 9in$$
$$0.884\ in^2 \geq\ 0.216\ in^2\ (\checkmark)$$

**6.4.2.1.2: Area of Steel in Top Face of Top Slab to Resist Negative Moment**

In the top slab, a negative moment results in tension in the top face of the slab and compression in the bottom face of the slab. Therefore, the amount of steel in the top face of the top slab is dictated by the maximum negative moment. The maximum factored negative moment in the top slab is:

$$M^{-}_{u,T} = -20.3918\ \text{ft} * kip * \left(\frac{12in}{ft}\right)$$
$$M^{-}_{u,T} = -244.702\ in * kip$$

The area of tension steel needed in the top face to resist the negative moment, assuming a value for the flexural strength reduction factor of 0.90 that will have to be checked later, is:

$$A^{-}_{S,T} = \frac{f_y d_{S,T} - \sqrt{\left(f_y d_{S,T}\right)^2 - 4\left(\frac{f_y^2}{1.7 f'_C b}\right)\left(\frac{M^{-}_{u,T}}{\phi_M}\right)}}{2\left(\frac{f_y^2}{1.7 f'_C b}\right)}$$

$$A^{-}_{S,T} = \frac{60\frac{kip}{in^2} * 6.625in - \sqrt{\left(60\frac{kip}{in^2} * 6.625in\right)^2 - 4\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)\left(\frac{244.70in * kip}{0.90}\right)}}{2\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)}$$

$$A^{-}_{S,T} = 0.746\ in^2$$

And the number of #6 bars needed is given by:

$$n^{-}_{\#6,T} = \frac{A^{-}_{S,T}}{A_{\#6}}$$
$$n^{-}_{\#6,T} = \frac{0.746in^2}{0.442in^2}$$
$$n^{-}_{\#6,T} = 1.69$$

So, we take the actual number of #6 bars as two:

$$n^{-}_{\#6,T_{Actual}} = 2$$

This makes the actual area of steel:

$$A^{-}_{S,T_{Actual}} = n^{-}_{\#6,T_{Actual}} * A_{\#6}$$
$$A^{-}_{S,T_{Actual}} = 2 * 0.442in^2$$

$$A_{S,T_{Actual}}^{-} = 0.884\ in^2$$

In order for our assumption of 0.90 for the flexural strength reduction factor to be valid, we must check that the following holds true:

$$A_{S,T_{Actual}}^{-} \leq \frac{0.85\beta_1 f_C' b d_{S,T} \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

$$A_{S,T_{Actual}}^{-} \leq \frac{0.85 * 0.85 * 4\frac{kip}{in^2} * 12in * 6.625in * 0.003}{(0.005 + 0.003) * 60\frac{kip}{in^2}}$$

$$A_{S,T_{Actual}}^{-} \leq 1.436\ in^2$$

$$0.884\ in^2 \leq 1.436\ in^2\ (\checkmark)$$

Finally, we must check that the minimum flexural reinforcement ratio is satisfied:

$$A_{S,T_{Actual}}^{-} \geq 0.002 * bt$$

$$A_{S,T_{Actual}}^{-} \geq 0.002 * 12in * 9in$$

$$0.884\ in^2 \geq\ 0.216\ in^2\ (\checkmark)$$

### 6.4.2.2: Moment Capacity of Bottom Slab

According to WisDOT guidelines, we can take clear cover as 2 inches from the top face of the bottom slab and 3 inches from the bottom face of the bottom slab. The thickness of the bottom slab is 10 inches. We are using #6 bars with a diameter of 0.75 inches.

#### 6.4.2.2.1: Area of Steel in Bottom Face of Bottom Slab to Resist Positive Moment

In the bottom slab, a positive moment results in compression in the top face of the slab and tension in the bottom face of the slab. Therefore, the positive moment capacity is dictated by the steel in the bottom face of the slab. The depth to the steel in the bottom face of the slab is constrained by the 3-inch cover requirement above. The depth to the positive moment tension steel from the top face of the bottom slab is given by:

$$d_{S,B}^{+} = t - h_{Clear} - \frac{1}{2}d_b$$

$$d_{S,B}^{+} = 10in - 3in - \frac{1}{2} * 0.75in$$

$$d_{S,B}^{+} = 6.625\ in$$

The amount of steel in the bottom face of the bottom slab is dictated by the maximum positive moment. The maximum factored positive moment in the bottom slab is:

$$M_{u,B}^{+} = 11.9068\ ft * kip * \left(\frac{12in}{ft}\right)$$

$$M_{u,B}^{+} = 142.882\ in * kip$$

The area of tension steel needed in the bottom face to resist the positive moment, assuming a value for the flexural strength reduction factor of 0.90 that will have to be checked later, is:

$$A_{S,B}^{+} = \frac{f_y d_{S,B}^{+} - \sqrt{\left(f_y d_{S,B}^{+}\right)^2 - 4\left(\frac{f_y^2}{1.7 f_C' b}\right)\left(\frac{M_{u,B}^{+}}{\phi_M}\right)}}{2\left(\frac{f_y^2}{1.7 f_C' b}\right)}$$

$$A_{S,B}^{+} = \frac{60\frac{kip}{in^2} * 6.625in - \sqrt{\left(60\frac{kip}{in^2} * 6.625in\right)^2 - 4\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)\left(\frac{142.88in * kip}{0.90}\right)}}{2\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)}$$

$$A_{S,B}^{+} = 0.419\ in^2$$

And the number of #6 bars needed is given by:

$$n_{\#6,B}^{+} = \frac{A_{S,B}^{+}}{A_{\#6}}$$

$$n_{\#6,B}^{+} = \frac{0.419in^2}{0.442in^2}$$

$$n_{\#6,B}^{+} = 0.95$$

So, we take the actual number of #6 bars as one:

$$n_{\#6,B_{Actual}}^{+} = 1$$

This makes the actual area of steel:

$$A_{S,B_{Actual}}^{+} = n_{\#6,B_{Actual}}^{+} * A_{\#6}$$

$$A_{S,B_{Actual}}^{+} = 1 * 0.442in^2$$

$$A_{S,B_{Actual}}^{+} = 0.442\ in^2$$

In order for our assumption of 0.90 for the flexural strength reduction factor to be valid, we must check that the following holds true:

$$A_{S,B_{Actual}}^{+} \leq \frac{0.85\beta_1 f_C' b d_{S,B}^{+} \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

$$A_{S,B_{Actual}}^{+} \leq \frac{0.85 * 0.85 * 4\frac{kip}{in^2} * 12in * 6.625in * 0.003}{(0.005 + 0.003) * 60\frac{kip}{in^2}}$$

$$A_{S,B_{Actual}}^{+} \leq 1.436\ in^2$$

$$0.442\ in^2 \leq 1.436\ in^2\ (\checkmark)$$

Finally, we must check that the minimum flexural reinforcement ratio is satisfied:

$$A_{S,B_{Actual}}^{+} \geq 0.002 * bt$$

$$A_{S,B_{Actual}}^{+} \geq 0.002 * 12in * 10in$$

$$0.442\ in^2 \geq\ 0.24\ in^2\ (\checkmark)$$

### 6.4.2.2.2: Area of Steel in Top Face of Bottom Slab to Resist Negative Moment

In the bottom slab, a negative moment results in tension in the top face of the slab and compression in the bottom face of the slab. Therefore, the negative moment capacity is dictated by the steel in the top face of the slab. The depth to the steel in the top face of the slab is constrained by the 2-inch cover requirement above. The depth to the negative moment tension steel from the bottom face of the bottom slab is given by:

$$d_{S,B}^{-} = t - h_{Clear} - \frac{1}{2} d_b$$

$$d_{S,B}^{-} = 10in - 2in - \frac{1}{2} * 0.75in$$

$$d_{S,B}^{-} = 7.625\ in$$

The amount of steel in the top face of the bottom slab is dictated by the maximum negative moment. The maximum factored negative moment in the bottom slab is:

$$M_{u,B}^{-} = -7.6437\ \text{ft} * kip * \left(\frac{12in}{ft}\right)$$

$$M_{u,B}^{-} = -91.724\ in * kip$$

The area of tension steel needed in the top face to resist the negative moment, assuming a value for the flexural strength reduction factor of 0.90 that will have to be checked later, is:

$$A_{S,B}^{-} = \frac{f_y d_{S,B}^{-} - \sqrt{\left(f_y d_{S,B}^{-}\right)^2 - 4\left(\frac{f_y^2}{1.7 f_c' b}\right)\left(\frac{M_{u,B}^{-}}{\phi_M}\right)}}{2\left(\frac{f_y^2}{1.7 f_c' b}\right)}$$

$$A_{S,B}^{-} = \frac{60\frac{kip}{in^2} * 7.625in - \sqrt{\left(60\frac{kip}{in^2} * 7.625in\right)^2 - 4\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)\left(\frac{91.724in * kip}{0.90}\right)}}{2\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)}$$

$$A_{S,B}^{-} = 0.228\ in^2$$

And the number of #6 bars needed is given by:

$$n_{\#6,B}^{-} = \frac{A_{S,B}^{-}}{A_{\#6}}$$

$$n_{\#6,B}^{-} = \frac{0.228in^2}{0.442in^2}$$

$$n_{\#6,B}^{-} = 0.52$$

So, we take the actual number of #6 bars as one:

$$n_{\#6,B_{Actual}}^{-} = 1$$

This makes the actual area of steel:

$$A^-_{S,B_{Actual}} = n^-_{\#6,B_{Actual}} * A_{\#6}$$
$$A^-_{S,B_{Actual}} = 1 * 0.442in^2$$
$$A^-_{S,B_{Actual}} = 0.442\ in^2$$

In order for our assumption of 0.90 for the flexural strength reduction factor to be valid, we must check that the following holds true:

$$A^-_{S,B_{Actual}} \le \frac{0.85\beta_1 f'_C b d^-_{S_B} \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$
$$A^-_{S,B_{Actual}} \le \frac{0.85 * 0.85 * 4\frac{kip}{in^2} * 12in * 7.625in * 0.003}{(0.005 + 0.003) * 60\frac{kip}{in^2}}$$
$$A^-_{S,B_{Actual}} \le 1.653\ in^2$$
$$0.442\ in^2 \le 1.653\ in^2\ (\checkmark)$$

Finally, we must check that the minimum flexural reinforcement ratio is satisfied:

$$A^-_{S,B_{Actual}} \ge 0.002 * bt$$
$$A^-_{S,B_{Actual}} \ge 0.002 * 12in * 10in$$
$$0.442\ in^2 \ge\ 0.24\ in^2\ (\checkmark)$$

### 6.4.2.3: Moment Capacity of Single Wall

As per WisDOT guidelines, we can take clear cover as 2 inches for the walls of a box culvert. The thickness of the single wall is 10 inches. We are using #6 bars with a diameter of 0.75 inches. Therefore, the depth to the tension steel from either direction is given by:

$$d_{S,W} = t - h_{Clear} - \frac{1}{2} d_b$$
$$d_{S,W} = 10in - 2in - \frac{1}{2} * 0.75in$$
$$d_{S,W} = 7.625\ in$$

In the walls, the positive and negative moment magnitudes are only off by a few percent. Therefore, we will reinforce both faces of the walls identically and save ourselves some calculation. To do this, we find the largest moment magnitude by comparing the largest positive moment in the single walls and the largest negative moment in the single walls and use that as our design moment:

$$M_{u,W} = max\begin{cases} 14.8123\ ft * kip \\ abs(-15.4969\ ft * kip) \end{cases}$$
$$M_{u,W} = 15.4969\ ft * kip * \left(\frac{12in}{ft}\right)$$
$$M_{u,W} = 185.963\ in * kip$$

The area of tension steel needed in both faces to resist the design moment, assuming a value for the flexural strength reduction factor of 0.90 that will have to be checked later, is:

$$A_{S,W} = \frac{f_y d_{S,W} - \sqrt{\left(f_y d_{S,W}\right)^2 - 4\left(\frac{f_y^2}{1.7 f_C' b}\right)\left(\frac{M_{u,W}}{\phi_M}\right)}}{2\left(\frac{f_y^2}{1.7 f_C' b}\right)}$$

$$A_{S,W} = \frac{60\frac{kip}{in^2} * 7.625in - \sqrt{\left(60\frac{kip}{in^2} * 7.625in\right)^2 - 4\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)\left(\frac{185.96in * kip}{0.90}\right)}}{2\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)}$$

$$A_{S,W} = 0.473\ in^2$$

And the number of #6 bars needed is given by:

$$n_{\#6,W} = \frac{A_{S,W}}{A_{\#6}}$$

$$n_{\#6,W} = \frac{0.473in^2}{0.442in^2}$$

$$n_{\#6,W} = 1.07$$

So, we take the actual number of #6 bars as two:

$$n_{\#6,W_{Actual}} = 2$$

This makes the actual area of steel:

$$A_{S,W_{Actual}} = n_{\#6,W_{Actual}} * A_{\#6}$$

$$A_{S,W_{Actual}} = 2 * 0.442in^2$$

$$A_{S,W_{Actual}} = 0.884\ in^2$$

In order for our assumption of 0.90 for the flexural strength reduction factor to be valid, we must check that the following holds true:

$$A_{S,W_{Actual}} \le \frac{0.85\beta_1 f_C' b d_{S,W} \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

$$A_{S,W_{Actual}} \le \frac{0.85 * 0.85 * 4\frac{kip}{in^2} * 12in * 7.625in * 0.003}{(0.005 + 0.003) * 60\frac{kip}{in^2}}$$

$$A_{S,W_{Actual}} \le 1.653\ in^2$$

$$0.884\ in^2 \le 1.653\ in^2\ (\checkmark)$$

Finally, we must check that the minimum flexural reinforcement ratio is satisfied:

$$A_{S,W_{Actual}} \ge 0.002 * bt$$

$$A_{S,W_{Actual}} \ge 0.002 * 12in * 10in$$

$$0.884\ in^2 \ge\ 0.24\ in^2\ (\checkmark)$$

### 6.4.2.4: Moment Capacity of Double Wall

Where the two culverts meet there are two walls of 10 inches each. We will assume that these two walls act monolithically as one 20-inch wall. As per WisDOT guidelines, we can take clear cover as 2 inches for the walls of a box culvert. We are using #6 bars with a diameter of 0.75 inches. Therefore, the depth to the tension steel from either direction is given by:

$$d_{S,D} = t - h_{Clear} - \frac{1}{2} d_b$$

$$d_{S,D} = 20in - 2in - \frac{1}{2} * 0.75in$$

$$d_{S,D} = 17.625\ in$$

As above, in the double wall, the positive and negative moment magnitudes are only off by a few percent. Therefore, we will reinforce both faces of the walls identically and save ourselves some calculation. To do this, we find the largest factored moment magnitude by comparing the largest positive moment in the double wall and the largest negative moment in the double wall and use that as our design moment:

$$M_{u,D} = max \begin{cases} 11.9558\ ft * kip \\ abs(-10.0395\ ft * kip) \end{cases}$$

$$M_{u,D} = 11.9558\ ft * kip * \left(\frac{12in}{ft}\right)$$

$$M_{u,D} = 143.470\ in * kip$$

The area of tension steel needed in both faces to resist the design moment, assuming a value for the flexural strength reduction factor of 0.90 that will have to be checked later, is:

$$A_{S,D} = \frac{f_y d_{S,D} - \sqrt{\left(f_y d_{S,D}\right)^2 - 4\left(\frac{f_y^2}{1.7 f_C' b}\right)\left(\frac{M_{u,D}}{\phi_M}\right)}}{2\left(\frac{f_y^2}{1.7 f_C' b}\right)}$$

$$A_{S,D} = \frac{60\frac{kip}{in^2} * 17.625in - \sqrt{\left(60\frac{kip}{in^2} * 17.625in\right)^2 - 4\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)\left(\frac{144in * kip}{0.90}\right)}}{2\left(\frac{\left(60\frac{kip}{in^2}\right)^2}{1.7 * 4\frac{kip}{in^2} * 12in}\right)}$$

$$A_{S,D} = 0.152\ in^2$$

And the number of #6 bars needed is given by:

$$n_{\#6,D} = \frac{A_{S,D}}{A_{\#6}}$$

$$n_{\#6,D} = \frac{0.152in^2}{0.442in^2}$$

$$n_{\#6,D} = 0.34$$

So, we take the actual number of #6 bars as one:

$$n_{\#6,D_{Actual}} = 1$$

This makes the actual area of steel:

$$A_{S,D_{Actual}} = n_{\#6,D_{Actual}} * A_{\#6}$$
$$A_{S,D_{Actual}} = 1 * 0.442 in^2$$
$$A_{S,D_{Actual}} = 0.442\ in^2$$

In order for our assumption of 0.90 for the flexural strength reduction factor to be valid, we must check that the following holds true:

$$A_{S,D_{Actual}} \leq \frac{0.85\beta_1 f'_C b d_{S,D} \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

$$A_{S,D_{Actual}} \leq \frac{0.85 * 0.85 * 4\frac{kip}{in^2} * 12in * 17.625in * 0.003}{(0.005 + 0.003) * 60\frac{kip}{in^2}}$$

$$A_{S,D_{Actual}} \leq 3.820\ in^2$$

$$0.442\ in^2 \leq 3.820\ in^2\ (✔)$$

Finally, we must check that the minimum flexural reinforcement ratio is satisfied:

$$A_{S,D_{Actual}} \geq 0.002 * bt$$
$$A_{S,D_{Actual}} \geq 0.002 * 12in * 20in$$
$$0.442\ in^2 \geq 0.48\ in^2\ (✘)$$

Since the minimum flexural reinforcement ratio was not satisfied, we must add more flexural reinforcement. For simplicity, we will add another #6 bar.

$$n_{\#6,D_{Actual}} = 2$$

This makes the actual area of steel:

$$A_{S,D_{Actual}} = n_{\#6,D_{Actual}} * A_{\#6}$$
$$A_{S,D_{Actual}} = 2 * 0.442 in^2$$
$$A_{S,D_{Actual}} = 0.884\ in^2$$

In order for our assumption of 0.90 for the flexural strength reduction factor to be valid, we must check that the following holds true:

$$A_{S,D_{Actual}} \leq \frac{0.85\beta_1 f'_C b d_{S,D} \varepsilon_{cu}}{(0.005 + \varepsilon_{cu}) f_y}$$

$$A_{S,D_{Actual}} \leq \frac{0.85 * 0.85 * 4\frac{kip}{in^2} * 12in * 17.625in * 0.003}{(0.005 + 0.003) * 60\frac{kip}{in^2}}$$

$$A_{S,D_{Actual}} \leq 3.820\ in^2$$

$$0.884\ in^2 \leq 3.820\ in^2\ (✔)$$

Finally, for completeness, we must check that the minimum flexural reinforcement ratio is satisfied:

$$A_{S,D_{Actual}} \geq 0.002 * bt$$
$$A_{S,D_{Actual}} \geq 0.002 * 12in * 20in$$
$$0.884\ in^2 \geq\ 0.48\ in^2\ (\checkmark)$$

### 6.4.3: Shear Capacity for One-Way Action

A box culvert slab with fill greater than 2 feet behaves under one-way action. As per the WisDOT manual, the nominal shear capacity for the top and bottom slabs is given by:

$$V_C = \left(0.0676\lambda\sqrt{f_C'} + 4.6\frac{A_S}{bd_S} * \frac{V_u d_S}{M_u}\right) bd_S \leq 0.126\lambda\sqrt{f_C'}bd_S$$

Where:

$$\frac{V_u d_S}{M_u} \leq 1$$

And:

$A_S = Area\ of\ reinforcing\ steel\ in\ the\ design\ width\ (in^2)$
$d_S = Depth\ from\ the\ extreme\ compression\ fiber\ to\ the\ tension\ steel\ centroid\ (in)$
$V_u = Load\ factored\ shear\ (kip)$
$M_u = Load\ factored\ moment, occuring\ simultaneously\ with\ V_u\ (kip * in)$
$b = Design\ width\ (in)$
$\lambda = Concrete\ density\ modification\ factor; 1.0\ for\ normal\ weight\ concrete$
$f_C' = 28\text{-}\ day\ design\ strength\ of\ concrete\ (ksi)$

The nominal shear capacity for the walls of the box culvert is given by:

$$V_C = 0.0316\beta\lambda\sqrt{f_C'}b_V d_V \leq 0.25 f_C' b_V d_V$$

Where:

$\beta = 2.0$
$b_V = Effective\ web\ width\ taken\ as\ the\ minimum\ web\ width\ within\ the\ depth\ d_V\ (in)$
$d_V = Effective\ shear\ depth. Perpendicular\ distance\ between\ tension\ and\ compression\ resultants\ (in)$
$f_C' = 28\text{-}\ day\ design\ strength\ of\ concrete\ (ksi)$

And:

$$d_V \geq max\begin{cases}0.9d_S \\ 0.72t\end{cases}$$

In order to resist the load, the following inequality must hold:

$$\phi_V V_C \geq V_u$$

According to WisDOT, the shear resistance reduction factor (ϕV) for reinforced concrete box structures can be taken as 0.85:

$$\phi_V = 0.85$$

### 6.4.4: Shear Capacity of Box Culvert Elements

### 6.4.4.1: Shear Capacity of Top Slab

The maximum factored shear force in the top slab is:

$$V_{u,T} = max\begin{cases} 13.264\ kip \\ abs(-13.941\ kip) \end{cases}$$

$$V_{u,T} = 13.941\ kip$$

The shear capacity for the top slab is given by:

$$V_{C,T} = \left(0.0676\lambda\sqrt{f_C'} + 4.6\frac{A_{S,T_{Actual}}}{bd_{S,T}} * \frac{V_{u,T}d_{S,T}}{M_{u,T}}\right)bd_{S,T} \leq 0.126\lambda\sqrt{f_C'}bd_{S,T}$$

Where:

$$\frac{V_{u,T}d_{S,T}}{M_{u,T}} \leq 1$$

This can be rewritten as:

$$V_{C,T} = 0.0676\lambda\sqrt{f_C'}bd_{S,T} + 4.6A_{S,T_{Actual}} * \frac{V_{u,T}d_{S,T}}{M_{u,T}} \leq 0.126\lambda\sqrt{f_C'}bd_{S,T}$$

This expression is complicated by the inclusion of terms for the factored shear and factored moment at a particular point along the top slab. In an attempt to simplify this, we will first look at only the first term to see if it alone is sufficient:

$$V_{C,T} = 0.0676\lambda\sqrt{f_C'}bd_{S,T}$$

$$V_{C,T} = 0.0676 * 1.0 * \sqrt{4\ ksi} * 12in * 6.625in$$

$$V_{C,T} = 10.748\ kip$$

In order to resist the shear force in the bottom slab, the following inequality must be satisfied:

$$\phi_V V_{C,T} \geq V_{u,T}$$

$$0.85 * 10.748\ kip \geq 13.941\ kip$$

$$9.14\ kip \geq 13.941\ kip\ (\text{✗})$$

Because this equality is not satisfied, the top slab does not have sufficient shear capacity from the first term of the shear capacity equation alone and we need to analyze further. To do this, we tabulate values for shear capacity using the full expression at different locations along the length of the top slab. We take these shear capacity values, reduce them by the shear strength reduction factor, and compare them to the factored shear forces at that location using the inequality above. This tabulation is displayed in the accompanying Excel spreadsheet.

Unfortunately, the top slab had several locations that were subject to larger shear forces than the shear capacity provided by the concrete slab. These locations are highlighted in red. Fortunately, these points of failure were located within the haunches of the culvert. Therefore, this problem might be able to be mitigated with a more detailed analysis of shear capacity at the haunches. If a more detailed analysis suggests that the haunches do not provide enough additional shear capacity to overcome the factored shear, we will have to reconsider our design. The most obvious course of action is to increase the thickness of the top slab. This requires an increase in thickness from 9 inches to 12 inches. Additionally, we can use higher strength concrete or add more flexural reinforcement. Or, we could do some combination of the three.

### 6.4.4.2: Shear Capacity of Bottom Slab

The maximum factored shear force in the bottom slab is:

$$V_{u,B} = max\begin{cases} 6.088\ kip \\ abs(-6.147\ kip) \end{cases}$$

$$V_{u,B} = 6.147\ kip$$

The shear capacity for the bottom slab is given by:

$$V_{C,B} = \left(0.0676\lambda\sqrt{f_C'} + 4.6\frac{A_{S,B_{Actual}}}{bd_{S,B}} * \frac{V_{u,B}d_{S,B}}{M_{u,B}}\right)bd_{S,B} \leq 0.126\lambda\sqrt{f_C'}bd_{S,B}$$

Where:

$$\frac{V_{u,B}d_{S,B}}{M_{u,B}} \leq 1$$

This can be rewritten as:

$$V_{C,B} = 0.0676\lambda\sqrt{f_C'}bd_{S,B} + 4.6A_{S,B_{Actual}} * \frac{V_{u,B}d_{S,B}}{M_{u,B}} \leq 0.126\lambda\sqrt{f_C'}bd_{S,B}$$

This expression is complicated by the inclusion of terms for factored shear and factored moment at a particular point along the bottom slab. In an attempt to simplify this, we will first look at only the first term to see if it alone is sufficient. Note that the lesser value for depth of steel in the bottom slab was used to ensure conservatism:

$$V_{C,B} = 0.0676\lambda\sqrt{f_C'}bd_{S,B}$$

$$V_{C,B} = 0.0676 * 1.0 * \sqrt{4\ ksi} * 12in * 6.625in$$

$$V_{C,B} = 10.748\ kip$$

In order to resist the shear force in the bottom slab, the following inequality must be satisfied:

$$\phi_V V_{C,B} \geq V_{u,B}$$

$$0.85 * 10.748\ kip \geq 6.15\ kip$$

$$9.14\ kip \geq 6.15\ kip\ (\checkmark)$$

Because this equality is satisfied, the bottom slab has sufficient shear capacity from the first term of the shear capacity equation alone and we do not need to analyze further.

### 6.4.4.3: Shear Capacity of Single Wall

The maximum factored shear force in the single wall is:

$$V_{u,W} = max\begin{cases} 6.386\ kip \\ abs(-7.085\ kip) \end{cases}$$

$$V_{u,W} = 7.085\ kip$$

The nominal shear capacity for the single wall is given by:

$$V_{C,W} = 0.0316\beta\lambda\sqrt{f_C'}b_V d_{V,W} \leq 0.25f_C'b_V d_{V,W}$$

The effective shear width is the 1-foot design width:

$$b_V = 12\ in$$

The effective shear depth is the perpendicular distance between tension and compression resultants in the single wall and is given by:

$$d_{V,W} = d_{S,W} - \frac{a_W}{2}$$

The depth to the tension steel from either direction is:

$$d_{S,W} = 7.625\ in$$

And the depth to the Whitney Stress Block is given by:

$$a_W = \frac{A_{S,W_{Actual}} f_y}{0.85 f_C' b}$$

The actual area of steel in the wall is given by:

$$A_{S,W_{Actual}} = 0.884\ in^2$$

So, the depth to the Whitney Stress Block is:

$$a_W = \frac{0.884 in^2 * 60\frac{kip}{in^2}}{0.85 * 4\frac{kip}{in^2} * 12 in}$$

$$a_W = 1.30\ in$$

And the effective shear depth is:

$$d_{V,W} = 7.625 in - \frac{1.30 in}{2}$$

$$d_{V,W} = 6.975\ in$$

However, WisDOT says the effective shear depth need not be taken less than the greater of $0.9d_S$ or 0.72t:

$$d_{V,W} \geq max\begin{cases}0.9 d_{S,W}\\ 0.72 t_W\end{cases}$$

$$d_{V,W} \geq max\begin{cases}0.9 * 7.625 in\\ 0.72 * 10 in\end{cases}$$

$$d_{V,W} \geq max\begin{cases}6.8625 in\\ 7.2 in\end{cases}$$

$$d_{V,W} \geq 7.2\ in$$

So, we take the effective shear depth as:

$$d_{V,W} = 7.2\ in$$

The shear capacity of the single wall is:

$$V_{C,W} = 0.0316 * 2.0 * 1.0\sqrt{4ksi} * 12 in * 7.2 in \leq 0.25 * 4\frac{kip}{in^2} * 12 in * 7.2 in$$

$$V_{C,W} = 10.92\ kip \leq 86.4\ kip$$

$$V_{C,W} = 10.92\ kip$$

In order to resist the shear force in the single wall, the following inequality must be satisfied:

$$\phi_V V_{C,W} \geq V_{u,W}$$

$$0.85 * 10.92\ kip \geq 7.085\ kip$$

$$9.282\ kip \geq 7.085\ kip\ (\checkmark)$$

### 6.4.4.4: Shear Capacity of Double Wall

The maximum factored shear force in the double wall is:

$$V_{u,D} = max\begin{cases} 0.851\ kip \\ abs(-1.19\ kip) \end{cases}$$

$$V_{u,D} = 1.19\ kip$$

The nominal shear capacity for the double wall is given by:

$$V_{C,D} = 0.0316\beta\lambda\sqrt{f_C'}b_V d_{V,D} \leq 0.25 f_C' b_V d_{V,D}$$

The effective shear width is the 1-foot design width:

$$b_V = 12\ in$$

The effective shear depth is the perpendicular distance between tension and compression resultants in the double wall and is given by:

$$d_{V,D} = d_{S,D} - \frac{a_D}{2}$$

The depth to the tension steel from either direction is:

$$d_{S,D} = 17.625\ in$$

And the depth to the Whitney Stress Block is given by:

$$a_D = \frac{A_{S,D_{Actual}} f_y}{0.85 f_C' b}$$

The actual area of steel in the wall is given by:

$$A_{S,D_{Actual}} = 0.884\ in^2$$

So, the depth to the Whitney Stress Block is:

$$a_D = \frac{0.884in^2 * 60\frac{kip}{in^2}}{0.85 * 4\frac{kip}{in^2} * 12in}$$

$$a_D = 1.30\ in$$

And the effective shear depth is:

$$d_{V,D} = 17.625in - \frac{1.30in}{2}$$

$$d_{V,D} = 16.975\ in$$

However, WisDOT says the effective shear depth need not be taken less than the greater of $0.9d_S$ or 0.72t:

$$d_{V,D} \geq max\begin{cases} 0.9d_{S,D} \\ 0.72t_D \end{cases}$$

$$d_{V,D} \geq max\begin{cases} 0.9 * 17.625in \\ 0.72 * 20in \end{cases}$$

$$d_{V,D} \geq max\begin{cases} 15.8625in \\ 14.4in \end{cases}$$

$$d_{V,D} \geq 15.8625\ in$$

So, we take the effective shear depth as:

$$d_{V,D} = 16.975\ in$$

The shear capacity of the double wall is:

$$V_{C,D} = 0.0316 * 2.0 * 1.0\sqrt{4ksi} * 12in * 16.975in \leq 0.25 * 4\frac{kip}{in^2} * 12in * 16.975in$$
$$V_{C,D} = 25.75\ kip \leq 203.7\ kip$$
$$V_{C,D} = 25.75\ kip$$

In order to resist the shear force in the double wall, the following inequality must be satisfied:

$$\phi_V V_{C,D} \geq V_{u,D}$$
$$0.85 * 25.75\ kip \geq 1.19\ kip$$
$$21.89\ kip \geq 1.19\ kip\ (\checkmark)$$

### 6.4.5: Design Summary

In an actual design, we would also determine the amount and spacing of transverse steel needed for temperature and shrinkage cracking control. We would also check the maximum spacing of the flexural reinforcement for cracking control criteria. If the maximum spacing of the flexural reinforcement is exceeded, we would need to use a larger number of smaller bars spaced closer together. Another consideration would be development length of the reinforcing steel and bar cutoffs to ensure that the flexural steel is not terminated prematurely. These issues were not considered due to time constraints. The following table summarizes location and number of #6 reinforcing bars.

**Table 7: Summary of Reinforcing Steel for Design**

| | Top Slab | Bottom Slab | Single Wall | Double Wall |
|---|---|---|---|---|
| **Thickness (in)** | 9 | 10 | 10 | 20 |
| **Number of #6 Bars in Outside Face per Transverse Foot** | 2 | 1 | 2 | 2 |
| **Cover from Outside Face (in)** | 2 | 3 | 2 | 2 |
| **Number of #6 Bars in Inside Face per Transverse Foot** | 2 | 1 | 2 | 2 |
| **Cover from Inside Face (in)** | 2 | 2 | 2 | 2 |

## 7.0: Geotechnical Design

### 7.1 Summary of Process

Using soil strata data obtained from the SPT borehole reports provided by VDOT, our team was able to determine the soil types at various elevations. However, no borings were performed in the stream bed underneath the proposed culvert foundation. We have therefore assumed that the change in soil strata is linear within the proposed foundation area, between the nearest boreholes. This can be seen in figure 2 below, where the dashed black line represents the proposed foundation.

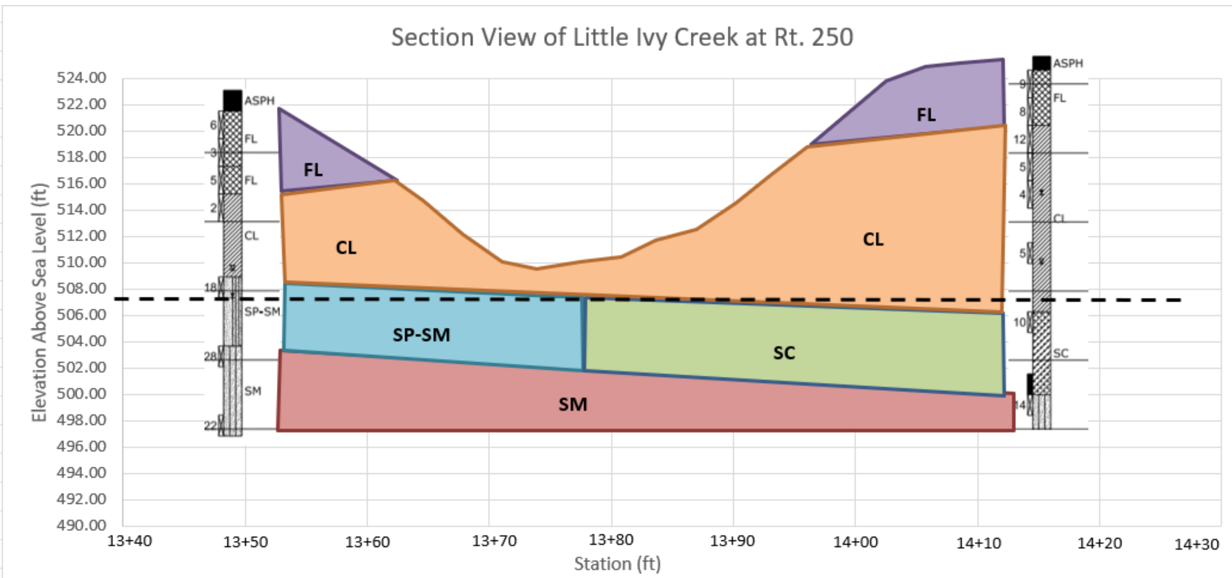



**Figure 2: Soil Strata for Culvert Foundation**

The structure foundation consists of two separate sections, each of which rests on a different soil type. Unit 1 will rest on silty sand. Unit 2 will rest on clay soil. N-values were determined for each of these soils. These N-values were then compared to Table A.11 in Budhu's "Soil Mechanics and Foundations", from which the soil unit weight and friction angle were found for each soil. Using this information, ultimate bearing capacity was calculated for the foundational soils under each unit.

Unit 1: $q_u$ = 1434.4 KPa = 29.98 ksf

Unit 2: $q_u$ = 1925.2 KPa = 40.24 ksf

Analysis of all dead and live loads upon the soil foundation has revealed the required bearing capacity to be 1.4 ksf, or 4.7% of the minimum available bearing capacity. By spreading the load across the large footprint of the culvert's floor slab, we have more than ensured suitable bearing capacity. In accordance with Section 302 of the 2016 VDOT Road and Bridge Specifications, Class I backfill shall be placed as bedding material to a minimum of 6 inches of compacted depth. The limits of this backfill shall be 1 foot beyond the limits of the culvert foundation slab. Backfill shall be either No.25 or No.26 crusher run aggregate, or 21A or 21B base material, and compacted in accordance with the same specification.

### 7.2 Unit 1 Calculations

$N$ value calculated from Figure 2:

$$N = 21$$

Estimated unit weight, $\gamma$,and friction angle, $\Phi_p$,values from Table A.11 (Budhu):

$$\gamma = 18\frac{kN}{m^3} \quad \Phi_p = 31^o$$

Geometric values of proposed foundation for a single unit; length, width, and depth of footing:

$$L = 35\,ft\; = 10.688\,m,\; B = 20.5\,ft = 6.246\,m\;, D_f\; = 4\,ft = 1.22\text{ m}$$

Effective Stress Analysis (ESA), or ultimate bearing capacity equation. The equation contains several unitless adjustment factors that will be explained below:

$$ESA = \; q_u = \gamma D_f(N_q - 1)s_q d_q i_q b_q g_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma b_\gamma g_\gamma$$

$N_q$is the bearing capacity factor:

$$N_q = e^{\pi tan\Phi_p}tan^2(45^o + \frac{\Phi_p}{2})$$

$$N_q = e^{\pi tan(31^o)}tan^2(45^o + \frac{(31^o)}{2})$$

$$N_q = 20.63$$

$s_q$is the shape factor, which accounts for the foundation shape relative to the friction angle:

$$s_q = 1 + \frac{B}{L}tan\Phi_p$$

$$s_q = 1 + \frac{(6.246\,m)}{(10.668\,m)}tan(31^o)$$

$$s_q = 1.352$$

$d_q$is the embedment depth factor:

$$d_q = 1 + 2tan\Phi_p(1 - sin\Phi_p \quad)^2 tan^{-1}(\frac{D}{B})$$

$$d_q = 1 + 2tan(31^o)(1 - sin(31^o \quad)^2 tan^{-1}(\frac{1.22\,m}{6.248\,m})$$

$$d_q = 1.05$$

$i_q$,$b_q$,$g_q$are the load, base, and ground inclination factors respectively. We assumed perfectly flat, level foundation surfaces, and a perpendicular load application to these surfaces. Therefore,

$$i_q = 1,\ b_q = 1\ , g_q = 1$$

$N_\gamma$is a calculated bearing capacity factor. $d_\gamma$, $b_\gamma$, $g_\gamma$are:

$$N_\gamma = (Nq - 1)tan(1.4\Phi_p)$$

$$N_\gamma = (20.63 - 1)tan(1.4(31^o)$$

$$N_\gamma = 18.56$$

$s_\gamma$ is a calculated shape factor similar to $s_q$:

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

$$s_\gamma = 1 - 0.4\frac{10.668\,m}{6.248\,m}$$

$$s_\gamma = 0.766$$

$d_\gamma$, $b_\gamma$, $g_\gamma$are additional geometric parameters that are all equal to 1 given our assumption of flat foundation surfaces and perpendicular load application:

$$d_\gamma = 1 \quad b_\gamma = 1 \quad g_\gamma = 1$$

Using all the above values, ultimate bearing capacity can be calculated:

$$q_u = \gamma D_f(N_q - 1)s_q d_q i_q b_q g_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma b_\gamma g_\gamma$$

$$q_u = (18\frac{kN}{m^3})(1.22\,m)(20.63 - 1)(1.352)(1.05)(1)(1)(1) + 0.5(18\frac{kN}{m^3})(6.248\,m)(18.56)(0.766)(1)(1)(1)$$

$$\boldsymbol{q_u = 1434.4\ kPa}$$

**7.3 Unit 2 Calculations**

$N$value calculated from Figure 2:

$$N = 17$$

Estimated unit weight, $\gamma$,and friction angle, $\Phi_p$,values from Table A.11 (Budhu):

$$\gamma = 18\frac{kN}{m^3}\Phi_p = 33^o$$

Geometric values of proposed foundation for a single unit; length, width, and depth of footing:

$$L = 35\ ft\ = 10.688\ m,\ B = 20.5\ ft = 6.246\ m\ , D_f\ = 4\ ft = 1.22\text{ m}$$

Effective Stress Analysis (ESA), or ultimate bearing capacity equation. The equation contains several unitless adjustment factors that will be explained below:

$$ESA =\ q_u = \gamma D_f(N_q - 1)s_q d_q i_q b_q g_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma b_\gamma g_\gamma$$

$N_q$is the bearing capacity factor:

$$N_q = e^{\pi tan\Phi_p} tan^2(45^o + \frac{\Phi_p}{2})$$

$$N_q = e^{\pi tan(33^o)} tan^2(45^o + \frac{(33^o)}{2})$$

$$N_q = 26.09$$

$s_q$is the shape factor, which accounts for the foundation shape relative to the friction angle:

$$s_q = 1 + \frac{B}{L} tan\Phi_p$$

$$s_q = 1 + \frac{(6.246\ m)}{(10.668\ m)} tan(33^o)$$

$$s_q = 1.38$$

$d_q$is the embedment depth factor:

$$d_q = 1 + 2tan\Phi_p(1 - sin\Phi_p\ \ )^2 tan^{-1}(\frac{D}{B})$$

$$d_q = 1 + 2tan(33^o)(1 - sin(33^o\ \ )^2 tan^{-1}(\frac{1.22\ m}{6.248\ m})$$

$$d_q = 1.05$$

$i_q$,$b_q$,$g_q$are the load, base, and ground inclination factors respectively. We assumed perfectly flat, level foundation surfaces, and a perpendicular load application to these surfaces. Therefore,

$$i_q = 1,\ b_q = 1\ , g_q = 1$$

$N_\gamma$ is a calculated bearing capacity factor. $d_\gamma$, $b_\gamma$, $g_\gamma$ are:

$$N_\gamma = (Nq - 1)tan(1.4\Phi_p)$$

$$N_\gamma = (20.63 - 1)tan(1.4(33^o)$$

$$N_\gamma = 26.16$$

$s_\gamma$ is a calculated shape factor similar to $s_q$:

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

$$s_\gamma = 1 - 0.4\frac{10.668\ m}{6.248\ m}$$

$$s_\gamma = 0.766$$

$d_\gamma$, $b_\gamma$, $g_\gamma$ are additional geometric parameters that are all equal to 1 given our assumption of flat foundation surfaces and perpendicular load application:

$$d_\gamma = 1 \quad b_\gamma = 1 \quad g_\gamma = 1$$

Using all the above values, ultimate bearing capacity can be calculated:

$$q_u = \gamma D_f(N_q - 1)s_q d_q i_q b_q g_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma b_\gamma g_\gamma$$

$$q_u = (18\frac{kN}{m^3})(1.22\ m)(26.09 - 1)(1.38)(1.05)(1)(1)(1) + 0.5(18\frac{kN}{m^3})(6.248\ m)(26.16)(0.766)(1)(1)(1)$$

$$\boldsymbol{q_u = 1925.2\ kPa}$$

## 7.4 Wing Wall Design

The wing walls were designed as concrete gravity retaining walls. The required wingwall height was determined from inspection to be 11 feet. The length was determined as follows:

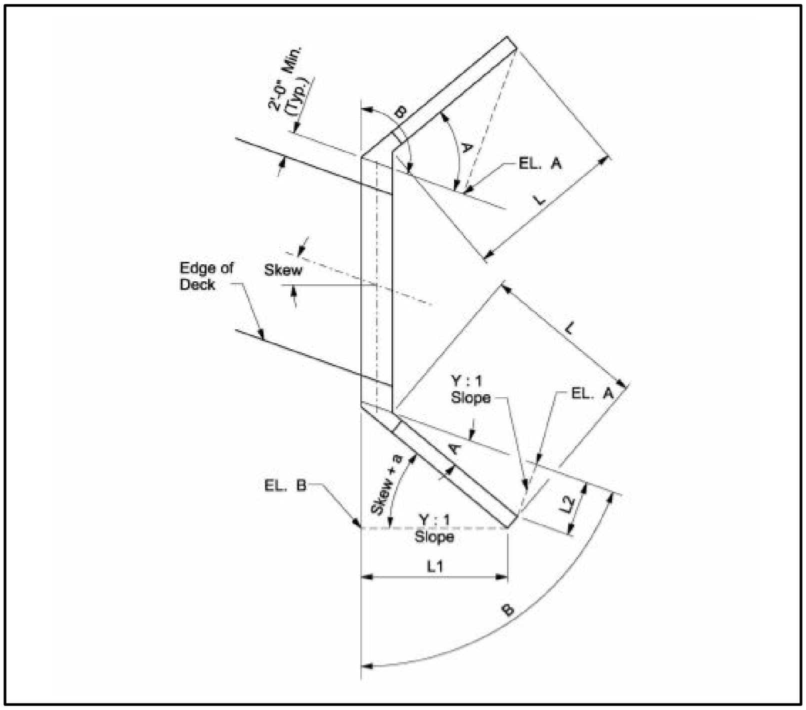



**Figure 3: Reference Diagram for Wing Wall Length Calculations**

For angle B < 90 deg:

$$L_1 + L_2 = (El_A - El_B)(Y)$$

$$cos(a - skew) = \frac{L_1}{L}$$

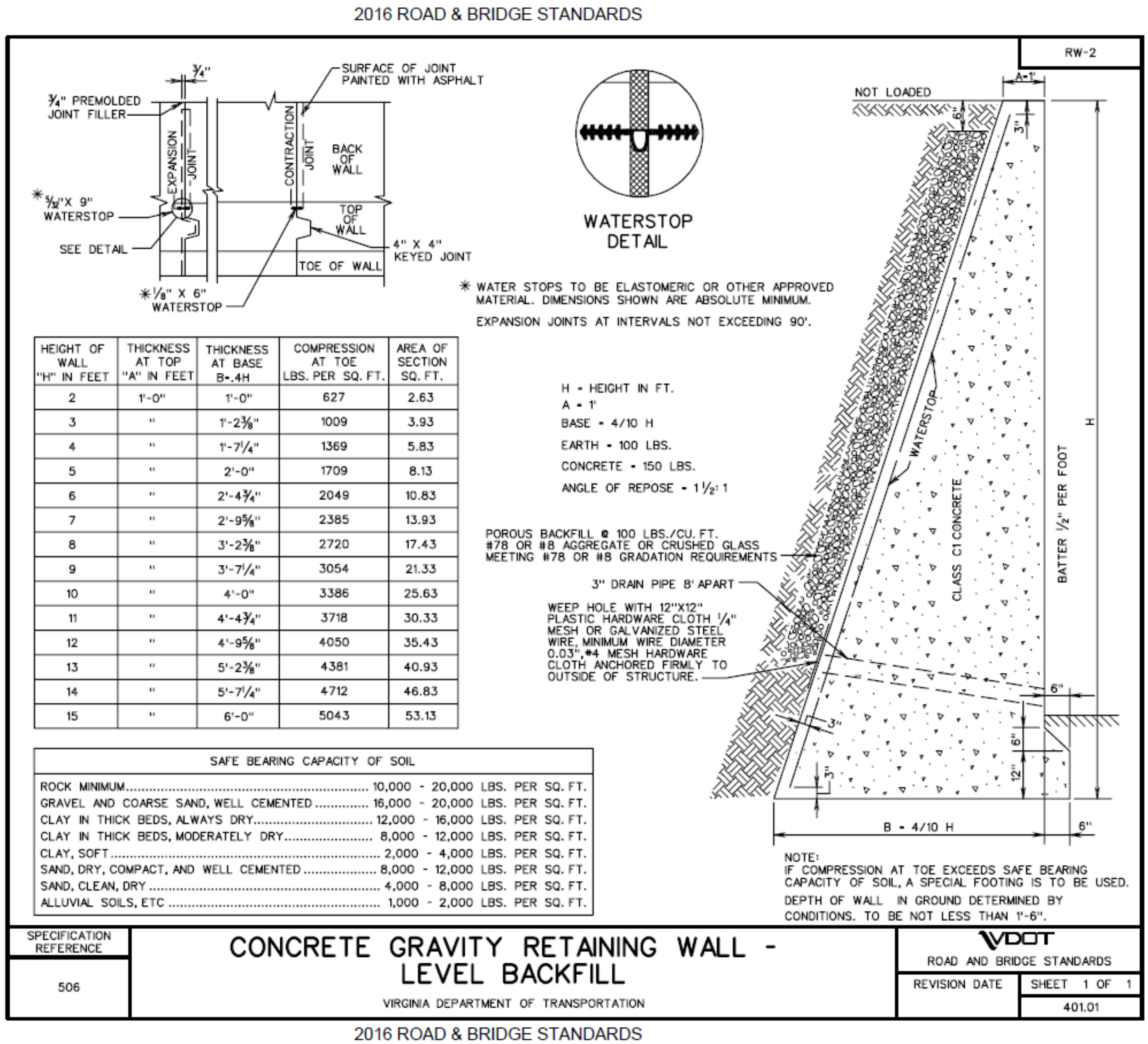
$$sin(a) = \frac{L_2}{L}$$

Assume: $skew = 3°, Y = 2\ (2{:}1\ slope\ on\ embankment), \Delta El = 12\ ft\ (from\ station\ elevations), a = 45°$

Therefore:

$$L = \frac{Y(El_A - El_B)}{cos(a - skew) + sin(a)} = \frac{(2)(12\ ft)}{cos(45-(3)) + sin(45)} \Rightarrow L = 16.5\ ft$$

VDOT 2016 Road and Bridge Standard sheet 401.01 presents design standards for concrete gravity retaining walls. This standard can be seen below in Figure 4, and a link can be found in appendix C.




| HEIGHT OF WALL "H" IN FEET | THICKNESS AT TOP "A" IN FEET | THICKNESS AT BASE B=.4H | COMPRESSION AT TOE LBS. PER SQ. FT. | AREA OF SECTION SQ. FT. |
|---|---|---|---|---|
| 2 | 1'-0" | 1'-0" | 627 | 2.63 |
| 3 | " | 1'-2⅜" | 1009 | 3.93 |
| 4 | " | 1'-7¼" | 1369 | 5.83 |
| 5 | " | 2'-0" | 1709 | 8.13 |
| 6 | " | 2'-4¾" | 2049 | 10.83 |
| 7 | " | 2'-9⅝" | 2385 | 13.93 |
| 8 | " | 3'-2⅜" | 2720 | 17.43 |
| 9 | " | 3'-7¼" | 3054 | 21.33 |
| 10 | " | 4'-0" | 3386 | 25.63 |
| 11 | " | 4'-4¾" | 3718 | 30.33 |
| 12 | " | 4'-9⅝" | 4050 | 35.43 |
| 13 | " | 5'-2⅜" | 4381 | 40.93 |
| 14 | " | 5'-7¼" | 4712 | 46.83 |
| 15 | " | 6'-0" | 5043 | 53.13 |

| SAFE BEARING CAPACITY OF SOIL | |
|---|---|
| ROCK MINIMUM | 10,000 - 20,000 LBS. PER SQ. FT. |
| GRAVEL AND COARSE SAND, WELL CEMENTED | 16,000 - 20,000 LBS. PER SQ. FT. |
| CLAY IN THICK BEDS, ALWAYS DRY | 12,000 - 16,000 LBS. PER SQ. FT. |
| CLAY IN THICK BEDS, MODERATELY DRY | 8,000 - 12,000 LBS. PER SQ. FT. |
| CLAY, SOFT | 2,000 - 4,000 LBS. PER SQ. FT. |
| SAND, DRY, COMPACT, AND WELL CEMENTED | 8,000 - 12,000 LBS. PER SQ. FT. |
| SAND, CLEAN, DRY | 4,000 - 8,000 LBS. PER SQ. FT. |
| ALLUVIAL SOILS, ETC | 1,000 - 2,000 LBS. PER SQ. FT. |

**Figure 4: VDOT Retaining Wall Standard Used for Design**

Using this standard, and the height and length determined above, the wingwalls may be designed:

*Height = 11 ft , Length = 16.5 ft, Thickness at Top = 1 ft, Thickness at Bottom = 4' - 4.75"*

*Toe embedment depth = 1.5 ft*

*Required bearing capacity = 3.718 ksf*

*Minimum bearing capacity of soil foundation,* $q_u$ *= 29.98 ksf*

*3.718 ksf < 29.98 ksf, foundation is suitable.*

$$\gamma_{concrete} = 150 \text{ lbs/ft}^3$$

$$\gamma_{backfill} = 100 \text{ lbs/ft}^3$$

The retaining wall shape shall be as shown in Figure 4. Elastomeric waterstop material shall be placed between the wall and backfill to protect the structure against infiltration. Backfill should be porous #78 or #8 stone to allow drainage and prevent a increase in backfill weight due to saturation. Care should be taken to ensure the backfill does not exceed $\gamma_{backfill} = 100$ lbs/ft$^3$, to prevent overturning of the wall. Weep holes should be provided as shown to ensure that any water that passes the waterstop has a path to be drained away from the structure. This design will be used to replace the existing four wingwalls.

## <u>8.0: Constructability Assessment</u>

This section will give an overview of the basic construction process for this project, and seek to identify any potential issues that may arise during construction. For the purposes of this assessment, we will assume this to be a design-build project, and "the contractor" to be responsible for all permitting. This should not be considered an exhaustive documentation of the construction process, only a rough overview.

### 8.1 Mobilization / Maintenance of Traffic (MoT)

The public must be made aware of the temporary changes to traffic patterns well in advance of the start of work. Businesses and residents local to the work-zone may be reached by mail, while both locals and commuters can be informed through use of portable changeable message signs (PCMS) placed along route 250. Given the relatively complicated road closure and detour required for this project, MoT plans signed and sealed by a professional engineer will likely be required. In addition, the Virginia Work Area Protection Manual (VWAPM) shall be followed. MoT will need to be maintained and monitored throughout the course of the project to ensure safety. The contractor should begin work by setting the full detour and closing Rt. 250 in the area of Little Ivy Creek bridge. Equipment and some materials can then be mobilized and staged within the closed roadway area. If additional space is required, it may be necessary for the contractor to reach an agreement with the owners of the nearby Exxon gas station to use some of their excess parking area.

### 8.2 Demo Existing Bridge / Place New Culvert

Before work can begin, environmental permits must be acquired. While all work will require coverage under the Virginia Pollutant Discharge Elimination System (VPDES) General Construction Permit, additional permitting will likely be required through the U.S. Army Corps of Engineers due to the direct intrusion of the work into a waterway. Applications for these permits will require a completed erosion and sediment control plan, stormwater management plan, and stormwater pollution prevention plan. It is suggested that the contractor use a temporary cofferdam and diversion dike to reroute the existing stream during construction, with designs submitted during permitting. Additionally, environmental studies may be required to check for impacts to protected animal species. These should be performed well in advance of the start of work.

Once all permits have been acquired, the contractor should begin demolishing the existing bridge. It is suggested that the contractor use hydraulic breaker attachments on 320 CAT excavators or equivalent for this purpose. These same excavators can then be used to load the rubble into trucks to be hauled off-site. The wing walls should be left in place to support the embankment directly next to the existing structure. This will allow the excavators to sit on the roadway embankment directly above the bridge during demolition, instead of within the existing waterway. Given the excavator's 22' boom, this should be suitably close. Next, culvert base material shall be placed as discussed in section 7, and lightly compacted using the bucket of the excavator. Each segment of the culvert shall be delivered on a flatbed truck and unloaded immediately into place by crane. The crane may also be placed on the roadway embankment. More information on crane sizing will be given in section 8.6.

### 8.3 Demolish Existing Wing Walls / Place New Wing Walls

Once the new culvert has been placed, the embankment contained within the wingwalls should be excavated and hauled off-site. The existing wingwalls can be demolished in the same manner as the existing bridge. Once demolition is complete, Class I backfill shall be placed as base material for the new wing walls, and compacted. The new wingwalls shall be shipped on-site using flatbed trucks and unloaded immediately by the crane and placed. Geotextile drainage fabric shall be attached to the culvert and wingwalls where they will be in contact with backfill material. Porous backfill material (open-graded stone, Section 204 of the VDOT Road and Bridge Specifications) shall be placed and compacted as embankment material in accordance with the VDOT Road and Bridge Specifications, and the wingwall standard as documented in Section 7.

### 8.4 Subgrade / Subbase / Asphalt Topping / Guardrail / Re-Striping

This work shall be performed in accordance with the following sections of the VDOT Road and Bridge Specifications:

Subgrade - Section 305. Previously excavated embankment material may be suitable for this purpose.

Subbase - Section 308.

Asphalt Concrete - Section 315

Guardrail - Section 505 (Specs) and Section 500 (Standards)

Re-Striping - Section 704

The approved plans should also be consulted for information in regards to these items; especially thickness requirements for subgrade, subbase, and asphalt, required guardrail, and roadway striping plans. While this work will not be explained in detail here, no unusual circumstances are expected to arise during the course of this work that would impact constructability.

### 8.5 Demobilize / Remove MoT

As soon as the roadway can safely be opened to traffic, MoT shall be removed and the road shall be opened to traffic. The contractor should then demobilize all remaining equipment and materials.

### 8.6 Summary

The primary areas of concern with regards to constructability have been identified as acquiring and complying with environmental permits, MoT, and safety and stability of the crane. The first two were discussed previously in this section. It has been determined that the following loads will need to be moved by the on-site crane:

Approximate weight of each box culvert = 4.33 Tons

Approximate weight of each wingwall = 37.4 Tons

The advertised maximum capacity for a TMS500-2 Grove truck mounted telescoping crane is 40 tons, with a 95 ft maximum boom length. From the above rough overview of the work, it can be expected that the contractor should require a boom length no greater than 50’ for any work performed on this project (distance to the center of gravity of the far wingwall), with a maximum required load of 37.4 tons. Therefore, a crane of this size should be acceptable for use on this project. In practice, this would need to be confirmed by more in-depth calculations at a later date. From the brief overview presented above, at this time we consider this project to present no excessive constructability concerns.

## 9.0 Preliminary Project Cost Estimate

A preliminary cost estimate was created following aspects from the constructability assessment. Using the plans from the old bridge and proposed dimensions for the new development, along with average price estimates found through research, estimates were made for all main components of the project. This includes the demolition of the old bridge, sitework, foundation work, substructure, superstructure, finishes, and equipment. Some assumptions and exclusions include the following:

- Estimate is in today’s dollars
- Material cost includes transportation cost of material
- Estimate only includes material and labor for physical aspects for the construction of the bridge

The total estimated construction cost totaled $508,362.
A breakdown of the cost estimate is shown below in Table 8

| **Table 8: Proposed Bridge Design Estimate** | | | | | | |
|---|---|---|---|---|---|---|
| | | | | | | |
| **Item** | **Quantity** | **Unit** | **Material Cost per Unit** | **Labor Cost per Unit** | | **Total Estimate Cost** |
| **Demolition/Sitework** | | | | | | |
| Bridge and Foundation Demo | 9.33 | cy | | $40 | | $373 |
| Cofferdam | 1 | ls | $28,000 | | | $28,000 |
| Existing Wingwall Removal | 2 | ea | | $500 | | $1,000 |
| **Foundation** | | | | | | |
| Excavation of Concrete | 96 | cy | | $100 | | $9,600 |
| Concrete | 480 | cy | $100 | $200 | | $144,000 |
| No. 25/26 Crusher Run Aggregate Backfill | 28.1 | cy | $20 | | | $562 |
| **Box Culvert** | | | | | | |
| Precast box culverts | 2 | ea | $75,000 | $1,000 | | $152,000 |
| | | | | | | |
| **Wing Wall** | | | | | | |
| Precast Wing Wall | 74 | cy | $430 | $90 | | $38,444 |
| Porous Backfill No. 78 | 19.6 | tons | $70 | $200 | | $15,372 |
| **Bridge** | | | | | | |
| Subbase | 47.3 | ton | $15 | $10 | | $1,183 |
| Asphalt Topping | 17.9 | ton | $115 | | | $2,059 |
| Guardrail | 70 | lf | $10 | | | $700 |
| Re-stripping | 70 | lf | $1 | | | $70 |
| Equipment | 1 | ls | $115,000 | | | $115,000 |
| **Total** | | | | | | **$508,362** |

## Appendix A, Conceptual Plans:

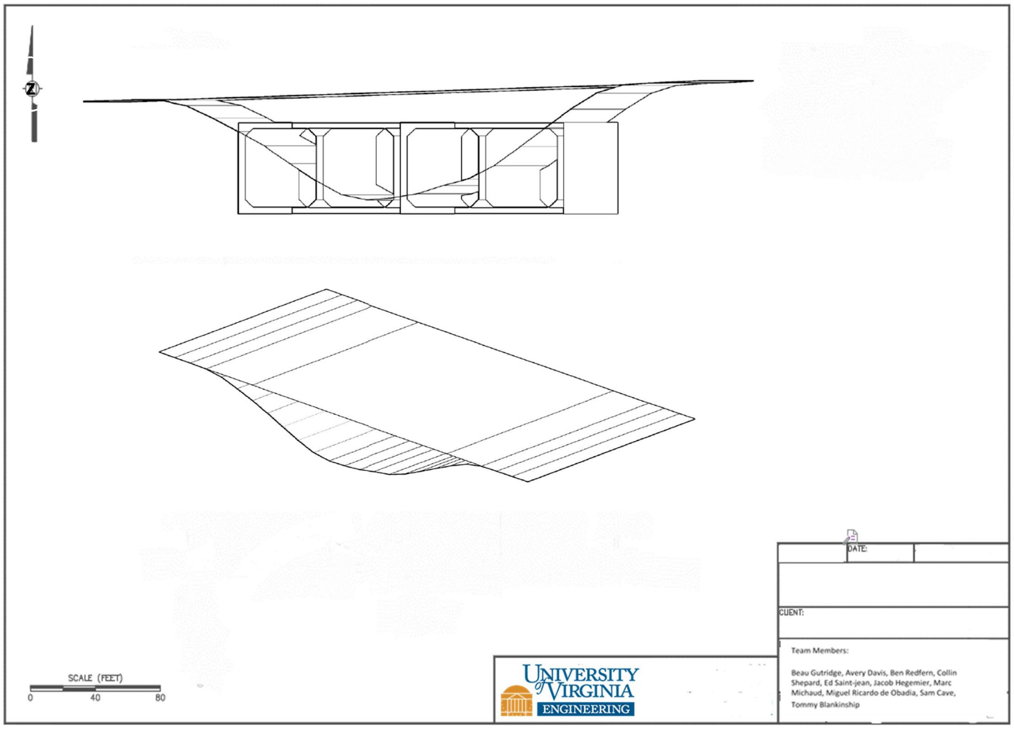



**Figure 5: Conceptual Plan/Profile Plan Sheet**

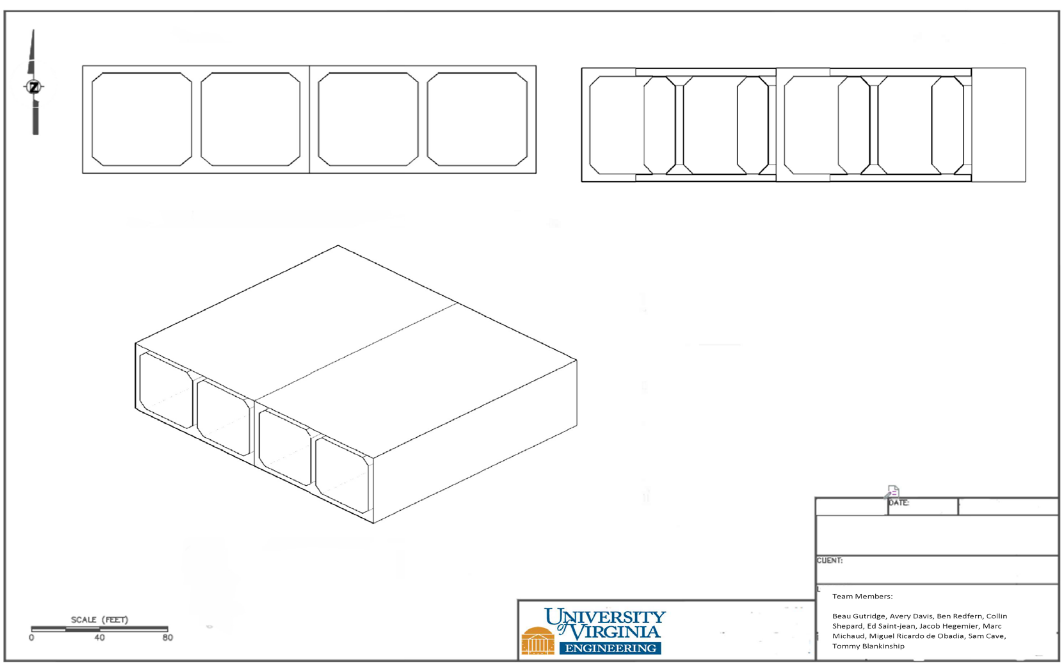



**Figure 6: Conceptual Profile Plan Sheet**

**Table 9: Minimum slab thickness according to MnDOT Guidelines.**

| Span (ft) | Minimum Top Slab Thickness (in) | Minimum Bottom Slab Thickness (in) |
|---|---|---|
| $6 \le$ Span $\le 8$ | 8 | 8 |
| Span > 8 | 9 | 10 |

**Table 10: Minimum wall thickness according to WisDOT Guidelines.**

| Minimum Wall Thickness (in) | Rise (ft) | Apron Wall Height Above Floor, $H_a$ (ft) |
|---|---|---|
| 8 | Rise < 6 | $H_a < 6.75$ |
| 9 | $6 \le$ Rise < 10 | $6.75 \le H_a < 10$ |
| 10 | Rise $\ge 10$ | $10 \le H_a < 11.75$ |
| 11 | | $11.75 \le H_a < 12.5$ |
| 12 | | $12.5 \le H_a < 13$ |

**Table 11: Height and equivalent height for live load surcharge according to WisDOT. Linear interpolation is used for values not listed in the table.**

| Height (ft) | $h_{eq}$ (ft) |
|---|---|
| $\le 5.0$ | 4.0 |
| 10.0 | 3.0 |
| $\ge 20$ | 2.0 |

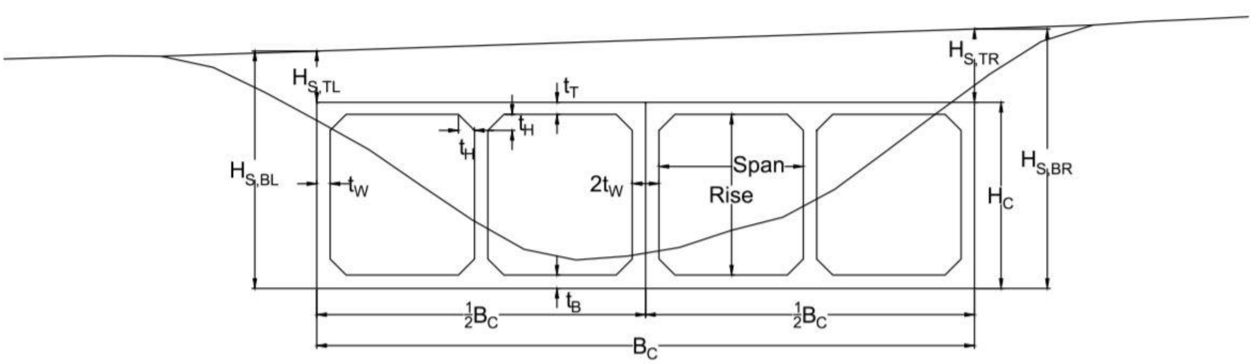



**Figure 7: CAD drawing of our two twin cell box culvert system explaining variable names. The sloped straight line on the top represents the roadway. The U-shaped section represents the elevation profile of the streambed.**

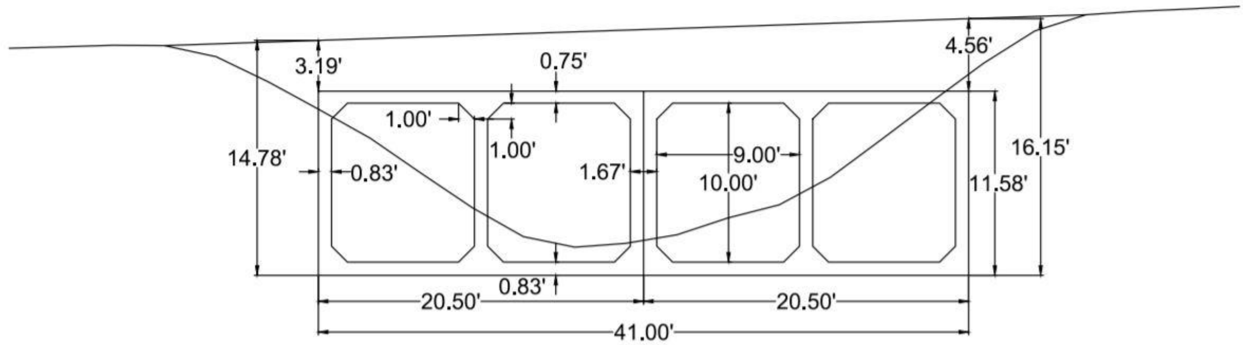



**Figure 8: CAD drawing of our two twin cell box culvert system with dimensions. The sloped straight line on the top represents the roadway. The U-shaped section represents the elevation profile of the streambed.**

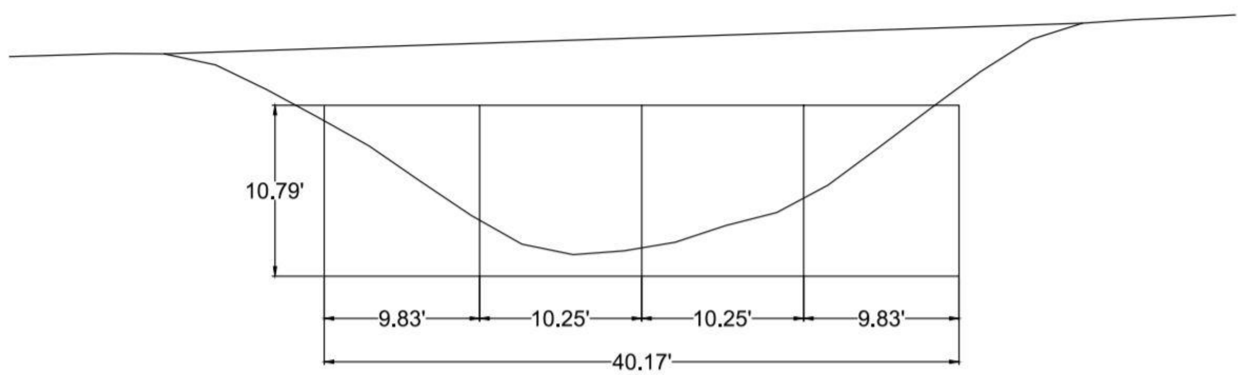



**Figure 9: CAD drawing of the rigid frame representation of our box culvert with dimensions. The members are drawn halfway between their respective walls and slabs. The sloped straight line on the top represents the roadway. The U-shaped section represents the elevation profile of the streambed.**

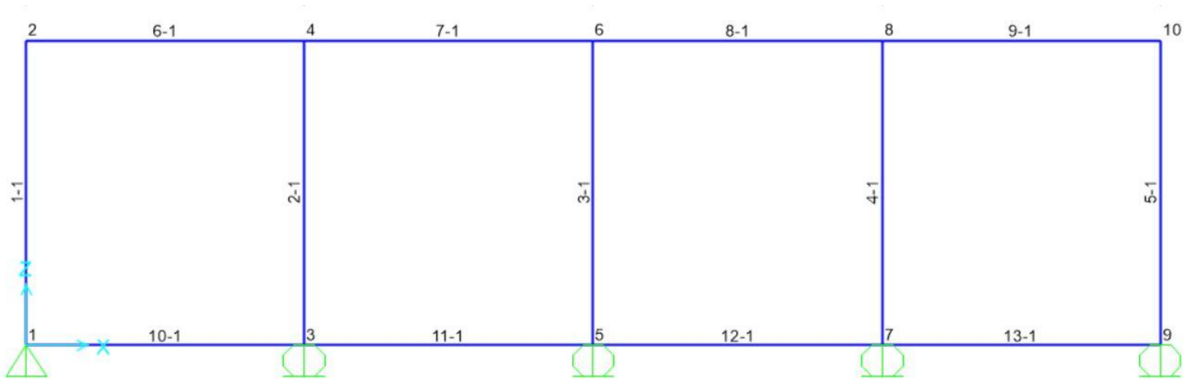



**Figure 10: Model rendering in SAP2000 with element and joint numbers.**

**<u>Appendix B, Figures:</u>**


**Figure 11: Aerial photograph of the area of the bridge. The section of roadway used for the elevation analysis is represented by a black line.**

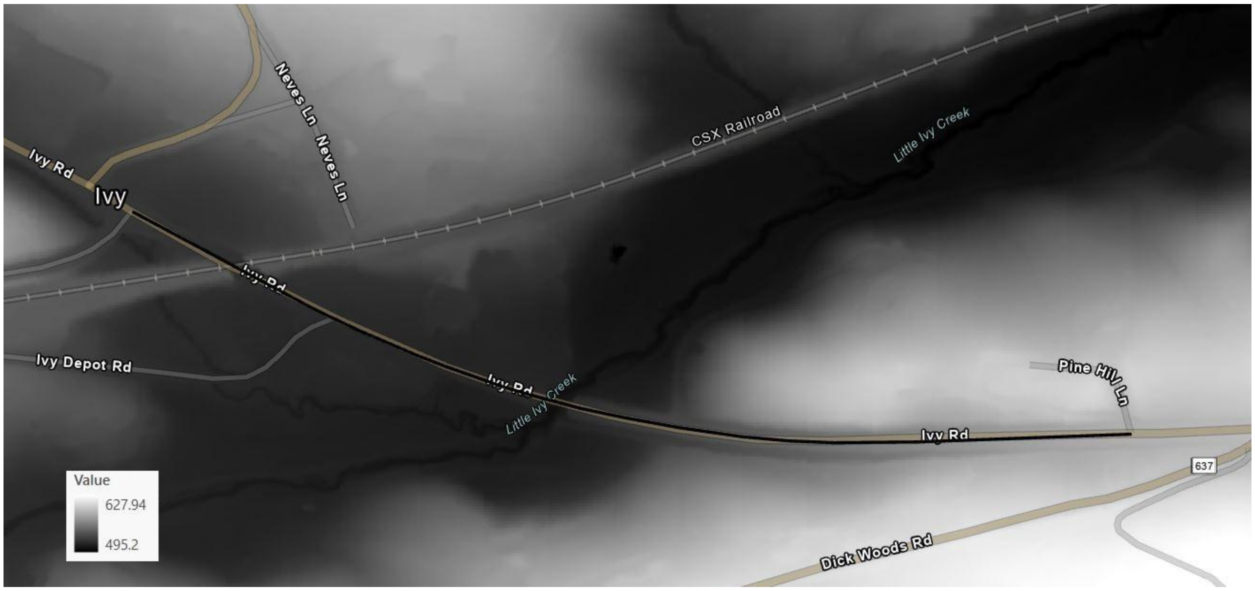



**Figure 12: The DEM of the area of the bridge used to create the graph showing distance along roadway versus the elevation above sea level shown below.**

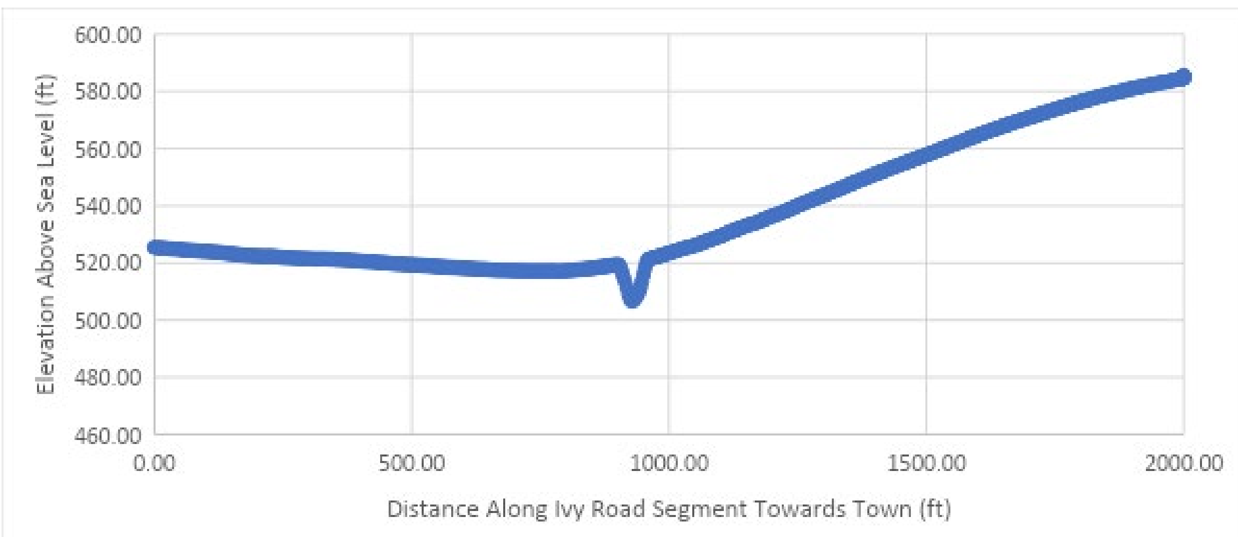



**Figure 13: Graph showing distance along roadway versus the elevation above sea level. This is for the entire segment shown above.**

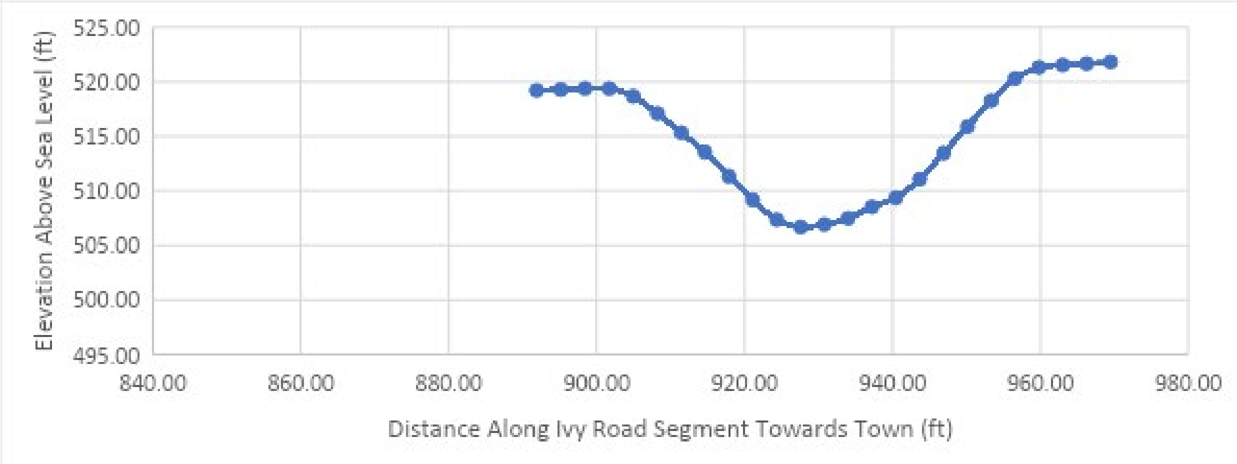



**Figure 14: Graph showing distance along roadway versus the elevation above sea level. This is for the segment including the stream and the hollow that needs to be spanned.**

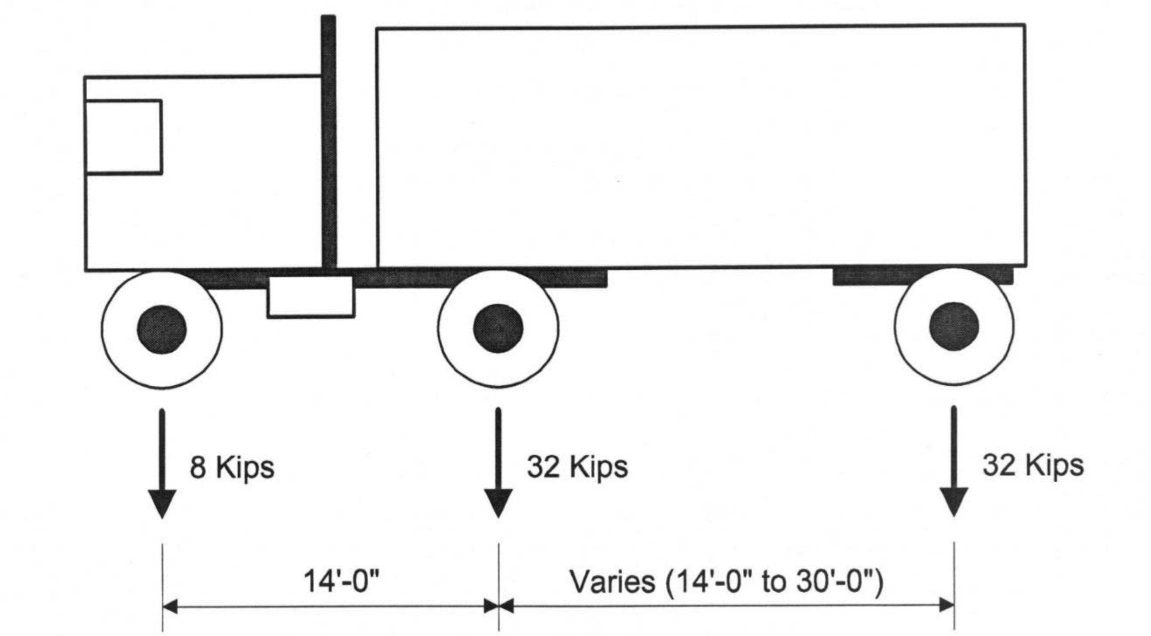



**Figure 15: AASHTO design truck sideview to illustrate the axle spacings and axle loadings.**

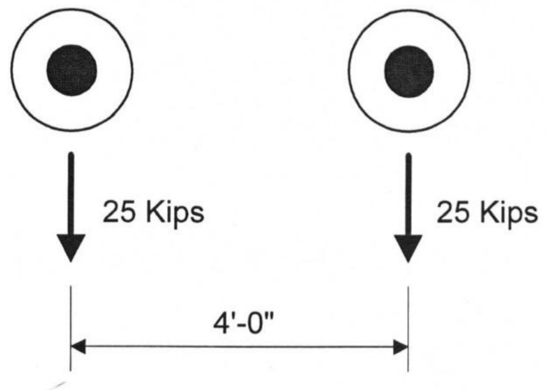



**Figure 16: AASHTO design tandem wheel sideview to illustrate the axle spacings and axle loadings.**

**Appendix C, References:**

Minnesota Department of Transportation LRFD Bridge Design Manual (September 2013). Retrieved from https://www.dot.state.mn.us/bridge/pdf/lrfdmanual/section12.pdf

Winsconsin Department of Transportation Bridge Manual (July 2018). Retrieved from https://wisconsindot.gov/dtsdManuals/strct/manuals/bridge/ch36.pdf

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