Project Report

CEEn-2018CPST-001: Bluffdale Bridge Options

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Structural Calculations and Options for a Roadway Bridge over Utah Canal
14400 S., Bluffdale, UT

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Introduction

A bridge at 14400 South over Utah Canal in Bluffdale, Utah has suffered significant damage due to corroding steel reinforcement within the bridge superstructure. A BYU capstone team was asked to analyze the current capacity of the bridge, estimate its remaining lifespan, propose options for rehabilitation or replacement, and provide possible funding sources for each recommended method.

Site visits were performed in October 2018 and January, February, and March 2019 to gather various pieces of data and to monitor the bridge over time. As part of the site visits and data gathering, a 3-dimensional model of this bridge was built using photogrammetric software. This model is available at http://prismweb.groups.et.byu.net/gallery2/ and may be used by the city of Bluffdale for any purpose.

We would like to recognize the great contributions made by Dr. Kevin Franke (team mentor), Dr. Fernando Fonseca (concrete analysis), Dr. Christine Isom (bridge loading and design), Dr. Jim Nelson (economic analysis), and Drs. Wayne Lee and Rollin Hotchkiss (capstone project coordinators) throughout the course of this project. We also recognize the contributions made by many third-party contractors who were willing to provide cost estimates for rehabilitation or replacement options.

Abstract

This project focused on a bridge constructed from prestressed double-tee beams, built and installed in 1986. The bridge has suffered heavy concrete damage on two girders and moderate damage on several others. Exposed rebar and prestressing strand are the primary concerns for the strength of this structure.

Data gathered from visual inspections was analyzed and compared to the original bridge design. Discrepancies were found between the original design specifications and the as-built condition, which likely had a large role in the early deterioration of this structure. Methods prescribed by the American Association of State Highway and Transportation Officials (AASHTO) and the Precast Concrete Institute (PCI) were used to analyze the current strength of the structure. Reports from the Utah Department of Transportation (UDOT) were used to form a time history of the bridge deterioration and to estimate the remaining life of the structure before terminal service condition is reached.

An economic analysis of two temporary repair options and two full bridge replacement options was performed. Because the damage to the bridge in question was likely amplified by both water seepage and thermal effects, a new type of bridge was proposed. This new bridge, made from precast concrete arches, will largely reduce the seepage and expansion problems found to be prevalent in the existing structure. This structure is more expensive than a bridge replacement of the same type, but its extended life expectancy makes it the most cost-efficient option. Advantages and disadvantages of each option are set forth in the economic analysis section.

Funding sources (apart from dedicated city funds) at both the local and state level are available. Several options at each level are presented in the funding sources section. Many of these sources are designed to aid cities in promotion of intermodal transportation. If this bridge were part of a larger project, repair and replacement funding would be readily available. All options, regardless of their tie to intermodal transportation, are set forth and analyzed.

Preface to Structural Analysis

This analysis encompasses several design and analysis techniques put forth by the American Association of State Highway and Transportation Officials (AASHTO) and the Precast Concrete Institute (PCI). The bridge beams in question have structural damage at the mid-span point, the critical area for bending moment stresses. The maximum shear stresses occur at the edge of the bridge. Because the edges of the bridge beams do not have major concrete section loss, nor has the prestressing steel been severely compromised in these locations, we have made the assumption that the bridge has its original design capacity for shear strength.

All analysis calculations focus on bending moment capacity due to the loss of prestressing strand and concrete sections. Various calculation methods were used to check the values included in this report; some methods of analysis were not included to maintain simplicity.

AASHTO bridge design specification section 4.6 allows for the use of static analysis in determining the strength of a structure. This analysis uses a simple statics method with appropriate fatigue limit factors. Because the loading patterns of this bridge are extremely simple, with two design lanes and a current weight restriction of 26,000lbs GVW, the basic bridge design falls outside standard AASHTO bridge loading specifications. Using a simple static analysis, together with dynamic loading factors prescribed in the bridge design specification, this analysis gives an approximation of the current state of the structure.

Steel and concrete deterioration are ongoing processes and cannot be accurately predicted. Corrosion of steel still embedded in concrete poses a special challenge for analysis, and the state of such reinforcement cannot be determined without a full impedance scan or other corrosion analysis of the structure. Such tests are costly and may not yield results with enough precision to be an economical option. Given the very small scale of the bridge and the challenges presented with connecting testing equipment to steel encased within the concrete, we do not recommend such scans or tests be done.

All structural calculations used to determine loading, capacity, and future projections will be presented. These calculations are estimations of the current structure condition but cannot be construed as precise and accurate figures. A large amount of variability exists based on unknown parameters. This evaluation uses conservative estimations, which are clearly explained hereafter.

Brigham Young University and the members of the capstone team representing the university are not liable for any consequences due to structural deficiencies. This report has not been stamped by a licensed engineer and only provides estimations and recommendations. Any failure of the structure before it is repaired or replaced is the sole liability of Bluffdale City and the Utah Department of Transportation (UDOT).

Demand Analysis

Lane Distribution

Lane distribution requirements were calculated based on equations from the 2012 AASHTO LRFD Bridge Design Specification Handbook. The distribution factor specifies how much of a design lane load is applied to each beam.

The bridge being examined consists of 12 beams: 2 on each side supporting a sidewalk that are raised 6" above the other beams, 8 interior beams supporting the roadway. Interior beam analysis was used for the beam in question. The end beams are only supporting pedestrian loads and do not show any signs of significant deterioration; thus, the interior beam loading distribution factors were used in the analysis.

Analysis

Table 4.6.2.2.2b-1 contains the distribution factor calculations for the interior beams. The double-tee section aligns with section i shown in Table 4.6.2.2.1-1 and is a valid shape for AASHTO bridge distribution calculations.

The bridge beams are anchored together with 3 small steel plates on each side. Beneath the bridge, rust stains are visible where some of the connecting plates are located. Because these stains are visible, we know that the plates are degrading. They can still be assumed to prevent relative vertical displacement at the interface of the beams (no evidence that the beams have differential displacement is visible) but cannot be assumed to be strong enough to make the entire bridge deck act as a single unit.

The distribution factor was calculated with the following equations:

$Dist.Factor = \frac{S}{D}$ where S = beam spacing, 60"

$C = K\left(\frac{W}{L}\right) \leq K$ where K = 2.0 (Table 4.6.2.2.2b-1, conservative values for preliminary design)

W = roadway width, 40 ft
L = clear span length, 24 ft

$$C = (2.0)\frac{40ft}{24ft} \leq 2.0$$

Using C = 2.0:

$D = 11.5 - NL + 1.4NL(1 - 0.2C)^2$ where NL = number of lanes, 3

$$D = 10.01\ ft$$

Total Distribution Factor:

$$Dist.Factor = \frac{S}{D} = \frac{5ft}{10.01ft} = 0.499 \approx 0.5$$

This bridge will only carry two lanes of traffic; however, due to the road width of approximately 40 feet, the bridge must be designed for three lanes of traffic.

The distribution factor applies to the live load due to the design axle load and the uniform distributed load across the road surface. Dead loads due to the wearing surface and the beam weight are not subject to the distribution factor.

Bending Moment Analysis

Dead Load Moment

The weight of each beam was calculated as a distributed dead load based on the original cross section. Though some concrete has chipped off the bottom, it is conservative to use the original cross-sectional area of the beam for the calculations.

$$A = b_w * t_w + 2(b_f * h_f)$$

Where b_w = 60 in. (59.5" concrete plus 0.5" of filler grout)
t_w = 6 in.
b_f = 6.5 in.
h_f = 10 in.

$$A = 60 * 6 + 2(6.5 * 10) = 487\ in^2$$

Assuming a concrete weight of 0.145kcf for a 1-foot length of beam:

$$w = 487in^2 * \frac{12in}{1ft} * \frac{0.145kcf}{(\frac{1728in^3}{ft^3})}$$

$$w = 0.4904kip/ft$$

Total moment on the beam due to self-weight:

$$M_{DC} = \frac{wl^2}{8} = \frac{\frac{0.4904k}{ft} * (24ft)^2}{8} = 35.31k\text{-}ft$$

Wearing Surface Moment:

$$w_s = 60in * 6in * \frac{12in}{ft} * \frac{0.145kcf}{(\frac{1728in^3}{ft^3})}$$

$$w_s = 0.3625\ k/ft$$

$$M_{DW} = \frac{\frac{0.3625k}{ft} * 24ft^2}{8} = 26.1k\text{-}ft$$

Live Load Moment

The bridge was originally designed for HS-20 loading, which consists of an 8 kip point load from the front axle of the design truck and a 32 kip point load from the back axle.

In addition to the point load from the axle, the design lane must be designed for a 0.64 kip/ft distributed live load.

Because the bridge is only 24 ft long and the spacing between the front and rear axle of a design truck is 14 ft, the front axle will no longer be on the bridge when the rear axle reaches the middle. The rear axle causes a larger moment at the center than the two axles combined at any point on the bridge. Only a single point load is used in the live load moment calculation because the front axle is not on the structure at the point of maximum moment.

$$PL = 32k$$

$$M_{PL} = \frac{PL}{4} = \frac{32k * 24ft}{4}$$

$$M_{PL} = 192k\text{-}ft$$

The distributed live load may be calculated in the same manner as the dead loads, with a 0.64k/ft distributed load.

$$M_{DL} = 46.1k\text{-}ft$$

Impact Factors and Limit States

Several limit states must be calculated to ensure safe operation of the bridge. All load combinations are taken from the AASHTO bridge design manual, section 3.4.1. Following is a list of applicable limit states and their purposes:

Service 1:	Load combination for normal operational use, with 55mph wind and nominal loads. This load combination is applicable.
Service 3:	Load combination to control cracks within prestressed concrete members.
Strength 1:	Load combination for normal vehicular use without wind.
Strength 4:	Load combination for high dead load to live load force effect ratios. This is not applicable to this bridge, as the main concern is live load rather than dead load.
Fatigue:	Fatigue and fracture load combination.

No extreme event load combinations were calculated. All other combinations are non-applicable for the scope of this project.

Table 1 summarizes the maximum bending moments produced by each load combination.

Table 1: HS-20 Limit States

Limit States		
Load	Max. Moment	
DC	35.3	k-ft
DW	26.1	k-ft
LL	238.1	k-ft
IM (Live)	0.33	
IM (Fatigue)	0.15	
Distribution Factor	0.500	lanes/beam
Service 1	219.7	k-ft
Service 3	188.1	k-ft
Strength 1	360.3	k-ft
Fatigue	123.2	k-ft
Notes		
1. Limit States are given in LRFD Table 3.6.2.1-1		

Shear Analysis

Shear stresses have not been considered in this design.

Maximum shear stresses are experienced at each end of the bridge. The concrete at each end of the beams has suffered very minimal damage so it can be assumed that the bridge still has the original design shear capacity. The primary concern with cracked concrete and exposed reinforcement at the center of the bridge is bending moment capacity, not shear capacity.

Shear and Bending Moment Diagrams

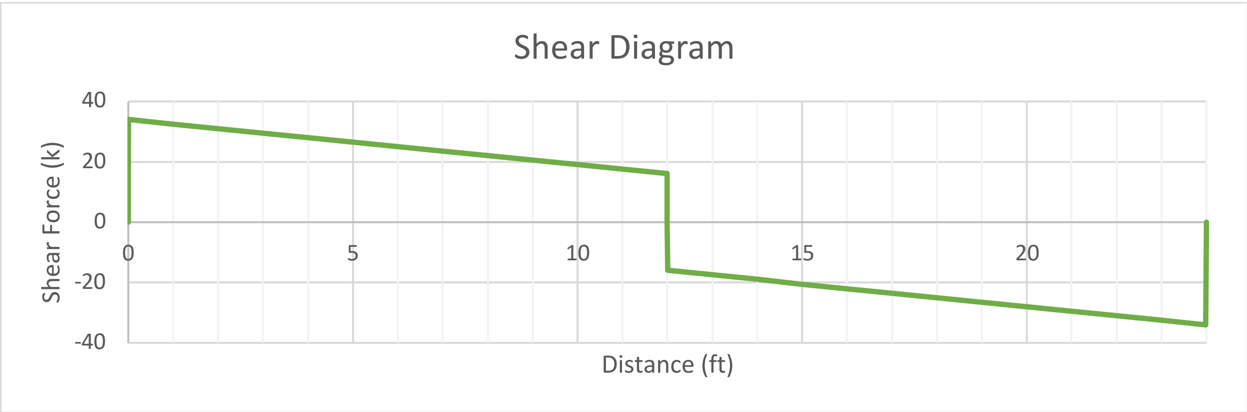


Figure 1: HS-20 Shear Diagram

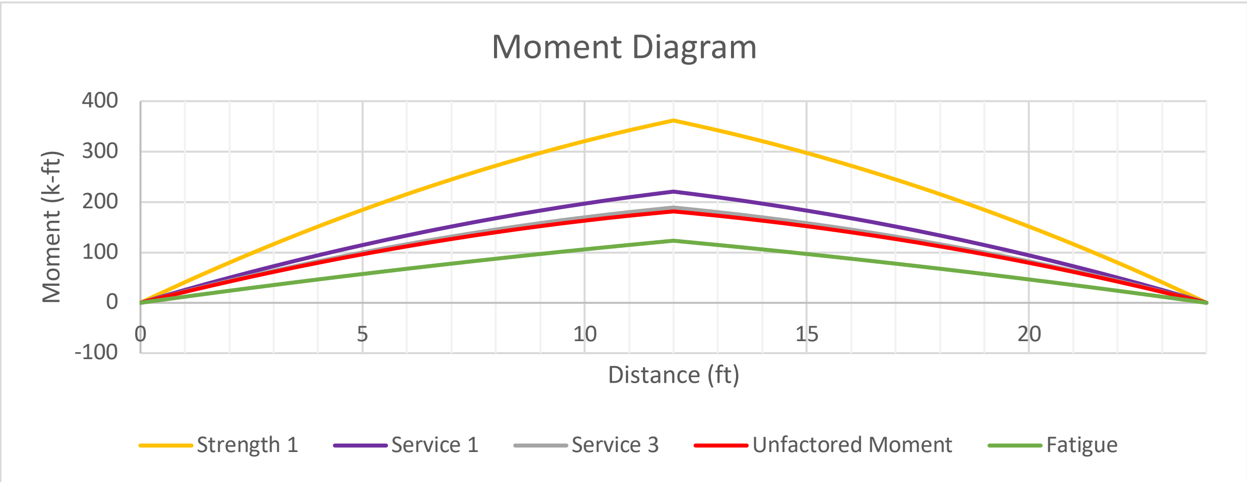


Figure 2: HS-20 Moment Diagram

Load Limits

Currently, 14400 South has a weight limit of 26,000lb, posted near the intersection at Redwood Road. This vehicular load limit reduces the demand on the bridge significantly. Rather than a 32k axle load, the rear axle load on a 26,000lb truck is roughly 22kips. The following tables and diagrams show the calculated demands on the bridge based on the current weight restriction.

Table 2: 26,000lb Loading Limit States

Limit States		
Load	Max. Moment	
DC	35.3	k-ft
DW	26.1	k-ft
LL	178.1	k-ft
IM (Live)	0.33	
IM (Fatigue)	0.15	
Distribution Factor	0.500	lanes/beam
Service 1	179.8	k-ft
Service 3	156.1	k-ft
Strength 1	290.5	k-ft
Fatigue	92.2	k-ft
Notes		
1. Limit States are given in LRFD Table 3.6.2.1-1		

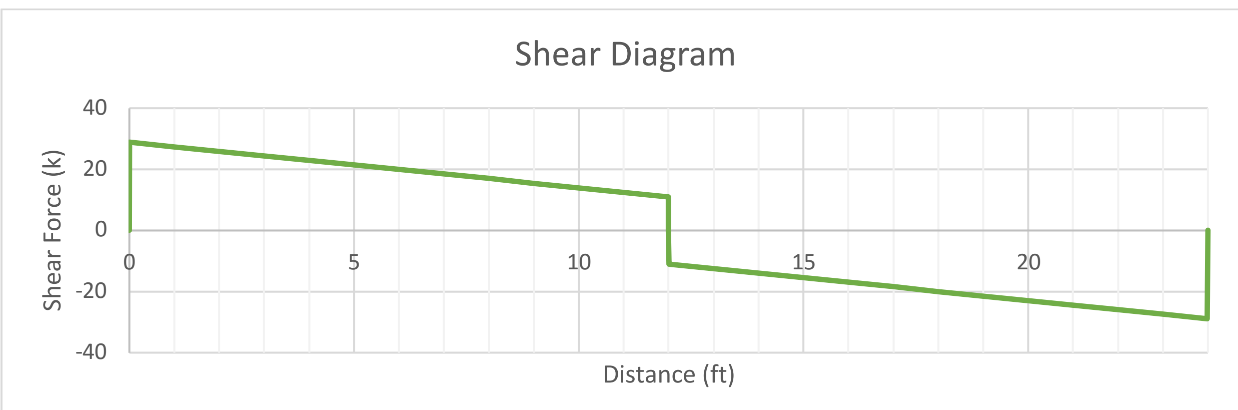


Figure 3: 26,000lb Loading Shear Diagram

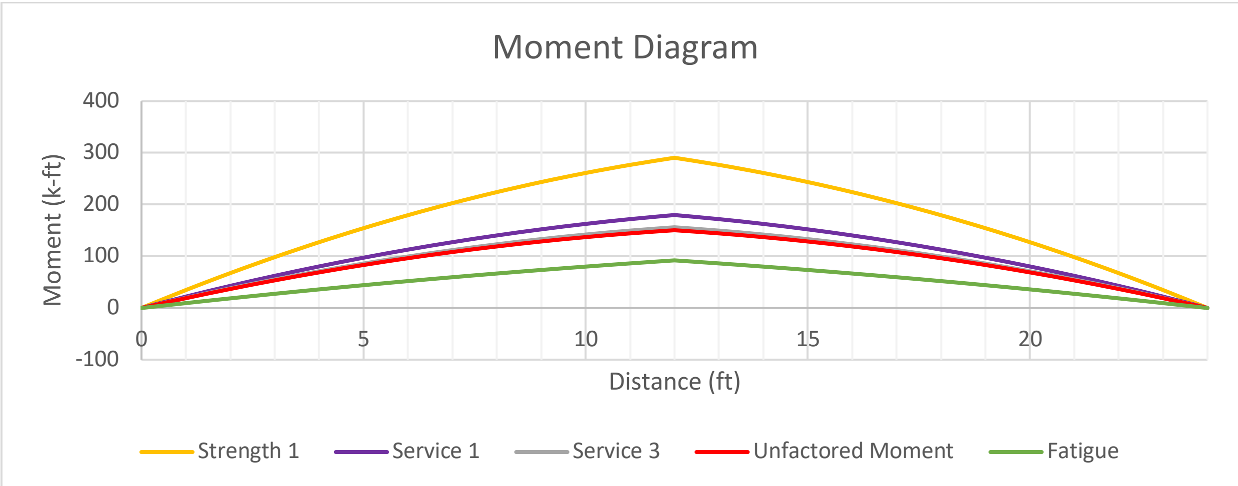


Figure 4: 26,000lb Loading Moment Diagrams

Strength Analysis

Cross Sectional Properties

Table 3 summarizes the cross-sectional properties of the bridge as originally designed. These measurements were used to determine the prestress losses associated with precast construction.

Table 3: Original Cross-Sectional Properties

Original Cross-Sectional Properties			
Flange Width	bf	59.5	in
Flange Thickness	tf	6	in
Web Width	bw	6.5	in
Web Height	hw	10	in
Total Area	Ag	487	in^2
Moment of inertia	I	8253	in^4
Centroid Location (from top)	y	5.14	in
Section Modulus	S	1607	in^3
Prestress Strand Area	Aps	1.53	in^2
Tension Reinforcement Area	As	0.20	in^2
Compression Reinforcement Area	As'	1.96	in^2

Prestress Concrete Analysis

Prestress Losses

Prestress Losses were calculated in accordance with the Federal Highway Administration (FHA) Prestressed Concrete Girder Superstructure Design handbook.

Prestress Losses are calculated with the following equation:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pCR} + \Delta f_{pR2}$$

Stresses in tendons prior to transfer:

$$\begin{aligned} f_{pt} + \Delta f_{pES} &= 0.75 f_{pu} \\ &= 202.5 ksi \end{aligned}$$

Elastic Shortening:

$$\Delta f_{pES} = \frac{A_{ps}f_{pbt}(I_g+e^2(A_g))-eM_gA_g}{A_{ps}(I_g+e^2(A_g))+\frac{A_gI_gE_{ci}}{E_p}}$$

Where e = eccentricity at midspan, 9.42"
f_{pbt} = stress prior to transfer, 202.5ksi
M_g = self-weight moment, 35.3 k-ft
E_{ci} = concrete modulus of elasticity, 4031ksi
E_p = prestressed strand modulus of elasticity, 29000ksi

$$\Delta f_{pES} = 21.97ksi$$

Note: eccentricity was calculated according to the original bridge plans with all (5) prestressing strands in a group at the bottom of each leg. Based on site observations, the strands were not installed in this pattern. However, for the purposes of this analysis, the original design was considered apart from site observations.

Prestressing stress at transfer:

$$f_{pt} = Stress\ prior\ to\ transfer - \Delta f_{pES}$$

$$= 180.53ksi$$

Prestressing force at transfer:

$$P_t = f_{pt}A_{ps}$$

$$= 276.2kip$$

Shrinkage Losses

$$\Delta f_{pSR} = (17.0 - 0.15H)ksi$$

Where H = average annual relative humidity, assumed to be 0.55

$$\Delta f_{pSR} = 8.75kip$$

Creep Losses

$$\Delta f_{pCR} = 12.0f_{cgp} - 7.0\Delta f_{cdp}$$

Where f_{cgp} = concrete stress at the center of gravity of the prestressing steel, 4.09ksi
f_{cdp} = change in concrete stress due to permanent loads

$$\Delta f_{pCR} = 30.75ksi$$

Relaxation Losses

$$\Delta f_{pR2} = 20.0 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})ksi$$

$$= 3.312ksi$$

Under the assumption that the strand is low-relaxation strand, the relaxation loss may be reduced by 70%. This will result in a relaxation loss of 0.99ksi.

Total Loss Calculations

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

$$= 62.47ksi$$

$$f_{pe_{max}} = 0.8f_{py} = 0.8(243ksi) = 194.4ksi$$

$$f_{pe} = 0.75f_{pu} - \Delta f_{pT}$$

$$= 140.03ksi < 194.4ksi$$

The stress loss calculated here is greater than half of the yield strength of the strand. However, with the analysis modified to incorporate the current cross-sectional properties of the beam and assumed layout of the prestressing strand, the losses in the strand are different. Table 4 and Table 5 show the results from the original analysis and the current section analysis, respectively.

When the prestressed losses exceed half the yield strength of the strand, the beam can be designed as a simply reinforced concrete double-tee beam. Because the bridge beams under investigation currently fit the criteria for design as a simply reinforced beam (strand stress is less than 135ksi), the analysis presented hereafter follows standard reinforced concrete principles rather than prestressed concrete.

Note: The current section eccentricity was calculated as if the prestressed strands were stacked on top of each other. The centroid of the concrete beam is currently 4.58 inches from the top of the beam. The centroid of the reinforcement was calculated by subtracting the strand edge distance (1.5 inches) and, assuming the strands are in contact with each other in a vertical pattern, the distance from the edge of the first strand to the center of the third strand (1.25 inches) from the original beam depth of 16 inches. This results in an eccentricity of 8.67 inches.

Table 4: Original Prestress Loss

Prestress Losses - Original Design			
Ultimate Stress	f_{pu}	270.0	ksi
Initial Tendon Stress	f_{pbt}	202.5	ksi
Prestress Area	A_{ps}	1.53	in^2
Self-Weight Moment	M_g	35.3	k-ft
Moment of Inertia	I_g	8253.0	in^4
Eccentricity	e	9.42	in
Total Cross-Sectional Area	A_g	487	in^2
Modulus of Elasticity	Ec	4031	ksi
	Es	29000	ksi
Elastic Shortening Loss	Δf_{pES}	21.97	ksi
Stress Prior to Transfer	f_{pt}	180.53	ksi
Prestress Force at Transfer	P_t	276.2	kip
Average Humidity	H	55%	
Shrinkage Loss	Δf_{pSR}	8.75	ksi
Concrete Stress	Δf_{cgp}	3.053	ksi
Center of Gravity Change	Δf_{cdp}	0.841	ksi
Creep Loss	Δf_{pCR}	30.75	ksi
Relaxation Loss	Δf_{pR2}	3.312	ksi
Low Relaxation Loss		0.99	ksi
Max f_{pe}	f_{pe}	194.40	ksi
Total Loss	Δf_{pT}	62.47	ksi
f_{pe}	f_{pe}	140.03	ksi

Table 5: Current Section Prestress Loss

Prestress Losses - Current Section			
Ultimate Stress	f_{pu}	270.0	ksi
Initial Tendon Stress	f_{pbt}	202.5	ksi
Prestress Area	A_{ps}	1.53	in^2
Self-Weight Moment	M_g	0.0	k-ft
Moment of Inertia	I_g	8253.0	in^4
Eccentricity	e	8.67	in
Total Cross-Sectional Area	A_g	487	in^2
Modulus of Elasticity	Ec	4031	ksi
	Es	29000	ksi
Elastic Shortening Loss	Δf_{pES}	22.16	ksi
Stress Prior to Transfer	f_{pt}	180.34	ksi
Prestress Force at Transfer	P_t	275.9	kip
Average Humidity	H	55%	
Shrinkage Loss	Δf_{pSR}	8.75	ksi
Concrete Stress	Δf_{cgp}	3.080	ksi
Center of Gravity Change	Δf_{cdp}	0.000	ksi
Creep Loss	Δf_{pCR}	36.96	ksi
Relaxation Loss	Δf_{pR2}	1.995	ksi
Low Relaxation Loss		0.00	ksi
Max f_{pe}	f_{pe}	194.40	ksi
Total Loss	Δf_{pT}	67.87	ksi
f_{pe}	f_{pe}	134.63	ksi

Beam Design (PCI)

Because the prestress losses are less than $0.5f_{pu}$, the stress in each strand is determined by strain compatibility methods. The following calculations are based on the current bridge condition, with tensile and compressive reinforcement determined from the original design drawings and site observations.

$$C\omega_{pu} = C\left(\frac{A_{ps}f_{pu}}{bd_p f'c}\right) + \frac{d}{d_p}(\omega - \omega')$$

$\omega = \frac{A_s f_y}{bdf'c}$ where $A_s = 0$ in^2 (tensile reinforcement is not providing strength)

$\omega' = \frac{A'_s f_y}{bd'f'c}$ where A’s = 3.73 in^2 (10 #4 bars, 4 #6 bars)

d’ = 3 inches (rebar locations estimated from structural drawings)
b = 59.5 inches (width of beam flange)
f’c = 4.5ksi

$\omega = 0$
$\omega' = 0.2801$

For bridges…… C = 1.06

$$C\omega_{pu} = 1.06\left(\frac{1.53in^2(270ksi)}{59.5in(13in)(4.5ksi)}\right) + \frac{0in}{13in}(0 - 0.2801)$$

$$C\omega_{pu} = 0.0925$$

Note: d was taken as 0 inches because there is no longer any remaining tensile rebar.

If d was assumed to be 14 inches as originally designed, with tensile reinforcement equal to 0.39 in^2, the ω value would increase to 0.02514, and the value of $C\omega_{pu}$ would be taken as 0. Because the higher $C\omega_{pu}$ value is conservative in concrete design, d was taken as 0 inches.

From Figure 4.12.3 in the PCI handbook with $C\omega_{pu}$ = 0.136,

$\varepsilon = 0.022$
$f_{ps} = 267ksi$

Area of compression:

$$a = \frac{A_{ps} + A_s f_y - A_s' f_y'}{0.85(f'c)b}$$

$$a = 0.812\ in$$

Check for tension-controlled design:

$$c = \frac{a}{\beta_1} \text{ where } \beta_1 = 0.825$$

$$c = 0.984$$

$$\frac{c}{d_t} = \frac{0.984in}{14in} = 0.0703$$

Because $\frac{c}{d_t}$ is less than 0.375, the $\emptyset$ factor may be taken as 0.9.

Since the compression zone depth (a) is less than the flange depth, the moment may be calculated with no further modifications

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) + A_s f_y \left(d - \frac{a}{2}\right) + A'_s f'_y \left(\frac{a}{2} - d'\right)$$

Using the values previously calculated:

$$M_n = 4340\ k\text{-}in$$
$$= 361.7\ k\text{-}ft$$

$$\emptyset M_n = 325.5\ k\text{-}ft$$

This value for the moment-resisting capacity of the beam reflects the total loss of the tensile rebar reinforcement, but no loss of prestressing force. This calculation gives a base capacity of the bridge before any strand rusting has occurred.

Estimates of the original strength and current strength are given in Tables 6-7 below. See the Comments on Analysis section below for explanations of assumptions made in this analysis.

Table 7: Original Design Capacity

Precast Concrete Beam Capacity Analysis - Original Design			
Prestressing Strand Strength	f_{ps}	267	ksi
Effective Stress in Steel	f_{se}	134	ksi
Number of Strands	N	10	
Prestress Area	A_{ps}	1.53	in^2
Tensile Steel Area	A_s	0.393	in^2
Tensile Steel Strength	fy	60	ksi
Concrete Strength	f'c	4.5	ksi
Width	b	59.5	in
Compression Reinforcing	A'_s	3.73	in^2
Compression Steel Strength	f'y	60	ksi
Centroid of Comp. Reinforcing	d'	4	in
Stress Block Depth	a	0.915	in
Stress Block Area	A_{comp}	54.45	in^2
Prestressing Steel Centroid	d_p	13.00	in
Depth to Extreme Tension	d_t	14.00	in
Factor	β	0.825	
Extreme Comp. to Neutral	c	1.109	in
	c/d_t	0.0792	< 0.375
	∅	0.9	
Nominal Moment	M_n	4650.2	k-in
		387.5	k-ft
Design Moment	$\emptyset M_n$	348.8	k-ft
Notes			
1. All calculations were performed according to ACI 318-95 specifications			
2. Original cross section calculations do not incorporate any steel or concrete section loss due to corrosion			
3. A centroid depth of 12" was assumed for the prestressing strand. The five strands in each leg are likely stacked vertically, not in a horizontal group.			

Table 6: Current Capacity

Precast Concrete Beam Capacity Analysis - Current Section			
Prestressing Strand Strength	f_{ps}	267	ksi
Effective Stress in Steel	f_{se}	134	ksi
Number of Strands	N	8	
Prestress Area	A_{ps}	1.224	in^2
Tensile Steel Area	A_s	0.00	in^2
Tensile Steel Strength	fy	60	ksi
Concrete Strength	f'c	4.5	ksi
Width	b	59.5	in
Compression Reinforcing	A'_s	3.73	in^2
Compression Steel Strength	f'y	60	ksi
Centroid of Comp. Reinforcing	d'	4	in
Stress Block Depth	a	0.453	in
Stress Block Area	A_{comp}	26.93	in^2
Prestressing Steel Centroid	d_p	13.00	in
Depth to Extreme Tension	d_t	14.00	in
Factor	β	0.825	
Extreme Comp. to Neutral	c	0.549	in
	c/d_t	0.0392	< 0.375
	∅	0.9	
Nominal Moment	M_n	3330.0	k-in
		277.5	k-ft
Moment Capacity	$\emptyset M_n$	249.7	k-ft
Notes			
1. All calculations were done as per ACI 318-95 requirements			
2. Current cross-section capacity analysis incorporates the loss of one pre-stressing strand per girder, including a change in centroid location.			

Comments on Analysis

The calculated beam capacity shown above (325.5 k-ft) is well below the original design requirement for the HS-20 Strength 1 loading requirement (361.7 k-ft). In the calculations presented, the depth from the top of the beam to the centroid of the prestressing strand was assumed to be 13 inches. The reasons for this assumption are demonstrated in Figures 5-7 below.

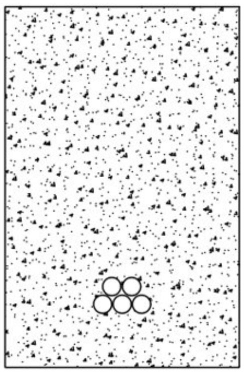


Figure 5: Original Strand Design (3-2 Layout)

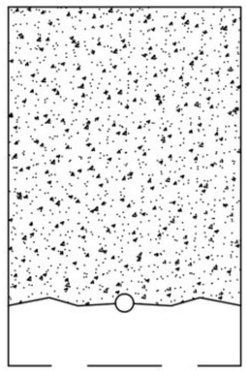


Figure 6: Known Design

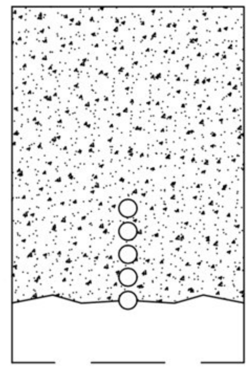


Figure 7: Probable Strand Layout

The most probable strand layout (shown in Figure 7) was used in the design calculations. This layout is not detailed in the structural drawings, so distances between strands were assumed. Because we can clearly see that there are not three exposed strands, we must be conservative and assume that the strands were placed in a vertical pattern instead of a grouped pattern. The bottom strand has a clear cover of 1.5 inches from the bottom of the beam to the edge of the strand, verified from site visits. Assuming that all the strands touch each other, the centroid of the 5-strand system is located 2.75 inches from the bottom of the beam, producing a d_p value of 13.25 inches. If the bottom strand is removed entirely, the centroid of the 4-strand system is located exactly 3 inches from the bottom of the beam. This d_p value of 13 inches was used in all beam capacity calculations.

The original plans show the strands stacked 2” apart from each other at the ends of the beams but grouped together in the middle. It is very likely that the strands were placed correctly on the ends but stacked vertically in the middle to make construction easier.

All modifications to the assumed strand layout are conservative estimates and may not actually reflect the construction of the beams. The exact construction cannot be determined without rebar imaging tests performed by other professional organizations. Without these tests, the conservative assumptions made in this analysis will give a general idea of the beam condition and strength but will not yield exact values.

Strength Projections

Figure 8 shows the calculated bridge strength as a function of the prestressing strand area remaining, combined with the code requirements. This graph shows that the bridge strength was never sufficient to satisfy the HS-20 Strength 1 load combination. If the strand were arranged according to the original design, as shown in Figure 5, the strength would be above the code requirement. However, with the modification to the design, with the strand layout likely following the pattern shown in Figure 7, the maximum strength never reaches the uppermost requirement.

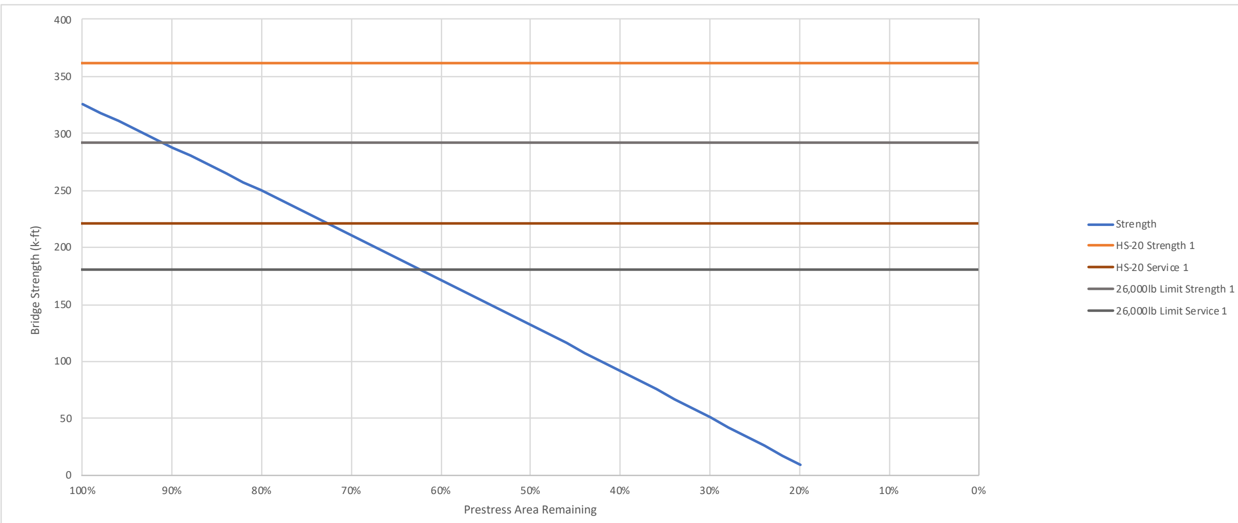


Figure 8: Strength vs Steel Area

It is estimated that the amount of prestressed strand remaining is roughly 75%. This estimate is rooted in the assumption that one strand is completely lost from one leg of each beam. To simplify calculations and remove unbalanced forces within the concrete due to torsional effects, we must assume that both legs of the double-tee are balanced. Thus, with the loss of one strand,

See the Photo Summary section of this report for further explanation of the origins of this estimate. As the steel continues to corrode, the strength will go down.

The bridge is currently estimated to have a strength of 210k-ft. This is sufficient to carry the 26,000lb gross vehicle weight requirement currently placed on the road. However, as the steel continues to corrode, the strength will rapidly decrease and will no longer be sufficient.

Figure 9 shows an estimate of the rate of steel corrosion with time. There is no way to precisely determine the current rate of steel corrosion; however, the area lost due to corrosion is an exponential function of time. Each labeled year refers to December of the given year, the time of the biennial UDOT inspection. This chart roughly matches our previous estimate of 75% percent section remaining at the end of 2018.

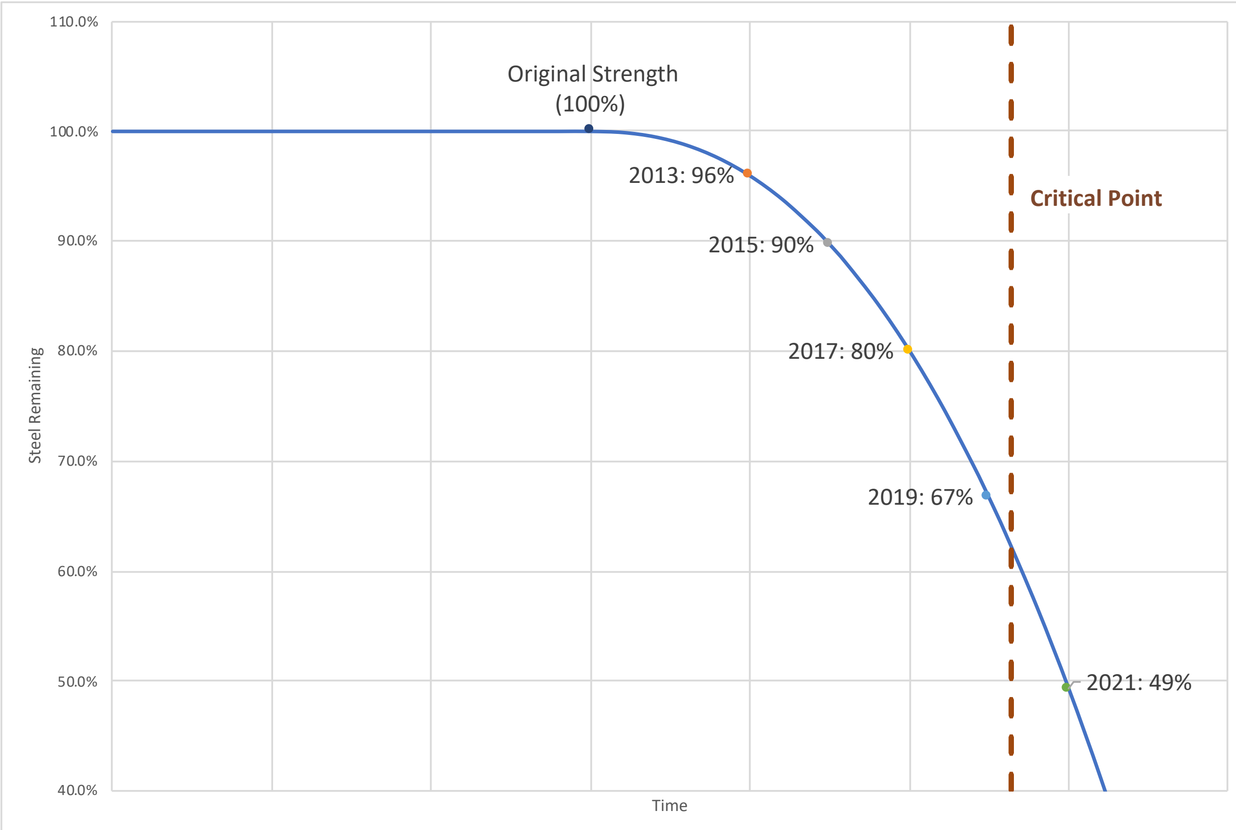


Figure 9: Estimated Steel Corrosion Timeline

To account for the exponential loss of steel section area, Figure 8 and Figure 9 may be combined to produce a more accurate estimation of the bridge strength. Figure 10 shows the resulting strength approximation with time. This approximation is based only on estimations gathered from site visits and may not reflect the actual strength curve for the bridge.

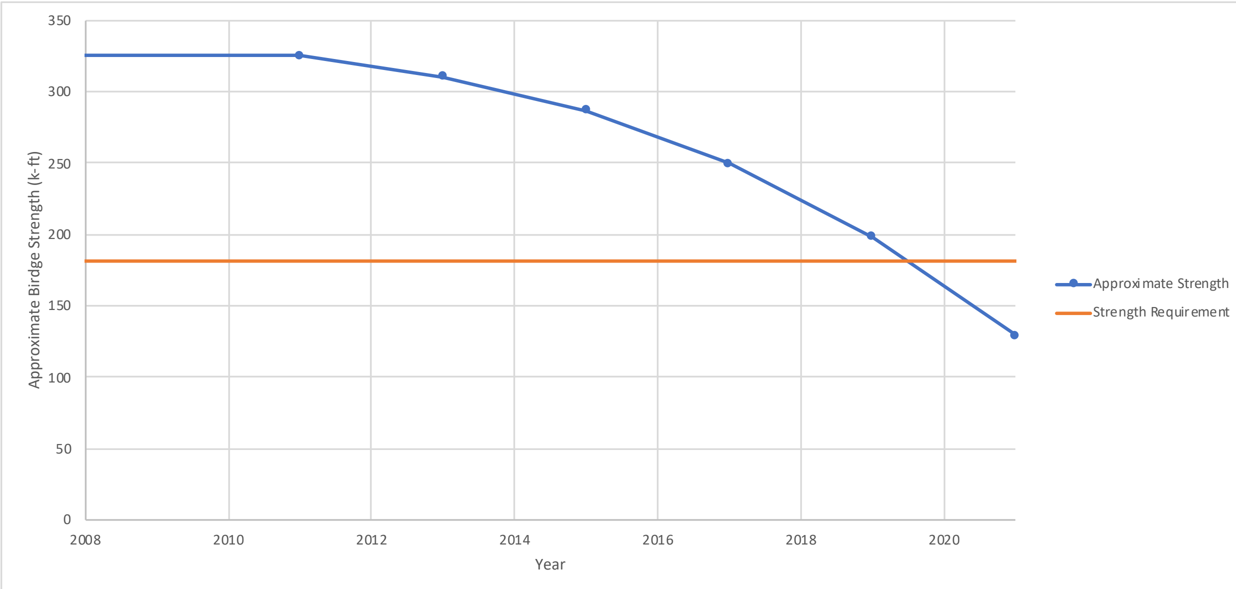


Figure 10: Adjusted Strength Approximation

At the end of the 2018-2019 winter, the bridge has an estimated capacity of 210k-ft. The rate of corrosion is likely lower during the dry summer months than the winter and spring months due to the decreased presence of water and road salts; however, corrosion will continue to occur.

It is anticipated that in the spring of 2020, the bridge beams will no longer have the required capacity to support traffic loads. Replacement of the deteriorating beams must be done as soon as possible so as to avoid structural failure.

Cracking on the bottom of the bridge beams was first reported in December 2011. At this point, only the rebar at the bottom of the bridge was likely corroding. Once the cracking occurred, more water and deicing salt was able to enter the concrete. When the rebar was first noted as exposed in 2013, the concrete cover remaining over the prestressing strand was likely at a very minimal level. However, with the passage of time, the concrete has continued to spall, and the prestressing strand has continued to rust. Because rust is highly expansive, rusting causes increased cracking; increased cracking allows more water and road salt to penetrate through the concrete and contact the steel; increased water and salt amplifies the rate of rusting. Once this spiral of degradation begins, it is very difficult to stop it. It may have been possible many years ago to combat the corrosion, but it has progressed to a point where it is impossible to stop it. Replacement of the damaged beams is mandatory.

Photo Summary

The purpose of this section is to provide images showing the current condition of the bridge and progression of deterioration over the duration of the investigation.

The team took four sets of photographs of the bridge on the following dates: 19 October 2018, 25 January 2019, 22 February 2019, & 5 March 2019. This section also includes pictures from the 14 December 2017 UDOT Inspection Report for comparison; the full report can be found in the Appendix.

Unfortunately, the photos in the UDOT inspection report and those taken in October 2018 and February 2019 were not intended to be used in a detailed comparison. Despite this, these photos still clearly show the rapidly progressing deterioration occurring within a relatively short time frame.

Center-Span Cracking

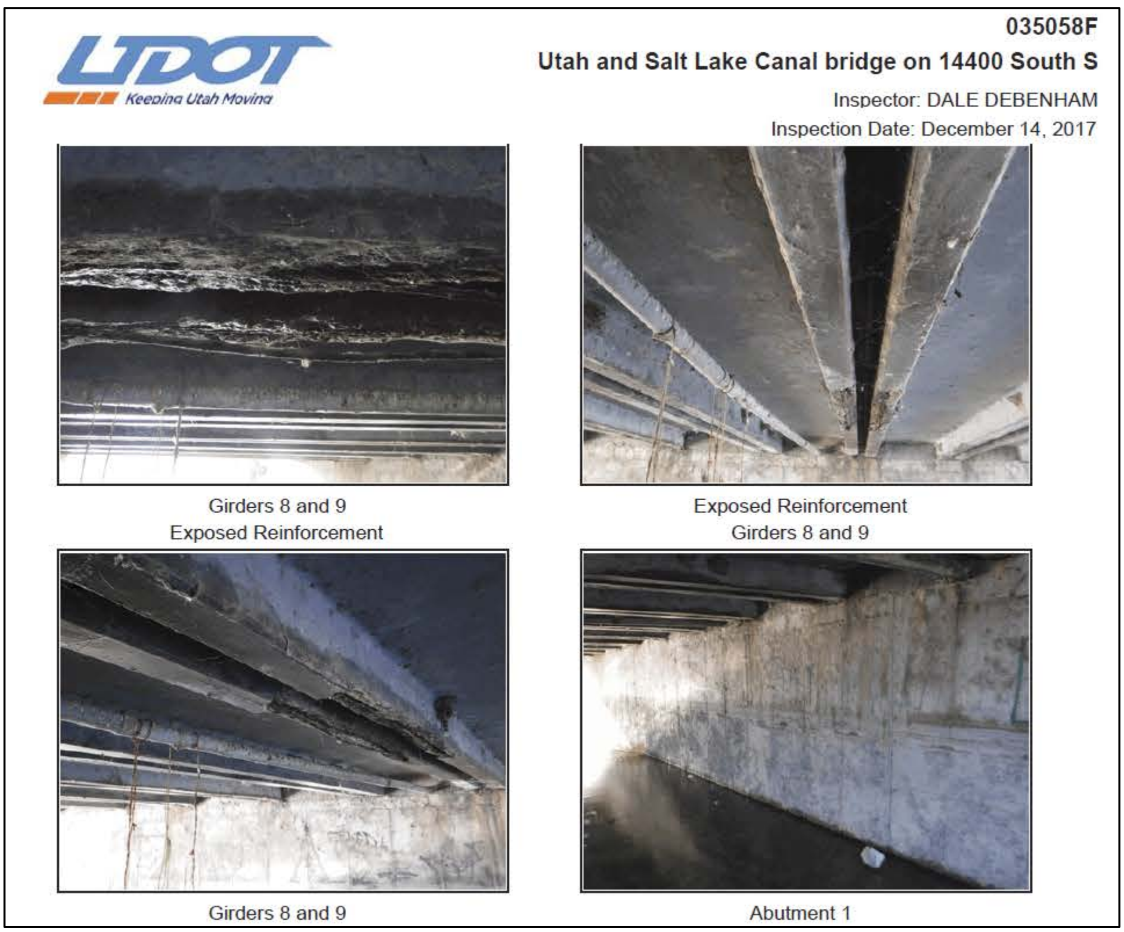


Figure 11: UDOT Report Photos, 14 December 2017

A comparison between Figure 12 and Figure 13 shows that additional corrosion has taken place during the winter. The rebar on girder 9, the beam in the foreground, has sagged significantly more. This sagging indicates active degradation of the bond between the steel and the concrete, and similar Comparing Figure 12 and Figure 14, it is clear to see that a large piece of concrete has fallen off the left side of the beam. Concrete will continue to crack off as time progresses, exposing more rebar to the atmosphere. Once the steel is exposed, the rate of corrosion is intensified.

Concrete cracking off is a major concern for the future strength of the bridge. The concrete protects the reinforcing steel form corrosion; once the concrete cracks off, there is nothing left to protect the steel. As shown in the strength analysis section, corroding steel causes a major decrease in strength.

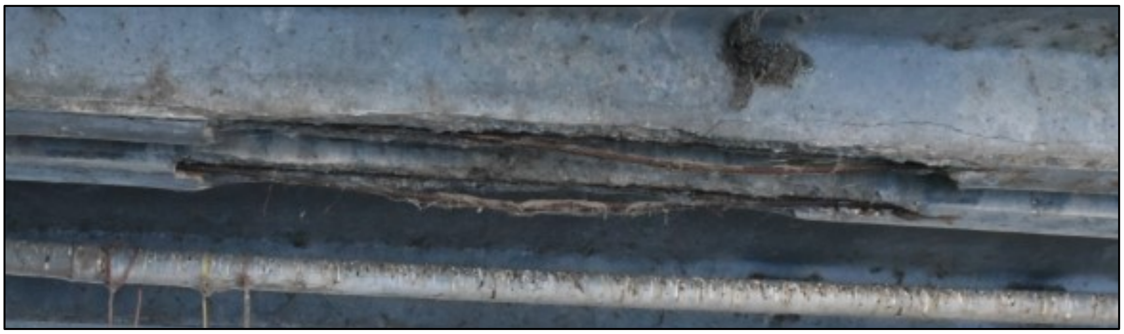


Figure 12: Corrosion on Girders 8 & 9, October 2018

Figure 13: Corrosion on Girders 8 & 9, January 2019

Figure 14: Close-Up of Corrosion, January 2019

Figure 13 also clearly shows a line of discoloration in the concrete. All the concrete near the spalling is dark in color, while the concrete above is lighter. This is likely caused by efflorescence, water seeping through the concrete and carrying away minerals. This is cause for concern because it is evidence that water is actively penetrating the concrete, especially surrounding the spalled region. With the water line coinciding well with the location of the prestressing strand, it is very likely that all the strands have an active and sufficient supply of water. The steel is at high risk of corrosion within the beam because of this penetration.

Figure 15 shows a large crack developing in the beam. This piece of concrete is likely to detach completely from the beam in a very short time and expose more reinforcing steel. This crack is approximately 18 inches long—if it were to fall, it would expose a significant amount of prestressing steel and rebar.

Figure 15: Large Developing Crack, January 2019

According to the December 2011 UDOT inspection report, these girders that now exhibit severe spalling showed only longitudinal cracking; thus, the six-feet of complete delamination and spalling of concrete occurred within a 6-year time period.

The concrete continues to spall at a very fast rate; we observed 4-6 inches of additional spalling of Girders 8 & 9 within just the time frame of our project (5 months). These girders are expected to lose a significant portion of concrete within the next few months.

Cracking Near Abutments

In addition to the significant deterioration of Girders 8 and 9, other girders show signs of cracking at the ends of the beams. The figures below show examples of this cracking, almost all of which show evidence of rust staining.

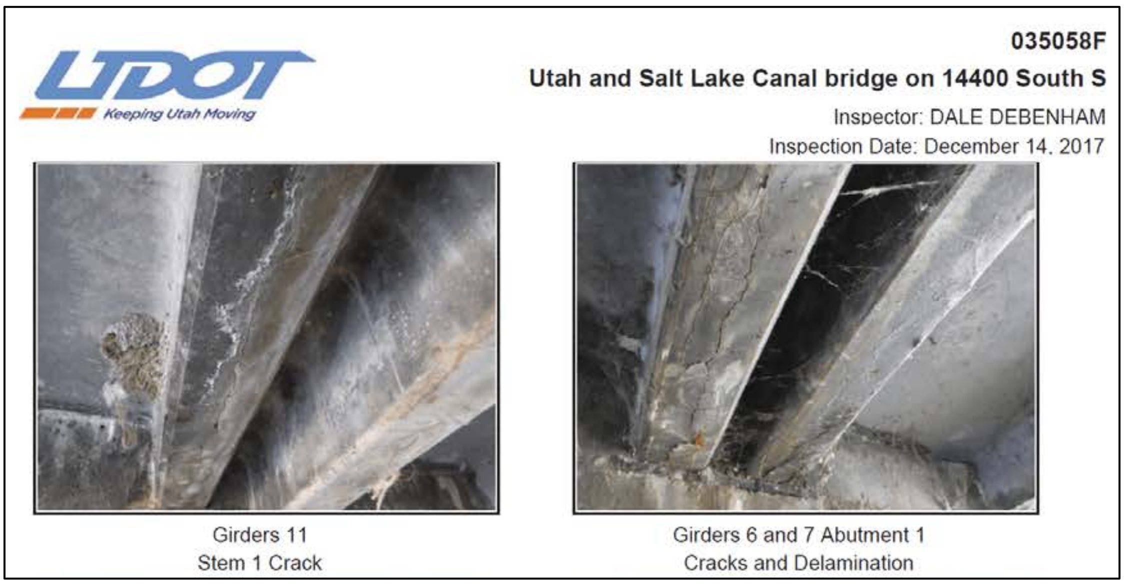


Figure 16: UDOT Inspection Report Cracking, 14 December 2017

Figure 17: Girder 6 Cracking Near Mid-Span, January 2019


Figure 18: Girders 6 & 7 Cracking and Delamination, January 2019


Figure 19: Other Cracking and Delamination, January 2019

Figure 20: Girders 10 & 11 Cracking, March 2019

The cracking in the pictures above are expected to produce significant problems in the near future. According to the UDOT inspection report, the deterioration shown in girders 8 & 9 began as middle longitudinal cracking in 2011, similar to the current cracks shown in girder 11 in Figure 16 and girder 6 in Figure 17Figure 17: Girder 6 Cracking Near Mid-Span, January 2019. Additionally, Figures 18 and 19 show the same longitudinal cracking at the ends of the girders. While the type of failure may not be exactly the same, we can assume that these cracks will develop in a similar manner and time frame as girders 8 & 9. This indicates that girders 6, 7, 10 & 11 are expected to have a similar deterioration as girders 8 & 9 within approximately 5 years.

Figure 21 shows a major concern for the future structural integrity of the bridge. The two photos shown are of opposite sides of the same beam flange. This crack on each side is approximately 3” from the base of the beam. These cracks are very concerning because of their proximity to the prestressing steel strands.

In the Comments on Analysis section, probable layouts for the prestressing strand were presented. If the strand is constructed in a vertical pattern as was assumed, this crack located 3” from the bottom of the beam approximately aligns with the location of the second and third prestressing strands from the bottom. If it is constructed as drawn in the original plans, the crack is located at the top of all 5 strands. In either case, a crack on both sides of the beam indicates that water and oxygen may be readily penetrating through the concrete to reach multiple steel members.

Figure 21: Primary Crack Concern, January 2019

The cracks seen in these photos occur at multiple points along the beam. The wide range of locations affected by such cracking necessitate the use of conservative assumptions regarding the corrosion of the prestressing strand within the concrete. It is highly unlikely that, given the frequency and size of these cracks, that the reinforcement contained in the concrete has been unaffected up to this point in time.

The structural analysis calculations assume that this reinforcing steel has begun to corrode. On each double-tee beam, only one leg exhibits such cracking and exposed reinforcement. However, in the analysis, we must assume that each leg has an equal balance of strength. With the bottom strand considered to be entirely lost, 1/5 of the steel in one of the legs, the overall strength of the beam must be assumed to be 4/5, or 80%. If additional corrosion has occurred, the percentage decreases further. Our estimate is that roughly 70-75% of the original cross-section of the strand is remaining.

Rehabilitation Options and Economic Analysis

Introduction

Four solutions are proposed in this report. The advantages, disadvantages and cost estimates of each solution are discussed below. This report does not contain a comprehensive list of all available solutions—only the most viable options are presented. These options include short-term solutions to the problem and long-term solutions. The methods used for calculating the Value of Time (VOT) costs are presented in Appendix A: Economic Analysis References

All figures presented for the cost per year of service were calculated including the cost of VOT. Because the time of bridge users is extremely valuable, it is important to consider the time of bridge closure in every option.

Some communication has taken place with consulting firms and contractors outside of Brigham Young University. Any proposals or solutions received from these companies are presented in the appendices of this report.

The cost associated with each solution is only an estimation of the total cost— actual costs will vary between now and the time an option is selected. Rough bid proposals were received from several companies for various phases of bridge replacement. These bids are not legally binding and must be re-evaluated by the providing companies. Any selected method of repair or replacement should be thoroughly inspected and approved by the city engineers.

Solution 1: Leave the bridge as-is

Due to the rapid deterioration of the concrete and strands in beams 8 and 9, the bridge is anticipated to reach its terminal service condition (TSC) in approximately one year. At that time, rehabilitation or replacement will be required. TSC is not synonymous with a catastrophic failure; rather, it is the point where engineering practice and highway bridge code necessitates the decommission of the structure. The bridge is not likely to collapse within one year, but it must be replaced or repaired when it reaches its TSC.

This option is not considered to be a viable option for the future of the bridge. It is included in this report as a temporary solution rather than a long-term solution.

Solution 2: Repair the damaged concrete and apply Tyfo

Fyfe Engineering manufactures an epoxy resin polymer called Tyfo. This product has been proven to increase the durability and strength of concrete that has suffered damage, especially on the bottom of bridge decks. This product can be applied to the surface of the concrete as a replacement for the reinforcing strength that has been lost. Engineers at Fyfe Engineering create a specific design for each structure. This is not a general-use product that is readily available for use—it is a highly specialized product meant specifically for situations like this.

Advantages

Using Tyfo is the cheapest temporary option available, and the bridge will only need to be closed for one day while it is applied. Using Tyfo will also restore some strength that has been lost due to corrosion of the reinforcing strands. It will add durability to the concrete and may extend the life of beams 8 & 9 for a few more years until additional funding for a full bridge replacement can be acquired.

Disadvantages

The application of the Tyfo epoxy resin will not prevent water from penetrating the concrete. Rather, it is a temporary solution that will simply prolong the amount of time before TSC is reached. Each beam will continue to corrode, and the rate of corrosion may even increase due to the halo effect of repaired concrete. This is a large downfall of repairing the concrete to add Tyfo.

The halo effect is the phenomena that occurs when reinforcing steel in the concrete surrounding a concrete patch experiences a rapid rate of corrosion immediately after the patch is installed. In the case of this bridge, the concrete that has already fallen from the beams will be repaired, covering the currently exposed reinforcement. This seems like a good practice at first glance, but we expect that once the patch is installed, the steel immediately surrounding the concrete patch will corrode very quickly, causing concrete on either side of the patch to crack and fall at an accelerated rate. The halo effect can be very dangerous and may harm the beams more than it helps them. If the concrete is repaired, a sacrificial zinc anode must be installed with it to prevent the bridge from additional damage. This anode must be maintained and replaced periodically throughout the remaining life of the bridge.

The exact strength gain provided by the application of Tyfo is unknown. A licensed engineer will analyze the bridge and estimate the exact strength gain and life span of the bridge after application. Our email conversations with Fyfe engineering did not allude to an accurate estimate of the exact strength gain or durability provided by such repairs. With the benefit of such a repair very much unknown, it may not be wise to pursue this route.

Cost

The cost for repairing the beams with Tyfo is estimated to be \$18,000 - \$22,000. Closing the bridge for one day will add a VOT cost of about \$7,000. Therefore, the total cost will range from \$25,000 - \$29,000.
Given an estimated five years of remaining service life after applying Tyfo, the cost per year of service is between \$5,000 and \$6,000 including the VOT cost. If money was borrowed at an interest rate of 6% over a 30-year loan term, the yearly payments would be roughly \$1,500/year for 30 years.

Solution 3: Replace beams 8 and 9

Beams 8 & 9, the two beams that have been severely damaged, may be replaced without replacing the entire bridge—this is one advantage of a double-tee bridge. The new beams would be designed to be identical to the existing beams so replacement will be as smooth as possible.

Advantages
This solution is a great temporary solution because the new beams will have the full strength the bridge was originally designed for. It would only require a portion of the roadway to be replaced rather than the entire bridge. It may also be possible to replace the beams without entirely closing the roadway, depending on the beam location.

This has a relatively low cost for a temporary solution. Replacing only the most critically-damaged beams will not negate the need for a full bridge replacement in the near future, but it will prolong the life of the bridge until additional funding can be obtained. It will also increase the life of the bridge for approximately 5 years, which will provide adequate time for a full bridge replacement to be designed and obtained.

Disadvantages
The bridge would still need to be closed for several days in order to replace the beams. The road surface would need to be removed prior to beam replacement and reconstructed once the beams are in place. Due to the unknown condition of the decks and steel connections between beams, installing the new beams may require difficult construction retrofits. It may take as little as 2-3 days to replace the beams if all goes well, but it could take 2 weeks if there are problems with the beam or connection designs.

Replacing only the two beams with the largest degree of damage is a very temporary solution. Most of the other beams that make up this structure are currently exhibiting moderate corrosion near the abutment walls. These beams are not yet near the terminal service condition like beams 8 and 9, but it is estimated that they could reach a similar point of deterioration within the next 5 years. Replacing two beams would be a temporary solution that would last until the corrosion in the other beams reaches a critical level. At the time when the full bridge is replaced, the two new beams would be scrapped with the rest of the bridge.

Cost
Forterra Engineering estimated a total cost of $78 per square foot of deck area for the new beams. The total cost of the new beams is:

$$\$78 * 240\ ft^2 \approx \$19{,}000$$

In addition, tearing out the road prior to placing the beams and replacing the asphalt after can be estimated to cost $150,000 (This is determined from W. W. Clyde's cost estimate of replacing the bridge; see Appendix B: W.W. Clyde Construction Proposal).The estimated time required to complete the project is approximately one week, making the VOT cost around $52,000. The total cost of replacing beams 8 and 9 is estimated as:

$$\$19{,}000 + \$150{,}000 + \$52{,}000 = \$221{,}000$$

With an estimated five years of life added to the bridge for replacing beams 8 and 9, the yearly cost of this option over that time is roughly $44,000. If a loan were obtained to pay for such a repair, annual payments over 30 years would be approximately $12,000.

Solution 4: Full double-tee bridge replacement

Rather than simply replace beams 8 and 9, which would not solve the underlying problem of corrosion occurring in all of the beams, it may be worthwhile to replace the bridge entirely with one of a similar design.

Advantages

A full bridge replacement will solve the problem of corrosion occurring in all the beams. Since this bridge will have the same design as the current bridge, a service life of 30 years is anticipated for the new bridge. It is possible that, through use of sacrificial anodes or other corrosion prevention measures, the new bridge design may last 50-75 years. In this case, a full replacement with the same type of bridge currently in place would be a very cost-effective option.

Disadvantages

A full replacement has a much higher up-front cost than a temporary repair solution. It is estimated to take one week to remove the current bridge, one to two weeks to place the abutments and beams, and an additional week to reconstruct the roadway. In total, this replacement may require 3-4 weeks of road closure to remove the existing structure, pour concrete abutment walls, replace the bridge superstructure, and pave the roadway above the bridge.

Cost

At \$78 per square foot of deck area, and given a total area of the bridge of 1,440 ft^2, the total cost of the new beams is:

$$\$78 * 1{,}440\ ft^2 \approx \$113{,}000$$

The cost of construction for this bridge replacement has been estimated from the construction proposal given by WW Clyde for the Contech bridge replacement. Though the two bridge designs are very different, this will provide a general idea of how expensive the bridge will be. Given a total installation cost of \$390,000 and a VOT cost of \$208,000 (based on the maximum four weeks of closure), the total cost of the bridge replacement is:

$$\$113{,}000 + \$390{,}000 + \$208{,}000 = \$711{,}000$$

This solution is estimated to provide 30-50 years of service life, similar to the life of the current bridge. The cost per year of operation for this bridge will range from \$14,000 to \$24,000. With a 6% loan over the course of 30 years, the yearly payments will be approximately \$37,000.

Alternative to Solution 4: Replace Beams 8 and 9 Now, Full Bridge Replacement in 5 Years

If sufficient funding cannot be obtained immediately, an alternative solution may be to replace beams 8 and 9 now, then replace the bridge entirely in five years. Replacing the beams costs \$221,000 (see Solution 2) and replacing the bridge in five years is estimated to cost \$785,000, assuming that inflation is 2% each year. Therefore, the total cost of this option is:

$$\$785{,}000 + \$221{,}000 \approx \$1{,}000{,}000$$

The cost per year of this option is between $18,000 and $29,000, with yearly payments of $57,000 at 6% interest.

Solution 5: Contech bridge replacement

Contech has provided a design and cost estimate for a new bridge using the ConSpan B-series design (design and estimated included in Appendix C: Contech Bridge Proposal). The design is significantly different from that of the current bridge and is suitable for use over the canal.

The ConSpan bridge utilizes arched concrete beams similar to a large concrete culvert. Soil is backfilled over the arches, and the roadway is constructed on the compacted soil. The arched concrete bridge was designed specifically for short-span applications like we have in this location.

Advantages

This option is highly recommended because it solves the underlying problem in which water is seeping through cracks in the concrete and corroding the steel reinforcement. Most of the strength in the bridge comes from the concrete rather than the steel, so any corrosion that will occur in the future is not likely to push the bridge to terminal service condition before its design life has been met. Given that no other solutions fully remove the potential for steel corrosion, even though they cost less, the future costs of other solutions will likely be greater as the beams continue to be damaged by water and salt corroding the steel.

Thermodynamic effects likely played a role in the deterioration of this bridge. Concrete expands and contracts with changes in temperature—the expansion and contraction occur in the direction of the largest dimension. Most bridges are longer than they are wide, so temperature changes cause the bridge to get longer or shorter as they expand or contract. Joints are provided at each end of the bridge to allow for this expansion; this is one large reason for the break in the driving surface between any bridge and the roadway leading up to it. The bridge being analyzed is 60 feet wide and only 24 feet long, so the expansion and contraction forces tend to make the bridge wider and narrower instead of shorter and longer. This can cause major distress to the asphalt pavement at the surface. When the beams are subjected to lateral loads, they will also experience unnatural stresses and may corrode more quickly. Thermal cracking was observed in the asphalt pavement before the roadway was rebuilt in the fall of 2018, so this is a large and active problem.

The Contech ConSpan bridge is designed for applications where the bridge is wider than it is long. Because the roadway sits on a bed of soil rather than directly on the concrete, it will not suffer the thermal cracking that has been noted previously. This will reduce the amount of water seeping through the roadway and contacting the concrete beneath.

The total cost for this bridge is slightly above that of the double-tee bridge, but because it largely eliminates the issues of thermal cracking and water contacting reinforcing steel, its design life is expected to be significantly longer than the double-tee bridge with less maintenance.

Disadvantages

This bridge replacement is more expensive than the double-tee replacement option. If adequate funding is unavailable, other options may need to be considered.

Like Solution 4, the Contech bridge replacement is estimated to require three to four weeks of bridge closure. This contributes a significant VOT cost to the construction project.

Cost

The bid proposal showing the cost estimates for the delivered materials and installation of the bridge can be found in Appendix B: W.W. Clyde Construction Proposal and Appendix C: Contech Bridge Proposal. The cost of materials is estimated to be $132,000 and the cost of installation is estimated to be $390,000. Given a maximum of four weeks of bridge closure, the VOT cost will be approximately $208,000. The total cost for the bridge replacement is estimated to be:

$$\$132{,}000 + \$390{,}000 + \$208{,}000 = \$730{,}000$$

The cost per year of this option is roughly $10,000, with yearly payments of $38,000 at 6% interest.

Summary

Table 8 provides a summary of the costs, remaining service life, and whether each solution is short-term or long-term. The cost per year of service was determined by dividing the total interest-free cost of each option by its expected service life. If money is borrowed and payments are made with an annual interest rate rather than paying for the bridge with one lump sum, the values shown in these tables will not exactly match the cost per year of service.

Table 8: Summary of Solutions

Solution	Total Cost	Cost Per Year of Service	Life Expectancy	Short-Term or Long-Term Solution
Leave As-is	None	None	1 year	Short-term
Repair Concrete & Apply Tyfo	$25,000 - $29,000	$5,000 - $6,000	5 years	Short-term
Replace Beams 8 & 9	$221,000	$44,000	5 years	Short-term
Double Tee Replacement	$711,000	$14,000 - $24,000	30-50 years	Long-term
Replace Beams 8 & 9 Now, Double Tee Bridge Replacement in 5 years	$1,000,000	$18,000 - $29,000	35-55 years	Long-term
Contech Replacement	$730,000	$10,000	75 years	Long-term

Assigning a categorical weight to each option allows us to calculate a desirability score for each solution, with the highest score signifying the best choice. Table 9 assigns scores based on service life being the most important criteria, and Table 10 assigns scores with cost being the most important The “leave as-is” solution was left out as it is not considered a viable option.

Table 9: Life-Focused Weighted Scores

Solution	Cost Per Year (1)	Life Expectancy (3)	Short-term or Long-term? (1)	Weighted Scores
Repair Concrete & Apply Tyfo	5	1	0	8
Replace Beams 8 & 9	1	1	0	4
Double-Tee Bridge Replacement	3	2	1	11
Repair Now, Replace Later	2	3	1	13
Contech Replacement	4	4	1	**18**

Table 10: Cost-Focused Weighted Scores

Solution	Cost Per Year (3)	Life Expectancy (2)	Short-term or Long-term? (1)	Weighted Scores
Repair Concrete & Apply Tyfo	5	1	0	17
Replace Beams 8 & 9	1	1	0	5
Double-Tee Bridge Replacement	3	2	1	14
Repair Now, Replace Later	2	3	1	13
Contech Replacement	4	4	1	**21**

When considering service life as the most important factor, the Contech replacement is the best option. When cost is the most important factor, both repairing the concrete with Tyfo and the Contech replacement received high scores. We recommend the Contech bridge replacement because this solution will provide a long service life and minimal maintenance costs during that time; other solutions, despite having a lower cost, may result in higher and more frequent maintenance costs, in addition to a shorter service life. If sufficient funding cannot be acquired within the year, we recommend either replacing beams 8 and 9 even though the Tyfo had a higher short-term score. Due to the complications associated with the halo effect and the uncertainty of the strength of Tyfo, we recommend beam replacement rather than repair. A bridge replacement can then be done in five years when sufficient funds can be obtained.

Funding Sources

This section provides several recommended funding sources that could be used to fund a bridge replacement or rehabilitation. This report does not provide comprehensive information about each funding source; the names, timelines, and goals/activities of each funding source are listed below in various tables. The information provided, however, should be sufficient in allowing the City to determine eligibility and enough context to begin preparing appropriate applications.
The Team faced several challenges while identifying possible funding sources. Few sources specifically address a bridge replacement, and those that do often have requirements that are not met by the Bluffdale bridge. Therefore, some of the bridge funding sources are contingent upon incorporation the bridge rehabilitation into a larger development plan.

According to the *Draft Phased 2019-2050 Regional Transportation Plan*, the area around the bridge is poised for several development projects in the near future (see "In-Progress"). This includes adding bicycle/pedestrian facilities on 14400 South in Phase 1 (2019-2030) and creating the Bluffdale Transit Connector as part of Phase 3 (2041-2050). These projects could easily incorporate a bridge rehabilitation, greatly expanding the pool of funding available. Other projects not explicitly stated in the *Regional Transportation Plan* may also provide viable methods to incorporate the cost of a bridge repair.

Local Sources

Table 11 summarizes the possible local funding sources to cover the costs of a bridge replacement or rehabilitation. All these sources can be accessed through the Wasatch Front Regional Council (WFRC); see the Appendix for links to more information found online.

Table 11: Summary of Possible Local Funding Sources

NAME	ELIGIBLE ACTIVITIES/GOALS	TIMELINE
Surface Transportation Program	**Bridge replacement**; intersection improvements; projects that reduce traffic (e.g. transit capital improvements/active transportation	Open Oct, current cycle for 2025
Transportation Alternatives Program	Construction of on/off-road trails for pedestrians, bicyclists, and other **safety-related infrastructure** that will provide safe routes for non-drivers). Includes Safe Routes to School infrastructure projects.	Open Oct, current cycle for 2025
Transportation and Land Use Connection	Help communities implement changes to reduce traffic on roads and enable more people to easily walk, bike, and use transit.	Open Oct

Source: http://wfrc.org/

Surface Transportation Program

As listed on the WFRC website, the Surface Transportation Program (STP) provides funding for "federal-aid highways and bridges, transit capital improvements and projects, and active transportation projects" (see "Surface"). This is the only project where funding for bridge replacement is explicitly listed. In addition to bridge replacement, funding can be used for the following:

- Constructing new streets or widening, improving, or reconstructing existing streets classified as Federal Aid Eligible (FAE) freeways, highways, arterials, or collectors;
- Intersection improvements;
- Projects which reduce traffic demand, such as transit capital improvements and active transportation.

As STP funding encompasses many different infrastructure projects, it may be more difficult to receive funding solely for bridge replacement or repair. While not strictly necessary, it may be beneficially to incorporation the cost of bridge replacement/repair into a larger infrastructure development.

Unfortunately, the next earliest application cycle for STP provides funding for construction in 2025. Given the current condition of the bridge and the expected point of failure, this funding source may not be able to meet the urgent funds needed for bridge rehabilitation. Application forms and instructions can be found on the STP website (see "Surface").

Transportation Alternatives Program

Transportation Alternatives Program (TAP) provides funding for construction and planning of bicycle and pedestrian facilities. Eligible projects include the construction, planning, and design of the following:

- On-road and off-road trail facilities for pedestrians, bicyclists, and other non-motorized forms of transportation (sidewalks, bicycle infrastructure, pedestrian and bicycle signals, traffic calming techniques, lighting, etc.)
- Other safety-related infrastructure that will provide safe routes for non-drivers, and in an effort to achieve compliance with the Americans with Disabilities Act of 1990
- Safe Routes to School infrastructure projects (see "Transportation Alternatives Program").

Unlike STP, the next funding cycle for TAP provides funds for 2021. Thus, it is possible for the City to receive the required funding for this project in the near future. Note that bridge replacement/repair is not explicitly listed as an eligible project. It may be possible, however, for a bridge replacement to be interpreted as necessary "safety-related infrastructure" for safe bicycle and pedestrian routes, making this funding source a viable way to receive the necessary funds. For example, a bridge replacement could be a part of project to create safer school routes for Summit Academy Elementary. Application forms and instructions can be found on the TAP website (see "Surface").

Transportation and Land Use Connection

The Transportation and Land Use Connection (TLC) program is a partnership between the Wasatch Front Regional Council (WFRC), Salt Lake County, Utah Department of Transportation (UDOT), and Utah Transit Authority (UTA), with the goal to "implement changes to the built environment that reduce traffic on roads and enable more people to easily walk, bike, and use transit" (see "Transportation and Land Use").
The conditions to obtain funding through this source is similar to TAP above; it may be possible to obtain funding only if the bridge replacement is incorporated into a larger development plan.

The application process can be found on the website (see “Transportation and Land Use”). One difficulty of TLC is the amount of funding typically awarded. According to the Salt Lake County Website, the average funding awarded is around $80,000. This is insufficient for the needs of a bridge replacement/rehabilitation and merits some consideration when determining the viability of TLC.

Federal Sources

In addition to local funding sources, several federal funding sources may be available for a bridge rehabilitation. Once again, these sources do not explicitly state bridge rehabilitation as an eligible project; it may be possible, however, to incorporate the cost of bridge revetments into a larger project. Table 12 below shows a brief overview of the different options found.
The application for these federal grants can be found on grants.gov.

Table 12: Summary of Possible Federal Funding Sources

NAME	CRITERIA	TIMELINE
Better Utilizing Investments to Leverage Development (BUILD)	Safety, State of Good Repair, Economic Competitiveness, Environmental Protection, Qualityof Life, Innovation, Partnership	Open Jun, Close Jul
Infrastructure for Rebuilding America (INFRA)	Improve Safety, Generate National/Regional Economic Benefits, Reduce Highway Congestion/Bottlenecks, Improve Connectivity	Open Jan, Closes Mar

Better Utilizing Investments to Leverage Development

The Better Utilizing Investments to Leverage Development (BUILD) Transportation program (previously known as TIGER grants) are given to projects that will have a significant local or regional impact (see “BUILD”). Eligible projects are listed below:

- Road or bridge projects eligible under title 23, United States Code;
- Public transportation projects eligible under chapter 53 of title 49, United States Code;
- Passenger and freight rail transportation projects;
- Port infrastructure investments (including inland port infrastructure and land ports of entry); and
- Intermodal projects (see “BUILD”)

According to the BUILD website, funding from previous years have been used to “repair bridges or improve infrastructure to a state of good repair.” While the scope of the bridge repair may not be the same, this appears to be a viable federal funding source. The application cycle occurs on an annual basis.

Infrastructure for Rebuilding America

The goal of Infrastructure for Rebuilding America (INFRA) grants is to provide funding for projects addressing “critical issues facing our nation’s highways and bridges” (See “Infrastructure”). Recently the focus has changed to promote innovation in the process of building such projects. These projects are eligible for INFRA grants:

- a highway freight project carried out on the National Highway Freight Network (23 U.S.C. 167)

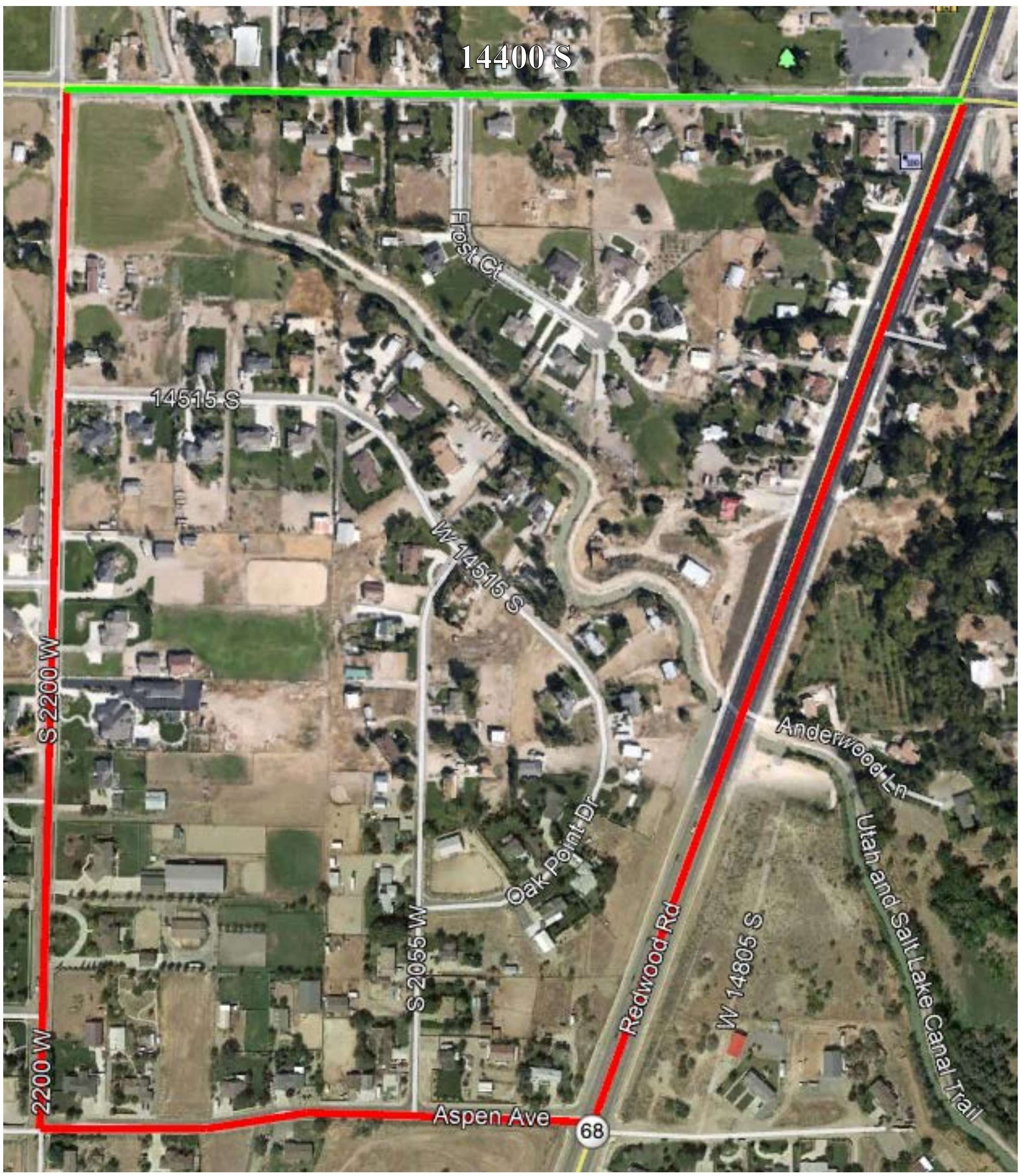
- a highway or bridge project carried out on the National Highway System (NHS) including projects that add capacity on the Interstate System to improve mobility or projects in a national scenic area
- a railway-highway grade crossing or grade separation project; or
- a freight project that is:
  1. an intermodal or rail project, or
  2. within the boundaries of a public or private freight rail, water (including ports), or intermodal facility, is a surface transportation infrastructure project necessary to facilitate direct intermodal interchange, transfer, or access into or out of the facility, and will significantly improve freight movement on the National Highway Freight Network. For these projects Federal funds can only support project elements that provide public benefits

Based on the eligibility requirements of this grant, INFRA grants may be more difficult than the other grants mentioned in receiving funding for the Bluffdale bridge. However, it may be possible to incorporate the bridge revetment into other freight and rail projects that the City may be currently pursuing.

Appendix A: Economic Analysis References

VOT Costs

Value of time (VOT) costs, or the cost incurred to drivers due to detours during construction, were estimated assuming an Individual Hourly Cost (IHC) of $18/hr. per passenger (not per vehicle) and a Truck Hourly Cost (THC) of $54/hr. per truck (This estimate comes from the Texas Transportation Institute). The average number of passengers per vehicle is about 1.6, according to the Office of Energy Efficiency & Renewable Energy; therefore, the IHC can be multiplied by 1.6 to get $29/vehicle. The expected detour for solutions requiring bridge closure is shown in the image below with a red line, while the normal route is shown with a green line.



Using Google Maps, it was determined that the detour would add about 5 extra minutes of driving time. Therefore, the total costs for each passenger vehicle and commercial truck are:

$$5\ min * \left(\frac{\$29}{60\ min}\right) = \$2.41/vehicle$$
$$5\ min * \left(\frac{\$54}{60\ min}\right) = \$4.50/truck$$

The Annual Daily Traffic (ADT) for 14400 S for the year 2010 was 2715 cars/day and is expected to be 3390 cars/day in the year 2030, with the percent of trucks at 1.0% (This was obtained from the 2017 UDOT inspection report of the bridge). Interpolating gives an ADT for 2019 of 3019 cars/day and 30 trucks/day. Therefore, the total cost per day is

$$(3019 * \$2.41) + (30 * \$4.50) = \$7411/day$$

So, to find the total VOT cost for *n* days of bridge closure,

$$VOT\ cost = \$7411/day\ * n\ days$$

Yearly Payments

For each option, yearly payments were estimated based on the estimated total cost and an estimate interest rate of 6%. The payment (PMT) function was used in Excel, and the value for number of periods (NPER) corresponds to the estimated service life associated with each solution. The estimated total cost corresponds to the present value (PV) used in the PMT function.

Appendix B: W.W. Clyde Construction Proposal

BID PROPOSAL

W. W. CLYDE & CO.
P.O. BOX 350
SPRINGVILLE, UTAH 84663
Contact: **Tyson McClellan**
Phone: **(801) 802-6800**
Fax: **(801) 802-6830**

Quote To: Shane Oh
Phone:
Fax:

Job Name: Conspan-bluffdale
Bid Date:
Addenda:

ITEM	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	AMOUNT
10	MOBILIZATION	1.00	LS	79,000.00	79,000.00
20	STRUCTE EXCAVATION/BACKFILL	1.00	LS	122,000.00	122,000.00
25	CAST IN PLACE FOOTINGS	1.00	LS	116,000.00	116,000.00
30	INSTALL BOX CULVERT/GROUT KEYWAY	1.00	LS	48,000.00	48,000.00
40	RECONSTRUCT ROADWAY OVER BOX	1.00	LS	25,000.00	25,000.00
GRAND TOTAL					**$390,000.00**

NOTES:
This quote includeds the following:

Excavate for Structure & Foundation
Support for 2 utility lines(assumed)
Construct Cast in Place Foundations
Unload and set structure
Grout unit legs and wingwalls into keyway
Supply and apply joint sealing material
Backfill the structue
Reconstruct 4" HMA roadway over culvert

Appendix C: Contech Bridge Proposal

CONTECH ENGINEERED SOLUTIONS

Contech Engineered Solutions LLC
9025 Centre Pointe Drive, Suite 400
West Chester, OH 45069
Phone: (513) 645-7000
Fax: (513) 645-7993
www.ContechES.com

Date: 11/8/2018

Project: 24FT SPAN CON/SPAN B-SERIES / BLUFFDALE, UT

The following is a CON/SPAN B-Series Bridge System ENGINEER'S COST ESTIMATE. This ESTIMATE is intended for preliminary estimating purposes only and should **not** be interpreted as a final QUOTATION. The information presented is based on the most current data made available to CONTECH.

CONTECH will fabricate and deliver the following described CON/SPAN Bridge components and appurtenances:

DESCRIPTION OF SUPPLIED MATERIALS:

- 65.75 LF of 24'-0" Span x 6'-0" Rise CON/SPAN B-Series Precast Concrete units
- Uncoated reinforcement in arch units (black steel)
- (2) – Attached precast parapet headwalls 1'-6" Tall x 10" Thick / No Impact
- (4) – Precast wingwalls with mounting hardware 12'-0" Long x 7'-10" Tall to 4'-3" Tall
- Joint sealant material
- Masonite shims
- Filter fabric and perforated drain tile
- On-site consultation during installation

ESTIMATE - $ 131,500.00 Delivered (F.O.B.)

ESTIMATED HEAVIEST CRANE PICK = 22.5 TONS

These costs do not include the foundation, or installation costs. As part of the construction process, the contractor is to perform the items listed below in accordance with the installation drawings:

- Excavate for the structure & foundations
- Construct cast-in-place foundations
- Unload and set structure utilizing crane
- Grout the unit legs and wingwalls into the keyway
- Apply all joint sealing material
- Backfill the structure

Please contact me at 801-851-0420 should you have any questions or need additional information. Thank you for your interest in the CON/SPAN Bridge System.

Respectfully,

Chad Kitchen
Bridge Consultant

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NOTES

GENERAL NOTES:

1. THIS BRIDGE HAS BEEN DESIGNED FOR GENERAL SITE CONDITIONS. THE PROJECT ENGINEER SHALL BE RESPONSIBLE FOR THE STRUCTURE'S SUITABILITY TO THE EXISTING SITE CONDITIONS AND FOR THE HYDRAULIC EVALUATION -- INCLUDING SCOUR AND CONFIRMATION OF SOIL CONDITIONS.

2. PRIOR TO CONSTRUCTION, CONTRACTOR MUST VERIFY ALL ELEVATIONS SHOWN THROUGH THE ENGINEER.

3. ONLY CONTECH ENGINEERED SOLUTIONS LLC, THE CON/SPAN® APPROVED PRECASTER IN UTAH MAY PROVIDE THE STRUCTURE DESIGNED IN ACCORDANCE WITH THESE PLANS.

4. THE USE OF ANOTHER PRECAST STRUCTURE WITH THE DESIGN ASSUMPTIONS USED FOR THE CON/SPAN® STRUCTURE MAY LEAD TO SERIOUS DESIGN ERRORS. USE OF ANY OTHER PRECAST STRUCTURE WITH THIS DESIGN AND DRAWINGS VOIDS ANY CERTIFICATION OF THIS DESIGN AND WARRANTY. REP ASSUMES NO LIABILITY FOR DESIGN OF ANY ALTERNATE OR SIMILAR TYPE STRUCTURES.

5. ALTERNATE STRUCTURES MAY BE CONSIDERED, PROVIDED THAT DRAWINGS AND CALCULATIONS SIGNED AND SEALED BY A PROFESSIONAL ENGINEER, REGISTERED IN THE STATE OF UTAH , EMPLOYED BY THE PRECAST CONCRETE BRIDGE SUPPLIER, ARE SUBMITTED TO THE ENGINEER 2 WEEKS PRIOR TO THE BID DATE FOR REVIEW AND APPROVAL.

6. ALTERNATE STRUCTURES MAY BE CONSIDERED, PROVIDED THAT THE ALTERNATE DESIGN DOES NOT REDUCE THE HYDRAULIC OPENING OF THE STRUCTURE AS SHOWN ON THE DRAWINGS. AT A MINIMUM THE ALTERNATE STRUCTURE MUST PROVIDE THE SAME OR LARGER SPAN AND RISE AS THE STRUCTURE SHOWN ON THE DRAWINGS.

7. THE PRECAST ARCH SUPPLIER MUST ATTEND THE PRE-BID MEETING, IF ONE IS HELD.

8. SUPPLIER OF PROPOSED ALTERNATES TO A CON/SPAN® BRIDGE SYSTEM MUST SUBMIT AT LEAST TWO (2) INDEPENDENTLY VERIFIED FULL SCALE LOAD TESTS THAT CONFIRM THE PROPOSED DESIGN METHODOLOGY OF THE THREE SIDED/ARCH STRUCTURE(S). THE PROPOSED ALTERNATE, UPON SATISFACTORY CONFIRMATION OF DESIGN METHODOLOGY, MAY BE CONSIDERED AN ACCEPTABLE ALTERNATE.

9. PROPOSED ALTERNATE STRUCTURES MAY BE CONSIDERED, PROVIDED THAT THE PRECAST CONCRETE BRIDGE STRUCTURES ARE PROVIDED BY A SUPPLIER THAT HAS A MINIMUM OF TWO (2) REGISTERED PROFESSIONAL ENGINEERS ON STAFF THAT ARE DEDICATED TO THE DESIGN OF THESE TYPES OF STRUCTURES. SUPPLIER MUST PROVIDE THESE NAMES, P.E. LICENSE NUMBERS AND DATES OF HIRE AT TIME OF ALTERNATE SUBMITTAL.

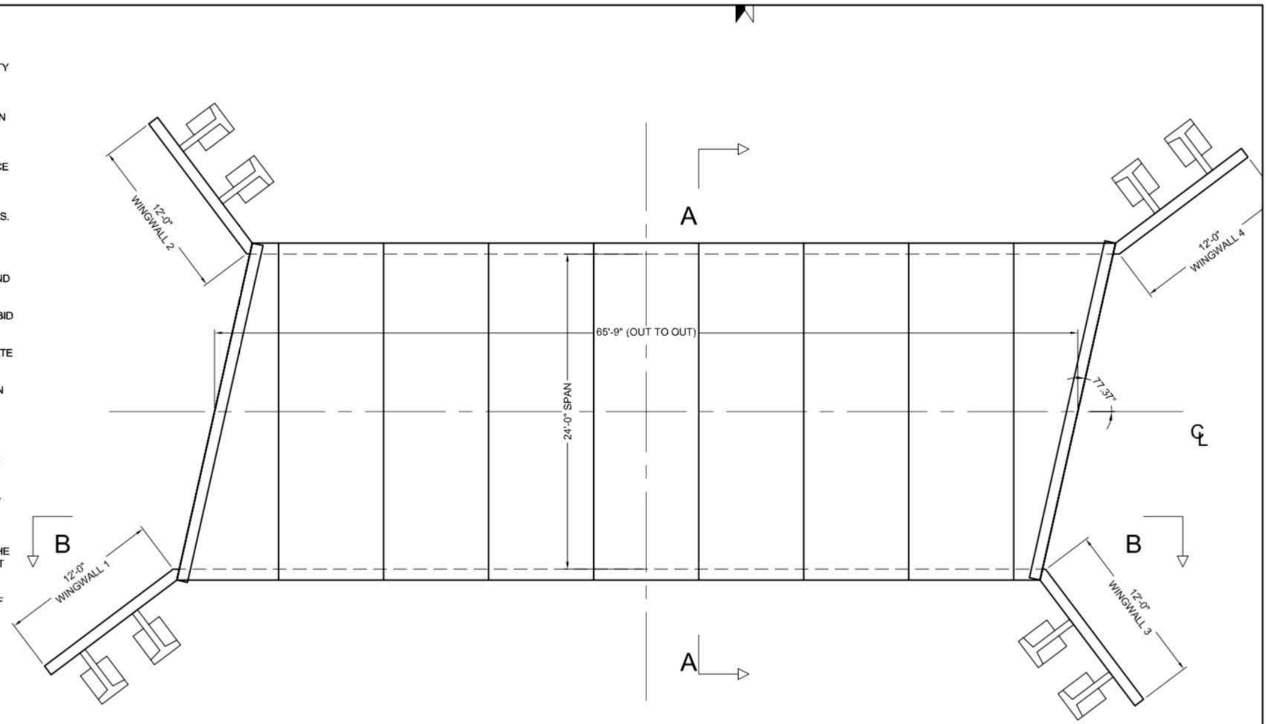
DESIGN DATA

DESIGN LOADING:
BRIDGE UNITS: HL-93
HEADWALLS: EARTH PRESSURE + LIVE LOAD SURCHARGE
WINGWALLS: EARTH PRESSURE + LIVE LOAD SURCHARGE
DESIGN FILL HEIGHT: 1'-0" TO 2'-0"
FROM TOP OF CROWN TO TOP OF PAVEMENT.
DESIGN METHOD: LOAD RESISTANCE FACTOR DESIGN PER AASHTO LRFD SPECIFICATION
ASSUMED NOMINAL BEARING RESISTANCE: 4000 PSF
ASSUMED FACTORED BEARING RESISTANCE: 4000 PSF

*AT THE TIME OF DESIGN, A GEOTECHNICAL REPORT FOR THE PROJECT SITE WAS NOT AVAILABLE. IT IS THE PROJECT ENGINEER'S, OWNER'S AND/OR THE CONTRACTOR'S RESPONSIBILITY TO VERIFY THAT THE ACTUAL SITE CONDITIONS AT THE TIME OF CONSTRUCTION ARE CONSISTENT WITH THE ASSUMED ALLOWABLE SOIL BEARING PRESSURE WITH A GEOTECHNICAL INVESTIGATION FROM A QUALIFIED GEOTECHNICAL ENGINEER.

MATERIALS

PRECAST UNITS SHALL BE CONSTRUCTED AND INSTALLED IN ACCORDANCE WITH CON/SPAN® SPECIFICATIONS. CONCRETE FOR FOOTINGS SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 4000 PSI. REINFORCING STEEL FOR FOOTINGS SHALL CONFORM TO ASTM A615 OR A996-GRADE 60.



BRIDGE PLAN

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CONTECH®
ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH®
STRUCTURAL PLATE

PROPOSAL
DRAWING

CONSPAN B SERIES 24'-0" SPAN
BLUFFDALE BRIDGE
2160 W. 14400 S.
BLUFFDALE, UT

PROJECT No.: 777777	SEQ. No.: ----	DATE: 11/6/2018
DESIGNED: ----		DRAWN:
CHECKED:		APPROVED:
SHEET NO.: 1 OF 5		

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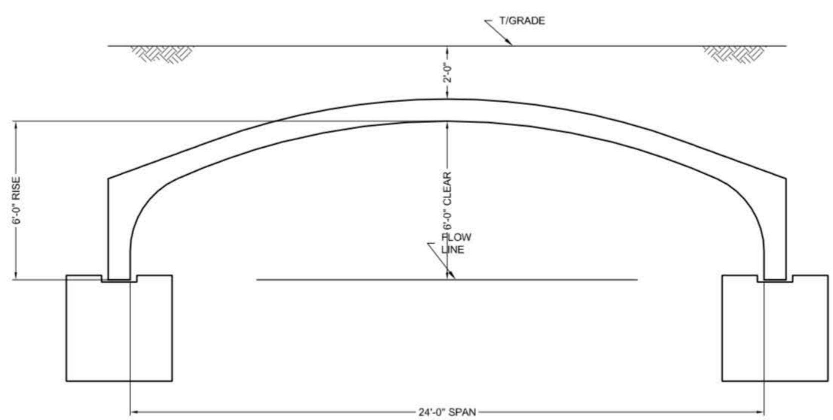
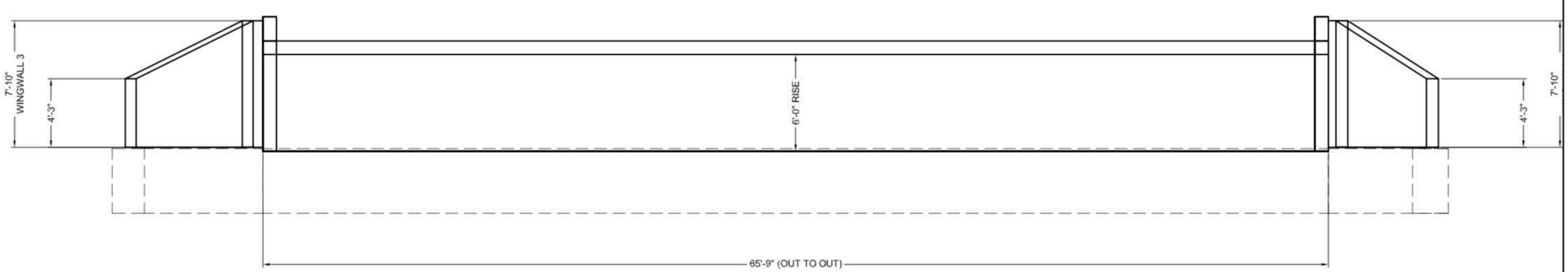


CROSS SECTION A-A

Approximate Area: 119 sq. ft. used, 119 sq. ft. total



PROFILE SECTION B-B

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800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® STRUCTURAL PLATE

PROPOSAL DRAWING

CONSPAN B SERIES 24'-0" SPAN
BLUFFDALE BRIDGE
2160 W. 14400 S.
BLUFFDALE, UT

PROJECT No.: 777777	SEQ. No.: ----	DATE: 11/6/2018
DESIGNED: ----		DRAWN:
CHECKED:		APPROVED:
SHEET NO.: 2 OF 5		

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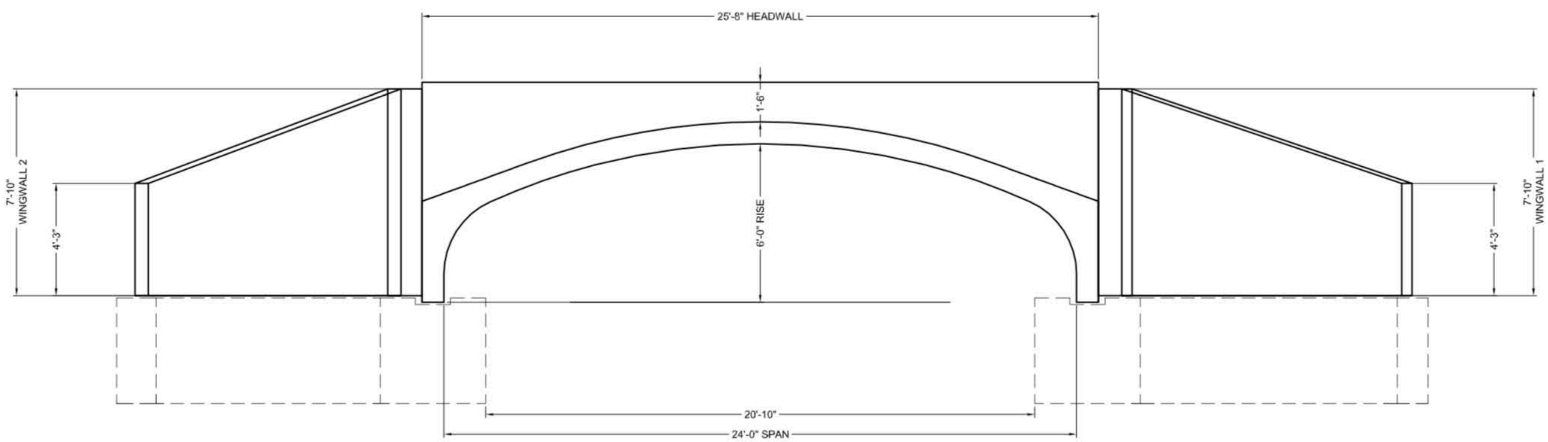

TYPICAL END ELEVATION

MARK	DATE	REVISION DESCRIPTION	BY
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CONTECH®
ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH® STRUCTURAL PLATE

PROPOSAL
DRAWING

CONSPAN B SERIES 24'-0" SPAN
BLUFFDALE BRIDGE
2160 W. 14400 S.
BLUFFDALE, UT

PROJECT No.: 777777	SEQ. No.: ----	DATE: 11/6/2018
DESIGNED: ----		DRAWN:
CHECKED:		APPROVED:
SHEET NO.: 3 OF 5		

1. DESCRIPTION
   1.1. TYPE - THIS WORK SHALL CONSIST OF FURNISHING AND CONSTRUCTING A CON/SPAN® BRIDGE SYSTEM IN ACCORDANCE WITH THESE SPECIFICATIONS AND IN REASONABLY CLOSE CONFORMITY WITH THE LINES, GRADES, DESIGN AND DIMENSIONS SHOWN ON THE PLANS OR AS ESTABLISHED BY THE ENGINEER. IN SITUATIONS WHERE TWO OR MORE SPECIFICATIONS APPLY TO THIS WORK, THE MOST STRINGENT REQUIREMENTS SHALL GOVERN.
   1.2. DESIGNATION - PRECAST REINFORCED CONCRETE CON/SPAN® BRIDGE UNITS MANUFACTURED IN ACCORDANCE WITH THIS SPECIFICATION SHALL BE DESIGNATED BY SPAN AND RISE. PRECAST REINFORCED CONCRETE WINGWALLS AND HEADWALLS MANUFACTURED IN ACCORDANCE WITH THIS SPECIFICATION SHALL BE DESIGNATED BY LENGTH, HEIGHT, AND DEFLECTION ANGLE. PRECAST REINFORCED CONCRETE EXPRESS™ FOUNDATION UNITS MANUFACTURED IN ACCORDANCE WITH THIS SPECIFICATION SHALL BE DESIGNATED BY LENGTH, HEIGHT AND WIDTH.

2. DESIGN
   2.1. SPECIFICATIONS - THE PRECAST ELEMENTS ARE DESIGNED IN ACCORDANCE WITH THE "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES" 17TH EDITION, ADOPTED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, 2002. A MINIMUM OF ONE FOOT OF COVER ABOVE THE CROWN OF THE BRIDGE UNITS IS REQUIRED IN THE INSTALLED CONDITION. (UNLESS NOTED OTHERWISE ON THE SHOP DRAWINGS AND DESIGNED ACCORDINGLY.)

3. MATERIALS
   3.1. CONCRETE - THE CONCRETE FOR THE PRECAST ELEMENTS SHALL BE AIR-ENTRAINED WHEN INSTALLED IN AREAS SUBJECT TO FREEZE-THAW CONDITIONS, COMPOSED OF PORTLAND CEMENT, FINE AND COARSE AGGREGATES, ADMIXTURES AND WATER. AIR-ENTRAINED CONCRETE SHALL CONTAIN 6 ± 2 PERCENT AIR. THE AIR- ENTRAINING ADMIXTURE SHALL CONFORM TO AASHTO M154. THE MINIMUM CONCRETE COMPRESSIVE STRENGTH SHALL BE AS SHOWN ON THE SHOP DRAWINGS.
      3.1.1. PORTLAND CEMENT - SHALL CONFORM TO THE REQUIREMENTS OF ASTM SPECIFICATIONS C150-TYPE I, TYPE II, OR TYPE III CEMENT.
      3.1.2. COARSE AGGREGATE - SHALL CONSIST OF STONE HAVING A MAXIMUM SIZE OF 1 INCH. AGGREGATE SHALL MEET REQUIREMENTS FOR ASTM C33.
      3.1.3. WATER REDUCING ADMIXTURE - THE MANUFACTURER MAY SUBMIT, FOR APPROVAL BY THE ENGINEER, A WATER-REDUCING ADMIXTURE FOR THE PURPOSE OF INCREASING WORKABILITY AND REDUCING THE WATER REQUIREMENT FOR THE CONCRETE.
      3.1.4. CALCIUM CHLORIDE - THE ADDITION TO THE MIX OF CALCIUM CHLORIDE OR ADMIXTURES CONTAINING CALCIUM CHLORIDE WILL NOT BE PERMITTED.
      3.1.5. MIXTURE - THE AGGREGATES, CEMENT AND WATER SHALL BE PROPORTIONED AND MIXED IN A BATCH MIXER TO PRODUCE A HOMOGENEOUS CONCRETE MEETING THE STRENGTH REQUIREMENTS OF THIS SPECIFICATION. THE PROPORTION OF PORTLAND CEMENT IN THE MIXTURE SHALL NOT BE LESS THAN 564 POUNDS (6 SACKS) PER CUBIC YARD OF CONCRETE.
   3.2. STEEL REINFORCEMENT
      3.2.1. THE MINIMUM STEEL YIELD STRENGTH SHALL BE 60,000 PSI, UNLESS OTHERWISE NOTED ON THE SHOP DRAWINGS.
      3.2.2. ALL REINFORCING STEEL FOR THE PRECAST ELEMENTS SHALL BE FABRICATED AND PLACED IN ACCORDANCE WITH THE DETAILED SHOP DRAWINGS SUBMITTED BY THE MANUFACTURER.
      3.2.3. REINFORCEMENT SHALL CONSIST OF WELDED WIRE FABRIC CONFORMING TO ASTM SPECIFICATION A 185 OR A 497, OR DEFORMED BILLET STEEL BARS CONFORMING TO ASTM SPECIFICATION A 615, GRADE 60. LONGITUDINAL DISTRIBUTION REINFORCEMENT MAY CONSIST OF WELDED WIRE FABRIC OR DEFORMED BILLET-STEEL BARS.
   3.3. STEEL HARDWARE
      3.3.1. BOLTS AND THREADED RODS FOR WINGWALL CONNECTIONS SHALL CONFORM TO ASTM A 307. NUTS SHALL CONFORM TO AASHTO M292 (ASTM A194) GRADE 2H. ALL BOLTS, THREADED RODS AND NUTS USED IN WINGWALL CONNECTIONS SHALL BE MECHANICALLY ZINC COATED IN ACCORDANCE WITH ASTM B695 CLASS 50.
      3.3.2. STRUCTURAL STEEL FOR WINGWALL CONNECTION PLATES AND PLATE WASHERS SHALL CONFORM TO AASHTO M 270 (ASTM A 709) GRADE 36 AND SHALL BE HOT DIP GALVANIZED AS PER AASHTO M111 (ASTM A123).
      3.3.3. INSERTS FOR WINGWALLS SHALL BE 1" DIAMETER TWO-BOLT PRESET WINGWALL ANCHORS AS MANUFACTURED BY DAYTON SUPERIOR CONCRETE ACCESSORIES, MIAMISBURG, OHIO, (800) 745-3700 AND SHALL BE MECHANICALLY ZINC COATED IN ACCORDANCE WITH ASTM B695 CLASS 50.
      3.3.4. FERRULE LOOP INSERTS SHALL BE F-64 FERRULE LOOP INSERTS AS MANUFACTURED BY DAYTON SUPERIOR CONCRETE ACCESSORIES, MIAMISBURG, OHIO, (800) 745-3700.
      3.3.5. HOOK BOLTS USED IN ATTACHED HEADWALL CONNECTIONS SHALL BE ASTM A307.
      3.3.6. INSERTS FOR DETACHED HEADWALL CONNECTIONS SHALL BE AISI TYPE 304 STAINLESS STEEL, EXPANDED COIL INSERTS AS MANUFACTURED BY DAYTON SUPERIOR CONCRETE ACCESSORIES, MIAMISBURG, OHIO, (800) 745-3700. COIL RODS AND NUTS USED IN HEADWALL CONNECTIONS SHALL BE AISI TYPE 304 STAINLESS STEEL. WASHERS USED IN HEADWALL CONNECTIONS SHALL BE EITHER AISI TYPE 304 STAINLESS STEEL PLATE WASHERS OR AASHTO M270 (ASTM A709) GRADE 36 PLATE WASHERS HOT DIP GALVANIZED AS PER AASHTO M111 (ASTM A123).
      3.3.7. MECHANICAL SPLICES OF REINFORCING BARS SHALL BE MADE USING THE DOWEL BAR SPLICER SYSTEM AS MANUFACTURED BY DAYTON SUPERIOR CONCRETE ACCESSORIES, MIAMISBURG, OHIO, (800) 745-3700, AND SHALL CONSIST OF THE DOWEL BAR SPLICER (DB-SAE) AND DOWEL-IN (DI).

4. MANUFACTURE OF PRECAST ELEMENTS - SUBJECT TO THE PROVISIONS OF SECTION 5, BELOW, THE PRECAST ELEMENT DIMENSION AND REINFORCEMENT DETAILS SHALL BE AS PRESCRIBED IN THE PLAN AND SHOP DRAWINGS PROVIDED BY THE MANUFACTURER.
   4.1. FORMS - THE FORMS USED IN MANUFACTURE SHALL BE SUFFICIENTLY RIGID AND ACCURATE TO MAINTAIN THE REQUIRED PRECAST ELEMENT DIMENSIONS WITHIN THE PERMISSIBLE VARIATIONS GIVEN IN SECTION 5 OF THESE SPECIFICATIONS. ALL CASTING SURFACES SHALL BE OF A SMOOTH MATERIAL.
   4.2. PLACEMENT OF REINFORCEMENT
      4.2.1. PLACEMENT OF REINFORCEMENT IN PRECAST BRIDGE UNITS - THE COVER OF CONCRETE OVER THE OUTSIDE CIRCUMFERENTIAL REINFORCEMENT SHALL BE 2" MINIMUM. THE COVER OF CONCRETE OVER THE INSIDE CIRCUMFERENTIAL REINFORCEMENT SHALL BE 1½" MINIMUM, UNLESS OTHERWISE NOTED ON THE SHOP DRAWINGS. THE CLEAR DISTANCE OF THE END CIRCUMFERENTIAL WIRES SHALL NOT BE LESS THAN 1" NOR MORE THAN 2" FROM THE ENDS OF EACH SECTION. REINFORCEMENT SHALL BE ASSEMBLED UTILIZING SINGLE OR MULTIPLE LAYERS OF WELDED WIRE FABRIC (NOT TO EXCEED 3 LAYERS), SUPPLEMENTED WITH A SINGLE LAYER OF DEFORMED BILLET-STEEL BARS, WHEN NECESSARY. WELDED WIRE FABRIC SHALL BE COMPOSED OF CIRCUMFERENTIAL AND LONGITUDINAL WIRES MEETING THE SPACING REQUIREMENTS OF 4.3, BELOW, AND SHALL CONTAIN SUFFICIENT LONGITUDINAL WIRES EXTENDING THROUGH THE BRIDGE UNIT TO MAINTAIN THE SHAPE AND POSITION OF THE REINFORCEMENT. LONGITUDINAL DISTRIBUTION REINFORCEMENT MAY BE WELDED WIRE FABRIC OR DEFORMED BILLET-STEEL BARS AND SHALL MEET THE SPACING REQUIREMENTS OF 4.3, BELOW. THE ENDS OF THE LONGITUDINAL DISTRIBUTION REINFORCEMENT SHALL BE NOT MORE THAN 3" AND NOT LESS THAN 1½" FROM THE ENDS OF THE BRIDGE UNIT.
      4.2.2. BENDING OF REINFORCEMENT FOR PRECAST BRIDGE UNITS - THE OUTSIDE AND INSIDE CIRCUMFERENTIAL REINFORCING STEEL FOR THE CORNERS OF THE BRIDGE SHALL BE BENT TO SUCH AN ANGLE THAT IS APPROXIMATELY EQUAL TO THE CONFIGURATION OF THE BRIDGE'S OUTSIDE CORNER.
      4.2.3. PLACEMENT OF REINFORCEMENT FOR PRECAST WINGWALLS AND HEADWALLS - THE COVER OF CONCRETE OVER THE LONGITUDINAL AND TRANSVERSE REINFORCEMENT SHALL BE 2" MINIMUM. THE CLEAR DISTANCE FROM THE END OF EACH PRECAST ELEMENT TO THE END OF REINFORCING STEEL SHALL NOT BE LESS THAN 1½" NOR MORE THAN 3". REINFORCEMENT SHALL BE ASSEMBLED UTILIZING A SINGLE LAYER OF WELDED WIRE FABRIC, OR A SINGLE LAYER OF DEFORMED BILLET-STEEL BARS. WELDED WIRE FABRIC SHALL BE COMPOSED OF TRANSVERSE AND LONGITUDINAL WIRES MEETING THE SPACING REQUIREMENTS OF 4.3, BELOW, AND SHALL CONTAIN SUFFICIENT LONGITUDINAL WIRES EXTENDING THROUGH THE ELEMENT TO MAINTAIN THE SHAPE AND POSITION OF THE REINFORCEMENT. LONGITUDINAL REINFORCEMENT MAY BE WELDED WIRE FABRIC OR DEFORMED BILLET-STEEL BARS AND SHALL MEET THE SPACING REQUIREMENTS OF 4.3, BELOW.
      4.2.4. PLACEMENT OF REINFORCMENT FOR PRECAST FOUNDATION UNITS - THE COVER OF CONCRETE OVER THE BOTTOM REINFORCEMENT SHALL BE 3 INCHES MINIMUM. THE COVER OF CONCRETE FOR ALL OTHER REINFORCEMENT SHALL BE 2 INCHES MINIMUM. THE CLEAR DISTANCE FROM THE END OF EACH PRECAST ELEMENT TO THE END OF REINFORCING STEEL SHALL NOT BE LESS THAN 2 INCHES NOR MORE THAN 3 INCHES. REINFORCEMENT SHALL BE ASSEMBLED UTILIZING A SINGLE LAYER OF WELDED WIRE FABRIC OR A SINGLE LAYER OF DEFOREMED BILLET-STEEL BARS. WELDED WIRE FABRIC SHALL BE COMPOSED OF TRANSVERSE AND LONGITUDINAL WIRES MEETING THE SPACING REQUIREMENTS OF 4.3, BELOW, AND SHALL CONTAIN SUFFICIENT LONGITUDINAL WIRES EXTENDING THROUGH THE ELEMENT TO MAINTAIN THE SHAPE AND POSITION OF THE REINFORCEMENT. LONGITUDINAL REINFORCEMENT MAY BE WELDED WIRE FABRIC OR DEFORMED BILLET-STEEL BARS AND SHALL MEET THE SPACING REQUIREMENTS OF 4.3, BELOW.
   4.3. LAPS, WELDS, SPACING
      4.3.1. LAPS, WELDS, AND SPACING FOR PRECAST BRIDGE UNITS - TENSION SPLICES IN THE CIRCUMFERENTIAL REINFORCEMENT SHALL BE MADE BY LAPPING. LAPS MAY BE TACK WELDED TOGETHER FOR ASSEMBLY PURPOSES. FOR SMOOTH WELDED WIRE FABRIC, THE OVERLAP SHALL MEET THE REQUIREMENTS OF AASHTO 5.11.2.5.2 AND 5.11.6.2. FOR DEFORMED WELDED WIRE FABRIC, THE OVERLAP SHALL MEET THE REQUIREMENTS OF AASHTO 5.11.2.5.1 AND 5.11.6.1. THE OVERLAP OF WELDED WIRE FABRIC SHALL BE MEASURED BETWEEN THE OUTER-MOST LONGITUDINAL WIRES OF EACH FABRIC SHEET. FOR DEFORMED BILLET-STEEL BARS, THE OVERLAP SHALL MEET THE REQUIREMENTS OF AASHTO 5.11.2.1 FOR SPLICES OTHER THAN TENSION SPLICES, THE OVERLAP SHALL BE A MINIMUM OF 1'-0" FOR WELDED WIRE FABRIC OR DEFORMED BILLET-STEEL BARS. THE SPACING CENTER TO CENTER OF THE CIRCUMFERENTIAL WIRES IN A WIRE FABRIC SHEET SHALL BE NOT LESS THAN 2" NOR MORE THAN 4". THE SPACING CENTER TO CENTER OF THE LONGITUDINAL WIRES SHALL NOT BE MORE THAN 8". THE SPACING CENTER TO CENTER OF THE LONGITUDINAL DISTRIBUTION STEEL FOR EITHER LINE OF REINFORCING IN THE TOP SLAB SHALL BE NOT MORE THAN 1'-4".
      4.3.2. LAPS, WELDS, AND SPACING FOR PRECAST WINGWALLS, HEADWALLS AND FOUNDATIONS - SPLICES IN THE REINFORCEMENT SHALL BE MADE BY LAPPING. LAPS MAY BE TACK WELDED TOGETHER FOR ASSEMBLY PURPOSES. FOR SMOOTH WELDED WIRE FABRIC, THE OVERLAP SHALL MEET THE REQUIREMENTS OF AASHTO 5.11.2.5.2 AND 5.11.6.2. FOR DEFORMED WELDED WIRE FABRIC, THE OVERLAP SHALL MEET THE REQUIREMENTS OF AASHTO 5.11.2.5.1 AND 5.11.6.1. FOR DEFORMED BILLET-STEEL BARS, THE OVERLAP SHALL MEET THE REQUIREMENTS OF AASHTO 5.11.2.1. THE SPACING CENTER-TO-CENTER OF THE WIRES IN A WIRE FABRIC SHEET SHALL BE NOT LESS THAN 2" NOR MORE THAN 8".
   4.4. CURING - THE PRECAST CONCRETE ELEMENTS SHALL BE CURED FOR A SUFFICIENT LENGTH OF TIME SO THAT THE CONCRETE WILL DEVELOP THE SPECIFIED COMPRESSIVE STRENGTH IN 28 DAYS OR LESS. ANY ONE OF THE FOLLOWING METHODS OF CURING OR COMBINATIONS THERE OF SHALL BE USED:
      4.4.1. STEAM CURING - THE PRECAST ELEMENTS MAY BE LOW-PRESSURE STEAM CURED BY A SYSTEM THAT WILL MAINTAIN A MOIST ATMOSPHERE.
      4.4.2. WATER CURING - THE PRECAST ELEMENTS MAY BE WATER CURED BY ANY METHOD THAT WILL KEEP THE SECTIONS MOIST.
      4.4.3. MEMBRANE CURING - A SEALING MEMBRANE CONFORMING TO THE REQUIREMENTS OF ASTM SPECIFICATION C309 MAY BE APPLIED AND SHALL BE LEFT INTACT UNTIL THE REQUIRED CONCRETE COMPRESSIVE STRENGTH IS ATTAINED. THE CONCRETE TEMPERATURE AT THE TIME OF APPLICATION SHALL BE WITHIN +/- 10 DEGREES F OF THE ATMOSPHERIC TEMPERATURE. ALL SURFACES SHALL BE KEPT MOIST PRIOR TO THE APPLICATION OF THE COMPOUNDS AND SHALL BE DAMP WHEN THE COMPOUND IS APPLIED.
   4.5. STORAGE, HANDLING & DELIVERY
      4.5.1. STORAGE - PRECAST CONCRETE BRIDGE ELEMENTS SHALL BE LIFTED AND STORED IN "AS-CAST" POSITION. PRECAST CONCRETE HEADWALL AND WINGWALL UNITS ARE CAST, STORED AND SHIPPED IN A FLAT POSITION. THE PRECAST ELEMENTS SHALL BE STORED IN SUCH A MANNER TO PREVENT CRACKING OR DAMAGE. STORE ELEMENTS USING TIMBER SUPPORTS AS APPROPRIATE. THE UNITS SHALL NOT BE MOVED UNTIL THE CONCRETE COMPRESSIVE STRENGTH HAS REACHED A MINIMUM OF 2500 PSI, AND THEY SHALL NOT BE STORED IN AN UPRIGHT POSITION.
      4.5.2. HANDLING - HANDLING DEVICES SHALL BE PERMITTED IN EACH PRECAST ELEMENT FOR THE PURPOSE OF HANDLING AND SETTING. SPREADER BEAMS MAY BE REQUIRED FOR THE LIFTING OF PRECAST CONCRETE BRIDGE ELEMENTS TO PRECLUDE DAMAGE FROM BENDING OR TORSION FORCES.
      4.5.3. DELIVERY - PRECAST CONCRETE ELEMENTS MUST NOT BE SHIPPED UNTIL THE CONCRETE HAS ATTAINED THE SPECIFIED DESIGN COMPRESSIVE STRENGTH, OR AS DIRECTED BY THE DESIGN ENGINEER. PRECAST CONCRETE ELEMENTS MAY BE UNLOADED AND PLACED ON THE GROUND AT THE SITE UNTIL INSTALLED. STORE ELEMENTS USING TIMBER SUPPORTS AS APPROPRIATE.
   4.6. QUALITY ASSURANCE - THE PRECASTER SHALL DEMONSTRATE ADHERENCE TO THE STANDARDS SET FORTH IN THE NPCA QUALITY CONTROL MANUAL. THE PRECASTER SHALL MEET EITHER SECTION 4.6.1 OR 4.6.2
      4.6.1. CERTIFICATION - THE PRECASTER SHALL BE CERTIFIED BY THE PRECAST/PRESTRESSED CONCRETE INSTITUTE PLANT CERTIFICATION PROGRAM OR THE NATIONAL PRECAST CONCRETE ASSOCIATION'S PLANT CERTIFICATION PROGRAM PRIOR TO AND DURING PRODUCTION OF THE PRODUCTS COVERED BY THIS SPECIFICATION.
      4.6.2. QUALIFICATIONS, TESTING AND INSPECTION
         4.6.2.1. THE PRECASTER SHALL HAVE BEEN IN THE BUSINESS OF PRODUCING PRECAST CONCRETE PRODUCTS SIMILAR TO THOSE SPECIFIED FOR A MINIMUM OF THREE YEARS. HE SHALL MAINTAIN A PERMANENT QUALITY CONTROL DEPARTMENT OR RETAIN AN INDEPENDENT TESTING AGENCY ON A CONTINUING BASIS. THE AGENCY SHALL ISSUE A REPORT, CERTIFIED BY A LICENSED ENGINEER, DETAILING THE ABILITY OF THE PRECASTER TO PRODUCE QUALITY PRODUCTS CONSISTENT WITH INDUSTRY STANDARDS.
         4.6.2.2. THE PRECASTER SHALL SHOW THAT THE FOLLOWING TESTS ARE PERFORMED IN ACCORDANCE WITH THE ASTM STANDARDS INDICATED. TESTS SHALL BE PERFORMED AS INDICATED IN SECTION 6 OF THESE SPECIFICATIONS.
            4.6.2.2.1. AIR CONTENT: C231 OR C173
            4.6.2.2.2. COMPRESSIVE STRENGTH: C31,C39,C497
         4.6.2.3. THE PRECASTER SHALL PROVIDE DOCUMENTATION DEMONSTRATING COMPLIANCE WITH THIS SECTION TO CONTECH® ENGINEERED SOLUTIONS AT REGULAR INTERVALS OR UPON REQUEST.
         4.6.2.4. THE OWNER MAY PLACE AN INSPECTOR IN THE PLANT WHEN THE PRODUCTS COVERED BY THIS SPECIFICATION ARE BEING MANUFACTURED.
      4.6.3. DOCUMENTATION - THE PRECASTER SHALL SUBMIT PRECAST PRODUCTION REPORTS TO CONTECH® ENGINEERED SOLUTIONS AS REQUIRED.

5. PERMISSIBLE VARIATIONS
   5.1. BRIDGE UNITS
      5.1.1. INTERNAL DIMENSIONS - THE INTERNAL DIMENSION SHALL VARY NOT MORE THAN 1% FROM THE DESIGN DIMENSIONS NOR MORE THAN 1½" WHICHEVER IS LESS.
      5.1.2. SLAB AND WALL THICKNESS - THE SLAB AND WALL THICKNESS SHALL NOT BE LESS THAN THAT SHOWN IN THE DESIGN BY MORE THAN ¼". A THICKNESS MORE THAN THAT REQUIRED IN THE DESIGN SHALL NOT BE CAUSE FOR REJECTION.
      5.1.3. LENGTH OF OPPOSITE SURFACES - VARIATIONS IN LAYING LENGTHS OF TWO OPPOSITE SURFACES OF THE BRIDGE UNIT SHALL NOT BE MORE THAN ⅝" IN ANY SECTION, EXCEPT WHERE BEVELED ENDS FOR LAYING OF CURVES ARE SPECIFIED BY THE PURCHASER.
      5.1.4. LENGTH OF SECTION - THE UNDERRUN IN LENGTH OF A SECTION SHALL NOT BE MORE THAN ½" IN ANY BRIDGE UNIT.
      5.1.5. POSITION OF REINFORCEMENT - THE MAXIMUM VARIATION IN POSITION OF THE REINFORCEMENT SHALL BE ±⅜". IN NO CASE SHALL THE COVER OVER THE REINFORCEMENT BE LESS THAN 1½" FOR THE OUTSIDE CIRCUMFERENTIAL STEEL OR BE LESS THAN 1" FOR THE INSIDE CIRCUMFERENTIAL STEEL AS MEASURED TO THE EXTERNAL OR INTERNAL SURFACE OF THE BRIDGE. THESE TOLERANCES OR COVER REQUIREMENTS DO NOT APPLY TO MATING SURFACES OF THE JOINTS.
      5.1.6. AREA OF REINFORCEMENT - THE AREAS OF STEEL REINFORCEMENT SHALL BE THE DESIGN STEEL AREAS AS SHOWN IN THE MANUFACTURER'S SHOP DRAWINGS. STEEL AREAS GREATER THAN THOSE REQUIRED SHALL NOT BE CAUSE FOR REJECTION. THE PERMISSIBLE VARIATION IN DIAMETER OF ANY REINFORCEMENT SHALL CONFORM TO THE TOLERANCES PRESCRIBED IN THE ASTM SPECIFICATION FOR THAT TYPE OF REINFORCEMENT.
   5.2. WINGWALLS & HEADWALLS
      5.2.1. WALL THICKNESS - THE WALL THICKNESS SHALL NOT VARY FROM THAT SHOWN IN THE DESIGN BY MORE THAN ½".
      5.2.2. LENGTH/HEIGHT OF WALL SECTIONS - THE LENGTH AND HEIGHT OF THE WALL SHALL NOT VARY FROM THAT SHOWN IN THE DESIGN BY MORE THAN ½".
      5.2.3. POSITION OF REINFORCEMENT - THE MAXIMUM VARIATION IN THE POSITION OF THE REINFORCEMENT SHALL BE ±½". IN NO CASE SHALL THE COVER OVER THE REINFORCEMENT BE LESS THAN 1½".
      5.2.4. SIZE OF REINFORCEMENT - THE PERMISSIBLE VARIATION IN DIAMETER OF ANY REINFORCING SHALL CONFORM TO THE TOLERANCES PRESCRIBED IN THE ASTM SPECIFICATION FOR THAT TYPE OF REINFORCING. STEEL AREA GREATER THAN THAT REQUIRED SHALL NOT BE CAUSE FOR REJECTION.
   5.3. FOUNDATION UNITS
      5.3.1. WALL THICKNESS - THE WALL THICKNESS SHALL NOT VARY FROM THAT SHOWN IN THE DESIGN BY MORE THAN ½".
      5.3.2. LENGTH/HEIGHT/WIDTH OF FOUNDATION SECTIONS - THE LENGTH, HEIGHT AND WIDTH OF THE FOUNDATION UNITS SHALL NOT VARY FROM THAT SHOWN IN THE DESIGN BY MORE THAN ½".
      5.3.3. POSITION OF REINFORCEMENT - THE MAXIMUM VARIATION IN THE POSITION OF THE REINFORCEMENT SHALL BE ±½". IN NO CASE SHALL THE COVER OVER THE REINFORCEMENT BE LESS THAN 1½".
      5.3.4. SIZE OF REINFORCEMENT - THE PERMISSIBLE VARIATION IN DIAMETER OF ANY REINFORCING SHALL CONFORM TO THE TOLERANCES PRESCRIBED IN THE ASTM SPECIFICATION FOR THAT TYPE OF REINFORCING. STEEL AREA GREATER THAN THAT REQUIRED SHALL NOT BE CAUSE FOR REJECTION.

6. TESTING/ INSPECTION
   6.1. TESTING
      6.1.1. TYPE OF TEST SPECIMEN - CONCRETE COMPRESSIVE STRENGTH SHALL BE DETERMINED FROM COMPRESSION TESTS MADE ON CYLINDERS OR CORES. FOR CYLINDER TESTING, A MINIMUM OF 4 CYLINDERS SHALL BE TAKEN FOR EACH BRIDGE ELEMENT. EACH ELEMENT SHALL BE CONSIDERED SEPARATELY FOR THE PURPOSE OF TESTING AND ACCEPTANCE.
      6.1.2. COMPRESSION TESTING - CYLINDERS SHALL BE MADE AND TESTED AS PRESCRIBED BY THE ASTM C39 SPECIFICATION. CYLINDERS SHALL BE CURED IN THE SAME ENVIRONMENT AS THE BRIDGE ELEMENTS. CORES SHALL BE OBTAINED AND TESTED FOR COMPRESSIVE STRENGTH IN ACCORDANCE WITH THE PROVISIONS OF THE ASTM C42 SPECIFICATION.
      6.1.3. ACCEPTABILITY OF CYLINDER TESTS - WHEN THE AVERAGE COMPRESSIVE STRENGTH OF ALL CYLINDERS TESTED IS EQUAL TO OR GREATER THAN THE DESIGN COMPRESSIVE STRENGTH, AND NOT MORE THAN 10% OF THE CYLINDERS TESTED HAVE A COMPRESSIVE STRENGTH LESS THAN THE DESIGN CONCRETE STRENGTH, AND NO CYLINDER TESTED HAS A COMPRESSIVE STRENGTH LESS THAN 80% OF THE DESIGN COMPRESSIVE STRENGTH, THEN THE ELEMENT SHALL BE ACCEPTED. WHEN THE COMPRESSIVE STRENGTH OF THE CYLINDERS TESTED DOES NOT CONFORM TO THESE ACCEPTANCE CRITERIA, THE ACCEPTABILITY OF THE ELEMENT MAY BE DETERMINED AS DESCRIBED IN SECTION 6.1.4, BELOW.
      6.1.4. ACCEPTABILITY OF CORE TESTS - THE COMPRESSIVE STRENGTH OF THE CONCRETE IN A BRIDGE ELEMENT IS ACCEPTABLE WHEN THE AVERAGE CORE TEST STRENGTH IS EQUAL TO OR GREATER THAN THE DESIGN CONCRETE STRENGTH. WHEN THE COMPRESSIVE STRENGTH OF A CORE TESTED IS LESS THAN THE DESIGN CONCRETE STRENGTH, THE PRECAST ELEMENT FROM WHICH THAT CORE WAS TAKEN MAY BE RE-CORED. WHEN THE COMPRESSIVE STRENGTH OF THE RE-CORE IS EQUAL TO OR GREATER THAN THE DESIGN CONCRETE STRENGTH, THE COMPRESSIVE STRENGTH OF THE CONCRETE IN THAT BRIDGE ELEMENT IS ACCEPTABLE.
         6.1.4.1. WHEN THE COMPRESSIVE STRENGTH OF ANY RECORE IS LESS THAN THE DESIGN CONCRETE STRENGTH, THE PRECAST ELEMENT FROM WHICH THAT CORE WAS TAKEN SHALL BE REJECTED.
         6.1.4.2. PLUGGING CORE HOLES - THE CORE HOLES SHALL BE PLUGGED AND SEALED BY THE MANUFACTURER IN A MANNER SUCH THAT THE ELEMENTS WILL MEET ALL OF THE TEST REQUIREMENTS OF THIS SPECIFICATION. PRECAST ELEMENTS SO SEALED SHALL BE CONSIDERED SATISFACTORY FOR USE.
         6.1.4.3. TEST EQUIPMENT - EVERY MANUFACTURER FURNISHING PRECAST ELEMENTS UNDER THIS SPECIFICATION SHALL FURNISH ALL FACILITIES AND PERSONNEL NECESSARY TO CARRY OUT THE TEST REQUIRED.
   6.2. INSPECTION - THE QUALITY OF MATERIALS, THE PROCESS OF MANUFACTURE, AND THE FINISHED PRECAST ELEMENTS SHALL BE SUBJECT TO INSPECTION BY THE PURCHASER.

7. JOINTS
   THE BRIDGE UNITS SHALL BE PRODUCED WITH FLAT BUTT ENDS. THE ENDS OF THE BRIDGE UNITS SHALL BE SUCH THAT WHEN THE SECTIONS ARE LAID TOGETHER THEY WILL MAKE A CONTINUOUS LINE WITH A SMOOTH INTERIOR FREE OF APPRECIABLE IRREGULARITIES, ALL COMPATIBLE WITH THE PERMISSIBLE VARIATIONS IN SECTION 5, ABOVE. THE JOINT WIDTH BETWEEN ADJACENT PRECAST UNITS SHALL NOT EXCEED ¾".

8. WORKMANSHIP/ FINISH
   THE BRIDGE UNITS, WINGWALLS, HEADWALLS AND FOUNDATION UNITS SHALL BE SUBSTANTIALLY FREE OF FRACTURES. THE ENDS OF THE BRIDGE UNITS SHALL BE NORMAL TO THE WALLS AND CENTERLINE OF THE BRIDGE SECTION, WITHIN THE LIMITS OF THE VARIATIONS GIVEN IN SECTION 5, ABOVE, EXCEPT WHERE BEVELED ENDS ARE SPECIFIED. THE FACES OF THE WINGWALLS AND HEADWALLS SHALL BE PARALLEL TO EACH OTHER, WITHIN THE LIMITS OF VARIATIONS GIVEN IN SECTION 5, ABOVE. THE SURFACE OF THE PRECAST ELEMENTS SHALL BE A SMOOTH STEEL FORM OR TROWELED SURFACE. TRAPPED AIR POCKETS CAUSING SURFACE DEFECTS SHALL BE CONSIDERED AS PART OF A SMOOTH, STEEL FORM FINISH.

9. REPAIRS
   PRECAST ELEMENTS MAY BE REPAIRED, IF NECESSARY, BECAUSE OF IMPERFECTIONS IN MANUFACTURE OR HANDLING DAMAGE AND WILL BE ACCEPTABLE IF, IN THE OPINION OF THE PURCHASER, THE REPAIRS ARE SOUND, PROPERLY FINISHED AND CURED, AND THE REPAIRED SECTION CONFORMS TO THE REQUIREMENTS OF THIS SPECIFICATION.

10. REJECTION
   THE PRECAST ELEMENTS SHALL BE SUBJECT TO REJECTION ON ACCOUNT OF ANY OF THE SPECIFICATION REQUIREMENTS. INDIVIDUAL PRECAST ELEMENTS MAY BE REJECTED BECAUSE OF ANY OF THE FOLLOWING:
   10.1. FRACTURES OR CRACKS PASSING THROUGH THE WALL, EXCEPT FOR A SINGLE END CRACK THAT DOES NOT EXCEED ONE HALF THE THICKNESS OF THE WALL.
   10.2. DEFECTS THAT INDICATE PROPORTIONING, MIXING, AND MOLDING NOT IN COMPLIANCE WITH SECTION 4 OF THESE SPECIFICATIONS.
   10.3. HONEYCOMBED OR OPEN TEXTURE.
   10.4. DAMAGED ENDS, WHERE SUCH DAMAGE WOULD PREVENT MAKING A SATISFACTORY JOINT.

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CONTECH STRUCTURAL PLATE

PROPOSAL DRAWING

CONSPAN B SERIES 24'-0" SPAN
BLUFFDALE BRIDGE
2160 W. 14400 S.
BLUFFDALE, UT

PROJECT No.: 777777	SEQ. No.: ----	DATE: 11/6/2018
DESIGNED: ----	DRAWN:	
CHECKED:	APPROVED:	
SHEET NO.: 4 OF 5		

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11. MARKING
EACH BRIDGE UNIT SHALL BE CLEARLY MARKED BY WATERPROOF PAINT. THE FOLLOWING SHALL BE SHOWN ON THE INSIDE OF THE VERTICAL LEG OF THE BRIDGE SECTION:
BRIDGE SPAN x BRIDGE RISE
DATE OF MANUFACTURE
NAME OR TRADEMARK OF THE MANUFACTURER

12. INSTALLATION PREPARATION
TO ENSURE CORRECT INSTALLATION OF THE PRECAST CONCRETE BRIDGE SYSTEM, CARE AND CAUTION MUST BE EXERCISED IN FORMING THE SUPPORT AREAS FOR BRIDGE UNITS, HEADWALL, AND WINGWALL ELEMENTS. EXERCISING SPECIAL CARE WILL FACILITATE THE RAPID INSTALLATION OF THE PRECAST COMPONENTS.

12.1. FOOTINGS
DO NOT OVER EXCAVATE FOUNDATIONS UNLESS DIRECTED BY SITE SOIL ENGINEER TO REMOVE UNSUITABLE SOIL.

THE SITE SOILS ENGINEER SHALL CERTIFY THAT THE BEARING CAPACITY MEETS OR EXCEEDS THE FOOTING DESIGN REQUIREMENTS, PRIOR TO THE CONTRACTOR POURING OF THE FOOTINGS.

THE BRIDGE UNITS AND WINGWALLS SHALL BE INSTALLED ON EITHER PRECAST OR CAST-IN-PLACE CONCRETE FOOTINGS. THE SIZE AND ELEVATION OF THE FOOTINGS SHALL BE AS DESIGNED BY THE ENGINEER. A KEYWAY SHALL BE FORMED IN THE TOP SURFACE OF THE BRIDGE FOOTING AS SPECIFIED ON THE PLANS. NO KEYWAY IS REQUIRED IN THE WINGWALL FOOTINGS, UNLESS OTHERWISE SPECIFIED ON THE PLANS.

THE FOOTINGS SHALL BE GIVEN A SMOOTH FLOAT FINISH AND SHALL REACH A COMPRESSIVE STRENGTH OF 2,000 PSI BEFORE PLACEMENT OF THE BRIDGE AND WINGWALL ELEMENTS. BACKFILLING SHALL NOT BEGIN UNTIL THE FOOTING HAS REACHED THE FULL DESIGN COMPRESSIVE STRENGTH.

THE FOOTING SURFACE SHALL BE CONSTRUCTED IN ACCORDANCE WITH GRADES SHOWN ON THE PLANS. WHEN TESTED WITH A 10'-0" STRAIGHT EDGE, THE SURFACE SHALL NOT VARY MORE THAN 1/4" IN 10'-0".

IF A PRECAST CONCRETE FOOTING IS USED, THE CONTRACTOR SHALL PREPARE A 4" THICK BASE LAYER OF COMPACTED GRANULAR MATERIAL THE FULL WIDTH OF THE FOOTING PRIOR TO PLACING THE PRECAST FOOTING.

THE FOUNDATIONS FOR PRECAST CONCRETE BRIDGE ELEMENTS AND WINGWALLS MUST BE CONNECTED BY REINFORCEMENT TO FORM ONE MONOLITHIC BODY. EXPANSION JOINTS SHALL NOT BE USED.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE CONSTRUCTION OF THE FOUNDATIONS PER THE PLANS AND SPECIFICATIONS.

13. INSTALLATION
13.1. GENERAL - THE INSTALLATION OF THE PRECAST CONCRETE ELEMENTS SHALL BE AS EXPLAINED IN THE PUBLICATION CON/SPAN BRIDGE SYSTEMS INSTALLATION HANDBOOK.
13.1.1. LIFTING - IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO ENSURE THAT A CRANE OF THE CORRECT LIFTING CAPACITY IS AVAILABLE TO HANDLE THE PRECAST CONCRETE UNITS. THIS CAN BE ACCOMPLISHED BY USING THE WEIGHTS GIVEN FOR THE PRECAST CONCRETE COMPONENTS AND BY DETERMINING THE LIFTING REACH FOR EACH CRANE UNIT. SITE CONDITIONS MUST BE CHECKED WELL IN ADVANCE OF SHIPPING TO ENSURE PROPER CRANE LOCATION AND TO AVOID ANY LIFTING RESTRICTIONS. THE LIFT ANCHORS OR HOLES PROVIDED IN EACH UNIT ARE THE ONLY MEANS TO BE USED TO LIFT THE ELEMENTS. THE PRECAST CONCRETE ELEMENTS MUST NOT BE SUPPORTED OR RAISED BY OTHER MEANS THAN THOSE GIVEN IN THE MANUALS AND DRAWINGS WITHOUT WRITTEN APPROVAL FROM CONTECH® ENGINEERED SOLUTIONS.
13.1.2. CONSTRUCTION EQUIPMENT WEIGHT RESTRICTIONS - IN NO CASE SHALL EQUIPMENT OPERATING IN EXCESS OF THE DESIGN LOAD (HS20 OR HS25) BE PERMITTED OVER THE BRIDGE UNITS UNLESS APPROVED BY CONTECH® ENGINEERED SOLUTIONS.
13.1.2.1. IN THE IMMEDIATE AREA OF THE BRIDGE UNITS, THE FOLLOWING RESTRICTIONS FOR THE USE OF HEAVY CONSTRUCTION MACHINERY DURING BACKFILLING OPERATIONS APPLY:
- NO CONSTRUCTION EQUIPMENT SHALL CROSS THE BARE PRECAST CONCRETE BRIDGE UNIT.
- AFTER THE COMPACTED FILL LEVEL HAS REACHED A MINIMUM OF 4" OVER THE CROWN OF THE BRIDGE, CONSTRUCTION EQUIPMENT WITH A WEIGHT OF LESS THAN 10 TONS MAY CROSS THE BRIDGE.
- AFTER THE COMPACTED FILL LEVEL HAS REACHED A MINIMUM OF 1'-0" OVER THE CROWN OF THE BRIDGE, CONSTRUCTION EQUIPMENT WITH A WEIGHT OF LESS THAN 30 TONS MAY CROSS THE BRIDGE.
- AFTER THE COMPACTED FILL LEVEL HAS REACHED THE DESIGN COVER, OR 2'-0" MINIMUM, OVER THE CROWN OF THE PRECAST CONCRETE BRIDGE, CONSTRUCTION EQUIPMENT WITHIN THE DESIGN LOAD LIMITS FOR THE ROAD MAY CROSS THE PRECAST CONCRETE BRIDGE.

13.2. LEVELING PAD/SHIMS - THE BRIDGE UNITS AND WINGWALLS SHALL BE SET ON HARDBOARD SHIMS CONFORMING TO ASTM D1037 OR PLASTIC SHIMS (DAYTON SUPERIOR P-80, P-81 OR APPROVED EQUAL) MEASURING 5" x 5", MINIMUM, UNLESS SHOWN OTHERWISE ON THE PLANS. A MINIMUM GAP OF 1/2" SHALL BE PROVIDED BETWEEN THE FOOTING AND THE BOTTOM OF THE BRIDGE'S VERTICAL LEGS OR THE BOTTOM OF THE WINGWALL. ALSO, A SUPPLY OF 1/8", 1/4" AND 1/2" THICK HARDBOARD OR PLASTIC SHIMS FOR VARIOUS SHIMMING PURPOSES SHALL BE ON SITE.
13.3. PLACEMENT OF BRIDGE UNITS - THE BRIDGE UNITS SHALL BE PLACED AS SHOWN ON THE ENGINEER'S PLAN DRAWINGS. SPECIAL CARE SHALL BE TAKEN IN SETTING THE ELEMENTS TO THE TRUE LINE AND GRADE. THE JOINT WIDTH BETWEEN ADJACENT PRECAST UNITS SHALL NOT EXCEED 3/4".

13.4. IT IS THE CONTRACTOR'S RESPONSIBILITY TO MAINTAIN THE STRUCTURE SPAN DURING ALL PHASES OF INSTALLATION. DUE TO THE ARCH SHAPE, BRIDGE ELEMENTS WILL TEND TO SPREAD UNDER SELF-WEIGHT. IT IS IMPERATIVE THAT ANY LATERAL SPREADING OF THE BRIDGE ELEMENTS BE AVOIDED DURING AND AFTER THEIR PLACEMENT. GENERALLY, HORIZONTAL CABLE TIES OR TIE RODS ARE SHIPPED IN THE LARGER BRIDGE ELEMENTS TO ASSIST IN PREVENTING THIS SPREADING. CABLE TIES/TIE RODS SHALL NOT BE REMOVED UNTILL BRIDGE UNITS ARE GROUTED AND GROUT HAS CURED. IT IS RECOMMENDED THAT TEMPORARY HARDWOOD BLOCKS BE USED IN CONJUNCTION WITH THE CABLE TIES/TIE RODS TO MAINTAIN SPAN. IF, HOWEVER, DUE TO SITE RESTRICTIONS, THESE CABLE TIES/TIE RODS MUST BE REMOVED PRIOR TO PLACEMENT OF THE BRIDGE ELEMANTS, THE CONTRACTOR MUST NOTIFY CONTECH (MANUFACTURER) AND REQUEST A SUGGESTED INSTALLATION PROCEDURE.

IN ADDITION, IF THE CABLE TIES/TIE RODS MUST BE REMOVED PRIOR TO SETTING ARCH UNITS, THE FOLLOWING QUALITY CONTROL PROCEDURE MUST BE FOLLOWED:
1) FIND "MEASURED SPAN" UPON ARCH UNIT'S DELIVERY TO SITE, PRIOR TO LIFTING FROM TRUCK AND REMOVING CABLE TIES/TIE RODS. "MEASURED SPAN" SHALL BE THE AVERAGE OF (3) SPAN MEASUREMENTS ALONG THE LAY LENGTH OF THE ARCH UNIT.
2) AFTER SETTING OF BRIDGE UNIT ON THE FOUNDATION, VERIFY THE SPAN. THIS "INSTALLED SPAN MEASUREMENT" SHALL NOT EXCEED THE MAXIMUM OF:
A) THE NOMINAL SPAN +1/2" OR
B) THE "MEASURED SPAN"
IF THE "INSTALLED SPAN MEASUREMENT" EXCEEDS THIS AMOUNT, THE ARCH UNIT SHALL BE LIFTED AND RE-SET UNTIL THE "INSTALLED SPAN MEASUREMENT" MEETS THE LIMITS.

13.5. PLACEMENT OF WINGWALLS, HEADWALLS AND FOUNDATION UNITS - THE WINGWALLS, HEADWALLS AND FOUNDATIONS SHALL BE PLACED AS SHOWN ON THE PLAN DRAWINGS. SPECIAL CARE SHALL BE TAKEN IN SETTING THE ELEMENTS TO THE TRUE LINE AND GRADE.
13.6. WATERPROOFING/JOINT PROTECTION AND SUBSURFACE DRAINAGE
13.6.1. EXTERNAL PROTECTION OF JOINTS - THE BUTT JOINT MADE BY TWO ADJOINING BRIDGE UNITS SHALL BE COVERED WITH A 7/8" x 1 3/8" PREFORMED BITUMINOUS JOINT SEALANT AND A MINIMUM OF A 9" WIDE JOINT WRAP. THE SURFACE SHALL BE FREE OF DIRT BEFORE APPLYING THE JOINT MATERIAL. A PRIMER COMPATIBLE WITH THE JOINT WRAP TO BE USED SHALL BE APPLIED FOR A MINIMUM WIDTH OF 9" ON EACH SIDE OF THE JOINT. THE EXTERNAL WRAP SHALL BE CS212 BY CONCRETE SEALANTS INC., EZ-WRAP RUBBER BY PRESS-SEAL GASKET CORPORATION, SEAL WRAP BY MAR MAC MANUFACTURING CO. INC. OR APPROVED EQUAL. THE JOINT SHALL BE COVERED CONTINUOUSLY FROM THE BOTTOM OF ONE BRIDGE SECTION LEG, ACROSS THE TOP OF THE BRIDGE AND TO THE OPPOSITE BRIDGE SECTION LEG. ANY LAPS THAT RESULT IN THE JOINT WRAP SHALL BE A MINIMUM OF 6" LONG WITH THE OVERLAP RUNNING DOWNHILL.
13.6.2. IN ADDITION TO THE JOINTS BETWEEN BRIDGE UNITS, THE JOINT BETWEEN THE END BRIDGE UNIT AND THE HEADWALL SHALL ALSO BE SEALED AS DESCRIBED ABOVE. IF PRECAST WINGWALLS ARE USED, THE JOINT BETWEEN THE END BRIDGE UNIT AND THE WINGWALL SHALL BE SEALED WITH A 2'-0" STRIP OF FILTER FABRIC. ALSO, IF LIFT HOLES ARE FORMED IN THE BRIDGE UNITS, THEY SHALL BE PRIMED AND COVERED WITH A 9" x 9" SQUARE OF JOINT WRAP.
13.6.3. DURING THE BACKFILLING OPERATION, CARE SHALL BE TAKEN TO KEEP THE JOINT WRAP IN ITS PROPER LOCATION OVER THE JOINT.
13.6.4. SUBSOIL DRAINAGE SHALL BE AS DIRECTED BY THE ENGINEER.

13.7. GROUTING
13.7.1. GROUTING SHALL NOT BE PERFORMED WHEN TEMPERATURES ARE EXPECTED TO GO BELOW 35° FOR A PERIOD OF 72 HOURS. FILL THE BRIDGE-FOUNDATION KEYWAY WITH CEMENT GROUT (PORTLAND CEMENT AND WATER OR CEMENT MORTAR COMPOSED OF PORTLAND CEMENT, SAND AND WATER) WITH A MINIMUM 28-DAY COMPRESSIVE STRENGTH OF 3000 PSI. VIBRATE AS REQUIRED TO ENSURE THAT THE ENTIRE KEY AROUND THE BRIDGE ELEMENT IS COMPLETELY FILLED. IF BRIDGE ELEMENTS HAVE BEEN SET WITH TEMPORARY TIES (CABLES, BARS, ETC.) GROUT MUST ATTAIN A MINIMUM COMPRESSIVE STRENGTH OF 1500 PSI BEFORE TIES MAY BE REMOVED.
13.7.2. ALL GROUT SHALL HAVE A MAXIMUM AGGREGATE SIZE OF 1/4".
13.7.3. LIFTING AND ERECTION ANCHOR RECESSES SHALL BE FILLED WITH GROUT.
13.7.4. AFTER GROUT HAS REACHED ITS DESIGN STRENGTH THE TEMPORARY HARDWOOD WEDGES SHALL BE REMOVED AND THEIR HOLES FILLED WITH GROUT.
13.8. BACKFILL
13.8.1. DO NOT PERFORM BACKFILLING DURING WET OR FREEZING WEATHER.
13.8.2. NO BACKFILL SHALL BE PLACED AGAINST ANY STRUCTURAL ELEMENTS UNTIL THEY HAVE BEEN APPROVED BY THE ENGINEER.
13.8.3. BACKFILL SHALL BE CONSIDERED AS ALL REPLACED EXCAVATION AND NEW EMBANKMENT ADJACENT TO THE PRECAST CONCRETE ELEMENTS. THE PROJECT CONSTRUCTION AND MATERIAL SPECIFICATIONS, WHICH INCLUDE THE SPECIFICATIONS FOR EXCAVATION FOR STRUCTURES AND ROADWAY EXCAVATION AND EMBANKMENT CONSTRUCTION, SHALL APPLY EXCEPT AS MODIFIED IN THIS SECTION.
13.8.4. BACKFILL ZONES:
- IN-SITU SOIL
- ZONE A: CONSTRUCTED EMBANKMENT OR OVERFILL.
- ZONE B: FILL THAT IS DIRECTLY ASSOCIATED WITH PRECAST CONCRETE BRIDGE INSTALLATION.
- ZONE C: ROAD STRUCTURE.
13.8.5. REQUIRED BACKFILL PROPERTIES
13.8.5.1. IN-SITU SOIL - NATURAL GROUND IS TO BE SUFFICIENTLY STABLE TO ALLOW EFFECTIVE SUPPORT TO THE PRECAST CONCRETE BRIDGE UNITS. AS A GUIDE, THE EXISTING NATURAL GROUND SHOULD BE OF SIMILAR QUALITY AND DENSITY TO ZONE B MATERIAL FOR MINIMUM LATERAL DIMENSION OF ONE BRIDGE SPAN OUTSIDE OF THE BRIDGE FOOTING.
13.8.5.2. ZONE A - ZONE A REQUIRES FILL MATERIAL WITH SPECIFICATIONS AND COMPACTING PROCEDURES EQUAL TO THAT FOR NORMAL ROAD EMBANKMENTS.
13.8.5.3. ZONE B - GENERALLY, SOILS SHALL BE REASONABLY FREE OF ORGANIC MATTER, AND, NEAR CONCRETE SURFACES, FREE OF STONES LARGER THAN 3" IN DIAMETER SEE CHARTS FOR DETAILED DESCRIPTIONS OF ACCEPTABLE SOILS.
13.8.5.4. ZONE C - ZONE C IS THE ROAD SECTION OF GRAVEL, ASPHALT OR CONCRETE BUILT IN COMPLIANCE WITH LOCAL ENGINEERING PRACTICES.
13.8.5.5. GEOTECHNICAL ENGINEER SHALL REVIEW GRADATIONS OF ALL INTERFACING MATERIALS AND, IF NECESSARY, RECOMMEND GEOTEXTILE FILTER FABRIC (PROVIDED BY CONTRACTOR)

13.8.6. PLACING AND COMPACTING BACKFILL
DUMPING FOR BACKFILLING IS NOT ALLOWED ANY NEARER THAN 3'-0" FROM THE BRIDGE LEG.

THE FILL MUST BE PLACED AND COMPACTED IN LAYERS NOT EXCEEDING 8". THE MAXIMUM DIFFERENCE IN THE SURFACE LEVELS OF THE FILL ON OPPOSITE SIDES OF THE BRIDGE MUST NOT EXCEED 2'-0".

THE FILL BEHIND WINGWALLS MUST BE PLACED AT THE SAME TIME AS THAT OF THE BRIDGE FILL. IT MUST BE PLACED IN PROGRESSIVELY PLACED HORIZONTAL LAYERS NOT EXCEEDING 8" PER LAYER.

THE BACKFILL OF ZONE B SHALL BE COMPACTED TO A MINIMUM DENSITY OF 95% OF THE STANDARD PROCTOR, AS REQUIRED BY AASHTO T-99.

SOIL WITHIN 1'-0" OF CONCRETE SURFACES SHALL BE HAND-COMPACTED. ELSEWHERE, USE OF ROLLERS IS ACCEPTABLE. IF VIBRATING ROLLER-COMPACTORS ARE USED, THEY SHALL NOT BE STARTED OR STOPPED WITHIN ZONE B AND THE VIBRATION FREQUENCY SHOULD BE AT LEAST 30 REVOLUTIONS PER SECOND.

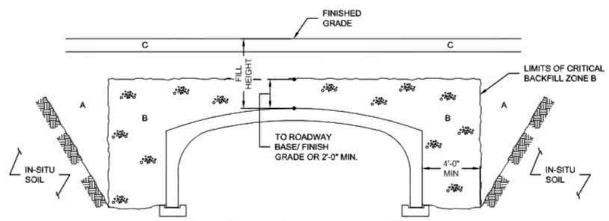
THE BACKFILL MATERIAL AND COMPACTING BEHIND WINGWALLS SHALL SATISFY THE CRITERIA FOR THE BRIDGE BACKFILL, ZONE B.

BACKFILL AGAINST A WATERPROOFED SURFACE SHALL BE PLACED CAREFULLY TO AVOID DAMAGE TO THE WATERPROOFING MATERIAL.
13.8.7. BRIDGE UNITS
FOR FILL HEIGHTS OVER 12 FEET (AS MEASURED FROM TOP CROWN OF BRIDGE TO FINISHED GRADE), NO BACKFILLING MAY BEGIN UNTIL A BACKFILL COMPACTION TESTING PLAN HAS BEEN COORDINATED WITH AND APPROVED BY CONTECH® ENGINEERED SOLUTIONS.
13.8.8. WINGWALLS
BACKFILL IN FRONT OF WINGWALLS SHALL BE CARRIED TO GROUND LINES SHOWN IN THE PLANS.
13.8.9. MONITORING
THE CONTRACTOR SHALL CHECK SETTLEMENTS AND HORIZONTAL DISPLACEMENT OF FOUNDATION TO ENSURE THAT THEY ARE WITHIN THE ALLOWABLE LIMIT PROVIDED BY THE ENGINEER. THESE MEASUREMENTS SHOULD GIVE AN INDICATION OF THE SETTLEMENTS AND DEFORMATIONS ALONG THE LENGTH OF THE FOUNDATIONS.

THE FIRST MEASUREMENT SHOULD TAKE PLACE AFTER THE ERECTION OF ALL PRECAST BRIDGE SYSTEM ELEMENTS, A SECOND AFTER COMPLETION OF BACKFILLING, AND A THIRD BEFORE OPENING OF THE BRIDGE TO TRAFFIC. FURTHER MEASUREMENTS MAY BE MADE ACCORDING TO LOCAL CONDITIONS.

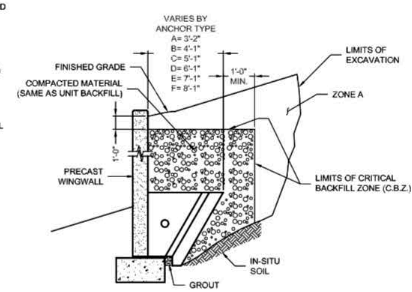
ACCEPTABLE SOILS FOR USE IN ZONE B BACKFILL

TYPICAL USCS MATERIALS	AASHTO GROUP	AASHTO SUBGROUP	PERCENT PASSING US SIEVE NO. #10	#40	#200	CHARACTER OF FRACTION PASSING NO. 40 SIEVE: LIQUID LIMIT	PLASTICITY INDEX	SOIL DESRIPTION
GW, GP, SP	A1	A-1a	50 MAX	30 MAX	15 MAX		6 MAX	LARGELY GRAVEL BUT CAN INCLUDE SAND AND FINES
GM, SW, SP, SM	A1	A-1b		50 MAX	25 MAX		6 MAX	GRAVELLY SAND OR GRADED SAND, MAY INCLUDE FINES
GM, SM, ML, SP, GP	A2	A-2-4			35 MAX	40 MAX	10 MAX	SANDS, GRAVELS WITH LOW-PLASTICITY SILT FINES
SC, GC, GM	A2	A-2-5			35 MAX	41 MIN	10 MAX	SANDS, GRAVELS WITH PLASTIC SILT FINES
SP, SM, SW	A3			51 MIN	10 MAX		NON-PLASTIC	FINE SANDS
ML, SM, SC	A4				36 MIN	40 MAX	10 MAX	LOW-COMPRESSIBILTY SILTS

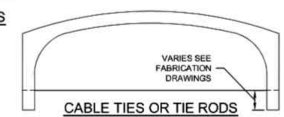


SPAN	FILL HEIGHT	ACCEPTABLE MATERIAL INSIDE ZONE B
≤ 24'-0"	≥ 12'-0"	A1, A3
≤ 24'-0"	< 12'-0"	A1, A2, A3, A4
> 24'-0"	ALL	A1, A3

BACKFILL REQUIREMENTS



WALL BACKFILL REQUIREMENTS



CABLE TIES OR TIE RODS
(> 24'-0" SPAN)

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MARK	DATE	REVISION DESCRIPTION	BY
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800-338-1122 513-645-7000 513-645-7993 FAX

CONTECH®
STRUCTURAL PLATE

PROPOSAL
DRAWING

CONSPAN B SERIES 24'-0" SPAN
BLUFFDALE BRIDGE
2160 W. 14400 S.
BLUFFDALE, UT

PROJECT No.: 777777	SEQ. No.: ----	DATE: 11/6/2018
DESIGNED: ----		DRAWN:
CHECKED:		APPROVED:
SHEET NO.: 5 OF 5		

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Appendix D: Funding Sources Cited

"Surface Transportation Program." *Wasatch Front Regional Council*, wfrc.org/programs/transportation-improvement-program/surface-transportation-program/.

"In-Progress 2019-2050 Regional Transportation Plan." *Wasatch Front Regional Council*, wfrc.org/vision-plans/regional-transportation-plan/progress-2019-2050-regional-transportation-plan/.

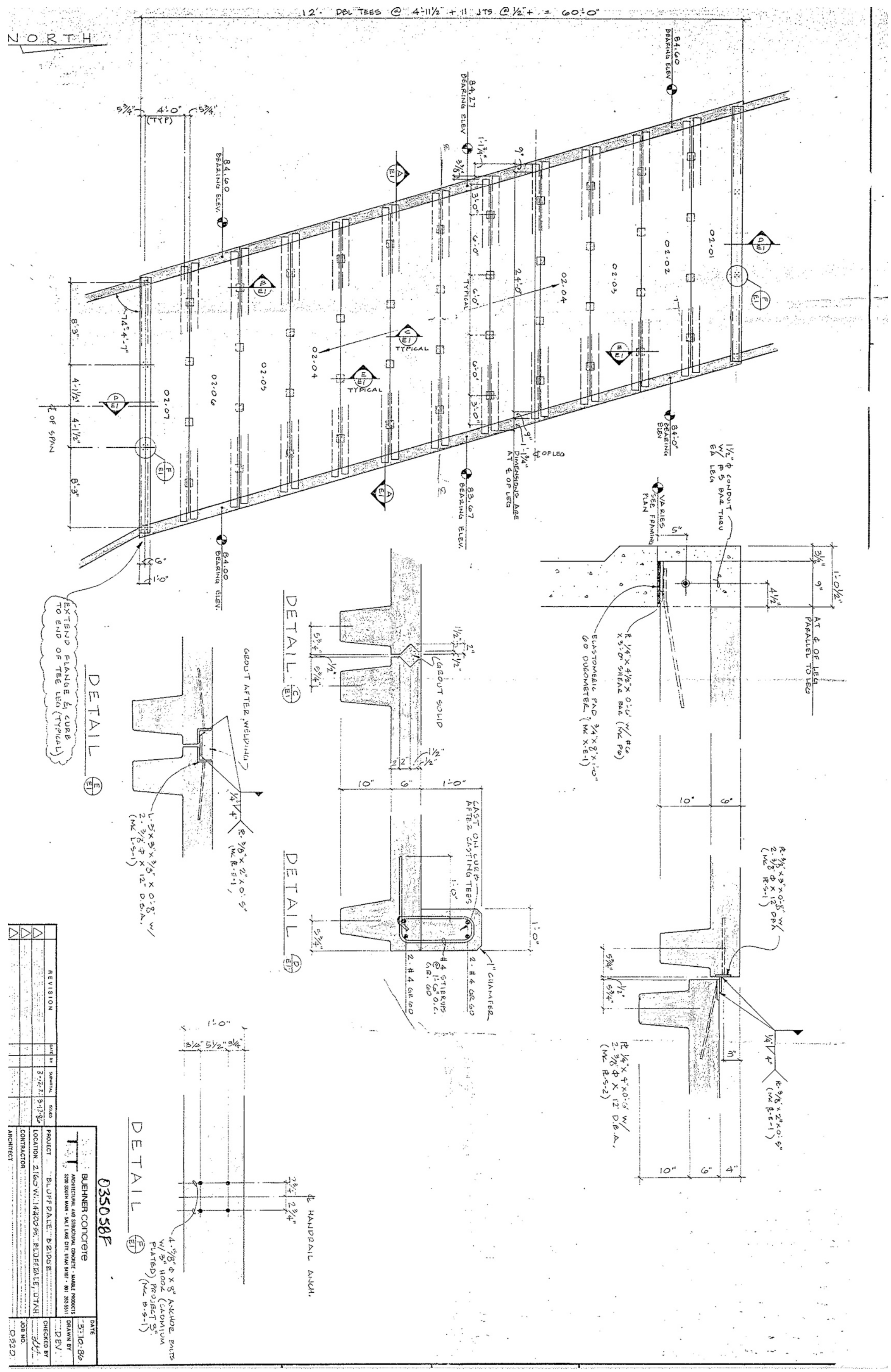
"Transportation Alternatives Program." *Wasatch Front Regional Council*, wfrc.org/programs/transportation-improvement-program/transportation-alternatives-program/.

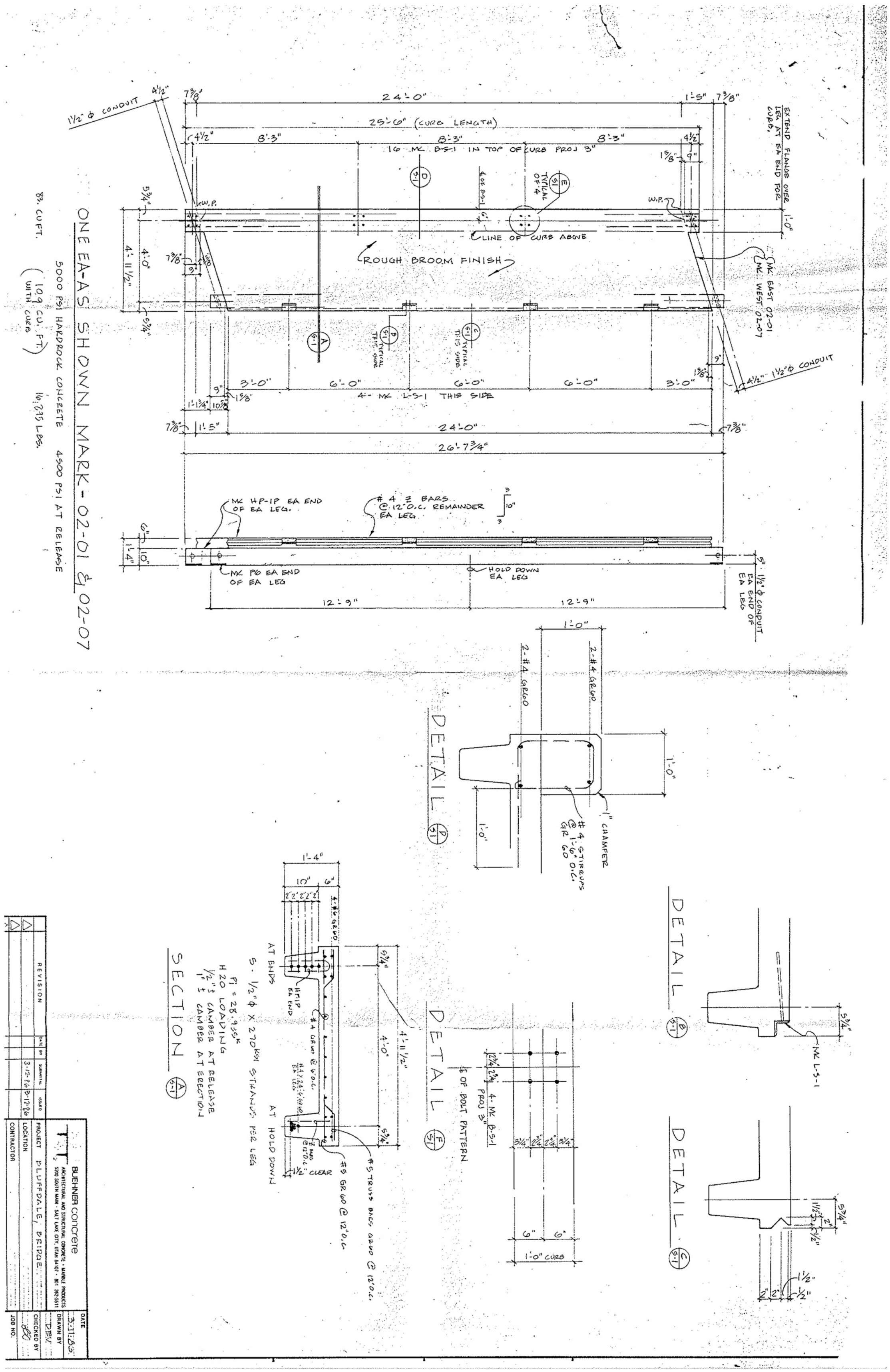
"Transportation and Land Use Connection." *Wasatch Front Regional Council*, wfrc.org/programs/transportation-land-use-connection/.

"BUILD Discretionary Grants." *US Department of Transportation*, United States Department of Transportation, 2 Mar. 2012, www.transportation.gov/BUILDgrants.

"Infrastructure For Rebuilding America." *US Department of Transportation*, United States Department of Transportation, 9 Feb. 2016, www.transportation.gov/buildamerica/infragrants.

Appendix E: Original Bridge Drawings (Applicable Sheeets)




ONE EA-AS SHOWN MARK-02-01 & 02-07
5000 PSI HARDROCK CONCRETE 4500 PSI AT RELEASE
ROUGH BROOM FINISH
LINE OF CURB ABOVE
SECTION
DETAIL
BUEHNER concrete
BLUFFDALE BRIDGE

Appendix F: 2017 UDOT Bridge Condition Report (Applicable Pages)

UDOT Keeping Utah Moving

035058F

Utah and Salt Lake Canal bridge on 14400 South S

Inspector: DALE DEBENHAM

Inspection Date: December 14, 2017

Condition Overview

Deck NBI:	7	**Culvert NBI:**	N	**BHI:**	83.12	**BHI Rank:**	875
Super NBI:	6	**Channel NBI:**	7	**PHI:**	83.12	**PHI Rank:**	875
Sub NBI:	7	**Scour NBI:**	5	**Structurally Defficent :**	No	**Year Built / Recon:**	1986 /

Bridge Issues

	Yes	No
This report identifies deficiencies requiring urgent corrective action. Details:	☐	☑
This bridge is scour critical:	☐	☑
This bridge contains fracture critical components:	☐	☑
This bridge needs a new load rating:	☐	☑
This bridge requires special inspection: Recommended Frequency:	☐	☑

Report Contents

☐ Desk Review Audit	☐ Critical Findings Report
☑ Condition Ratings Report	☐ Vertical Underclearance Report
☑ Element Level Inspection Report	☐ Cross Section Report
☑ Bridge Photographs	☑ Other: Double T Beam Stem Deterioration Chart

Type of Inspection

NBI	Element	Fracture Critical	Underwater	Complex	Other Special
☑	☑	☐	☐	☐	☐

Inspectors	Name	Date	P.E. Seal and Signature
Inspector of Record	Dale Debenham	12/14/2017	PROFESSIONAL ENGINEER NO. 9659500 TYSON J. SCHULTZ ELECTRONIC SEAL STATE OF UTAH
Field Checked	Randy Haider	12/14/2017	
Checked	Tyson Schultz	01/11/2018	
Back Checked	Dale Debenham	01/11/2018	
Corrected	Dale Debenham	01/11/2018	
Verified	Tyson Schultz	01/15/2018	
Independent Field QC Review			
QA Review			06/19/2018

035058F

Utah and Salt Lake Canal bridge on 14400 South S

Inspector: DALE DEBENHAM

Inspection Date: December 14, 2017

IDENTIFICATION

Bridge Key:	035058F
(8) **NBI Number:**	035058F
Structure Name:	Utah and Salt Lake Canal bridge on 1440
(9) **Location:**	2160 W.14400 S.,BLUFFDALE
(7) **Carries:**	14400 S.ST.FA#2038
(42A) **Service On:**	5 Highway-pedestrian
(6) **Feature Crossed:**	UTAH AND SALT LAKE CAN
(42B) **Service Under:**	5 Waterway
(4) **Placecode:**	Bluffdale City
(3) **County:**	Salt Lake
(1) **State:**	49 Utah
(2) **Region:**	Reg 2 Local
Station:	902 - Not Applicable
(16) **Latitude:**	40.49
(17) **Longitude:**	-111.95
(22) **Owner:**	City/Municipal Hwy Agenc
(21) **Custodian:**	04 City/Municipal Agenc
(27) **Year Built:**	1986
(106) **Year Recon:**	
(37) **Historical:**	5 Not eligible for NRHP

(98) Border State: Not Applicable (P)

(99) Border Number:

% Responsibility: 0

INSPECTION

(90) Date of Inspection:	12/14/2017
(91) Frequency:	24
Next Inspection:	12/14/2019

Inspection Type	(92)Freq	(93)Last Insp	Next Insp
Element	24	12/14/2017	12/14/2019
(A) Fracture Critical		N/A	N/A
(B) Underwater		N/A	N/A
(C) Special Insp		N/A	N/A

LOAD RATING, POSTING AND SIGNS

(41) **Posting Status:**	A Open, no restriction
(70) **Posting %:**	5 At/Above Legal Loads
Rating Date:	10/29/2014
(31) **Design Load:**	6 MS18(HS20)+mod
(63) **Opr Method:**	8 LRFR (HL93)
(64) **Opr Rating:**	1.31
(65) **Inv Method:**	6 Load Factor (MS18)
(66) **Inv Rating:**	0.74

Sign Legibility: N/A **Sign Visible:** N/A

Object Markers Reqd N/A

Deck Overlay

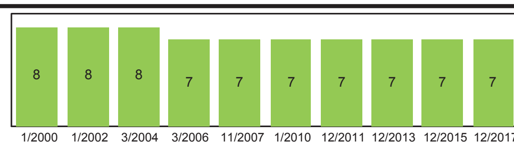
(108A) **Wearing Surface:**	6 Bituminous	**Year Applied:**	N/A
(108B) **Membrane:**	0 None	**Thickness:**	4.00 IN

Steel Coatings

Steel Overcoat Year:	N/A
Paint / Repaint Year:	N/A

DECK GEOMETRY

(68) **Deck Geometry:**	5 Above Tolerable
Deck Area:	1,628.00
(107) **Deck Type:**	2 Concrete Precast Panel
(108C) **Deck Protection:**	None
(52) **O. to O. Width:**	62.42
(51) **Curb to Curb Width:**	36.50
(50A) **Curb / Sidewalk Width L:**	8.83
(50B) **Curb / Sidewalk Width R:**	8.83
(33) **Median:**	0 No median

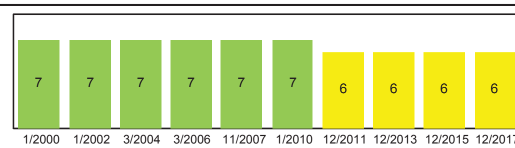


DECK CONDITION

(58) **Deck Rating:**	7 Good
(36A) **Bridge Rail:**	N N/A or not required
(36B) **Transition:**	N N/A or not required
(36C) **Approach Rail:**	N N/A or not required
(36D) **Approach Rail Ends:**	N N/A or not required

SUPERSTRUCTURE GEOMETRY

(45) **# of Main Spans:**	1
(46) **# of Approach Spans:**	0
(43 A) **Main Material:**	5 Prestressed Concrete
(43 B) **Main Design:**	04 Tee Beam
(48) **Max Span Length:**	24.75
(49) **Structure Length:**	26.08
(112) **NBIS Length:**	Long Enough
(103) **Temp Structure:**	Not Applicable (P)
(34) **Skew:**	5
(35) **Structure Flared:**	0 No flare



SUPERSTRUCTURE CONDITION

(59) **Superstructure Rating:**	6 Satisfactory
(67) **Structure Evaluation:**	6 Equal Min Criteria
(101) **Parallel Structure:**	No || bridge exists
(72) **Approach Alignment:**	8 Equal Desirable Crit
(32) **Approach Roadway Width:**	36.50

035058F

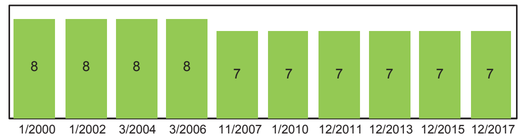
Utah and Salt Lake Canal bridge on 14400 South S

Inspector: DALE DEBENHAM

Inspection Date: December 14, 2017

SUBSTRUCTURE GEOMETRY

(38) **Navigation Control:**	NA-no waterway
(39) **Nav Vert Clearance:**	0.00
(40) **Nav Horiz Clearance:**	0.00
(111) **Pier Protection:**	Not Applicable (P)
(116) **Lift Bridge Vertical Clearance:**	0.00
(113) **Scour Rating:**	5 Stable w/in footing
(71) **Waterway Adequacy:**	8 Equal Desirable



SUBSTRUCTURE CONDITION

(60) **Substructure Rating:**	7 Good
(61) **Channel Rating:**	7 Minor Damage

ROUTE ON STRUCTURE: 14400 South Street (FA 2038)

ROADWAY LOCATION

(5A) **Pos Prefix:**	Route On Structure
(5B) **Kind of Hwy:**	5 City Street
(5C) **Level Service:**	0 None of the below
(5D) **Route Num:**	02038
(13A/B) **LRS Route:**	0000002038/00
(11) **Milepost:**	0.54 mi
(5E) **Suffix:**	0 N/A (NBI)
(102) **Direction:**	2 2-way traffic
(28A): **Lanes On**	2
(19) **Detour Length:**	4.97 mi (8.00 km)
(20) **Toll Facility:**	3 On free road

ROADWAY CLASSIFICATION

(26) **Funct Class:**	16 Urban Minor Arterial
(104) **NHS:**	0 Not on NHS
(110) **Nat Truck Net:**	
(100) **Defense Hwy:**	0 Not a STRAHNET hwy
(29) **ADT:**	2,715 Cars/Day
(109) **Pct Trucks:**	1.00%
(30) **ADT Year:**	2010
(114) **Future ADT:**	3,390 Cars/Day
(115) **Future ADT Year**	2030

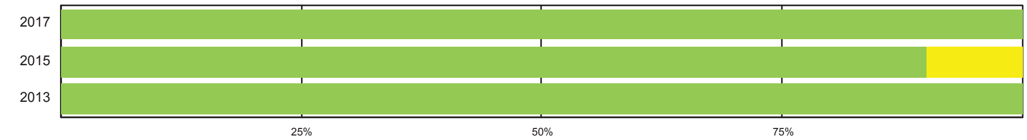
CLEARANCES

(10) **Vertical:**	99.99
(53) **Min Vert Over:**	99.99
(54b) **Min Vert Under**	0.00
(54A) **Vert Ref:**	N Feature not hwy or RF
(47) **Horizontal:**	36.50
(56) **Min Lat Left:**	0.00
(55B) **Min Lat Right:**	0.00
(55A) **Horiz Ref:**	N Feature not hwy or RI
(69) **Underclearance:**	N Not applicable (NBI)

Planning and Inspection Details

Available Plans:	Partial Plans	**Crane Req:**	No	**Follow Up Req:**	0	(94) Bridge Cost:	$1
Funding Avail:	NHPP, STP	**Last Crane Date**	1/1/1901 12:00:00	**Date Completed**		(95) Roadway Cost:	$1
Prime Funding:	NHPP_BR	**UT Req:**	No	**Follow Up**		(96) Total Cost:	$1
						(97) Year of Cost Estimate:	Unknown
Update POA:	No	**Last UT Date**		**Reason:**		(75)Type of Work:	Unknown (P)
						(76) Length of Improvement:	-1.0 ft

16 / 1	**Re Conc Top Flange**	Total: 1,628 sq.ft	CS1: 1,628 sq.ft (100%)	CS2: 0 sq.ft (0%)	CS3: 0 sq.ft (0%)	CS4: 0 sq.ft (0%)



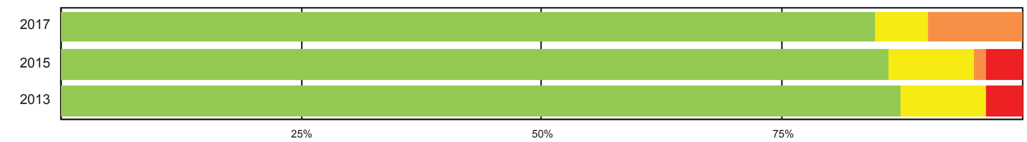
12/21/2015 Brandon Reda

Top surface is covered with asphalt overlay and not visible for inspection. Ride is smooth. Bottom surface overall in good condition with typical efflorescence, active leaking and water staining along seams. Both backwall joints are open. 10% meets CS2 due to staining.

12/14/2017 DALE DEBENHAM

The deck surface is covered with an asphalt wearing surface. There are no conditions observed that might indicate defects are developing in the deck surface. The deck is comprised of the double T beam top flanges. There are no defects found in the flange underside. The top flange element remains in CS1.

109 / 2	**Pre Opn Conc Girder/Beam**	Total: 313 ft	CS1: 265 ft (85%)	CS2: 17 ft (5%)	CS3: 31 ft (10%)	CS4: 0 ft (0%)



12/21/2015 Brandon Reda

Superstructure consists of double T beams. Typical active leaking and seepage along seams. Fourth and fifth from north end along west side have cracking and 1 foot exposed rebar. Minimal section loss is visible. Eighth and ninth beam midspan have exposed rebar 6 feet long on each beam. Rebar has dropped approximately 2.5" with no concrete intact around rebar. Up to 1/8" section loss. 12 feet meets CS4, 4 feet meets CS3 due to cracking and exposed rebar.

12/14/2017 DALE DEBENHAM

The bridge is constructed using 12 precast double T girder lines. Many of the beam stems have delaminations, longitudinal cracks, with 2 of the beam stems having spalled with exposed reinforcement. See attached T Beam Deterioration Chart. 17 feet was placed in CS2 for delaminations. The prestressed concrete girders have 18 linear feet of cracking and 13 feet of spalls that are placed in CS3. The remaining girder quantity remains in CS1.

1080 / 2	***Delamination/Spall/Patched Area***	*Total: 13 each*	*CS1: 0 each (0%)*	*CS2: 0 each (0%)*	*CS3: 13 each (100%)*	*CS4: 0 each (0%)*

12/14/2017 DALE DEBENHAM

Double T beams 8 and 9 have 6 foot long spalls with exposed reinforcement. The bars are deformed and sagging approximately 4 inches from the bottom of the stem. The #4 bar is rusted having approximately 75% section remaining. Beam 11 has a 1 foot spall noted near abutment 1. See attached Beam Deterioration Detail Sheet. 13 feet was added to CS3 for spalls with exposed reinforcement.

1110 / 2	***Cracking (PSC)***	*Total: 18 each*	*CS1: 0 each (0%)*	*CS2: 0 each (0%)*	*CS3: 18 each (100%)*	*CS4: 0 each (0%)*

12/14/2017 DALE DEBENHAM

The prestressed T beam stems have wide horizontal cracking near each abutment. The crack width varies from 0.01 to 0.05 inches wide. The crack width warrants 18 feet of wide cracking in CS3.

Change Notes (Prior to NBE's)

15-MAR-2004: 03/15/2004 Neal Pierce, Shane Jones - City
06-MAR-2006: 03/06/2006 Terri Taylor, Dale Debinham new set of photos taken.
15-NOV-2007: 11/15/2007 Team: CFB/AJC (Ayres). Routine NBI and Element insp.
04-JAN-2010: 01/04/2010 Brenda R., Dale D.
31-MAR-2011: 03/31/2011 TS Changed Scour Rating from a U to 7, due to the full implemented PPOA screening.
21-DEC-2011: 12/21/2011 Inspection party consists of Team Leader, Dale Debenham with Clint McCleery, Chad Cornia and Ron Rasmussen. Ron is the inspector of record for this bridge. Regular NBI inspection.
23-MAR-2012: 03/23/2012 DRA Item 113 rating was changed to 5 based on the results of a Scour POA, a copy of which is in the bridge folder. Ref. project F-ST99(139), Pin 9437.
02-DEC-2013: 12/02/13 Clint McCleery Chad Cornia Routine NBI & element level inspection. Inspection was performed by Clint McCleery.

Deck Notes (Prior to NBE's)

15-APR-1994: 04/15/94 Asphalt wearing surface has transverse cracking.
08-APR-1996: 4/08/96 looked ok at this time.
14-JAN-1998: 01/14/98 Still looks good.
11-JAN-2000: 01/11/00 Asphalt wearing surface continues to look good.
29-JAN-2002: 01/29/2002 The asphalt wearing surface looks good. There is leaking occuring between some of the construction joints.
15-MAR-2004: 03/15/2004 Asphalt wearing surface is in good condition. The underside of the deck looks very good. No moisture moving through the joints of the double T beams.
06-MAR-2006: 03/06/2006 Some minor leaking through the deck surface between the double tee beams. see photos
15-NOV-2007: 11/15/2007 The deck has a new asphalt overlay that is in good condition,
04-JAN-2010: 01/04/2010 Minor cracking in the wearing surface. Leakage between the tee beam joints with heavy staining in some areas.
21-DEC-2011: 12/21/2011 RLR Asphalt wearing surface has some cracking, but is basically in good condition. Steel railings and sidewalk areas are in good condition. Underside of deck slab portion of precast Double T beams are in good condition with no spalling.
21-DEC-2011: 12/21/2011 RLR There some light to moderate staining of the seams between the various T beam precast sections. No cracking or spalling is evident.
02-DEC-2013: 12/02/13 Clint McCleery Sidewalks are in good condition. Minor scaling of the concrete rub on the headwalls. Ped hand rail is in good condition. Wearing surface is in fair condition with minor unsealed cracks. Approx. 5 in. of asphalt at the crown.
02-DEC-2013: 12/02/13 CLM Underside of the deck between the beams is in good condition.

Approach Comments (Prior to NBE's)

23-APR-1992: 4/23/92 APPROACH ROADWAY AT BOTH BACKWALL JOINTS HAS SETTLED 1-2 INCHES, ROADWAY IS ROUGH AT THE ENDS OF THE STRUCTURE.
15-APR-1994: 04/15/94 Same as above.
08-APR-1996: 04/08/96 looked ok at this time.
14-JAN-1998: 01/14/98 Still looks good.
11-JAN-2000: 01/11/00 Continues to look good.
29-JAN-2001: 01/29/2001 Looks good.
06-MAR-2006: 03/06/2006 Joints are open.
15-NOV-2007: 11/15/2007 Alignmint looks good and there is a new asphalt overlay that ride smooth. Seweral metal plates on the approach road for some ongoing construction.
21-DEC-2011: 12/21/2011 RLR Ends of the bridge are behind the sidewalks on each side of the roadway and approach barrier is not needed. Some cracking of the roadway surfacing asphalt on both approaches.
02-DEC-2013: 12/02/13 Clint McCleery Ride across the structure is good. Approach asphalt is in fair condition with minor unsealed cracks.

Drainage Comments (Prior to NBE's)

15-NOV-2007: 11/15/2007 All four wingwalls has moderate erosion behind them
04-JAN-2010: 01/04/2010 It would appear that the wingwalls were never backfilled.
21-DEC-2011: 12/21/2011 RLR Bridge deck has no drainage problems.

Superstructure Comments (Prior to NBE's)

23-APR-1992: 4/23/92 GIRDER AT WEST ABUTMENT HAS BEEN DAMAGED DURRING ERECTION. CONCRETE BROKEN AWAY FROM REBAR ON BOTTOM OF TEE.
15-APR-1994: 04/15/94 Same as above.
08-APR-1996: 04/08/96 conditions are the same.
11-JAN-2000: 01/11/00 Same as above.
29-JAN-2991: 01/29/2991 Same.
21-DEC-2011: 12/21/11 RLR Bridge is made up of precast Double T beam units with the sections in the sidewalk areas, sitting higher in order to compensate for the curb and gutter. The bottom of the stems along the seam between the 4th & 5th units having long. cracking.
21-DEC-2011: 12/21/11 RLR Cracking on the bottom of the stems appears to be the result of the rusting of the rebar that runs the length of the stem. Cracking at this time is around 6 ft long, located within the center portion of the stem and is approx.1/8th in. wide.
21-DEC-2011: 12/21/11 RLR In addition to the cracking previously indicated in the center of the stems, all roadway area stems on the east side of the bridge, adjacent to the bearing seats have cracking and some rust staining. This cracking extends out about 6 inches.
21-DEC-2011: 12/21/2011 RLR Cracking adjacent to the bearing seats is occurring at the east abutment only, with none on the west side. Cracking is mostly on the bottom surface with some on the sides. There is no associated spalling at this time.
21-DEC-2011: 12/21/2011 RLR Because of the cracking that is occurring on the double T beam stems the NBI inspection rating was dropped from a 7 to a 6.
02-DEC-2013: 12/02/13 Clint McCleery Beams at center span under the WB travel lanes one has a four ft. spalls with exposed rebar with heavy rusting. Adjacent beam has similar condition but the concrete has not spalled of at this time.
02-DEC-2013: 12/02/13 CLM Beam at the NW corner has 10 in. spalls with minor surface rusting. A few remaining beam ends have minor cracking with 0.062. in. of separation. Superstructure NBI has not been dropped. Remaining beams are in good condition.
02-DEC-2013: 12/02/13 10 in. pipe utilities spanning under from abutment to abutment through the backwalls at the south side.

Substructure Comments (Prior to NBE's)

06-MAR-2006: 03/06/2006 Some minor leaking at the backwall.
15-NOV-2007: 11/15/2007 Minor stayning on the backwall from previous leakige.
04-JAN-2010: 01/04/2010 Leakage through the backwall joints.
21-DEC-2011: 12/21/2011 RLR Cast-in-place concrete abutments are in good condition with no areas of cracking or deterioration.
02-DEC-2013: 12/02/13 Clint McCleery Both abutments are in good condition. All four wingwalls are in good condition. Backwalls are in good condition.

Channel Comments (Prior to NBE's)

21-DEC-2011: 12/21/2011 RLR Canal channel is in good condition and since the removal of the large trees on the canal banks, the channel appears to be more stable. No riprap on the banks and there are no erosion problems.
02-DEC-2013: 12/02/13 Clint McCleery Channel banks are under construction at the SE corner. Remaining banks have minor slumping that is covered with grass type vegetation. Minor silting on top of the concrete floor.