SH 82 Over Castle Creek Bridge Feasibility Study

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City of Aspen

Structure No. H-09-B

Castle Creek Bridge

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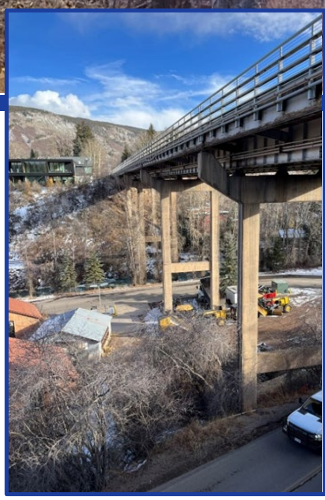
Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ABC	accelerated bridge construction
BDM	Bridge Design Manual
CCB	Castle Creek Bridge
CDOT	Colorado Department of Transportation
City	City of Aspen
eO	Engineering Operations
FO	functionally obsolete
kips	pound(s)-force
LRFD	Load and Resistance Factor Design
MASH	Manual for Assessing Safety Hardware
mph	mile(s) per hour
NBI	National Bridge Inventory
NSA	not self-arrested
PPC	polyester polymer concrete
ROW	right-of-way
RTD	Regional Transportation District
SH 82	State Highway 82
SIBC	slide-in bridge construction
SPMT	self-propelled modular transporter
vph	vehicle(s) per hour

1. Executive Summary



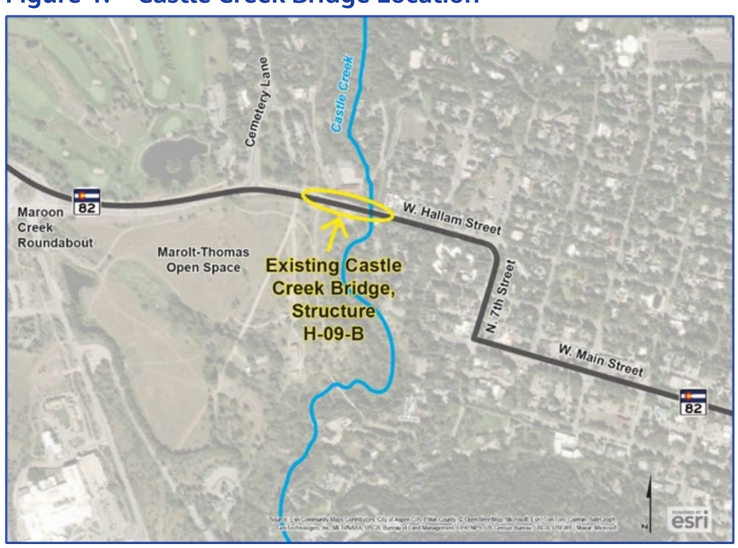
Purpose of this Feasibility Study

Why was this Feasibility Study prepared?

The City of Aspen is evaluating the feasibility of either rehabilitating or replacing the State Highway 82 (SH 82) bridge over Castle Creek and Power Plant Road in the City of Aspen, Colorado (City) (Figure 1). SH 82 is the single roadway that connects the City to other towns in the Roaring Fork Valley and beyond and, as such, serves as a vital link for local and regional travelers. Built in 1961, the Castle Creek Bridge (CCB) is a 5-span riveted steel plate girder bridge, with a reinforced concrete deck that rests on top of steel girders. It provides 2 travel lanes and sidewalks on both sides. A complex network of utilities run under the bridge.

> The existing CCB is a 63-year-old steel bridge on concrete supports. It was designed for vehicular loading less than today's AASHTO standard code requirements, for a design life of 50 years.

Figure 1. Castle Creek Bridge Location




Deck concrete spall with exposed rebar


Considerable girder corrosion


Hole in girder stiffener


Bearing pedestal crack


Movement of timber retaining wall

What issues have been identified with the existing bridge?

Recent inspections of the 63-year-old bridge have identified several issues, including signs of wear and major deterioration and corrosion of structural steel and concrete bridge components. In a 2022 routine bridge inspection, the Colorado Department of Transportation (CDOT) assigned the CCB a sufficiency rating of 50.3 (out of 100), which ranks the bridge according to structural condition, load rating, traffic data, and public importance. The National Bridge Inventory notes that a detour for the CCB is a 1-mile route that cannot accommodate present traffic volumes or oversize vehicles, impacting emergency response times. The length of this detour affects the sufficient rating of the bridge. The CCB also was designated as functionally obsolete, meaning the bridge and/or approach road alignment do not meet current standards for the highway system of which the bridge is an integral part.

A 2023 bridge special inspection ranked the bridge superstructure as "Fair" and revealed several issues with the CCB, particularly with the concrete deck and asphalt overlay and steel girders. The underside of the concrete deck exhibits signs of degradation and widespread surface cracking, and the steel girders show varying degrees of corrosion, with exterior girders displaying considerable corrosion and sag. Numerous tack welds and girder stiffeners exhibit cracks, and protective coatings on steel elements have failed, contributing to accelerated corrosion. Pier caps have water staining, delamination, and cracks. An inadequate joint sealing at Abutment 6 raises concerns about the bridge's structural performance, and notable movement of the bridge was observed through the existing conditions of the bearings. The timber retaining wall that supports the bike path at a bridge abutment requires yearly adjustments to keep the wall vertical (see Section 4.2 of this Feasibility Study for details).

Bridge Rehabilitation Analysis

What bridge rehabilitation measures are feasible?

Given the considerable deterioration of bridge components, a comprehensive rehabilitation plan is essential and would include bearing replacement, exterior girder replacement (requiring sidewalk replacement), protective steel coating rehabilitation, tack weld removal and monitoring, concrete deck repairs and asphalt overlay, pier cap repairs, joint seal replacement, and bridge rail replacement (see Section 4.3 for details). Regular monitoring and inspections would be crucial to evaluate the effectiveness of the rehabilitation measures and to promptly address issues as they emerge.

Will rehabilitation fix all the issues with the bridge?

The rehabilitation measures would address the bridge's immediate maintenance needs, prevent further deterioration, and maintain its structural integrity and safety while improving the bridge's long-term durability. Rehabilitation would not raise the load rating of the bridge to current standards, reduce maintenance needs, or address the limited functionality of the narrow roadway width. As such, the CCB would still be rated functionally obsolete. Further, the sufficiency rating of the bridge would not greatly increase because issues such as the narrow travel way width would not be addressed by rehabilitation measures. The extent to which rehabilitation measures would extend the bridge's service life would depend on factors such as routine maintenance (see Sections 4.3 and 4.4 for details). In short, rehabilitation measures would not substantially improve the bridge's condition to a level where total replacement would not be deemed necessary.

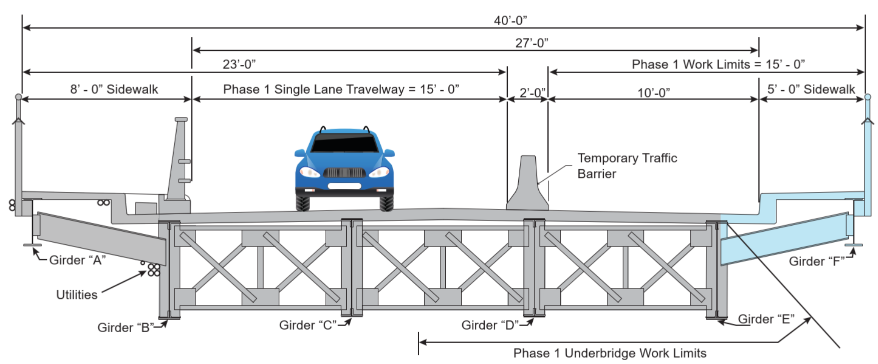
Rehabilitation would not address all issues with the current bridge.

How would you implement rehabilitation activities?

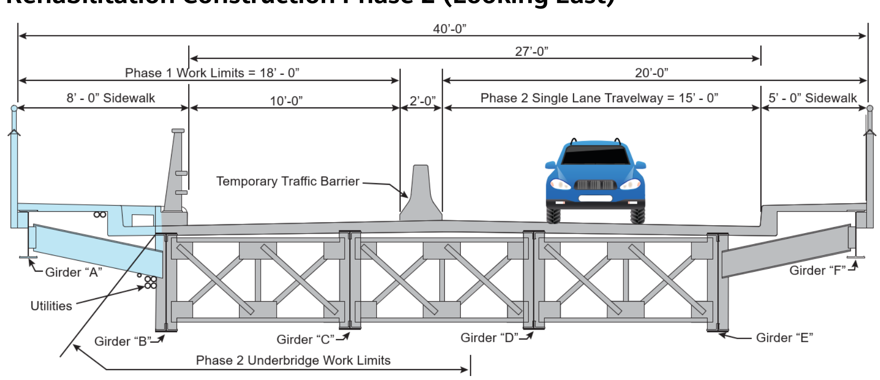
To minimize disruptions to traffic during construction, a phased construction approach would be most feasible. This approach would keep part of the bridge open during construction, with one lane open while

Figure 2. Rehabilitation Construction Phasing

Rehabilitation Construction Phase 1 (Looking East)



Rehabilitation Construction Phase 2 (Looking East)




One-inch bend in stiffener


Failure of protective coating


Bearing movement at Abutment 6


Inadequate joint seal at Abutment 6


Light scale cracking typical at all pier caps

construction is completed on one section of the bridge, then shifting traffic to the other side of the bridge to complete construction (Figure 2). Pedestrian access would be maintained on the bridge in a similar fashion. One temporary lane would be used and traffic movement would be maintained using either a signalized alternating lane or a single lane across the bridge in one direction paired with a companion detour in the other direction. The phasing options evaluated in this analysis could accommodate traditional snowplows and smaller wide loads (snowcats), but not larger snowcats due to space constraints (see Section 4.5 and 6 for details). Temporary or permanent utility relocations would be conducted in phases to maintain uninterrupted service during construction.

How long would it take to rehabilitate the bridge?

Bridge rehabilitation would be completed in 2 years. However, the bridge would remain in the "functionally obsolete" classification.

Rehabilitation would occur in two phases – each lasting approximately 4 to 6 months as dictated by area weather conditions and community events in Aspen and surrounding areas. One phase would be completed per year; therefore, bridge rehabilitation would be completed in two years.

Bridge Replacement Analysis

What bridge replacement options were evaluated?

The following two bridge width alternatives were evaluated for a bridge replacement:

- **Two-Lane Bridge Alternative**: This alternative would provide a two-lane bridge similar to the existing bridge, with one 10-foot sidewalk on the north side to accommodate the City's construction future planned trail. However, it would require of a temporary access lane that would be left in place, resulting in an approximately 48-foot-wide bridge, which would be only slightly narrower than the three-lane alternative.

Three-Lane bridge provides transit lane options now and into the future.

- **Three-Lane Bridge Alternative:** This alternative would provide a three-lane bridge with one 10-foot sidewalk on the north side, and provide the flexibility to designate one lane for immediate and future transit use. This bridge would be approximately 52 feet wide – only slightly wider than the two-lane alternative.

What type of bridge would be built?

Several bridge types were evaluated, including precast concrete, steel, and cast-in-place concrete bridges. The steep terrain and facilities under the bridge limit the space for large cranes, eliminating the ability to use precast concrete. Crane placement for steel requires closures of Power Plant Road. Therefore, only cast-in-place concrete is considered feasible because it provides the best constructability and limits impacts to the SH 82 profile. In addition, it was found that a four-span bridge would provide the best opportunity to control span lengths for a shallower structure depth that would accommodate traditional phased construction. Bridge replacement alternatives would be designed to current design standards and support heavier vehicle loads.

All phasing options would carry traffic on the bridge in a partial state throughout construction.

Full bridge demolition and construction is only an option if a new inbound/outbound detour is built to shift all traffic from bridge site to reduce traffic impacts.

How would you phase construction of a replacement bridge?

With the need to keep SH 82 partially open to traffic during construction, phased construction options were evaluated, all of which would keep portions of the existing bridge open to at least one lane of traffic during construction. (Full bridge demolition/construction would only be an option if a new inbound/outbound detour is built to shift all SH 82 traffic from the bridge site to reduce traffic impacts.) Traffic would be shifted from one part of the bridge to the other as portions of the new bridge are completed. An “overbuild” option was evaluated, which involves building a wider bridge than required for the final bridge in order to accommodate traffic during construction. This was eliminated as a feasible option because of spatial constraints at the bridge site and costly right-of-way that would be required.

Temporary lanes would be required on both portions of the existing and new bridge during construction to prevent a full closure of SH 82. The phasing options evaluated in this analysis could accommodate traditional snowplows and smaller wide loads (snowcats), but not larger snowcats due to space constraints. Pedestrian access would be provided either along the bridge or rerouted underneath the bridge, depending on the construction phase (see Section 5.3. for details). In all construction phasing options, utilities would be protected and relocated prior to demolition of existing bridge components.

Two-lane bridge construction phasing

One phasing option for this alternative was considered, because the only other option would be to fully close and replace the existing bridge. Four phases of construction would be required, but a single lane of traffic would be able to remain open during all construction phases while a detour lane would handle the other direction of travel. A temporary travel lane would be built on the bridge for use during construction and would remain in place after construction completion. As such, the width of the new two-lane bridge would be approximately 8 feet wider than the existing bridge. The new bridge would be located within the existing right-of-way limits; therefore, no right-of-way acquisition would be required.

Three-lane bridge construction phasing

Three construction phasing options were evaluated for the three-lane bridge replacement alternative, as summarized below.

The Three-Lane Centered bridge would provide the best overall scenario for construction; however, it would result in the most impacts to vehicular and pedestrian travelers.

- **Three-Lane Centered:** Phasing under this option is similar to the two-lane alternative. The main difference is that the bridge segments would be wider to accommodate the width for a third lane. The bridge would be located within the existing right-of-way limits; therefore, no right-of-way acquisition would be required.
- **Three-Lane Faster:** This option would demolish portions of the existing bridge early in the first phase to allow earlier construction of two temporary lanes, thus limiting the need for a single lane to one phase. However, pedestrians would be rerouted under the bridge in all phases. This option would shift the bridge approximately 3 feet to the south to avoid residences to the north, resulting in right-of-way impacts and removal of nearby trees. However, the south edge of the new bridge would almost be above the residence on Harbour Lane. Additional care would be required during construction to protect this residence.
- **Three-Lane Shifted:** This option would maintain two lanes on the bridge during all construction phases. Similar to Faster, this option would shift the new bridge to the south to avoid residences to the north, and as a result, the residence on Harbour Lane would nearly be under the bridge. The shift to the south would require rebuilding road segments at both ends of the bridge to align sidewalks. Like Faster, this option would extend outside existing right-of-way, affecting nearby residences and potentially requiring additional right-of-way acquisitions. A variation was considered that would provide pedestrian access during all phases by adding a pedestrian path on the bridge, but this would shift the bridge farther south, placing the bridge over a residence and resulting in right-of-way impacts. Therefore, it would not be feasible to accommodate pedestrians during all phases of this option.

The Three-Lane Shifted bridge would be the best scenario for vehicular and pedestrian travelers but would encounter substantial project risks and property impacts.

What would a new bridge look like?

Aesthetic guidelines for a replacement bridge have not been established. If a bridge replacement alternative is selected, aesthetic features would be incorporated into the bridge design as required by the City, CDOT, and other involved parties.

How long would it take to build a new bridge?

New bridge construction would be completed in 3 or 4 years, depending on phasing option chosen.

A construction phase would last approximately 4 to 6 months as dictated by area weather conditions and community events in Aspen and surrounding areas. One phase would be completed per year. Total construction duration would be four years for Three-Lane Centered and Shifted, and three years for Three-Lane Faster.

Use of ABC would be a good option if the only consideration was reducing impacts to the traveling public. However, the proximity of nearby residents, tight curves of the roadway below the bridge, and narrow footprint of the CCB make most ABC options very problematic. Traditional bridge construction phasing or full closure of SH82 are the only reasonable options.

Can accelerated bridge construction methods be used to build a new bridge?

Several accelerated bridge construction (ABC) techniques were analyzed to determine which, if any, would be a good fit for this spatially constrained site. ABC typically reduces onsite construction time and improves site constructability, total project delivery time, and work-zone safety for the traveling public. It can also reduce traffic impacts during construction and weather-related time delays. ABC methods considered include self-propelled modular transporter bridge move, bridge launch, bridge slide, and prefabricated bridge elements. These techniques involve various methods of building new bridge components off-site or near the bridge site and transporting/moving them into place once the new bridge substructure is built. It was determined that these ABC methods would not be successful for the CCB because of site terrain and space constraints for assembling and operating the large cranes required to move the heavy bridge components into place, larger construction footprint that impedes on ROW or other facilities, and/or lack of a viable detour during an extended closure of SH 82 (see Section 5.5 for details). Considering these issues, traditional bridge construction phasing or a full closure of SH 82 (where the existing bridge is demolished and rebuilt with traffic on a detour) are the only feasible options.

Traffic Impacts During Construction

Total bridge closure during rehabilitation is impractical – this would be avoided with a two-phased construction approach.

How would traffic be handled during construction?

Existing traffic volumes are highest during morning and evening peak travel hours. The 2022 West End Traffic Study (Fox Tuttle 2022) estimated outbound (westbound) traffic at 1,000 to 1,250 vehicles per hour (vph) on SH 82 and 600 to 650 vph at Power Plant Road during the evening peak hours. No recent estimates of inbound (eastbound) traffic volumes in the morning peak hour are available; however, inbound traffic backups and congestion commonly occur on SH 82 between 7:00 a.m. and 9:00 a.m. during the weekdays. Considering the critical need to minimize traffic flow disruptions to and from the City, total bridge closure is impractical. Therefore, phased construction approaches were evaluated that would keep at least one lane open on the bridge during bridge rehabilitation or replacement, as summarized below.

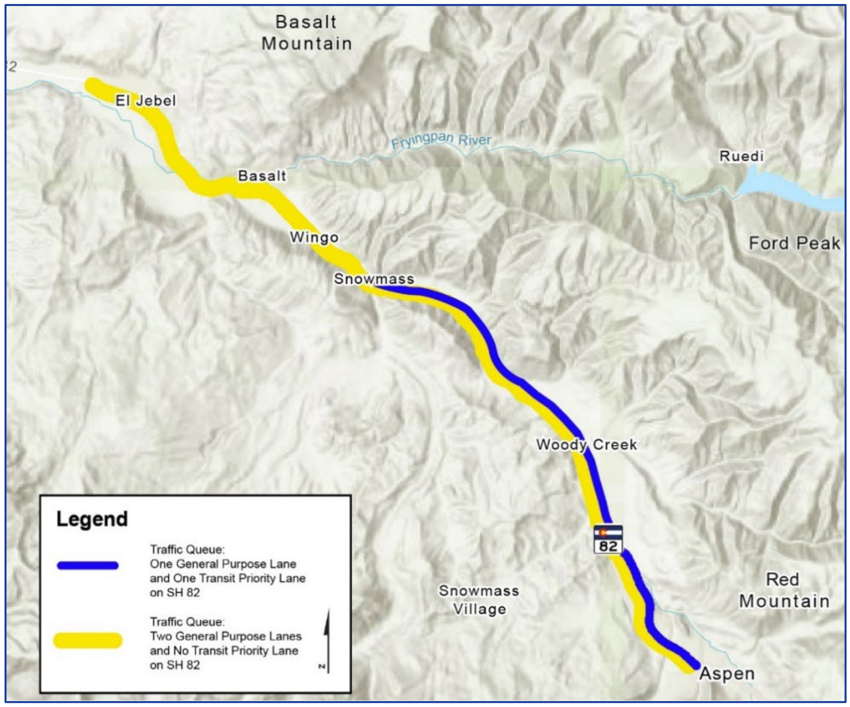
Bridge Rehabilitation and Two-Lane Bridge Replacement Alternative

Three options were considered to accommodate traffic during bridge rehabilitation or construction of a two-lane bridge replacement, as summarized below.

Alternating Single Lane phasing across the bridge would cause large traffic queues in the Inbound direction (reaching past Basalt, CO) and grid lock in the city.

- **Alternating Single Lane:** Temporary signals would be placed at each end of the bridge to operate an alternating-direction single lane on the bridge. While buses could be moved to the front of the queue where space permits, overall this option would result in substantial delays for transit and school buses and emergency response times. To accommodate emergency evacuations, outbound traffic would have right-of-way on the single lane; however, evacuation times would increase. Pedestrian access would be accommodated. As Figure 3 shows, this option would result in extremely long traffic queues and gridlock. Evacuation times also would be untenable and, therefore, this option was not deemed reasonable.

Figure 3. Alternating Single Lane Projected Traffic Queues



A West End Detour via Power Plant Road could be improved to serve as an outbound detour during construction.

Despite the improvements it would experience large travel delays and not be a reliable detour option.

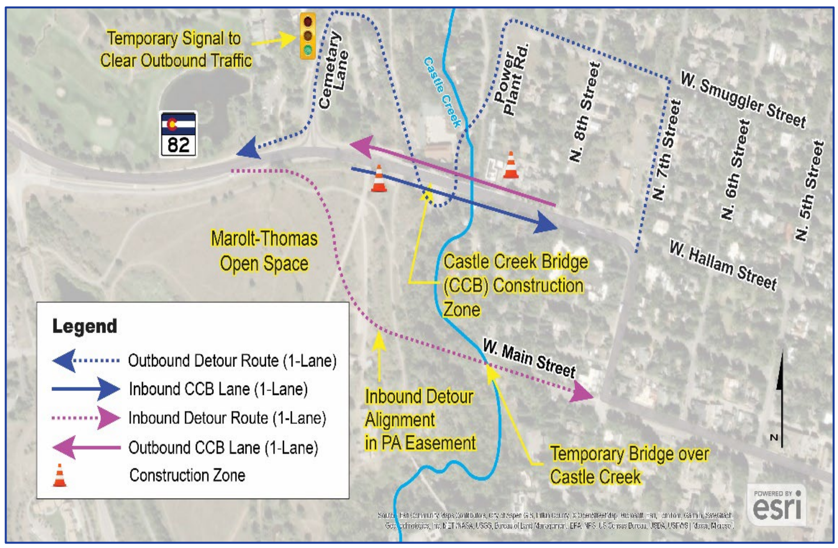
- **Inbound CCB Lane with Outbound Detour—West End Detour (Power Plant Road):** One lane of (outbound) traffic would detour down North 7th Street to West Smuggler Road and Power Plant Road while inbound traffic would use one lane over the bridge during phased construction (Figure 4). This option may require one-way movement and improvements to Power Plant Road to accommodate large vehicles and improve traffic capacity. Use of temporary signals and modifying existing signal cycles, as well as increasing bus service to the Brush Creek Intercept Lot, would be explored to improve traffic flow, however up to 5-hour travel delays would persist. Travelers accessing the hospital and high school or evacuating during emergencies would experience delays. Use of construction protocols such as transit and school bus priority on SH 82 and providing right-of-way to outbound traffic during an emergency evacuation would reduce travel delays for these vehicles and users. Also, both bridge construction or rehabilitation may require periodic closure of Power Plant Road, impacting the reliability of this detour.

The inbound detour across Marolt-Thomas is the most reliable option – minimizes travel delays, prioritizes transit services, provides continual safe pedestrian access, and doubles as evacuation route during construction.

If desired, this detour could also be evaluated for carrying two lanes of traffic (inbound and outbound), allowing for faster replacement or rehabilitation of the CCB.

- **Outbound CCB Lane with Inbound Detour—Temporary Detour Across Marolt-Thomas:** In this scenario, a temporary one lane detour route would be built along an existing transportation easement to split one lane of eastbound (inbound) traffic from SH 82 to the south across the Marolt-Thomas open space, span Castle Creek with a temporary bridge, and join SH 82 on West Main Street (Figure 4). This detour route could accommodate peak morning traffic volumes and maintain one lane on the CCB for westbound (outbound) peak evening traffic. Access to the hospital and high school would be similar to existing conditions. The outbound detour route would experience minor construction delays, but the inbound route would remain open during construction and experience no delays. The temporary detour could be removed after construction completion. For emergency evacuation, the inbound detour lane could be reversed and serve as outbound egress in conjunction with CCB outbound lane. This detour option would provide an additional evacuation route during construction and, if desired, the temporary bridge could remain in place for future evacuation needs. Safe pedestrian and bicycle traffic could be provided along this detour route.

Figure 4. Outbound and Inbound Detour Options During CCB Rehabilitation or Replacement



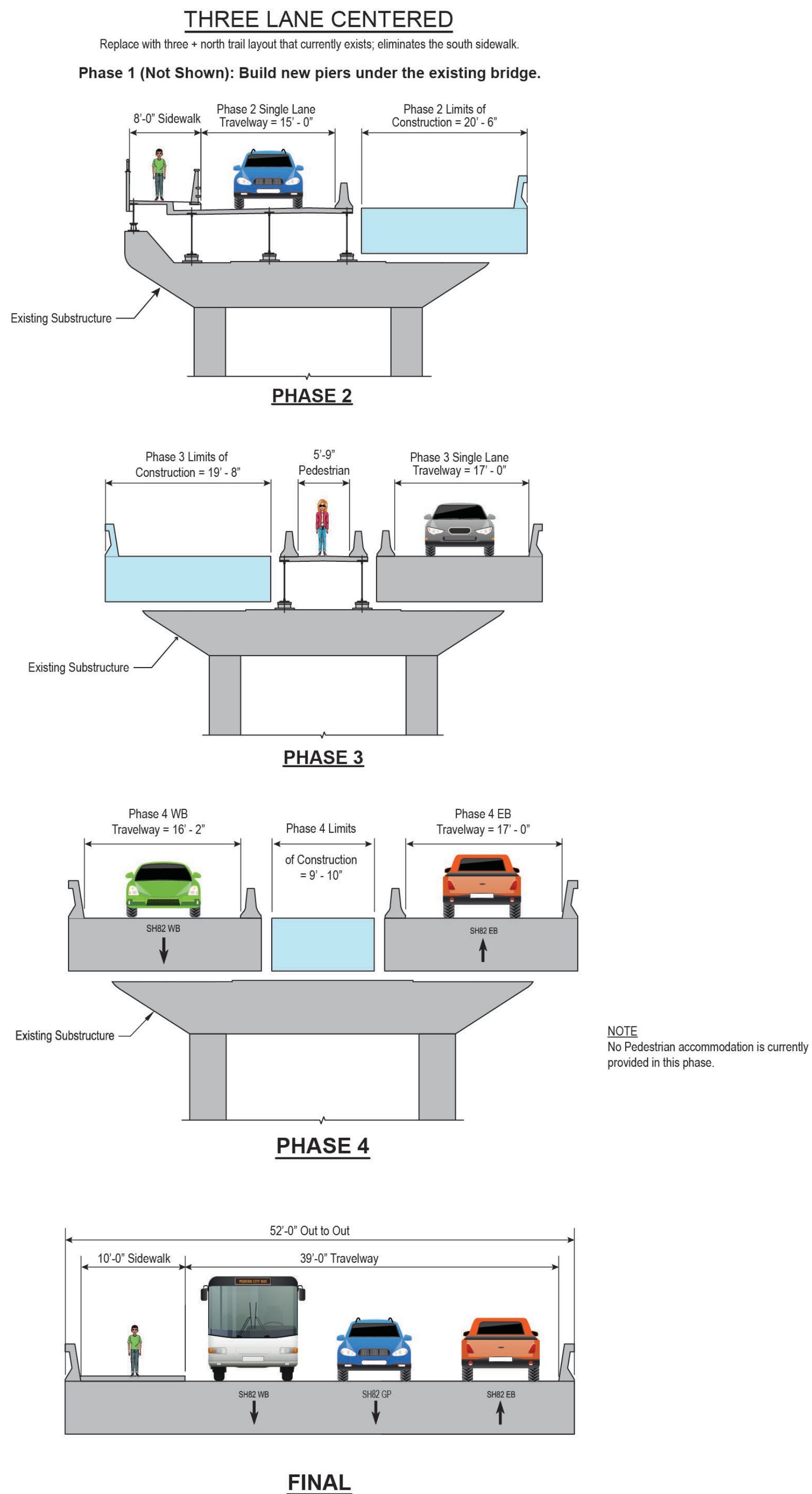
Three-Lane Bridge Replacement Alternative

Two options were evaluated that would use the open lane on the bridge in one direction and a companion detour in the other direction to accommodate traffic during construction. These options are summarized below and shown on Figure 4 (see Section 6.4 for details).

- **Centered (One-lane bridge during all construction phases with companion detour):** Under this option, the bridge would be optimally placed to minimize construction impacts. This option would provide a single lane of traffic on the bridge paired with an inbound detour, as described for the Bridge Rehabilitation and Two-Lane Bridge Replacement alternatives, and would result in similar traffic impacts during construction. Pedestrian access across the bridge would be maintained with the use of an outbound detour or diverted over to the inbound detour with minimal to no interruptions (see Section 6.4.1). Construction phasing for this option is shown on Figure 5.

- **Faster (One-Lane bridge during Phase 1):** This option would shift the bridge to the south to provide sufficient width to build two continuous lanes (inbound and outbound) that would be used during all construction phases except Phase 1. During Phase 1, a single outbound lane on the bridge in conjunction with an inbound detour would serve both directions of travel. As such, the detours and traffic impacts for Phase 1 would be the same as those described for the Bridge Rehabilitation and Two-Lane Bridge Replacement alternatives above. Traffic under all other phases would be similar to existing conditions, where both lanes would be converted to facilitate outbound flow during an evacuation event. Pedestrians would be rerouted under the bridge or over to the inbound detour for all phases, and pedestrian access would be impacted when construction impacts the path below the bridge (see Section 6.4.2).
- **Shifted (Two-lane bridge during all phases):** This option would require an overbuild of the replacement bridge. Two traffic lanes would be maintained during construction, resulting in minimal traffic impacts. Construction may constrain S-Curve traffic flow for short periods, but queues and delays would not be a noticeable change from existing conditions. Pedestrians would use the northern sidewalk until the final phase, during which they would be rerouted to Power Plant Road (see Section 6.4.3).

Figure 5. Three-Lane Centered Bridge Replacement



Existing Conditions in the Study Area

What potential environmental concerns are present in the study area?

An Environmental Assessment would be required to assess existing environmental conditions and potential impacts from bridge construction, operation, and maintenance.

A detailed assessment of the environmental setting of the study area was not conducted for this study. However, potential environmental concerns include effects to Castle Creek, wetlands, potentially hazardous materials (lead paint on bridge), recreational trails, open space, historic properties, and trees/vegetation. Construction activities could affect water quality and habitat for aquatic and terrestrial species in the short term, and trees and vegetation near the bridge would be impacted by bridge replacement alternatives. The CCB was deemed to not be eligible for the National Register of Historic Places. Long term impacts are not anticipated for the recreational trail, wildlife, air quality, water quality, archaeological, or paleontological resources. However, an environmental assessment would be required to assess existing environmental conditions and potential impacts from bridge construction, operation, and maintenance activities.

Costs

How much would bridge rehabilitation and bridge replacement alternatives cost?

Potential overall costs at this early feasibility stage are estimated at approximately $45 million for bridge rehabilitation, $69 million for a concrete Two-Lane Bridge Replacement, and $73 million for a concrete Three-Lane Centered Replacement Bridge. For more details, see Table 1 in this Executive Summary and Section 7.

Feasibility Analysis Summary

Bridge Rehabilitation Analysis

- Feasible rehabilitation measures to address current deterioration of steel and concrete bridge components (refer to Section 4.2) include replacing bearings, exterior girders, and bridge railing; rehabilitating steel protective coating; removing tack welds; and repairing pier columns and caps. Refer to Section 4.3.
- It is unlikely that rehabilitating the bridge would substantially improve its sufficiency rating, prolong its service life, or change its "functionally obsolete" classification. Refer to Sections 4.3 and 4.4.
- Bridge rehabilitation would require relocating several critical utilities running along the existing bridge that serve the City, which poses a considerable construction challenge. Refer to Section 4.5.
- Bridge rehabilitation would require closure of one lane of the existing bridge for approximately 4 to 6 months combined with the use of detour route, which

would temporarily impair traffic operations. Refer to Section 6.3 and Section 6.4.

Bridge Replacement

- It was determined that a four-span, post-tensioned, cast-in-place concrete girder bridge would be the most feasible option for a replacement bridge. Refer to Sections 5.2 and 5.3.
- Limited space is available to construct the new bridge because of the proximity of residential structures and prohibitive costs of potential right-of-way (ROW) requirements. Refer to Section 5.3.
- Phased construction would be required to maintain access to SH 82 during construction. Traffic would be reduced to one outbound lane over the bridge, and inbound detour also would be used. Pedestrian access across the bridge also would be limited during construction or rerouted along the inbound detour. Refer to Section 5.3 and Section 6.4
- The bridge replacement is estimated to take approximately 4 years of construction, working around the restrictions of major events and winter weather. Refer to Section 5.3.3.

Table 1 compares key features and elements of the bridge rehabilitation and replacement options. Color shading denotes how each option compares to others as follows: red (poor), yellow (fair), green (good).

Table 1. Bridge Feasibility Study Summary

Construction Issues	Rehabilitation	Two-Lane Bridge Replace	Three-Lane Bridge Centered
Maintenance of Traffic	• SH 82 remains accessible; traffic maintenance and inbound detour required. • Oversized loads (>14 feet) not accommodated on SH 82 CCB.	• SH 82 partially accessible; traffic maintenance and inbound detour required. • Oversized loads (>14 feet) not accommodated on SH 82 CCB.	• Same as Two-Lane Bridge Replace.
Traffic Travel Time Impacts	• Inbound detour (Marolt-Thomas), no substantial delay. • Outbound lane phased across existing CCB with delays similar to existing conditions.	• Inbound detour (Marolt-Thomas), no substantial delay. • Outbound lane phased across CCB with delays similar to existing conditions.	• Same as Two-Lane Bridge Replace.
Pedestrian and Bicycle Access	• Access via bridge provided during all phases	• Access via bridge not provided during all phases. • Access provided via reroute under bridge on existing trail or along Inbound detour (Marolt-Thomas).	• Same as Two-Lane Bridge Replace.

Construction Issues	Rehabilitation	Two-Lane Bridge Replace	Three-Lane Bridge Centered
Utilities (Gas, Fiber, Copper)	• No impacts to SH 82 traffic. • Utility relocation required.	• No impacts to SH 82 traffic. • Phase 1 duration extended to relocate utilities to new bridge.	• Same as Two-Lane Replace.
Schedule	• Construction period anticipated to be shorter than replacement alternatives. • Weather and summer event shutdown period restricts construction window, prolonging construction. • Construction completed in 2 years.	• Weather and summer event shutdown period restricts construction window, prolonging construction. • Longer construction period than rehabilitation alternative. • Construction completed in 4 years.	• Same as Two-Lane Bridge Replace.
Right-of-Way Impacts	• No ROW impacts anticipated. • Temporary construction easements (TCE) may be required for access.	• No ROW impacts anticipated for alternative shown in Appendix I. • ROW limits restrict construction north/south of existing bridge. • Temporary construction easements (TCE) may be required for access.	• No ROW impacts anticipated for alternative shown in Appendix J. • ROW limit restrictions and TCEs for access same as Two-Lane Bridge Replace.
Constructability	• Minimal impact to Power Plant Road and facilities under bridge. • Crane locations for girder erection are challenging around the existing facilities below the bridge.	• Falsework would accommodate facilities under bridge. • Facilities under bridge and nearby residences restrict construction method options.	• Same as Two-Lane Bridge Replace.
Enables Transit Priority and Future Transit	• Provides bus transit in existing general traffic lanes. • Cannot handle future Light Rail Transit (LRT) loads.	• Provides bus transit in new general traffic lanes. • Cannot handle future Light Rail Transit (LRT) loads.	• Provides bus transit priority lane in outbound direction. • Designed for future Light Rail Transit (LRT) loads.

Construction Issues	Rehabilitation	Two-Lane Bridge Replace	Three-Lane Bridge Centered
Bridge Service Life	• No substantial service life extension. • Would remain functionally obsolete for roadway width. • Would not meet current design code requirements.	• 75-year service life with standard bridge maintenance. • Would meet current design code requirements. • Future widening of bridge to accommodate future traffic and transit demands would be challenging.	• 75-year service life with standard bridge maintenance. • Would meet current design code requirements. • Would accommodate future traffic and transit demands.
Overall Project Costs (2024)*	$44 million	$69 million	$73 million
• Construction Costs	63%	62%	62%
• Planning and Design	8%	12%	13%
• ROW/TCE's	10%	7%	6%
• Construction Management/PI	18%	19%	19%

* See Section 7 for explanation of Overall Project Costs, including structural bridge costs, which are detailed in Section 4.7 and Section 5.4.

^percentage of total costs

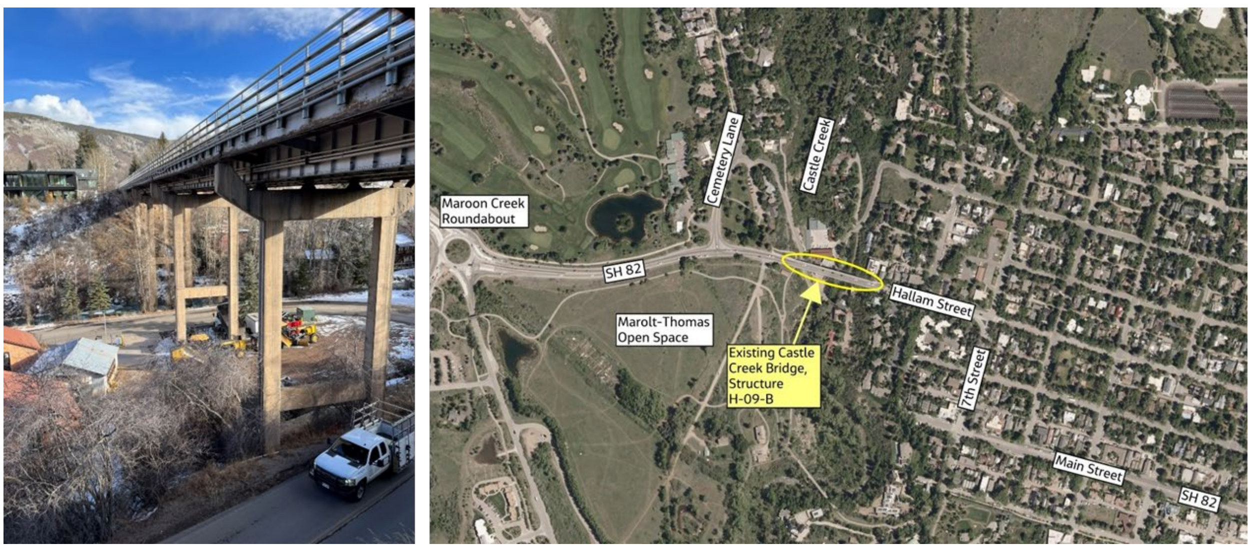
2. Site Description and Design Features

This section summarizes the site description and design features of CCB.

2.1 Existing Structure

CCB is a 5-span riveted steel plate girder continuous bridge that carries SH 82 over Castle Creek and Power Plant Road. Originally constructed in 1961, the superstructure is made of a reinforced concrete deck resting on top of steel girders. The bridge is 423.6 feet long and 40 feet wide (out-to-out), with two vehicular lanes and sidewalks on each side. The bridge uses steel plate girders under the vehicular portion of the deck and rolled steel wide flange girders to provide the main support for each sidewalk. Steel girders are supported by rocker bearings at Abutment 1 (West) and Piers 2, 3, 4, and 5. Pinned bearings support steel girders at Abutment 6 (East). There is a new approach slab with a modular expansion joint constructed in 2022 to replace the original failed backer rod–type expansion joint at Abutment 1 (West). The superstructure is supported by reinforced concrete piers resting on spread footings 3 feet thick, 8 feet wide, and 12 feet long. The piers have tapered columns and vary in height: Pier 2 stands at 55 feet, Pier 3 at 63 feet, and Pier 4 and Pier 5 at 68 and 40 feet, respectively.

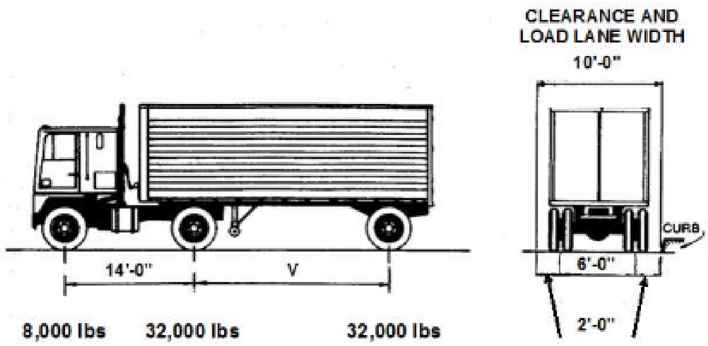
Figure 6. Existing Bridge



The current structure was designed to withstand H20-S16-44 vehicular live loading, which was renamed to HS20-44 loading after 1965. The H20-S16-44 designation indicates the vehicle tractor axles (two axles) combined are 20 tons, with semi-trailer weight of 16 tons, as published in 1944. Combined, the gross vehicle weight is 36 tons, as shown on

Figure 7. At the time of this bridge design, code required design for either a design truck or design lane load, which simulates a series of trucks. After 1993, the code changed to requiring design toward a design truck combined with a design lane load.

Figure 7. American Association of State Highway and Transportation Officials H20-S16-44 Truck



The existing Castle Creek Bridge is a 63-year-old steel bridge on concrete supports. The bridge was designed for vehicular loading less than today's American Association of State Highway and Transportation (AASHTO) standard code requirements for a design life of 50 years.

2.2 Traffic Detours

The detour length recorded in the National Bridge Inventory (NBI) for CCB is 0.6 mile. However, it involves vehicles descending into the Castle Creek area via Power Plant Road. This detour cannot accommodate present traffic volumes, impacting travel times and emergency response when used. Also, the detour route does not meet the current requirements of transit vehicles (buses) and oversize vehicles on SH 82. The length of this alternative route affects the sufficiency rating of the current bridge based on the Colorado Structure Element Level Coding Guide evaluation system conducted by the Colorado Department of Transportation (CDOT). Generally, when work is being performed on SH 82 in the bridge area, transit traffic receives priority through traffic management. For additional information related to the existing detour and potential alternative detours during construction, refer to Section 6.

Power Plant Road is currently the only detour for SH82 (Hallam Street). Roadway improvements would be required on Power Plant Road if it were relied upon as the detour route during bridge construction. Alternatively, a separate detour route could be constructed to accommodate traffic during construction.

2.3 Utilities

Being the singular linkage between Aspen and other towns in the Roaring Fork Valley, CCB accommodates several utilities essential for the operational support of Aspen. A complex network of City fiber optic lines run under the bridge. The 96-strand cables run directly along the north side of the bridge, connecting to a utility cabinet above the bike path on the west side at Abutment 1. These fibers play a pivotal role in networking and connectivity for City and county facilities, including the 911 dispatch center. The cables have minimal slack available to accommodate movement or relocation. Relocation of these communication lines will be part of the project cost.

A communication provider, Comcast, also runs their fiber infrastructure along the bridge, and any disruption to this service could lead to a loss of all services for the downtown and surrounding areas. Comcast has a 96-strand cable and 72-strand cable, for which a relocation from manhole to manhole could be a significant relocation.

Lumen Technologies, Inc., runs six conduits carrying both copper cables and fiber optics along the bridge for communications, with critical circuits that cannot be removed or disrupted. Ting, Inc., also leases fiber lines from Lumen Technologies, Inc., as a communications provider. Any relocation of these conduits would run vault to vault; however, there is currently no slack in the copper lines to easily accommodate that relocation. Relocation requires new 8-way duct, with an anticipated duration of 8 weeks for relocation.

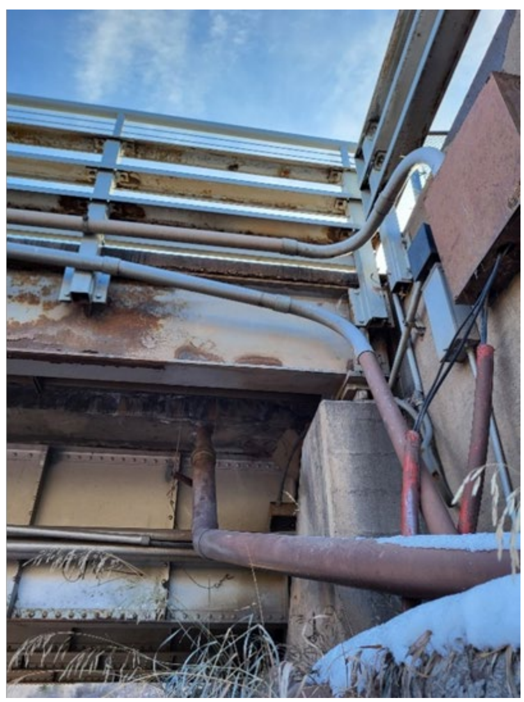
While no gas lines are directly attached to the bridge superstructure, a partially exposed high pressure gas line runs immediately in front of the west Abutment 1, the main gas feed to the City. This line will likely require relocation to ensure safety during construction. Also, a steel gas line is under Power Plant Road below the bridge. A gas regulator station is south of the west end of the bridge, with high pressure gas in/out of the facility.

Near the west approach to the bridge, CDOT has a weather sensor puck on the north side of SH 82. Any relocation of this weather sensor puck would be part of the project cost.

Finally, the Aspen Sanitation District notes an 8-inch diameter polyvinyl chloride (PVC) main sewer line under the bridge at the valley bottom in Harbour Lane in between Piers 4 and 5, susceptible to potential impacts from modifications to Harbour Lane or Power Plant Road. In addition, there are other sewer mains at either end of the bridge that may be affected by modifications to the bridge approaches.

For costs associated with the utility relocations, refer to Section 7.

Figure 8. Utilities Along Bridge and Connection at Abutment 1



Overall, existing utilities constitute a complex and interconnected web of service to the community that demands careful consideration in any construction or modification efforts on the existing CCB. Any bridge work impacting the support for the existing utilities will require relocation to another location on the bridge or to a separate temporary support structure for the utilities.

2.4 Geotechnical Summary

A geotechnical field investigation has not been conducted at this conceptual stage of the project. Boring information provided on the existing bridge as-builts (CDOH 1954) show sand and gravel is present between the existing grade and the bottom of the existing footings. It is anticipated deep foundations would be the proposed foundation type for the replacement alternatives considered. Drilled shafts and driven piles are commonly used on CDOT projects throughout the state. Deep foundations have the benefit of requiring less area for their construction compared to spread footing, which would be helpful in reducing impacts to the facilities and residents under the bridge. Deep foundations are also beneficial near waterways such as Castle Creek, mitigating instances of undermining a shallow foundation from water movement.

The rehabilitation work discussed in this report would not require work on or around the existing footings, and therefore, the existing soil conditions are not of concern.

2.5 Hydraulic Summary

No hydraulic report is available at this stage of the project. The replacement alternatives would remove the existing Pier 4 from Castle Creek to eliminate obstructions to the waterway. Free board is not a concern because of the height of the superstructure. Work within Castle Creek would be required to remove the existing pier, initiating a Section 404 permit for Waters of the U.S. and Wetlands.

The rehabilitation work discussed in this report would not affect the piers or foundation elements of the bridge. Therefore, hydraulics is not a concern for the rehabilitation alternatives. Scour does not present a concern based on the visual inspection.

2.6 Environmental Concerns

Investigation of environmental constraints and concerns was not conducted for this report. This section highlights known or potential environmental issues based on field observation.

Construction for both the rehabilitation and replacement alternatives would take place above Castle Creek with some work within the creek, likely requiring a Section 404 permit for temporary impacts to Waters of the U.S. A wetland delineation in the project would be conducted to further assess any impacts.

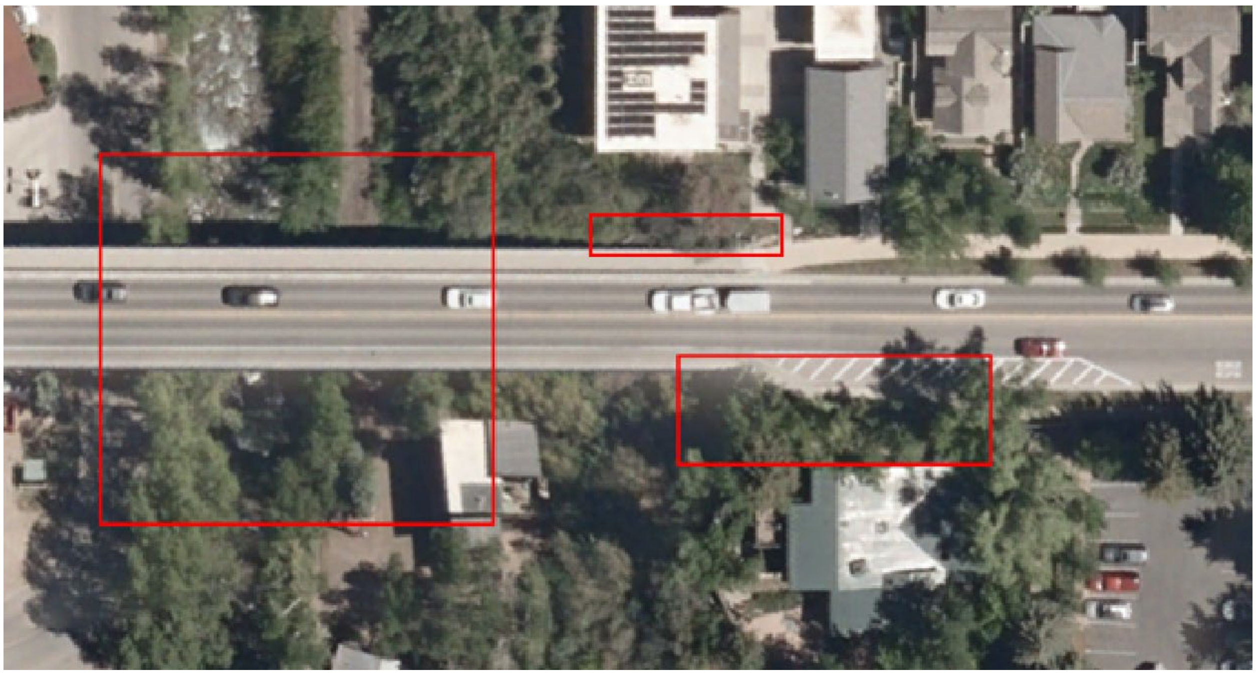
Replacement work associated with removing and replacing piers in the Aspen Streets Department parking lot would be near the fuel station and its associated storage tank. This could require hazardous material investigation before any subsurface work and careful consideration toward placing any foundation elements outside of any zones with hazardous materials present.

Because of the age of the bridge, lead paint may be present. Sample testing will be needed before any construction activities involving the existing steel girders.

During construction, temporary impacts are expected for the recreation trail near the west abutment underneath the bridge. Permanent impacts to the trail are not anticipated, but the temporary impacts would require evaluation.

Trees and surrounding vegetation may be impacted because of the bridge replacement alternatives. Trees near the northern and southern edge of the existing bridge at the east abutment would be near the proposed edge of deck for some of the phasing options considered in Section 5.3.2, as shown on Figure 9. Trees along the banks of Castle Creek, near Power Plant Road and Harbour Lane, may also be impacted during construction from the installation of new piers and falsework. Impacted trees may require removal or relocation and would need to be coordinated with the City.

Figure 9. Potential Tree Impact Areas



The bridge was reviewed in a CDOT-prepared statewide inventory of historic bridges and deemed to not be eligible to the National Register of Historic Places.

Short-term air quality impacts, including greenhouse gas impacts, would result from bridge construction. Impacts generally would be proportional to traffic delays and queues, with the highest increase in emissions caused by queuing traffic.

Longer term, air quality emissions from the three-lane bridge rehabilitation options are expected to be lower than two-lane options because of reduced congestion from three-lane options, thereby reducing emissions from queued traffic. The three lane options are not expected to induce travel demand (and higher emissions) because of the transportation management measures in place on the SH 82 corridor.

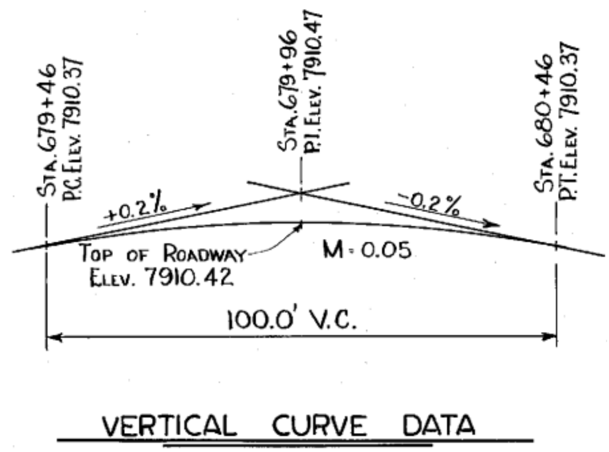
Short term impacts associated with construction activities could affect water quality and habitat for aquatic and terrestrial species. Long term impacts are not anticipated for wildlife, air quality, water quality, archaeological, or paleontological resources. However, environmental assessment would be required for any bridge action to move forward.

2.7 Roadway Design Features

The existing bridge carries SH 82 over Castle Creek, Power Plant Road, and Harbour Lane. Two sidewalks are 8 feet wide at the northern edge of the bridge and 5 feet wide at the southern edge. A project in 2018 widened the northern sidewalk from 5 feet and added a traffic barrier adjacent to the roadway. The roadway is 27 feet wide from curb to curb and accommodates two 11-foot lanes and two 2-foot-6-inch shoulders. The deck has a crowned cross-slope of 1% +/- that slopes away from the centerline of the roadway (City of Aspen 2017).

The bridge is built along a tangent section of the SH 82 alignment; however, beyond the bridge limits, there are horizontal curves. A 100-foot-long vertical curve at the center of the bridge alignment within Span 3 begins and ends on the bridge. The tangent that extends to the west has a slope of 0.2%, and the tangent to the east has a slope of 0.2% (CDOH 1954).

Figure 10. SH 82 Existing Profile



3. Structural Design Criteria

This section summarizes the structural design criteria for CCB.

3.1 Design Specification and Criteria

The bridge replacement alternatives considered in this report would be designed per the latest AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO 2020) and the CDOT Bridge Design Manual (BDM) (2023a). In considering rehabilitation, upgrading the bridge to meet current code might be cost prohibitive. Usually, the AASHTO Load Factor Design methodology serves as an alternative; however, discussions with the bridge owner are essential in this context.

Jacobs discussed with City staff the potential of the bridge alternatives carrying a future light rail guideway in and out of Aspen. The conceptual design does not preclude a transit component in the future, accommodating for light rail transit in the evaluation. The Regional Transportation District (RTD) Light Rail Facility Design Guidelines and Criteria (RTD 2018) was used for additional loading and design requirements because the Roaring Fork Transportation Authority does not currently have separate light rail design requirements to reference.

3.2 Loading

For a bridge replacement, LRFD would be used for the bridge design and other structural items such as retaining walls. This is the current design approach specified in the CDOT BDM (2023) and a CDOT technical memorandum dated December 7, 1998. HL-93 and permit live load vehicles, in addition to contributing dead loads and pedestrian loads, would be calculated for LRFD load combinations. The bridge elements will be designed for the applicable service, strength, and extreme limit states. If the bridge alternatives are required to carry light rail traffic in the future, additional live load cases would need to be considered to include the RTD Light Rail Vehicle.

Over time, AASHTO and CDOT have increased the design vehicle loadings to accommodate heavier vehicles that use roadways as compared to the original interstate system. Any rehabilitation will maintain the H20-S16-44 as the design vehicle for the bridge because the existing bridge will not provide sufficient capacity for the additional loading of an HL-93 vehicle. This same loading limitation was noted in the SH 82 Reversible Lane Feasibility Study (SGM 2008) done over 15 years ago before the HL-93 vehicle introduced in today's code.

3.3 Aesthetic Requirements

Aesthetic guidelines have not been established at this time. Should any replacement effort advance to preliminary design, aesthetic features can be incorporated into the design by Jacobs as required by the City, CDOT, and other involved parties.

Bridge replacement alternatives are designed in compliance with current design codes. A bridge rehabilitation cannot be upgraded to meet current design codes without significant cost implications. Over the remaining service life of the rehabilitated bridge, heavier vehicles introduced to the roadway system may be limited on this route.

4. Bridge Rehabilitation Feasibility

This section summarizes the feasibility of bridge rehabilitation.

4.1 Bridge Condition Assessment

Recent inspections of the bridge have highlighted areas of concern, indicating signs of wear, major deterioration in several girders, and localized structural concerns. Routine inspection carried out by CDOT in September 2022 (CDOT 2022) assigned a sufficiency rating of 50.3, which is a rating procedure with a numeric value ranging from 0 to 100 indicative of bridge sufficiency to remain in service. A bridge's sufficiency rating is a comprehensive assessment that considers factors such as structural condition, load rating, traffic data, and public importance. Calculated using a formula outlined by the Federal Highway Administration, the rating reflects the bridge's ability to remain in service and compares the existing bridge to a new one meeting current engineering standards. The same assessment also designated the bridge as functionally obsolete, meaning the deck geometry, load-carrying capacity, clearance, or approach roadway alignment no longer meet the current standards for the highway system of which the bridge is an integral part. Refer to Appendix A for CDOT's inspection report.

The sufficiency rating also considers the load rating of the bridge structure. All structures require a load rating defining their long term high frequency live load (traffic) capacity. The NBI rating for the CCB structure is 24.6 tons. The minimum inventory load rating goal for any structure on a state highway is 36 tons.

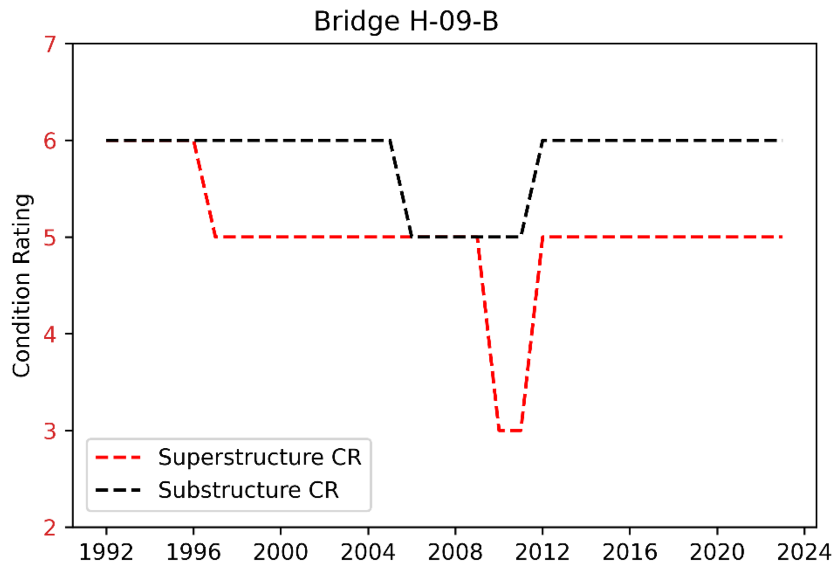
Each element of a bridge is coded during a bridge inspection, from 0 to 9 based on their condition state within NBI Standards. The code is dependent upon the defect location, frequency, and condition.

Table 2. National Bridge Inventory Standard Coding

Condition Code	Description	Current Code for CCB Major Elements
7-9	"Good": From Good to Excellent	
5-6	"Fair": From Fair to Satisfactory	Deck (6), Superstructure (5), Substructure (6)
0-4	"Poor": From Failed to Poor	

Figure 11 illustrates superstructure and substructure condition code history of the bridge based on the NBI database (NBI 2024). During a 2009 inspection, (CDOT 2009) a decline in the superstructure condition code to 3 ("Poor") was noted, necessitating immediate attention. According to CDOT records, extensive repairs and rehabilitation efforts were implemented on the bridge in 2011 to improve the condition code of the bridge. Despite these substantial rehabilitation efforts, they were only sufficient to elevate the superstructure to a "Fair" code.

Figure 11. Condition Rating History of the Bridge



Source: NBI 2024

It should be noted Figure 11 does not include the deck element, which has maintained a code of 6 for over 30 years. Because the deck has an asphalt overlay, inspectors can usually only assess the deck condition from the underside, meaning some issues may be covered by the overlay. City staff have confirmed deck repairs were performed during the 2018 project that milled off the existing overlay to place a new waterproofing membrane and overlay. This project uncovered some areas requiring full depth deck repair and additional reinforcing where corrosion or impact was noted. Replacing the overlay was challenging because of inconsistencies with the existing bridge deck surface.

From November 28 to November 30, 2023, a team of two inspectors used an Under Bridge Inspection Truck to conduct an arm's length inspection of the steel superstructure. The "In-depth Superstructure Investigation Report" (eO 2023) by Engineering Operations, LLC (eO), produced a comprehensive steel superstructure inspection that confirmed CCB's (H-09-B) superstructure is in fair condition, substantiating an NBI Item 59 rating of 5 per CDOT's 2022 inspection. The full inspection report is provided in Appendix B.

4.2 Summary of Field Inspection

This section summarizes the findings of the bridge inspection carried out by eO in November 2023 and the findings of the routine bridge inspection by CDOT in September 2022. While CDOT's inspection covered other bridge elements, including the concrete deck, columns, abutments, pier caps, protective coatings, slope protection, joints, bearings, wing walls, sidewalks, and railing, eO's primary focus was on the six steel girders. This section details the findings, including defect locations, severity, and quantities.

In summary, this bridge shows significant signs of deterioration in all the areas typical of a bridge of this age. Although the concrete deck has recently undergone rehabilitation in 2018, a mill and overlay with deck repairs may be warranted once again. The exterior girders are also in need of replacement, which could coincide with the deck repairs. To preserve the life of the bearings and abutments, the expansion joints should be replaced. The bearings at the abutments need to be replaced, and bearings at the piers need additional rehab. The bridge's concrete substructure shows significant signs of deterioration and in several locations requires immediate attention to prevent further overall damage or load carrying capacity.

4.2.1 Concrete Deck and Asphalt Overlay

The concrete deck totals 16,945 square feet and shows moderate signs of degradation with heavy map cracking, efflorescence, and spalling. Transverse cracks with efflorescence, rust staining, and map cracking are widespread throughout the underside of the deck. Specific crack quantities with efflorescence and rust are outlined in the 2022 CDOT Structure Inspection and Inventory Report, indicating the need for a closer inspection and potential repairs.

Figure 12. Deck Concrete Spall with Exposed Rebar

In conjunction with a new asphalt overlay, deck repair was performed on the bridge in 2018, which repaired several inspection and maintenance items. The wheel rutting in the overlay noted during the November 2023 inspection can lead to degradation of the deck over time if not properly maintained. A well-performing overlay system is the best defense for maintaining the integrity of the concrete deck underneath and extending its service life. The type of overlay is also important to enhancing the deck's resistance to corrosion. Cementitious and non-cementitious wearing surfaces are available. The concrete deck issues noted will need to be monitored regularly to confirm the 2018 deck repair effort stopped or significantly slowed down the observed deterioration of the underside of the deck. If it is determined deck repairs need to be performed once again, the deck repairs would follow the typical protocol CDOT uses for this situation.

Infiltration of chloride ions into concrete is the most common cause of corrosion initiating in reinforcing steel. Chloride exposure is primarily through the application of deicing salts, such as magnesium chloride. Deck repair and patching or the installation of new membranes and overlays must begin by first identifying the extent of chloride contamination of the deck. Chloride testing consists of taking cores of the concrete deck and analyzing them for chlorides. Depending on the results of the testing, future deck rehabilitation may consist of the replacement of all chloride-contaminated concrete with sound concrete, along with the replacement of the membrane and wearing surface. It may also consist of the installation of a barrier-type overlay on the deck or full replacement of the deck. The test process may significantly impact traffic on the bridge when samples are taken. In addition, the potential deck rehabilitation effort will impact traffic on the bridge and may include full closer of the bridge to complete the repair work.

Figure 13. Deck Concrete Spall with Exposed Rebar

Depending on the chloride testing results of the concrete deck, a new overlay system along with localized deck repairs may be required, or if the contamination is widespread, a full deck replacement may be needed. Full deck replacement would provide the greatest mitigation for corrosion and degradation of the deck. However, it is also the most intrusive activity regarding construction requirements. The bridge would require full closure to replace the deck in its entirety. In the future, if it's determined a full deck replacement is needed, it is recommended a full bridge replacement be considered, given the complicated nature of work involved with a full deck replacement.

4.2.2 Steel Girders

The steel girders show varying degrees of corrosion. Exterior rolled steel wide flange girders (North exterior Girder A and South exterior Girder F) under each sidewalk show significant corrosion on the bottom flanges and lower webs (refer to Figure 14). Several locations along the girder also have severe localized corrosion in the top part of the web. On average, exterior girders show 20% section loss in the web. Some localized areas show up to 40% section loss. Both exterior girders show a visible sag with approximately 3 inches of downward displacement at mid-span locations (refer to Figure 15).

Compared to the exterior girders, the interior girders (B through E) have less corrosion. Surface corrosion and minor pitting are observed at these girders, especially at piers under the deck joints (refer to Figure 16). Girder ends at the abutments exhibited corrosion with negligible section loss.

Figure 14. Significant Corrosion in Web and Top of Bottom Flange of North Exterior Girder A

Figure 15. Girder F Sagging Near Mid-span

Figure 16. Typical Surface Corrosion of Interior Girder – Girder E South Face at Pier 4

4.2.3 Girder Stiffener and Tack Welds

Tack welds, located at multiple stiffener locations for fit-up during the construction phase, have been inspected and classified based on crack propagation into self-arrested and not self-arrested (NSA) categories. Of particular concern are the NSA tack welds because their potential for crack migration into the girder base metal warrants focused attention. The extensive inspection performed by eO (2023) encompassed an estimated 3850 tack welds, revealing a distribution of 415 self-arrested cracks and 36 NSA cracks. Additionally, one specific instance of rivet shearing was identified.

One stiffener in Girder B exhibited a noteworthy 1-inch out of plane deflection, emphasizing the necessity for a more in-depth structural evaluation at this stiffener location (refer to Figure 17). Stiffeners in exterior girders follow the same corrosion pattern as that of exterior girders as described in an earlier section. Notably, the vertical web bearing stiffener of Girder F at Abutment 6 has 100% section loss (2-inch diameter hole) at the bottom of the stiffener (refer to Figure 18). The bearing stiffeners of the interior plate girders, located at both the abutments and piers, are constructed using double back-to-back angles. They exhibit pack rust between the faying surfaces with a thickness of up to 1/2 inch. This rust has caused bowing in the stiffener legs at various points, as illustrated on Figure 19. While section loss in these regions is minimal, the accumulation of pack rust poses a potential concern over time. The continued presence of pack rust can induce separation between the angles, leading to further distortion of the angle shape.

Figure 17. Deflection of Stiffener at North Face of Girder B

Figure 18. Section Loss in Base of Bearing Stiffener – Girder F at Abutment 6

Figure 19. Pack Rust Between Bearing Stiffeners of Interior Girder at Abutment 1

4.2.4 Steel Protective Coating

The protective coating on the steel elements has failed in areas because of corrosion, indicating a need for prompt attention. Approximately 80% of coating has deteriorated. This deterioration contributes to the accelerated corrosion of the steel components, emphasizing the urgency of addressing protective coating issues.

Figure 20 Failure of Protective Coating - Typical on All Steel Sections

4.2.5 Bearings

The primary girder bearings display surface corrosion without measurable section loss, with some bearings surrounded by soil and debris specifically at Abutment 1 (refer to Figure 21). Loose anchor bolt nuts are observed at all bearings, and pin bolts are backing out at Bearings 1C, 1D, 3E, and 3F. Furthermore, certain fixed bearings on Abutment 6 exhibit surface corrosion, with the grout pad breaking up under several bearings. It is imperative to address these issues urgently to prevent further deterioration and unintended movement. Overall, the bearing condition is generally fair, with surface corrosion and identified problems with anchor bolts. The interior girder bearings, especially the rocker bearings at Abutment 1, are in the expansion position, which is opposite of what is expected in the colder weather conditions during the inspection in November.

The abutment bearings on both Abutment 1 and Abutment 6 exhibit corrosion-related issues, including flaking and minor section loss. Debris accumulation around the rocker bearings on Abutment 1 is a concern because it can impede movement, trap moisture, and reduce the bearing assembly's lifespan. Abutment 6 experiences surface corrosion on all bearings, with additional problems such as a broken grout pad on Girder 6B and deteriorating grout pads under Bearings 6C to 6E. Despite previous rehabilitation efforts in 2011, the grout pad below Bearing F is broken, with significant bearing loss. The fixed bearing (6F) is beginning to tip longitudinally, as shown on Figure 22. The bearing pedestal for Bearing 6A has a significant vertical crack stemming from the anchor bolts. This crack has propagated through the bearing pedestal and created a large section of delaminated concrete. Immediate attention is necessary to address these structural concerns.

Figure 21. Rocker Bearing Covered in Dirt – Typical at Abutment 1

Figure 22. Movement of Bearing 6F at Abutment 6

Figure 23. Impending Spall in the Bearing Pedestal at Bearing 6A at Abutment 6

Figure 24. Loose Anchor Bolt Nuts – Typical at All Bearings

4.2.6 Diaphragms

The steel diaphragms exhibit satisfactory overall condition, except for surface corrosion identified in the C-Channel diaphragms at the piers. Notably, the C-Channel diaphragms between the exterior and interior girders at piers show corrosion with 10% to 30% section loss of the webs.

Figure 25. Surface Corrosion of C-Channel Diaphragms – Typical at All Diaphragms

4.2.7 Pier Caps

Pier caps exhibit moderate to heavy water staining, light scaling, delamination, and various severity of cracks. Specific issues include a 4-square-foot spall with exposed, corroded rebar on Pier 2, rear face under Girder E. Pier 3 cap shows delamination and cracks with efflorescence, while Pier 4 cap displays delamination, shallow spalls, and horizontal and diagonal cracking below Bay 3C. Pier 5 cap is starting to delaminate on the right side under Bay 4D.

Figure 26. Exposed Corroded Rebar on Pier Cap at Pier 2

Figure 27. Light Scale Cracking at Pier Cap – Typical at All Pier Caps

4.2.8 Abutments

Abutment 1 is covered in debris from previous rehabilitation projects and possibly from an effort to cover the high-pressure gas line. A portion of the backwall appears to have been removed approximately 3 feet from the top during previous construction projects, then recasted to a thinner section thickness. Vertical rebars along the front face of the existing backwall have been cut and are exposed at some locations. Abutment 6 has some light scale, delamination, and water staining.

Figure 28. A Portion of Abutment 1 Backwall Was Removed During Previous Construction

Figure 29. Light Scale, Delamination and Water Staining at Abutment 6

4.2.9 Expansion Joints

Observing inadequate joint sealing at Abutment 6 raises concerns about the bridge's structural performance. Despite the presence of fixed bearings at this abutment, notable movement of the bridge has been observed through the existing conditions of the bearings, as shown on Figure 22 (Section 4.2.5). One plausible explanation is the potential impact of the partially buried rocker bearings at Abutment 1. The bearings at Abutment 1 were intended to absorb thermal movements of the bridge, whereas the bearings at Abutment 6 were intended to remain stationary on the abutment seat. It is possible the partially buried bearings at Abutment 1 may be restricting the performance of the rocker bearings, thereby contributing to unintentional movement at the fixed end of Abutment 6. Another factor under consideration is the relocation of the expansion joint from the backwall of Abutment 1 completed in late 2022, which may be contributing to the observed issues. A comprehensive analysis is needed to identify the root cause of the unexpected movement behavior of the bridge bearings and determine the most effective remedial measures.

In addition to the structural concerns, the inadequacy of the joint seal at Abutment 6 has exacerbated the situation. Water infiltration from the pavement to the abutment has been observed, resulting in deterioration of the concrete below. If left unaddressed, this issue could lead to significant deterioration of the concrete and bearings in the future.

Figure 30. Inadequate Joint Seal at Abutment 6

4.2.10 Slope Protection

According to the CDOT inspection report for 2022, maintenance personnel in Aspen have reported the need to annually adjust or reposition the timber wall supporting the bike path to a vertical orientation at Abutment 1. This recurring issue has prompted the City to initiate a rehabilitation project aimed at addressing the structural concerns associated with the timber wall. Mitigation of the retaining wall and bike path helps the continued protection of Abutment 1.

Figure 31. Movement of Timber Retaining Wall at Abutment 1

4.3 Rehabilitation Recommendations

The bridge inspections (CDOT 2022; eO 2023) revealed significant deterioration in various elements that require immediate attention to enhance the long-term durability, functionality, and safety of the bridge. However, it is crucial for the owner, users, and local community to understand rehabilitation is a substantial effort. Further, while rehabilitation can address certain issues, it may not be a cure-all for every issue linked to the bridge.

The following work is highly recommended to rehabilitate the bridge:

1. Bearing Replacement and Maintenance:

   It is important to completely replace the bearings at Abutments 1 and 6. This measure ensures the restoration of proper load distribution and minimizes structural stress from thermal movements of the bridge. Cleaning and repainting all pier bearings prevents further corrosion and can help extend the lifespan. The replacement or insertion of missing nuts, pins, and bolts, along with grout pad replacement where necessary, enhances the overall stability and performance of the bridge. And finally, cleaning the bearing seats is an easy way to prevent moisture buildup and debris from preventing the bearings to function as designed.

2. New Bearing Pedestal at Abutment 6:

   Replacing the cracked north side exterior girder bearing pedestal at Abutment 6 is crucial to ensure the bridge's stability and load-carrying capacity of the sidewalk.

3. Exterior Girder Replacement:

   The extensive section loss in both exterior girders warrants replacement. This action not only restores the load-bearing capacity of the bridge but also ensures the elimination of compromised elements, safeguarding against potential structural failures. Replacing the exterior girders would also trigger the replacement of the sidewalks above and potentially the bridge railing.

4. Steel Protective Coating Rehabilitation:

   A durable high-performance coating is recommended to protect the steel elements from corrosion. It is apparent the existing paint is at or near its design life for the structure. The existing protective coating has failed on multiple bridge elements. Reapplication of protective paint is essential to prevent further corrosion. Protective paint provides a barrier against environmental factors, such as corrosion and preserves the integrity of the steel components.

5. Tack Weld Removal and Monitoring:

   Removal of cracked tack welds, especially those not considered self-arrested, is crucial for eliminating potential weak points in the structure. If funds allow, removing all tack welds from the structure avoids future close monitoring at higher frequencies. If it is not economically viable to remove all tack welds, it is recommended for continuous monitoring of the welds to be carried out during routine inspections to ensure timely identification and management of any emerging issues.

6. Concrete Deck and Asphalt Overlay:

   Addressing spalls, cracks, and delamination through concrete deck repairs is vital for maintaining the bridge's overall long term structural integrity. The initial step would be to determine the condition of the deck through chloride testing to determine the extents of repair required. Assuming a new overlay and deck repair activities, the Contractor would chain drag the deck and mark locations that are delaminated. These locations would then receive a Class 2 or Class 3 deck repair depending on the severity of the degradation. Then a thin polyester polymer concrete (PPC) overlay would be constructed over the repaired deck to provide a barrier against chloride infusion. The new PPC overlay would replace the current asphalt and membrane system as a more effective overlay system for a compromised deck to extend the service life.

   NOTE: If the chloride testing indicates extensive infiltration of chloride ions in the deck, a deck replacement is likely needed. Performing a deck replacement at CCB is extremely difficult if traffic needs to be maintained during construction. Further, the existing 6.5-inch deck thickness is atypical of current design code minimum deck thickness. The remaining superstructure and substructure are not currently designed for additional loading to support a thicker deck. Therefore, if a deck replacement is warranted, a full bridge replacement is recommended, as described in Section 5.

7. Pier Cap Repairs:

   Repairing the observed spalls and cracks on the pier substructure elements will also prevent further degradation of the components and improve the overall long term durability of the bridge. Leaving the defects unchecked or repaired increases the potential for water infiltration and subsequent corrosion of the steel reinforcement.

8. Joint Seal Replacement:

   The replacement or addition of open-joint seals at all sidewalks and joints at all piers and Abutment 6 is essential to prevent water infiltration. This rehabilitation mitigates the risk of water-induced damage, including delamination, preserving the structural components.

9. Bridge Rail Replacement:

   The current bridge rails do not meet AASHTO Manual for Assessing Safety Hardware (MASH) (AASHTO 2016) criteria. Replacing the bridge rails with MASH compliant bridge rails will further improve the safety and sufficiency rating for the bridge. While typically a bridge rail replacement can be problematic on older bridge decks, the replacement of the exterior girders facilitates replacement of the bridge rails by rebuilding the deck overhangs to accommodate the design loads associated with the new railings at the same time.

The recommended rehabilitation measures are an attempt to maintain structural integrity and safety while improving the long-term durability of the bridge. Although implementation of these measures will help provide a prolonged service life, it is challenging to estimate how much service life will be added to the bridge. It is important to acknowledge these interventions primarily focus on preventing further deterioration of the bridge rather than providing substantial improvements in the bridge's load-carrying capacity. Regular monitoring and follow-up inspections will be crucial to evaluate the effectiveness of these rehabilitation measures and promptly address emerging issues as they appear.

Rehabilitation measures discussed in this section are intended to improve the service life of the bridge by addressing the structure's immediate maintenance needs. It is important to note the following issues would not be mitigated as part of the proposed rehabilitation:

a. Increasing the Load Rating of the Bridge:

   The proposed rehabilitation measures do not address the challenge of increasing the load rating of the bridge deck and girders to meet current design standards. The existing bridge was designed using an AASHTO live load of H20-S16-44. Updating the structure to adhere to the current AASHTO and CDOT's Live Load standard (BDM 2023) would require extensive rehabilitation and strengthening, including structural evaluation of the substructure, which has its own unique limitations.

b. Significantly Reducing Current Maintenance Demands:

   The proposed rehabilitation measures do not substantially reduce the ongoing maintenance demands of the bridge. Despite regular maintenance efforts over the past three decades, the bridge's condition rating has consistently remained at "Fair." This extended duration of "Fair" condition implies persistent structural and maintenance concerns, suggesting the proposed measures may not result in a notable decrease in routine maintenance requirements.

c. Removing the "Functionally Obsolete" Categorization:

   The proposed rehabilitation measures do not address the challenge of removing the bridge from the "functionally obsolete" (FO) categorization. The current functional obsolescence is attributed to the inadequate roadway width, which cannot accommodate the current traffic volume. This FO status contributes to a reduced sufficiency rating. Despite proposed rehabilitation interventions, the bridge will retain its FO status. Additionally, these rehabilitation measures do not improve other deficiencies of the bridge, such as the limited viable detours.

Bridge rehabilitation is recommended for nine key bridge elements. While the rehabilitation aims to extend the service life of the bridge, three specific issues cannot be remedied by a rehabilitation, including accommodating heavier vehicle loadings, reducing maintenance needs, and eliminating the limited functionality of the narrow roadway width.

4.4 Sufficiency Rating Calculation after Proposed Rehabilitation

The sufficiency rating computed in this section considers the rehabilitation interventions outlined in the preceding section. It is assumed the superstructure's condition rating increases from the existing 5 to 6, while the deck condition rating remains at 6. Providing a new wearing surface to the deck will help protect the deck but will not change the condition of the underside of deck. The latest condition rating of the deck was based on the underside of deck because the top was not inspectable. Replacing the bridge rails eliminates the “special reductions” applied in the sufficiency rating. All other parameters remain unaffected by the proposed rehabilitation efforts.

After factoring in the condition rating increase from the proposed rehabilitation, the increase in sufficiency rating was found to be modest from 50.3 to 64.7 (refer to Appendix C for sufficiency rating calculations), which, in the broader context of the Highway Bridge Replacement and Rehabilitation Program criteria, does not constitute a significant improvement.

The sufficiency rating is not greatly increased by the rehabilitation because of other constraints on the bridge, specifically the vehicle travelway width. The validity of the rehabilitation to extending the bridge service life is also dependent on factors such as routine maintenance.

4.5 Construction Phasing

The proposed construction phasing for the bridge's rehabilitation considers the critical need to minimize disruptions to traffic flow to and from Aspen. Recognizing the available substandard large vehicle access and inconvenient alignment of the required detour roadway, a total closure of the bridge during rehabilitation is deemed impractical. Instead, a phased approach is adopted, allowing for partial opening of the bridge to traffic. This strategy involves completing construction on one section of the bridge before shifting traffic to the other side to help eliminate significant traffic disruptions. One temporary lane, configured with a minimum width of 11 feet is proposed during construction, enabling anticipated speeds of up to 15 to 20 miles per hour (mph) for one direction of travel. Continuous maintenance of traffic using either a signalized alternating lane or a single lane across the bridge in one direction paired with a companion detour in the other direction would be required to complete the proposed rehab activities. Refer to Section 6, Traffic Impacts, for further information on maintenance of traffic options and associated impacts during construction.

Figure 32. Rehab Construction Phasing – Phase 1 (Looking East)

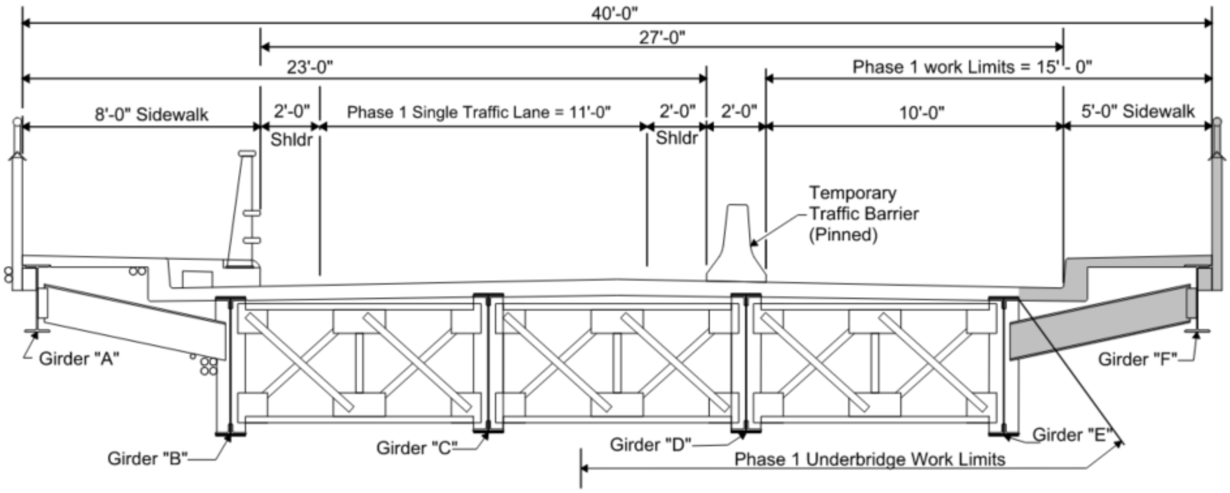
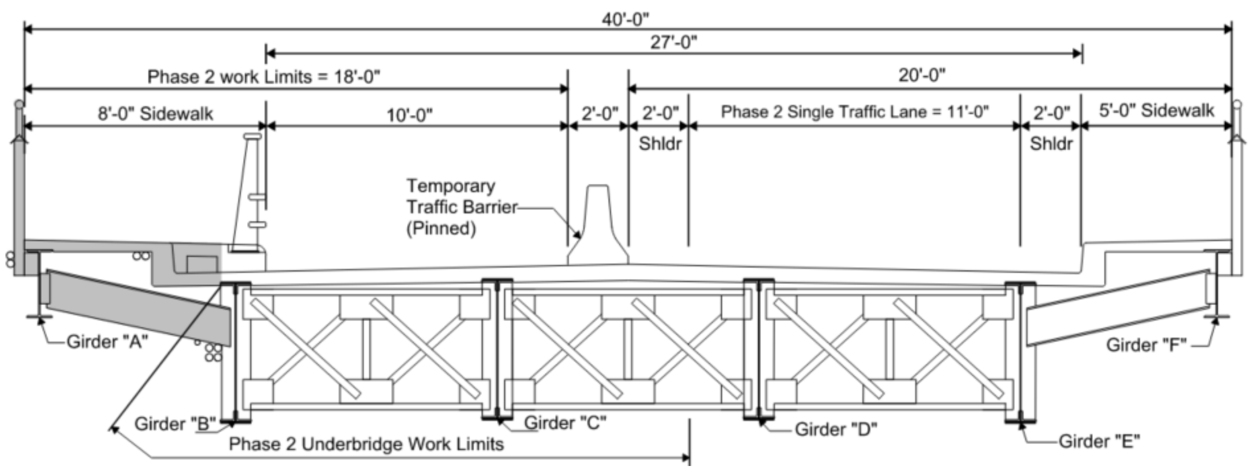


Figure 33. Rehab Construction Phasing – Phase 2 (Looking East)



In the proposed construction phasing, Phase 1 will involve the replacement of the exterior girders on the southern side. This includes the replacement of bearings for Girders D, E, and F at both abutments. Additionally, all other bearings on Girders D, E, and F will be cleaned, the bearing grout pads will be regrouted where necessary, and any instances of deck spalling will be patched. The subsequent phase will mirror this rehabilitation process, focusing on the northern side of the bridge. Additionally, Phase 2 will involve the replacement of Bearing Pedestal for Girder A at Abutment 6.

Larger vehicles, such as snow cats and plows, also use the bridge to access the ski resorts and Independence Pass. Traditional snowplows and smaller snowcats can travel over the bridge during construction, but larger snowcats (up to 19.5 feet wide with the blade) cannot be accommodated during the rehabilitation because of spatial constraints.

Pedestrian access can be maintained throughout rehabilitation work. Specifically, work will be carried out on the exterior girder supporting the south sidewalk during one phase, while the exterior girder supporting the north sidewalk will be the focus in another phase. This phased approach ensures pedestrians can access the bridge throughout the construction process.

Phasing utilities during bridge rehabilitation involves strategic planning to determine the most effective sequencing of construction activities. Utilities directly supported on the bridge will require permanent or temporary relocation while bridge rehabilitation activities are performed. The construction phasing provided in Appendix H, completing the south side of the bridge first to accommodate utility relocation before rehabilitation on the north side, is the recommended sequence. Relocation will require installation of approximately 10 new conduits, using utility boring under SH 82 at the bridge approaches to reroute conduits on the south side of the bridge.

Bridge rehabilitation will significantly affect local traffic for the duration of the work. A single traffic lane is provided during each phase of the construction.

4.6 Schedule

The rehabilitation of the SH 82 Bridge is proposed to be conducted within a restricted timeframe, dictated by weather conditions and the need to adapt to various community events in Aspen and the surrounding areas. The phased rehabilitation plan will maintain traffic in one lane across the bridge and use a companion detour carrying another lane allowing traffic into Aspen during morning peak hours and away from Aspen during evening peak hours. Anticipating a construction duration of 4 to 6 months per phase, the proposed schedule aims to complete one phase per year and the entire rehabilitation to be completed within 2 years.

Table 3. Rehabilitation Schedule

Phasing	Total Construction Duration	SH 82 Impact Duration	Maintenance of Traffic Duration
Phase 1	4-6 months	4-6 months	4-5 months
Phase 2	4-6 months	4-6 months	4-5 months

4.7 Cost Estimate

The preliminary bridge cost estimates outlined in this feasibility study are initial approximations and should be viewed as a general indicator of cost rather than conclusive figures. The primary purpose of this cost estimate is to give a general "ballpark" idea of costs associated with the prescribed rehabilitation measures. The preliminary cost estimate, which encompasses construction costs and a high-level assessment of costs to relocate utilities during construction, is $5,900,000. This does not represent a full project cost because project costs for mobilization, traffic control, site civil work for roadway approaches, and any other non-structural items are not included. Section 7 discusses and calculates the overall project costs associated with the rehabilitation option. Refer to Appendix D for the cost estimate for the proposed rehab activities.

4.8 Summary and Conclusions

Recent inspections have revealed significant concerns about the CCB, with girders showing varying signs of deterioration, the underside of deck showing signs of distress, and other localized structural issues. The sufficiency rating is currently at 50.3. Despite substantial rehabilitation efforts in 2011 and 2018, the bridge only achieved a "Fair" rating. A recent hands-on inspection confirmed the assigned "Fair" condition with a superstructure condition rating (NBI Item 59) of 5.

The field inspection highlighted various issues, particularly in the underside of concrete deck and steel girders. The underside of concrete deck exhibits signs of degradation and widespread surface cracking.

Steel girders display varying degrees of corrosion, with exterior girders showing significant corrosion and sag. Numerous tack welds and girder stiffeners exhibit cracks, and protective coatings on steel elements have failed, contributing to accelerated corrosion.

Given the significant deterioration, a comprehensive rehabilitation plan is essential. Proposed measures include bearing replacement, exterior girder replacement, protective coating rehabilitation, tack weld removal, pier column and cap repairs, joint seal replacement, and bridge rail replacement. While these measures aim to slow down and prevent further deterioration, they are not expected to bring improvements in the load-carrying capacity of the bridge or significantly extend the service life of the bridge. Proposed rehabilitation plans are provided in Appendix H. Regular monitoring will be crucial to assess the performance and effectiveness of the proposed rehabilitation measures over the life of the bridge.

The proposed rehabilitation interventions would result in a modest increase to the sufficiency rating. The proposed measures would not significantly improve the bridge condition to a level where total replacement is not deemed necessary. Challenges such as increasing the load rating, reducing inspection/maintenance demands, and improving the roadway width will not be addressed by the proposed rehabilitation, suggesting possible replacement of the bridge may be necessary to address these issues.

5. Bridge Replacement Feasibility

This section summarizes the feasibility of bridge replacement.

5.1 Bridge Width Alternatives

Two alternatives were considered for the feasibility of replacing the existing bridge as requested by the City. The first alternative considers that two lanes are provided on the new bridge, the southern sidewalk is removed, and the northern sidewalk is replaced. The second alternative considers that the bridge be widened to accommodate three lanes and a sidewalk on the northern side of the bridge. In both alternatives, the northern sidewalk is considered to be replaced with a 10-foot-wide sidewalk, which is understood to be the preference of the City Parks Department because it will accommodate future demands as a trail.

The three-lane alternative would provide the flexibility to have one lane designated for transit (bus or light rail) in the future. The presence of a transit lane on the three-lane alternative would result in increased live load effects on the bridge and would require increased superstructure depths. When determining approximate superstructure depths in Section 5.2, the AASHTO span-to-depth ratios are amplified by a factor of 1.30. The previous SH 82 Reversible Lane Feasibility Study (SGM 2008) has already documented the challenges and insufficiencies of trying to add a third lane to the existing bridge. Therefore, only a bridge replacement is considered for a three-lane bridge.

The two-lane alternative aims to maintain a similar width to the existing bridge. However, as discussed in Section 5.3, the space required to provide temporary lanes for access during construction is limited and requires an overall width of 48 feet 10 inches for the two-lane alternative. This is only slightly less than the 52 feet required for the three-lane alternative.

5.2 Structure Type

The following subsection describe the CCB structure.

5.2.1 Span Configurations

The existing bridge consists of five spans for a total length of 420 feet from bearing to bearing. The end spans have lengths of 75 feet, and the center three spans have lengths of 90 feet. The bridge passes over a pedestrian/bike trail at the west abutment, Power Plant Road in two locations, Castle Creek, and Harbour Lane near the east abutment. Additionally, near the piers under the bridge are fuel pumps belonging to the City at their maintenance building, and there are homes toward the east abutment along Harbour Lane.

Figure 34. Site Overview

The bridge replacement alternatives considered these constraints under the bridge when determining possible pier locations. Per the City, piers can be anywhere within the parking lot of the Aspen Streets Department building if there are no impacts to the fuel station or storage tanks. The existing bridge has a pier within Castle Creek. It is recommended this pier is removed and not replaced to avoid further impacts to the waterway and permitting issues. With that, piers at the east side of the bridge can be placed on the east bank of Castle Creek, west of Harbour Lane, or along the slope east of Harbour Lane.

Reducing the number of piers subsequently reduces construction cost and schedule. Because of the height of superstructure above the valley, the piers are anticipated to be costly because larger columns and foundation elements will be required. However, the cost savings realized from eliminating piers needs to be compared against the additional costs of a deeper superstructure to achieve the longer spans.

Additional superstructure costs arise from requiring more material for deeper members, and the extra depth may also require the profile on the bridge to be raised.

To maintain traffic on SH 82 during construction, the existing bridge will need to remain partially open while portions of the new bridge are constructed. To support the existing roadway, the existing bridge piers need to remain in place while the new bridge superstructure is constructed over the existing piers. This means that if the new bridge superstructure is deeper than the existing, the profile of the bridge would need to be raised. Raising the profile of the bridge would place the new bridge deck above the existing deck and require extensive reconstruction of the roadway approaches to tie into the existing roadway profile. With the proximity of homes near the bridge and roadway at the east abutment, the cost of construction and ROW impacts would be significantly increased; therefore, a profile raise is not considered feasible.

This limit to the structure depth eliminated a one span and two span bridge from consideration because the span lengths would result in significantly deeper superstructures. Three- and four-span layouts were considered; their feasibility depends on the type of superstructure and whether a profile raise would be required. For more information, refer to Section 5.2.2.

When laying out preliminary options for the three- and four-span pier locations, the limits set forth by CDOT BDM Section 5.5.1.9 (2023a) regarding the shipping and handling of girders were considered. This section limits the maximum length of a single girder segment to 154 feet and the maximum weight to 240 pounds-force (kips) (240,000 pounds). This only impacts the alternatives using precast concrete or steel because these members are prefabricated and shipped to the site for erection.

Figure 35. Three-span Configuration

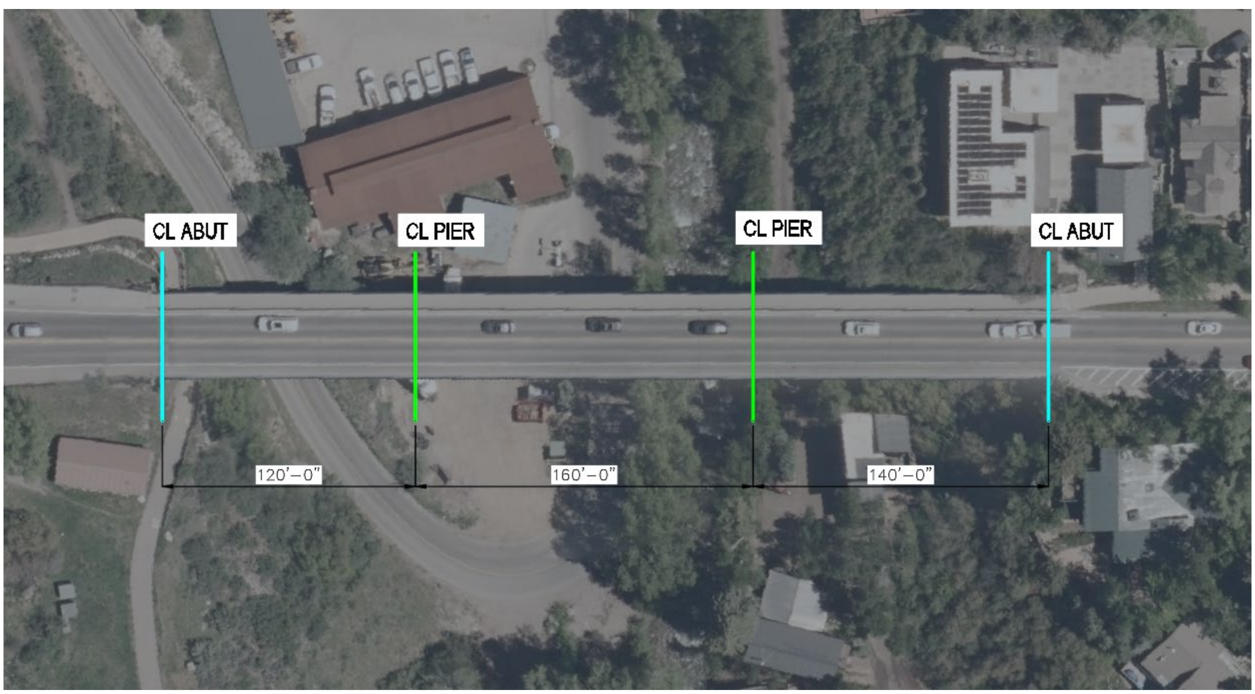
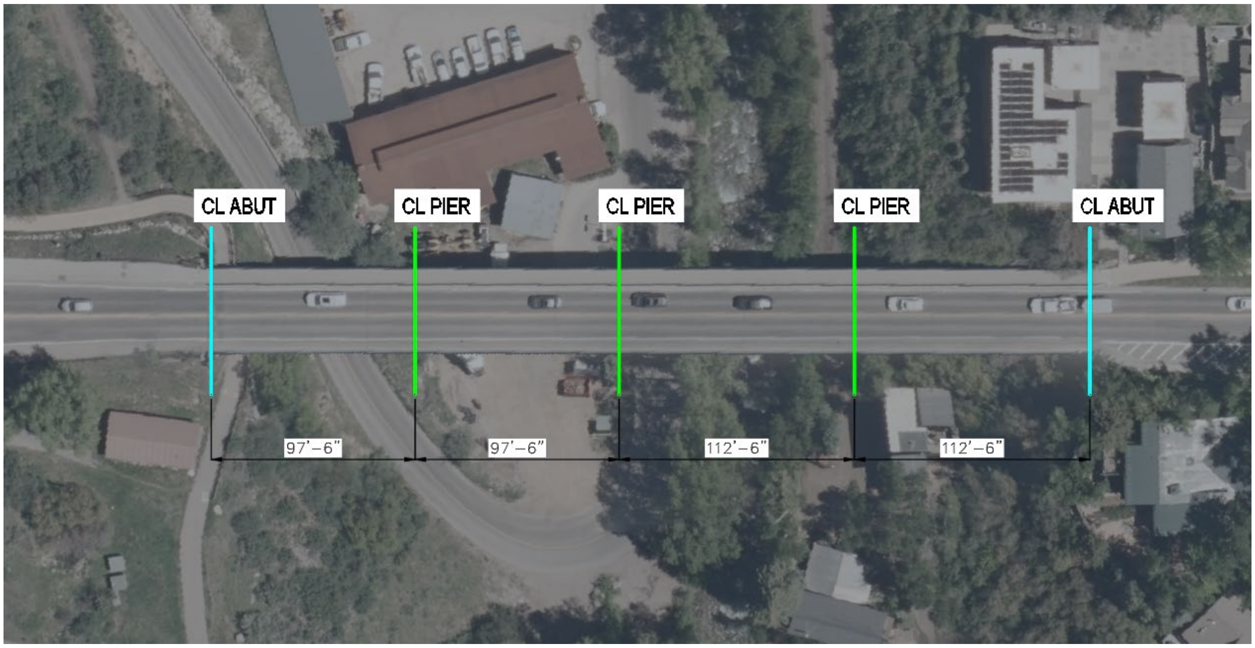


Figure 36. Four-span Configuration



The four-span configuration provided on Figure 36is presented in the conceptual drawings and used for cost estimates for both the two- and three-lane alternatives because it capable of keeping the structure depths approximately the same as the existing bridge. The three-span layout is possible in terms of structure depth and shipping restrictions if a steel girder superstructure were used, but because of construction restraints (discussed in Section 5.2.2), it is not feasible, and the three-span bridge is not considered further.

The four-span bridge configuration provides the best opportunity to control span lengths for a shallower structure depth that will accommodate traditional phased construction.

5.2.2 Materials

The following subsections describe the materials reviewed for the feasibility study.

5.2.2.1 Precast Concrete

Precast concrete girders are fabricated at two precast facilities in the Denver Metro area and surrounding states. These girders have their concrete cast in form beds around pretensioned high-strength steel strands. When the concrete reaches a desired compressive strength, the strands are cut, and they compress the girder to achieve its capacity. The girders are then stored at the precast facility until they are ready to be shipped to the site for erection.

Precast concrete girders are flexible when it comes to span capabilities because they can take various shapes and depths. As discussed in the previous subsection, the maximum length is 154 feet, and the maximum weight is 240 kips. In situations where these shipping limits start to govern (for larger depth girders), several girder segments can be spliced together using post-tensioned strands at the bridge site to achieve longer spans.

However, once the precast girders arrive onsite, they would need to be set into place using cranes. This can be done in several ways for a standard project. Cranes could be placed on the existing bridge, and the girders can be picked from trucks to swing into position. However, because of the limited space available for temporary lanes, this is not a viable option because SH 82 would require a full closure for the girders to be erected (assuming sufficient space for the cranes to operate from the existing bridge). For this operation, the girders would need to travel across the bridge using temporary lanes, and the cranes would need to mobilize into position. After setting a girder, the cranes would need to mobilize off the bridge and repeat this process, resulting in lengthy periods of full closures of SH 82, which is not feasible.

Another option for setting the girders would be to construct a large lattice crane beside the existing bridge to lift the girders into place. This becomes challenging for various reasons, including the facilities under the bridge and the weight of the girders.

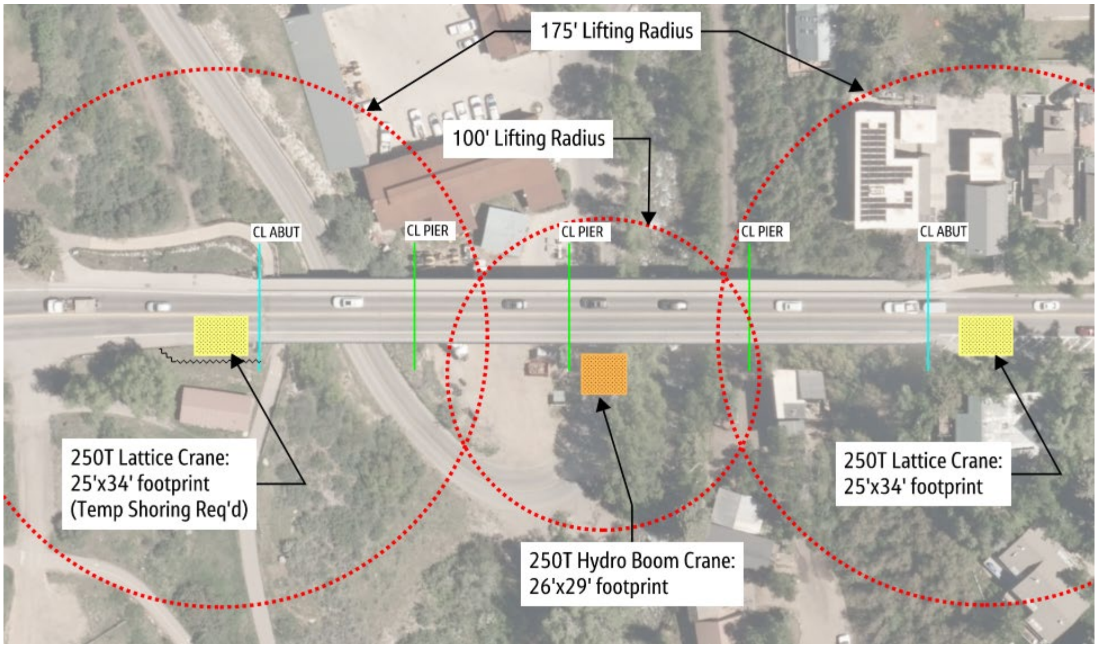
With Power Plant Road, Castle Creek, and Harbour Lane under the bridge, the locations where a large crane could be placed is limited. A crane would most likely need to be constructed south of the bridge, in/near the Aspen Streets Department parking lot. Placing the girders at the western side of the bridge would not be as difficult, but the girders at the eastern side of the bridge would be challenging. This would require a large radius for the crane to reach, and it would be carrying girders over the facilities under the bridge and near residential structures by the east abutment. This would require closures of the roads under the bridge, and safety measures would be required to protect the residential structures adjacent to the bridge.

Compounding on this problem is the weight of precast girders because they are typically heavier than other alternatives such as steel. With the long reach required, as the girder becomes heavier, the size would be required also increases. **Because of this, erection becomes problematic, and the precast concrete girders would not be feasible for this location.**

5.2.2.2 Steel

Steel girders are like precast girders because they are fabricated offsite, shipped, and erected via the use of cranes. The same concerns regarding the challenging erection of the precast concrete girders are present for the steel girders; however, steel girder members are typically lighter (50% to 60% less) than precast concrete members of similar lengths. Steel girder construction requires the splicing of several girder segments and would necessitate additional falsework to be constructed, so the splice connections can be installed. The use of these splice connections can be helpful in reducing the weight of the girder segments being lifted by a crane. These spliced connections are common practice for steel construction; whereas, spliced precast girder segments is not as common in Colorado. These lighter weights of girders make the steel erection a more practical operation but would still be challenging and require a large lattice crane to be used. It is estimated a 250- to 300-ton crane would be required for the erection of the outer spans. The lattice boom would need to be assembled in the roadway, which would require a lane closure. For the inner spans, the use of a 250-ton telescopic crawler crane would need to be transported, assembled, and positioned below on the south side of the structure. Both crane sizes and locations are shown on Figure 37, identifying the lifting radius from each crane. SH 82 would require full closure at night when the center span girders would need to be offloaded from the existing bridge and erected.

Figure 37. Approximate Crane Layout Needed to Erect Steel Girders



Overall, the steel girders are a potential option for the replacement of the existing bridge; however, the erection of the girders may be a limiting factor for the reasons stated in this subsection.

5.2.2.3 Cast-in-place Concrete

Cast-in-place concrete girders are constructed onsite and require falsework to be constructed to form the concrete. After the concrete has reached a desired strength, high-strength steel strands are run through ducts placed inside the concrete girder and tensioned. These post-tensioned strands compress the girder to provide its capacity, similar to the precast pretensioned concrete girders previously discussed.

The benefit of the cast-in-place concrete girders are that heavy girder segments are not lifted into place. However, falsework would need to be constructed along the entire length of the bridge to form and support the concrete throughout the duration of construction. Before the post-tensioning, the concrete is not capable of spanning between the abutments and piers and requires external support. The falsework would need to be designed to provide openings that allow for access to Power Plant Road, Castle Creek, and Harbour Lane under the bridge, similar to Figure 38 where bays are open for traffic flow. While the falsework is constructed, smaller cranes and temporary closures/lane shifts of the facilities under the bridge would be required. However, it is anticipated this would be less impactful than the steel girder erection operations previously described.

Figure 38. Example Falsework Photo for Cast-in-place Concrete Construction

A cast-in-place concrete superstructure with post-tensioning would pose some challenges if any future widening of the structure is explored. A typical widening project would remove the overhangs such that additional girders can be added to expand the deck. Because of the post-tensioning, the concrete is in a stressed condition that prevents the concrete from cracking, providing it strength against external loads. Removing the overhang would change the section properties of exterior girders significantly and could result in damage to the superstructure. Steel or precast concrete girder systems can be widened in a simpler manner because the overhangs are not stressed by post-tensioned strands. Because of this, the three-lane alternative would be beneficial should the cast-in-place concrete option be used because it could accommodate future traffic and transit demands.

The cast-in-place concrete alternative is a viable option for replacement of the existing bridge. While the erection concerns are eliminated, falsework construction under the bridge would be required. This alternative is shown in the attached conceptual drawings because it is anticipated to have the best constructability.

5.2.2.4 Structure Depth

Approximate depths of the superstructure were determined using the span-to-depth ratios defined by AASHTO Table 2.5.2.6.3-1 (AASHTO 2020). This table provides guidance regarding traditional minimum depths of superstructures that depend on the type of construction and span lengths. The resulting depths from this table are typically conservative when compared with final design member depths determined from detailed calculations that follow the AASHTO LRFD specifications (2020). As discussed in Section 5.1, the span-to-depth ratios for the three-lane alternative are increased by 30% to account for additional loading from the potential future light rail transit on the bridge. Table 44 presents the approximate superstructure depths, girder height, plus deck thickness for the two- and three-lane alternatives. Not feasible (NF) is provided for alternatives where a profile raise would be required and the alternative is not feasible.

Table 4. Superstructure Depths

Superstructure Type	Two-lane		Three-lane	
	Three-span	Four-span	Three-span	Four-span
Cast-in-place Concrete	6.40 feet (NF)	5.08 feet	8.32 feet (NF)	5.92 feet
Steel	5.08 feet	5.08 feet	6.66 feet (NF)	5.08 feet

As presented in Table 4, the cast-in-place concrete alternatives would require profile raises for the three-span configuration provided on Figure 35. Therefore, the cast-in-place concrete superstructure type is only recommended for the four-span configuration shown on Figure 36. For steel, a profile raise is only required for the three-lane alternative when the three-span configuration is used.

With that, the conceptual drawings in Appendices I and J provide the four-span configuration with a cast-in-place concrete superstructure. Figure 39 and Figure 40 present the typical section for the cast-in-place concrete superstructure for the two- and three-lane alternatives. This structure type is anticipated to provide the best constructability and would not require a profile raise. The other options presented in this report may also be reasonable but would require further analysis not in the current scope.

Figure 39. Two-lane Alternative Cast-in-place Concrete Typical Section

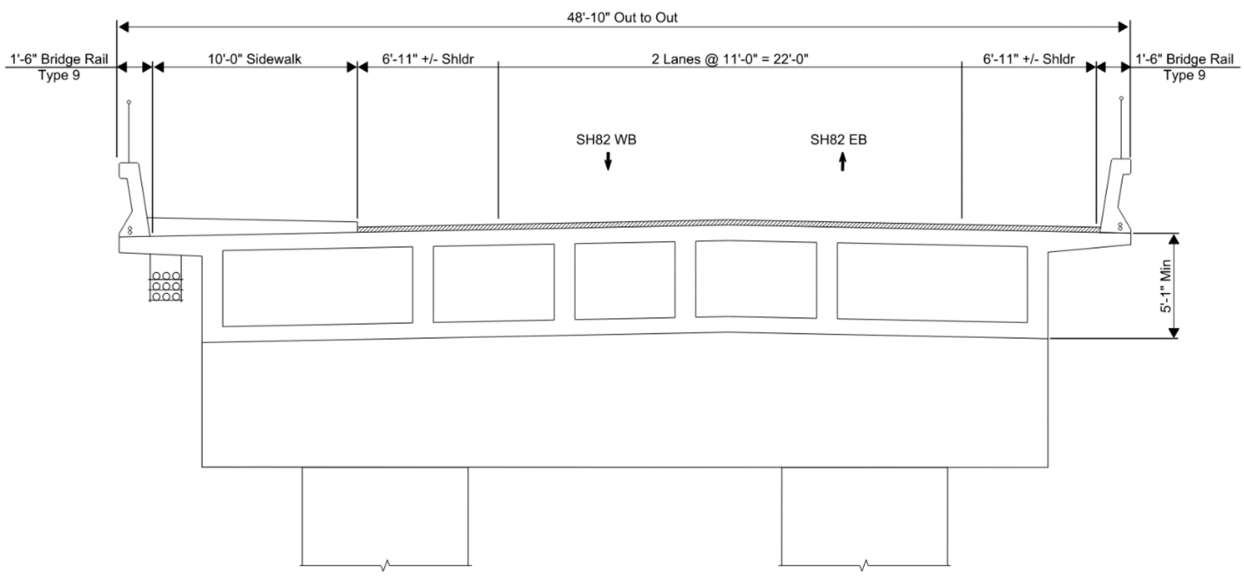
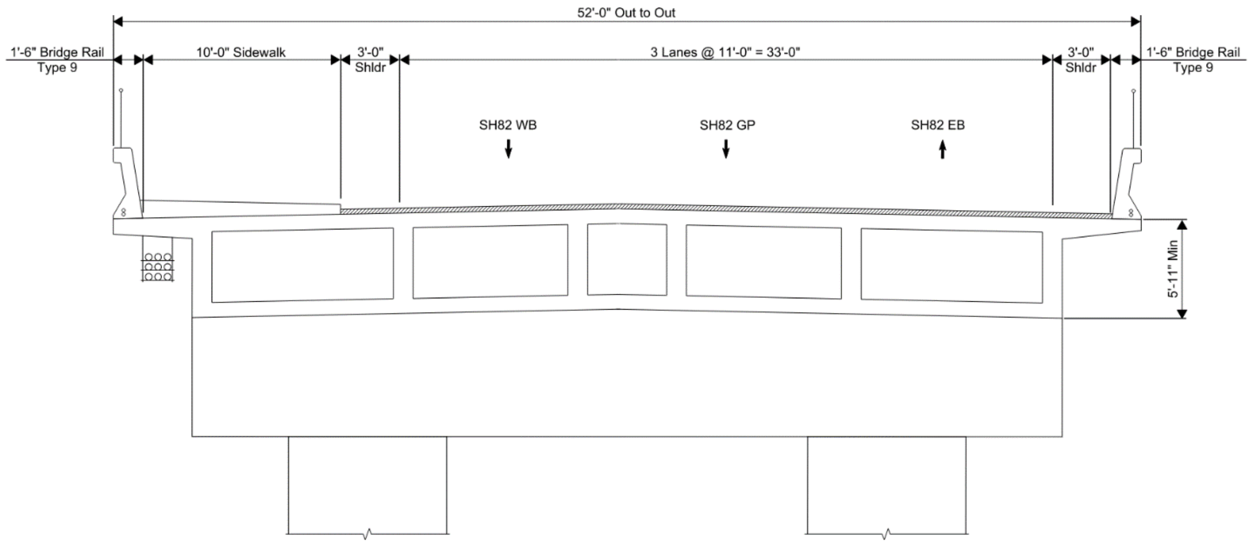


Figure 40. Three-lane Alternative Cast-in-place Concrete Typical Section



Precast concrete, steel, and cast-in-place concrete were evaluated for structure type feasibility. The steep terrain and facilities under the bridge limit the space for large cranes, eliminating the ability to use precast concrete. Crane placement for steel requires closures of Power Plant Road. Therefore, only cast-in-place concrete is advanced further because it provides the best constructability and limits impacts to the SH 82 profile.

5.3 Construction Phasing

This section details construction phasing possibilities for both the two- and three-lane alternatives.

5.3.1 Service During Construction

With SH 82 being the primary access in and out of Aspen, constructing the replacement bridge in phases is recommended. This would allow for portions of the existing bridge to remain open and provide access to vehicular and pedestrian traffic while the new bridge is constructed. As portions of the new bridge are completed, traffic can be shifted off the existing bridge and onto the new bridge. Availability of vehicular lanes and a pedestrian walkway across the bridge during construction will be dependent on the bridge phasing, which is further discussed in Section 5.3.2 for each option evaluated.

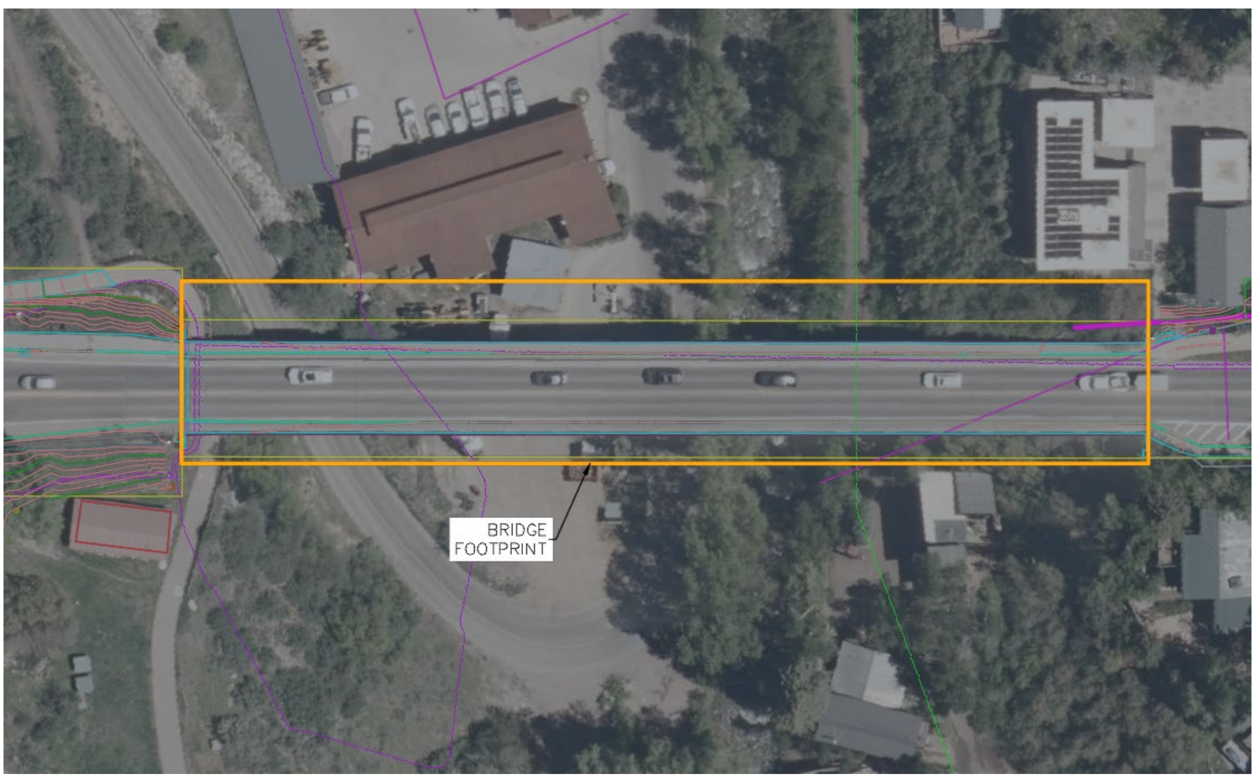
All phasing options considered in this report rely on the existing bridge to carry traffic in a partial state throughout the duration of construction. Full demolition and reconstruction of the bridge is only available as an option if a new detour route is built for all traffic to shift away from the existing bridge site and all impacts to travelers are eliminated. This is discussed further in Section 6 for traffic impacts.

The following is a list of constraints and assumptions considered when developing a bridge phasing plan:

- SH 82 traffic movement is paramount. Because a full bridge closure is not an option, bridge construction needs to be traditionally phased with temporary lanes.
- 5-foot wide (minimum) pedestrian access is required during all phases of construction either on or below the bridge. For more details, refer to Section 5.3.2.

- Traditional snowplows and smaller snowcats can travel over the bridge during construction, but larger snowcats (up to 19.5 feet wide with the blade) cannot be accommodated because of spatial constraints.
- The new bridge footprint is ideally inside the ROW limits because additional ROW acquisition is cost prohibitive.
- Temporary barriers need to be pinned to the deck to provide space for 11-foot wide lanes with 2-foot shoulders during maintenance of traffic, which ultimately controls the width of each construction phase.
- Short term closures on SH 82 and Power Plant Road will be required to accommodate bridge construction. A protective canopy can be installed to protect traveling public below the bridge.
- Utility relocations are required before demolition of the north side of the bridge.
- New pier construction can occur before the existing bridge is demolished.
- An "overbuild" is when the new bridge is built wider than the required final condition and is often used to accommodate traffic patterns during construction phasing. A bridge overbuild to the north and south was investigated because it would allow for uninterrupted traffic flow on SH 82. However, spatial constraints at the bridge site prevent an overbuild. Any significant ROW acquisition would be very costly and ultimately eliminate an overbuild as a feasible option (refer to Figure 41).

Figure 41. Bridge Footprint Required to Overbuild New Bridge Outside of Existing, Deemed Not Feasible



5.3.2 Phasing Options

Traditional construction phasing options are provided in the following subsections. While several phasing options were initially considered, the four options presented were advanced for further consideration.

Table 5. Phasing Options Advanced for Consideration

Phasing Option	Opportunity
Two-lane Replace	Replicates the existing condition for a comparison.
Three-lane Centered	Provides the least impact to the bridge site.
Three-lane Faster	Provides the shortest phased construction duration.
Three-lane Shifted	Provides the least impact to the traveling public (vehicular and pedestrian).

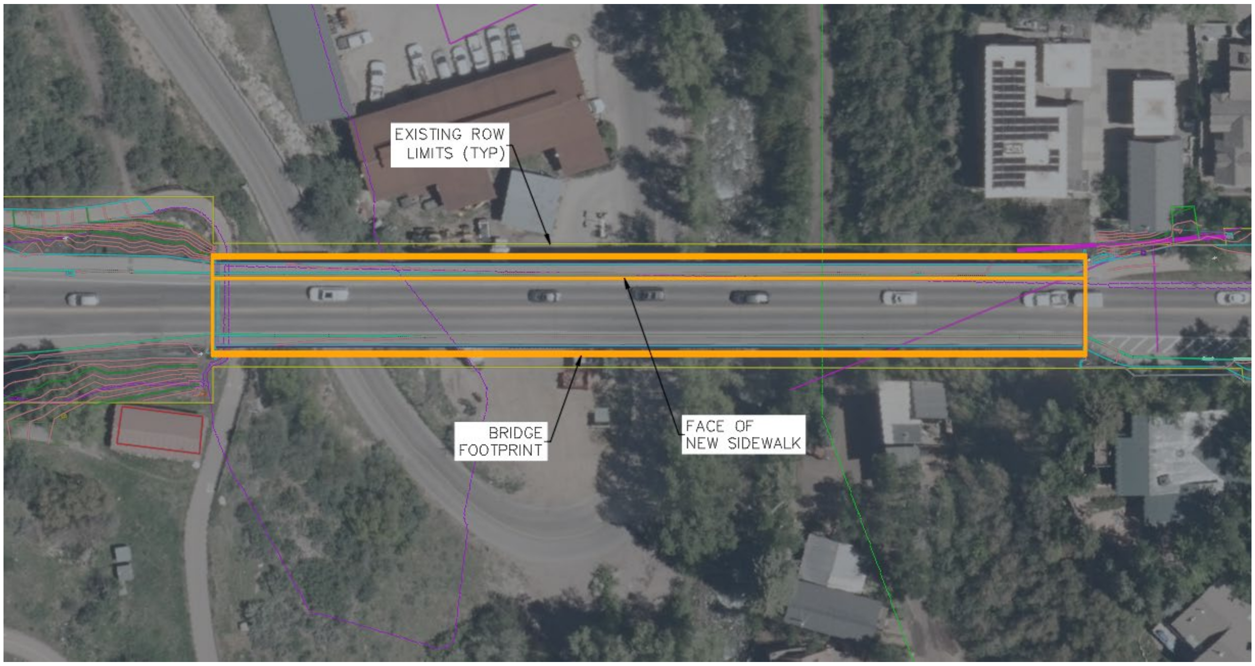
5.3.2.1 Two-lane Replace

The two-lane alternative aims to replace the bridge with a new bridge of similar width and footprint. Only one option was considered for this alternative because the only other option to maintain the same footprint of the bridge would be to fully close and replace the existing bridge. The bridge phasing considered for the two-lane alternative allows for the front face of the northern sidewalk to remain in approximately the same location as the existing. The southern edge of deck would move to the south, which would require some roadway reconstruction to tie into the adjacent roadway segments; however, it would be minimal.

For this option, four phases of construction would be required to replace the bridge, but it allows for a single lane of traffic to be open during all phases of construction. For more details, refer to the two-lane diagrams provided in Appendix G.

The final width of the two-lane bridge is 8 feet10 inches wider than the existing bridge while providing sufficient room for the 10-foot sidewalk at the northern edge of deck and two 11-foot lanes. The additional width is a result of the space required to fit the temporary travel ways at various phases as described in Section 5.3.1. While wider than the existing bridge, no ROW acquisitions are anticipated because the footprint of the bridge deck is within the ROW limits from a previous survey provided to Jacobs by the City (City of Aspen n.d.).

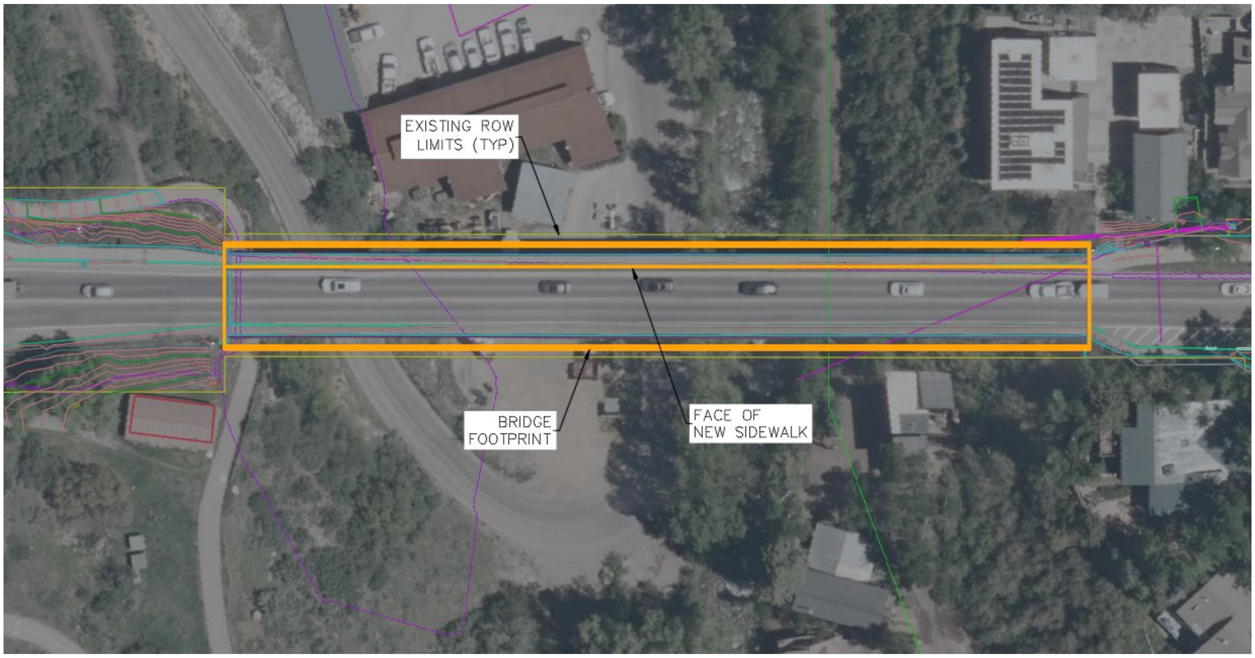
Figure 42. Two-lane Replace, Bridge Footprint



5.3.2.2 Three-lane Centered

This option is essentially the same as that described for the two-lane alternative, and all previous discussion are applicable to this option. Similar to the two-lane phasing, this results in the northern face of sidewalk remaining in approximately the same location as that of the existing bridge. Minimal roadway work would be required to tie into adjacent roadway segments. The only difference is the exterior segments constructed during Phases 1 and 2 are wider to accommodate the additional width needed for the third lane. This final configuration can accommodate a 10-foot sidewalk, three 11-foot lanes, and two 3-foot shoulders. While the bridge is wider, no ROW acquisitions are anticipated because the bridge footprint is within the ROW limits. For more details, refer to the Three-lane Centered diagrams provided in Appendix G.

Figure 43. Three-lane Centered, Bridge Footprint



5.3.2.3 Three-lane Faster

This option proposes the removal of the exterior segments of the existing bridge during Phase 1 of construction. This allows for sufficient width to be constructed such that two temporary lanes can be provided earlier, making the single traffic lane only required for one phase. However, pedestrians would need to be rerouted under the bridge because there would not be sufficient width to accommodate pedestrian access on the bridge during any phase. For more details, refer to the Three-lane Faster diagrams provided in Appendix G.

Phase 1 of construction would require SH 82 to be reduced to a single lane, so the exterior segments of the existing bridge can be demolished and replaced. Refer to Section 6 Traffic Impacts for further information on the traffic impacts of using only one lane during construction.

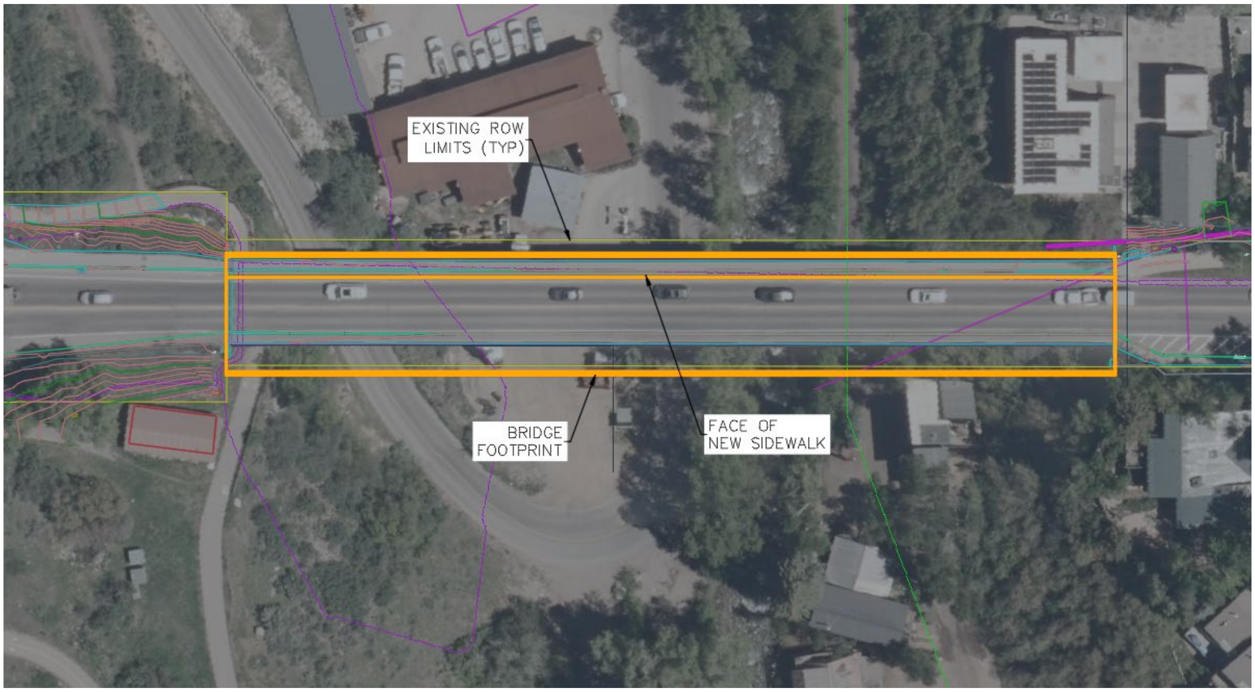
Once the exterior segments of the new bridge are complete, SH 82 eastbound and westbound traffic can be split and carried by the new bridge segments. The remaining center segment of the bridge could then be removed and replaced.

Once the center segment is complete, closure pours can be placed to connect the three segments of the new bridge. The sidewalk at the north can be constructed, and the SH 82 lanes can be placed on the new bridge.

This option requires the new bridge to shift to the south by approximately 3 feet 8 inches beyond the ROW limit and would require ROW acquisition. Trees in this region may also be impacted and require removal.

As presented on Figure 44, the southern edge of the bridge is nearly above the residential structure on Harbour Lane. Additional care would be required during pier and superstructure construction to protect this residence.

Figure 44. Three-lane Faster, Bridge Footprint



5.3.2.4 Three-lane Shifted

The intention of this option is to maintain two lanes on the bridge during all phases of construction. To accomplish this, most of the existing bridge needs to remain in place to be able to carry two temporary lanes while the first portion of the new bridge is constructed. The southern exterior segment of the new bridge is proposed to be constructed first because this will cause the bridge to be shifted to the south, similar to Three-Lane Faster, and will avoid conflicts with the residential structures to the north. For more details, refer to the Three-lane Shifted diagrams presented in Appendix G.

Because of the bridge shifting to the south, the face of the northern sidewalk does not align with the existing sidewalk face for this option. This would require reconstruction of the adjacent roadway segments on both sides of the roadway. Like the Faster option, the southern edge of the deck is beyond the ROW limit by approximately 4 feet 6 inches, and trees in this area would be impacted. As shown on Figure 45, the house on Harbour Lane is nearly under the bridge and would require care during construction to protect the residence.

Figure 45. Three-lane Shifted, South, Bridge Footprint

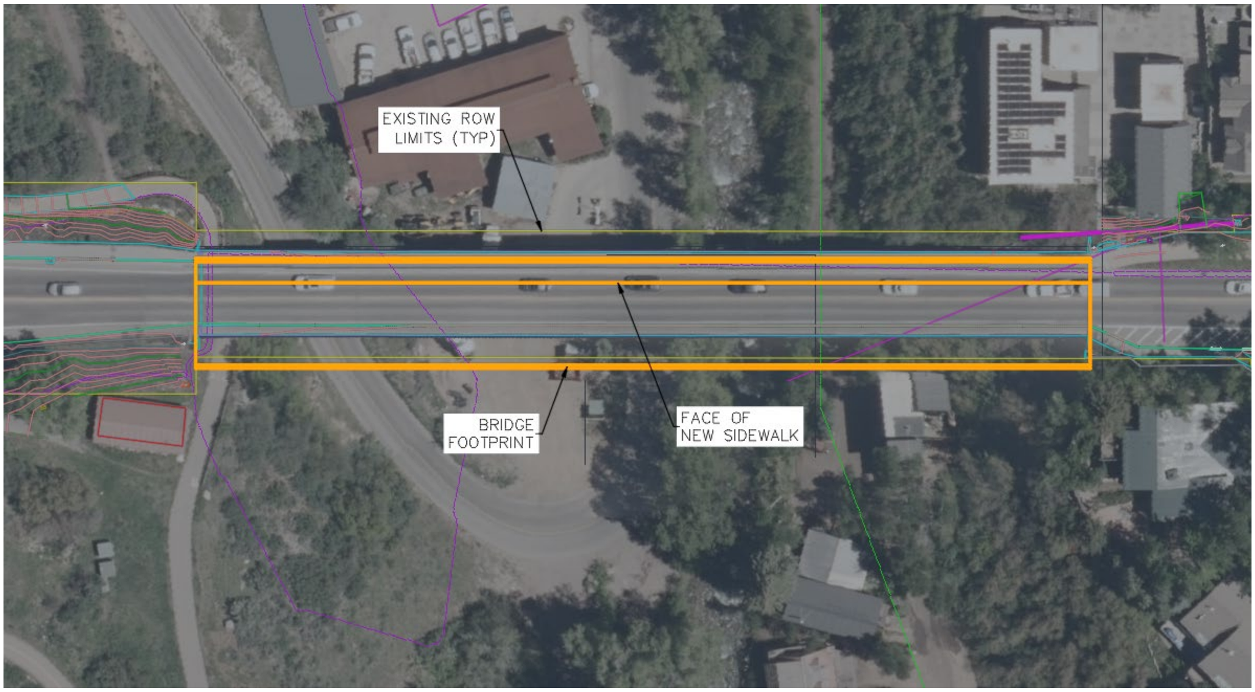
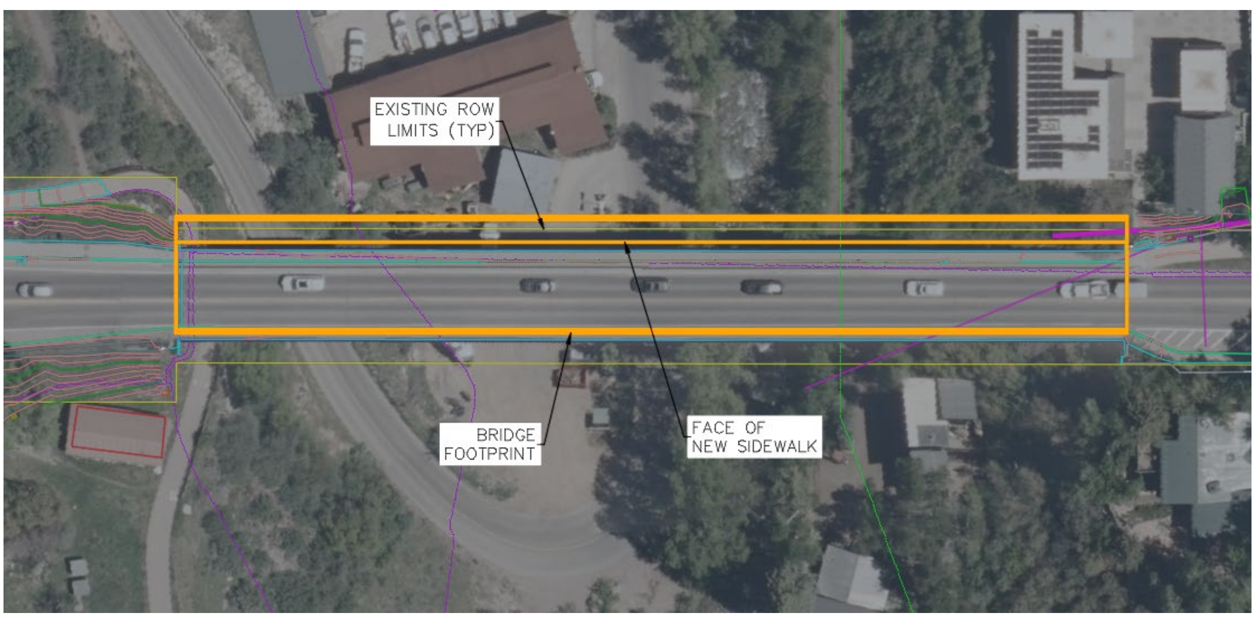


Figure 46 presents the resulting Three-Lane Shifted bridge footprint. The result is the bridge shifting to the north and being in proximity to the residential structures near the east abutment. Additionally, abutment and wingwall construction would be very close to the structures and is not recommended. Because of this, shifting the bridge to the north is not considered feasible.

Figure 46. Three-lane Shifted, North, Bridge Footprint



A variation of the Shifted option was considered to accommodate pedestrians in all phases. Adding a 5-foot pedestrian path shifts the entire bridge further south, placing it over the residential structure and requiring almost 12 feet of additional ROW. Because of these impacts to the residential structure and the significant increase in ROW acquisitions, accommodating pedestrians across the CCB in all phases of the Shifted option is not considered feasible.

5.3.2.5 Phasing Option Summary

Table 6 summarizes the impacts and constraints of the four phasing options described in this report. As discussed, only one option was considered for the two-lane alternative, and the phasing described is presented on the conceptual drawings. For the three-lane alternative, Centered is anticipated to be the least impactful, even though a single lane of traffic is required for two construction phases. This option eliminates the need to shift the bridge, reduces the amount of roadway reconstruction, does not require ROW acquisitions, and eliminates the risk associated with constructing the bridge above the residential structure on Harbour Lane.

Table 6. Phasing Option Impact Summary

Phasing Option	Impacts				
	Single Traffic Lane	No Pedestrian Access on the Bridge	ROW Acquisition	Construction Nearby/Above Residential Structures	Adjacent Roadway Realignment
Two-lane Replace	High	Medium	Low	Medium	Medium
Three-lane Centered	High	Medium	Low	Medium	Medium
Three-lane Faster	Medium	High	Medium	High	Medium
Three-lane Shifted	Low	Medium	High	High	High
Legend:	Low to Zero Impacts: Green		Medium Impacts: Orange		High Impacts: Red

Only one phasing option applies to the two-lane bridge alternative. For the three-lane bridge alternative, the Three-lane Centered provides the best overall scenario for construction. However, this option also creates the most impact to travelers (vehicular and pedestrian). The Three-lane Shifted is the best scenario for travelers, but it encounters considerable project risks and property impacts.

5.3.3 Schedule

Because of the local weather patterns, the available window for construction is limited. The estimated timeframe in which construction can progress is from the beginning of April to the end of October, with potential bleed into March and November when the weather is favorable. The City indicated a period of downtime, June 15 to July 11, to accommodate events and festivals held in Aspen. With that, it is estimated there is approximately 5 to 6 months each year in which construction activities can occur. Based on the superstructure types considered for the replacement, it is estimated each phase would take between 4 to 6 months to complete. This works out to about one phase of construction completed per calendar year.

When considering the impacts of a single traffic lane on the accessibility of SH 82 into or out of Aspen, the duration in which this lane would be in place is important to understand. For the two-lane replace and the three-lane centered option, the single traffic lane would be required for approximately 2 years. As discussed for the two-lane replace option, pedestrian access could be maintained on the bridge throughout the duration of construction, but it would require the single traffic lane for an additional year. Also discussed was the removal of the pedestrians from Phase 2 of the two-lane alternative and for the three-lane centered option. This would allow for one phase to be removed and would reduce the overall duration of construction to 3 years.

Table 7 presents a summary of the total construction duration and impacts to SH 82 for the construction phasing options considered. The SH 82 Impact Duration considers the time in which SH 82 would require maintenance of traffic control, slower speeds, or shifted lane locations.

Table 7. Summary of Construction Duration and Impacts

Phasing Option	Total Construction Duration	SH 82 Impact Duration	Single Traffic Lane Duration
Two-lane Replace	4 years	3 years	2 years
Three-lane Centered	4 years	3 years	2 years
Three-lane Faster	3 years	2 years	1 year
Three-lane Shifted	4 years	3 years	0 years

NOTE: Assumes a typical calendar year with the following months and partial months for construction:

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
✗	✗	✗ ✓	✓	✓	✓ ✗	✗ ✓	✓	✓	✓	✓ ✗	✗

For all options, construction duration ranges from 3 years to 4 years, with only a portion of each calendar year open to construction. Construction duration for the bridge replacement option is primarily a function of the available detour routes. If all traffic could be routed to an offline location to allow for full bridge closure, the bridge could be replaced quickly when compared to having to keep the bridge open to traffic during the replacement. Construction phasing of the bridge quadruples the time required for replacement.

5.4 Cost Estimate

Preliminary bridge cost estimates prepared for this feasibility study are high-level estimates and should not be considered final. These estimates are more so to compare the different phasing options and superstructure types. Cost per square foot of bridge were the basis of these cost estimates. This data is available from CDOT; however, as steel and cast-in-place concrete girders are not common in Colorado, cost data from other states such as California, Washington, and Wisconsin were referenced. (CALTRANS 2019; CDOT 2023b; WSDOT 2020; WISDOT, 2023) Additionally, a bridge cost estimate specialist from

Jacobs was consulted to determine how the complexities associated with this site and location would factor into the bridge cost estimates.

In addition to the cost of the new bridge, ROW acquisitions for approach tie-ins were also included. As discussed in Section 5.3.2, ROW acquisitions were required for the three-lane faster and shifted options. Refer to Section 7 for additional costs for Temporary Construction Easements.

Table 8 provides a summary of the costs for the replacement alternatives. The costs of the cast-in-place concrete and steel superstructures are fairly similar, with steel being slightly more expensive. The two-lane replace and the three-lane centered options are also similar, while the three-lane is slightly more expensive because of the additional width. However, it may be more economical to construct the three-lane centered, because the total cost of the bridge during its service life is anticipated to be less than the two-lane replace, given no future widening would be needed to accommodate the third lane. Three-lane Faster and Shifted have additional ROW acquisitions and are significantly more expensive as a result.

Table 8. Summary of Alternative Cost Estimates

Material	Alternative	Bridge Cost per Square Foot	Bridge Area (Square Foot)	Bridge Cost	ROW Acquisition (Square Foot)
Cast-in-place Concrete	Two-lane Replace	$450	21,125	$9,500,000	0
	Three-lane Centered	$450	22,048	$10,000,000	0
	Three-lane Faster	$450	24,557	$11,100,000	574
	Three-lane Shifted	$450	22,048	$10,000,000	673
Steel	Two-lane Replace	$475	21,125	$10,000,000	0
	Three-lane Centered	$475	22,048	$10,500,000	0
	Three-lane Faster	$475	24,557	$11,700,000	574
	Three-lane Shifted	$475	22,048	$10,500,000	673

The costs in Table 8 are not complete project costs because other costs for mobilization, traffic control, site civil work for roadway approaches, and any other non-structural items are not included. Section 7 discusses and calculates the overall project costs associated with each alternative.

5.5 Accelerated Bridge Construction

Accelerated bridge construction (ABC) uses innovative planning, design, materials, detours, and construction methods in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges. ABC improves site constructability, total project delivery time, and work-zone safety for the traveling public. In the most ideal cases, ABC also reduces traffic impacts, onsite construction time, and weather-related time delays, which can be significant in Colorado.

This project investigated several ABC techniques and analyzed each to determine which, if any, would be a good fit for this spatially constrained site. Using ABC on projects will typically save construction time while adding construction cost. Each project needs to decide if this trade-off, along with the added cost, is worth it.

5.5.1 Self-propelled Modular Transporter Move

First, the project investigated if a self-propelled modular transporter (SPMT) bridge move was an option. This option would build the superstructure offline on temporary supports and then move it into place after the existing bridge is demolished for the new substructure to be built. The SPMT includes hundreds of wheels to move the bridge in place and is controlled by a computer. An example of this type of construction is shown on the images in Figure 47. The superstructure is moved off the temporary supports with the SPMT in the left photo, and the SPMT drives the superstructure into position to rest on the new substructure in the right photo.

Figure 47. Self-propelled modular transporter construction on Minnesota Department of Transportation Maryland Avenue Bridge

This method requires ample site space nearby to stage and build the new superstructure. The site must be flat terrain to “drive” the superstructure into place and set it on the new substructure, with all the wheels of the SPMT working together. The Castle Creek site does not have enough space to build the new superstructure, and the steep terrain surrounding the bridge is not conducive to an SPMT move.

5.5.2 Incremental Bridge Launch

With steep terrain, an incremental bridge launch could be a beneficial ABC option. For a bridge launch, the bridge is typically built on the same alignment as the final bridge layout, then incrementally launched out to slide over each pier as it goes from one abutment to the next. A launch pit is built in the roadway area ahead of the bridge location, where the bridge sections are aligned, connected, and then pushed forward. Figure 48shows an example incremental steel bridge launch using hydraulic jacks.

Figure 48. Incremental Steel Bridge Launch at the Athabasca River Bridge

With SH 82 (Hallam Street) being the main route for Aspen, an incrementally launched bridge at this location is less feasible, requiring a full shutdown of the road for months while the bridge is built along the alignment to launch. Within the shutdown for construction, there will also be periods of time when Power Plant Road will require closure for safety critical activities, such as existing bridge demolition and steel launching. Closures on Power Plant Road then cut off the only existing detour to SH 82 during construction. The construction limits are also extended to accommodate the launch pit, excavated in the roadway ahead of the bridge location. The lack of an existing detour and an extended full closure period are major conflicts, negating the benefits of an incremental launch at this site. Because of the major constraints, an incremental launch was not considered further as a viable method. An incremental launch sequence is shown on Figures 43a and 43b in Appendix E, indicating the conflicts for this site.

5.5.3 Slide-in Bridge Construction

Another ABC option investigated was the bridge slide, which is built offline similar in nature to the SPMT bridge move. With a bridge slide, the new superstructure is built directly adjacent to the existing bridge on temporary supports. The new substructures (piers and abutments) are built in their permanent location. The bridge is then slid into place, transferring the superstructure from the temporary supports to the permanent substructure. Slide-in bridge construction (SIBC) was used on the State Highway 266 over Holbrook Canal Bridge, as shown on the images in Figure 49. The left photo shows the bridge superstructure sliding into place with a hydraulic jack, while the right photo highlights the temporary supports that were built for the superstructure before sliding into the final position shown in the background. SIBC is most advantageous when used on single span bridges.

Figure 49. Slide-in Bridge Construction at the Colorado Department of Transportation State Highway 266 over Holbrook Canal Bridge

While the piers could primarily be built underneath the existing bridge, the abutments must be built with lane closures. Like the SPMT option, a typical bridge slide requires a 6- to 8-day shutdown to complete the operation. However, at this location, a shutdown of SH 82 would be about 1 month because most of the abutment and approach work cannot be performed ahead of the closure. Given the height of the existing bridge, the temporary piers and abutments will be both costly and impractical at the SH 82 site. The following site constraints must be considered for a bridge slide of the CCB:

- Gas Regulator Station: An existing gas regulator station is adjacent to the southwest corner of the existing bridge. The temporary bridge location would likely interfere with the gas regulator station, which contains high pressure gas for much of the City area. Impacts to this station and the lines from it is highly dangerous, and relocating the station would be both expensive and impactful to the environmental site because it is in an open space area.
- Temporary Pier Locations: The location of the temporary piers must align with the proposed piers. With Power Plant Road crossing underneath the bridge in two locations (horizontal curve), locating piers to avoid Power Plant Road in both the temporary and permanent construction will be difficult without impeding traffic on Power Plant Road.
- Utilities: Currently, nine utility conduits run along the existing bridge, carrying utilities for multiple providers. To accommodate a bridge slide, these utilities would need to be either temporarily or permanently relocated to a separate support, similar to small bridge directly north of the existing bridge.
- Residential Properties: To build offline to the south on the temporary supports, the bridge superstructure would be built overtop of at least one residential property. This is a significant risk for the project, and contractors will avoid this interaction and risk.
- Power Plant Road Access: Currently, Power Plant Road is the only detour route to SH 82. As soon as the existing bridge demolition starts, Power Plant Road will need to be closed to traffic for safety during demolition. The road will also be closed during the slide. With SH 82 and Power Plant Road concurrently closed for extended periods during the construction, there will be no emergency route available.

Given these site constraints, SIBC would present more impacts than benefits. A typical bridge slide is shown on Figure 49a, and major conflicts toward using this method are shown on Figure 49b, both in Appendix F.

5.5.4 Prefabricated Bridge Elements and Systems

All precast elements are considered forms of ABC, and this project investigated using precast concrete deck forms, along with precast/prestressed girders. Precast elements allow for the casting of the concrete to be performed offsite instead of having to wait for long curing times at the bridge site. This has a dramatic impact on project schedule and is the construction method of choice in the State of Colorado. Both Colorado Bulb-T's and Colorado Decked Bulb-T's were considerations for this project. However, all precast girder options have been eliminated because of their heavy pick weights when compared to other superstructure types. Precast, prestressed concrete girders that accommodate a three- or four-span configuration require two large cranes to pick and place each girder. Both cranes would need to be brought to the site in pieces and transported into place from the top of the existing bridge to the ground below, where it would be assembled. Picking and placing these heavy girders with such large cranes is not impossible, but it is challenging and costly enough for the team to look at alternative options. Because precast/prestressed concrete girders are eliminated, so are their counterparts, precast concrete deck panels.

Other ABC options worth considering were prefabricated bridge elements and systems like pre-decked girder systems. These are preassembled girder pairs with a concrete deck placed on top. The system is picked and placed on the newly constructed substructure units and require closure pours between each of the elements. This system was also eliminated for the same reason the concrete girders were; the sheer weight of the girder picks requires very large cranes that cannot fit within the construction footprint area.

Table 9. Accelerated Bridge Construction Summary

ABC Method	Brief Description	Fatal Flaw at CCB
SPMT Move	"Driving" the superstructure into position to set on the new substructure	Steep terrain is not conducive to SPMT machine
Incremental Bridge Launch	"Pushing" the superstructure across from one side to the other side for the bridge length	Extended full closure of SH 82 cuts off access to the City
SIBC	"Sliding" the superstructure over from temporary supports to the new substructure	Existing facilities under SH 82 conflict with the temporary bridge location
Prefabricated Bridge Elements and Systems	Using prefabricated (typically precast concrete) elements to expedite construction	Existing facilities under SH 82 prohibit needed space for cranes to erect precast elements

While not considered an ABC method by definition, a full closure of the bridge would provide the fastest construction method. This requires a new, viable detour for all traffic for full demolition of the existing bridge before reconstruction on the same alignment. Refer to Sections 5.3 and 6 for additional discussion related to schedule and detours.

In summary, the CCB replacement is a viable candidate for ABC when only considering impacts to the traveling public. However, the proximity of the residents nearby, tight curves of the roadway below, and narrow footprint of the existing bridge make most ABC options untenable. Traditional bridge construction phasing or a full closure of SH 82 are the only options remaining to consider.

6. Traffic Impacts

This section summarizes potential traffic impacts caused by the bridge reconstruction or rehabilitation.

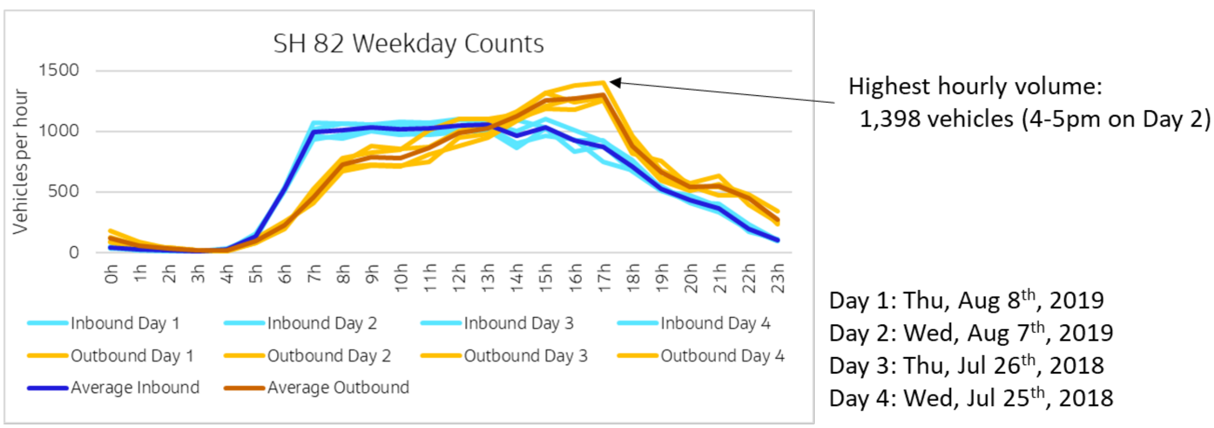
6.1 Existing Traffic Conditions

CCB supports SH 82, which is functionally classified as a Principal Arterial. SH 82 in this area contains one travel lane in each direction and has a posted speed limit of 25 mph.

Understanding traffic impacts for construction activities starts with reviewing existing traffic conditions in the busiest time period(s) of the day, which are the peak hours. This study uses evening peak-hour traffic counts derived from the West End Traffic Study to derive the peak hour volumes traveling outbound or west. This study estimated 600 to 650 vehicles per hour (vph) on Power Plant Road and 1,000 to 1,250 vph on SH 82 (Fox Tuttle 2022).

No recent studies have estimates of inbound traffic volumes in the morning peak hour, but inbound traffic backups and congestion commonly occur on SH 82 between 7:00 to 9:00 a.m. during the weekdays. These backups often extend past the Aspen Airport. Commuters into Aspen now try to avoid the backup on SH 82 by detouring over McClain Flats Road. In addition, CDOT's traffic counter data from 2018 and 2019 indicate no substantial difference in directional vehicle volume inbound and outbound between the hours of 10:00 a.m. and 3:00 p.m. The counter is west of Cemetery Lane.

Figure 50. Weekday Traffic Counts on SH 82 between Maroon Creek Road and Cemetery Lane



Source: CDOT 2024

While CDOT's counter and the West End Traffic Study counts were not in the same location, comparison of the data indicates CDOT's 2018 to 2019 peak hour volume data and the West End Traffic Study SH 82 peak hour volume data are similar in showing the S-curves, the Maroon Creek roundabout, and other traffic constrictions reduce capacity on SH 82 in this area to between 1,000 to 1,400 vph. From the West End Traffic Study, it appears Power Plant Road acts as a reliever route serving outbound traffic bypassing SH 82 by approximately 600 to 650 vph in the evening peak hour.

Transit service is another key piece for getting workers and visitors into the town. Currently, 814 buses cross Castle Creek on weekdays, and 841 buses cross on weekends. The Roaring Fork Transportation Authority anticipates the weekday number will rise to 841 crossings in 2025. It will be critical in any rehabilitation or reconstruction scenario that transit be prioritized as much as possible.

Three Aspen School District routes cross CCB twice per day for elementary students and again for the older kids. Aspen Country Day School also has at least one route crossing twice per day. In total, there are at least 14 school bus crossings.

6.2 Maintenance of Traffic Options

Bridge construction would require lane closures and greatly disrupt traffic movement along the already-congested SH 82. The following subsections discuss various detour and bridge options to manage traffic and prioritize bus services during construction or rehabilitation of CCB.

A summary of the traffic impacts for the alternatives is as follows:

1. Bridge Rehabilitation and Two-lane Bridge Construction
    a. Alternating single lane
    b. Inbound CCB lane with outbound detour—West End Detour (Power Plant Road)
    c. Outbound CCB lane with inbound detour—Temporary Detour across Marolt-Thomas
2. Three-lane Bridge Construction
    a. Centered: One-lane bridge during all construction phases with companion detour
    b. Faster: One-lane bridge during Phase 1 construction with companion detour; two-lane traffic across bridge during other construction phases
    c. Shifted: Two traffic lanes across bridge during all construction phases

Figure 51. Outbound and Inbound Detour Options during Castle Creek Bridge Reconstruction or Rehabilitation

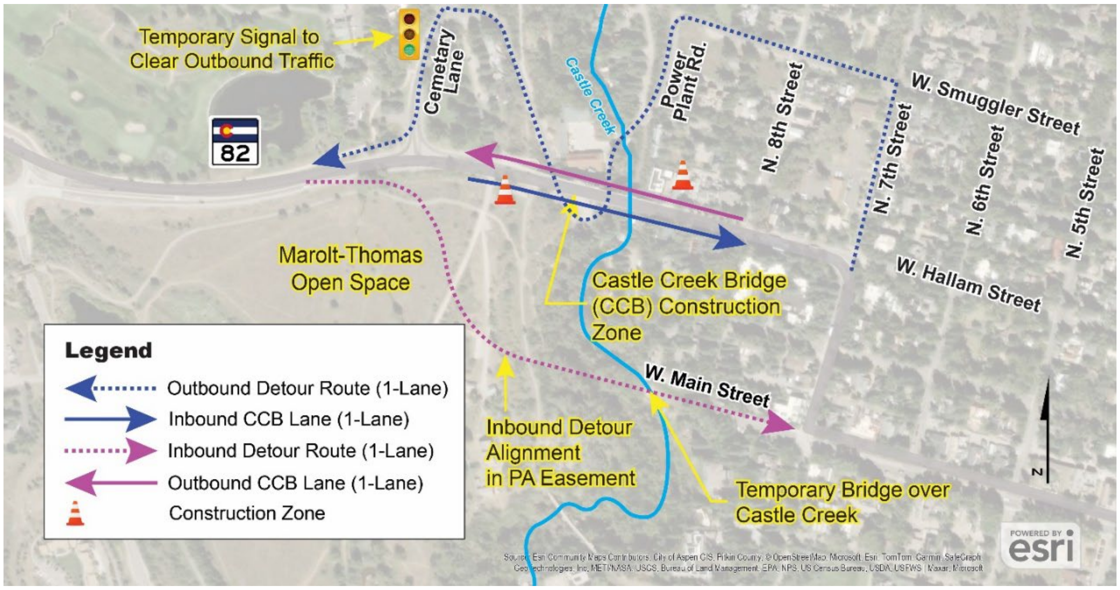


Table 10 summarizes the detour scenarios and their performance.

Table 10. Summary of Maintenance of Traffic Options and Performance

Maintenance of Traffic Scenario	Handles Outbound Peak Period	Handles Inbound Peak Period	Handles Emergency Egress	Periodic Closures of Detour	Pedestrians and Bicycle Impacts	Environmental Impacts	Maintenance of Traffic Cost	Transit/School Bus Delays	General Vehicle Travel Delays**
Existing Conditions (Baseline)	✓	✓	✓	N/A	Low	N/A	N/A	Low	½ hr
Alternating Single Lane*					Med	Low	Med	High	7 hrs
Inbound CCB Lane with Outbound Detour – West End Detour (Power Plant Road)*		✓		✓	High	Low	Med	Med	5 hrs
Outbound CCB Lane and Inbound Detour – Temporary Detour Marolt-Thomas	✓	✓	✓		Low	Med	High	Low	½ hr
Phased Two Way Traffic Across CCB	✓	✓	✓		Low	Low	Low	Low	½ hr
Legend:	Low to Zero Impacts: Green			Medium Impacts: Orange			High Impacts: Red		

*Alternating Single Lane and West End Detour deemed not feasible due to extended traffic delays & gridlock

**Maximum Estimated Delay in Queue. Actual travel delays would require a traffic model.

6.3 Bridge Rehabilitation and Two-lane Bridge Construction

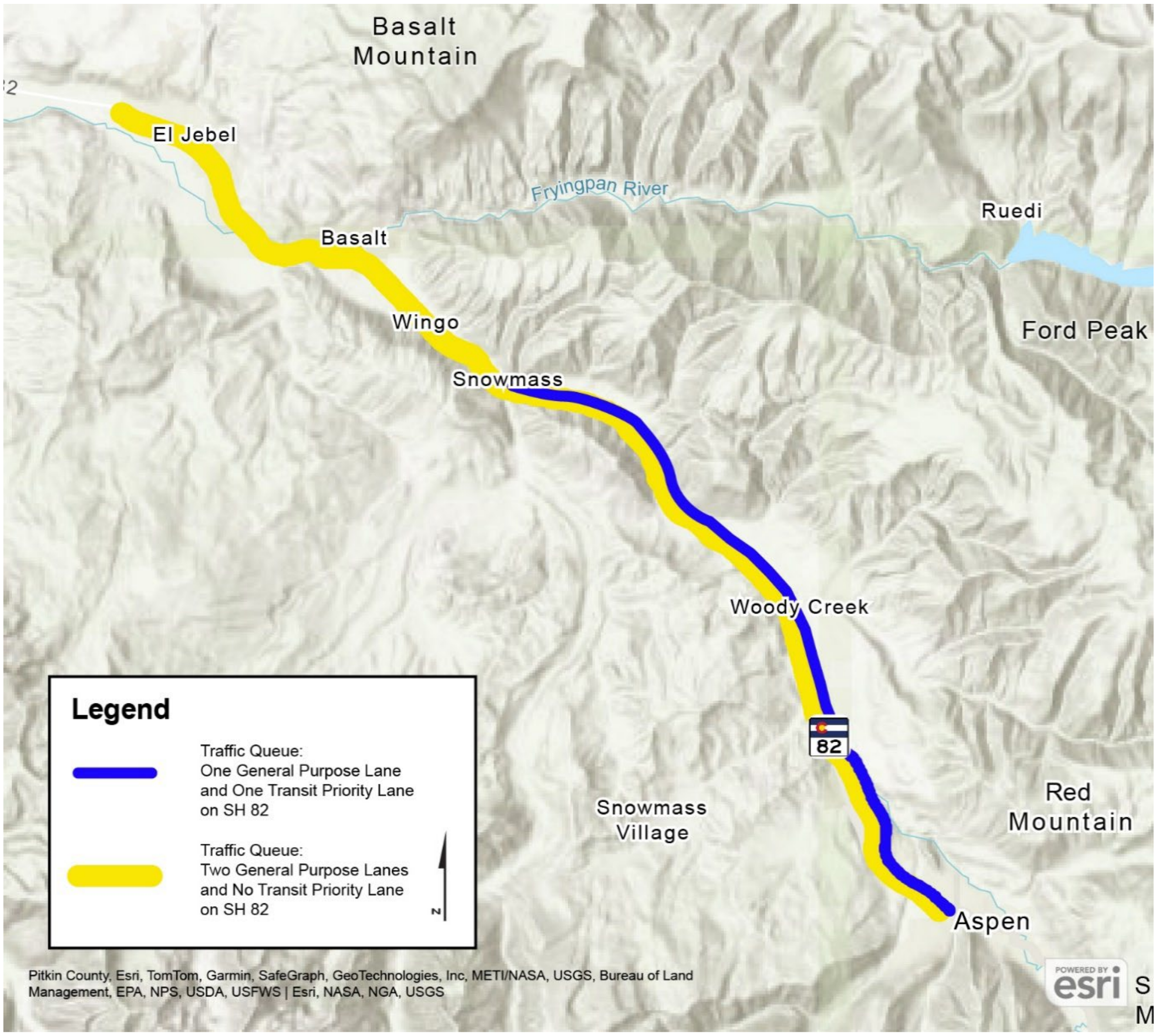
Options to accommodate traffic during a two-lane bridge rehabilitation or two-lane bridge construction are detailed in the following subsections.

6.3.1 Alternating Single Lane

A common approach to managing traffic during bridge construction is operating and signalizing an alternating single lane. This operation is applicable for maintaining one lane of traffic across the bridge for either rehabilitation or reconstruction work. The Highway Capacity Manual (TRB 2010) indicates a single lane work zone can pass 1,400 vph. Under a single lane two-way operation, that capacity would be split between each direction of travel. Assuming 15% of bridge capacity is lost to the safe clearance bridge traffic, each direction of travel is estimated to have a capacity of roughly 600 vph under an alternating (inbound/outbound) single lane operation.

As shown on Figure 51, inbound traffic counts exceed 600 vph between 7:00 a.m. and 7:00 p.m., while outbound traffic counts exceed 600 vph between 8:00 a.m. and 8:00 p.m. Therefore, without efforts to mitigate or reduce travel demand, any alternating single lane configuration would generate a continuously increasing backup condition on either side of construction. Weekday queues in the inbound direction would begin at 7:00 a.m. and last into the evening, with several thousand vehicles in a queue extending beyond the town of Basalt. Weekday queues in the outbound direction would begin around 8:00 a.m. and last into the evening. Several thousand vehicles would queue on SH 82 in Aspen and on surrounding Aspen streets, causing a gridlock condition in west Aspen and SH 82. Refer to Figure 52 for a visualization of the inbound queue length.

Figure 52. Inbound Queue Length with Alternating Single Lanes Across the Bridge



For this scenario, temporary signals would be placed at each end of the bridge to facilitate the alternating single lane operations. Where space permits, buses could be moved to the front of the signal queue, both transit and school bus service would be impacted and delayed up to 1 hour each day. Pedestrian traffic across the bridge would be maintained or accommodated, depending on whether rehabilitation or the two-lane bridge replacement is chosen.

Emergency service response times in an alternating single lane configuration would be severely impacted and delayed by the construction traffic, but the usual protocol of moving to the side to allow emergency vehicles to pass would still be applicable and allow access to the hospital, albeit with slower response times.

To plan for a wildfire or other emergency evacuation, a construction protocol would be developed to require inbound traffic to be stopped and outbound traffic to have right of way on the single lane on the bridge for as long as it took to clear town. Current estimates from the City indicate the time needed to evacuate the town would take over 11 hours. This time estimate factors reversing both existing lanes across the bridge and through the roundabout to facilitate outbound flow. All bridge rehabilitation and single lane construction options would limit outbound capacity to a single lane across the bridge, reducing the outbound flow and increasing the amount of time to evacuate.

The alternating single lane configuration is not a viable option for maintaining normal traffic flow, transit priority, or providing emergency egress.

Because of the long queues, gridlock, and egress risks, the alternating single lane option is untenable for SH 82 travelers and destinations served by these users, including local businesses.

Removing the alternating signals would still allow traffic to navigate the bridge in one direction but would need a companion detour in the other direction. The following subsections describe potential one-lane detour used in conjunction with a single lane across phased bridge work (Figure 51).

6.3.2 Inbound Castle Creek Bridge Lane with Outbound Detour—West End Detour (Power Plant Road)

One lane of (outbound) traffic could detour down north 7th Street to West Smuggler Road and Power Plant Road (currently limited by restrictions on oversize vehicles) while the inbound lane uses one lane over the bridge during phased construction. Along Power Plant Road, turning radii, pavement widths, curve widening, vertical grades, and the small bridge over Castle Creek would be evaluated to determine whether a one-way (outbound) condition would allow the use of oversize vehicles on the route. As noted previously in this section, the current evening peak hour traffic volume on this route is approximately 600 to 650 vph; however, roadway improvements and an added signal on Cemetery Lane would enhance the capacity of this detour to between 800 and 1000 vph. However, Jacobs estimates there would still be a substantial backup into the west end neighborhoods because the total demand can't be served. Maximum delays to travelers are estimated to be up to 5 hours, in the evening peak hour for each day of construction.

Mitigation options are available, such as a temporary signal at the intersection of Cemetery Lane and Power Plant Road, and modifications to the signal cycle at SH 82 and Cemetery Lane to help clear traffic faster. Other travel demand mitigation options, such as increased bus service to the Brush Creek Intercept Lot, would also be explored. Depending on the detour scope and needs, improvement costs to Power Plant Road is estimated to be $3 to $5 million. And during construction or rehabilitation of CCB, Power Plant Road, which winds underneath the bridge, may be closed periodically, limiting its use as a reliable detour. Closures could be related to falsework placement or adjustments under the bridge or the movement of equipment and materials into staging areas below the bridge. Falsework is needed for the preferred cast-in-place concrete construction technique. If a steel solution was picked, then cranes would be occupying portions of Power Plant Road.

Under this scenario, travelers trying to access the hospital and high school and those trying to evacuate for emergencies would experience similar delays as those described in the alternating single lane configuration. Pedestrian traffic would be accommodated across the bridge or along Power Plant Road, depending on which construction alternative is selected.

Transit and school bus priority could be managed by keeping these essential services on the current SH 82 route and using flaggers at each end of the bridge to send them across the bridge with delays kept to less than 30 minutes. This would be similar to how the city manages transit priority when outbound traffic is shifted to Power Plant Road during CCB maintenance or repairs.

To plan for a wildfire or other emergency evacuation, a construction protocol would be developed to require inbound traffic to be stopped and outbound traffic to have right of way on the single lane on the bridge for as long as it took to clear town. Current estimates from the City indicate the time needed to evacuate the town would take over 11 hours. This time estimate factors reversing both existing lanes across the bridge and through the roundabout to facilitate outbound flow. All rehabilitation or reconstruction options that offer a single lane on CCB would limit outbound evacuations to a single lane across the bridge causing a choke point that would increase evacuation time, thereby creating a large risk for the City and its citizens.

Using a West End Detour (Power Plant Road) will not be a reliable, cost-effective detour considering travelers would experience up to 5-hour outbound delays and periodic detour closures.

6.3.3 Outbound Castle Creek Bridge Lane with Inbound Detour—Temporary Detour across Marolt-Thomas

A temporary one lane detour route could be constructed to split one lane of eastbound (inbound) traffic from SH 82 to the south, using an existing transportation easement across the Marolt-Thomas open space, and then spanning Castle Creek with a temporary bridge to join SH 82 on West Main Street into Aspen. This detour route, while similar in alignment to the preferred alternative, could be constructed across the existing easement to meet the peak morning incoming volumes of nearly 1,100 vph while maintaining a single lane on the existing bridge serving westbound (outbound) traffic. Depending on detour scope and needs, construction costs for this inbound detour are estimated at $13 million. Unlike the outbound detour with its periodic closures, the inbound route would remain open during construction or rehabilitation of CCB. This option temporarily impacts open space; however, the detour could be removed at the end of the CCB construction and the open space restored to its natural state.

In this scenario, access to the hospital and the high school would be similar to the existing condition. However, the inbound detour lane could be reversed and serve as outbound emergency evacuation egress in conjunction with CCB. This detour option provides the town an additional evacuation route during construction, and if desired, the temporary bridge could also be left in place for facilitating future evacuation efforts.

With an inbound detour pedestrian and bicycle traffic could be accommodated across Castle Creek via the temporary bridge and follow along the detour to make their trail connections. This would be a safer path for pedestrians since they would not need to cross a bridge under construction or rehabilitation.

Transit and school bus priority would be no different than current SH 82 conditions with minimal delays expected in normal peak period congestion.

During construction of the CCB, the inbound detour across the Marolt-Thomas open space is the most reliable detour for minimizing travel delays, prioritizing transit service, and providing safe pedestrian access and reliable evacuation routing. If the City determined it was a high priority to minimize the bridge project's duration and impacts to the community, this detour route could be evaluated for carrying two lanes of traffic (inbound and outbound).

6.4 Three-lane Bridge Construction

Options to accommodate traffic during a three-lane bridge construction are detailed in the following subsections.

6.4.1 Centered—One-lane Bridge During All Construction Phases with Companion Detour

For this option, the bridge is optimally placed to minimize construction impacts. The alternating single lane configuration and the West End Detour option are not a viable for maintaining normal traffic flow or providing emergency egress. Therefore, for this construction option, a single lane of outbound traffic across the bridge is maintained in conjunction with an inbound detour, as described in Section 6.3.3. All traffic impacts and mitigations identified in Section 6.3.3 apply in this scenario.

Pedestrian access could still be maintained across the bridge with minimal interruptions or diverted over the inbound detour with no interruptions and a safer path.

In this scenario, access to the hospital, high school, and emergency evacuation egress would be similarly delayed or mitigated according to Section 6.3.3.

6.4.2 Faster—One-lane Bridge During Phase 1 with Companion Detour; Two-lane Traffic During Subsequent Phases

For this option, the bridge is shifted to the south, allowing for sufficient width to provide continuous two lanes (inbound and outbound) during construction. However, in Phase 1, a single lane of outbound traffic across the bridge is required in conjunction with an inbound detour, described in Section 6.3.3. All impacts and mitigations defined in Section 6.3.3 apply in Phase 1 only. In all other phases, this option would function similarly to two-way existing conditions. Refer to Section 6.4.3 for additional traffic information.

Pedestrians and bicyclists would be routed under the bridge in all phases subsequent to Phase 1 to allow the width for two temporary lanes. Pedestrian access would be impacted when bridge construction impacts the pedestrian path below the bridge. However, with a separate inbound detour, safer and uninterrupted pedestrian access could be facilitated by rerouting pedestrians and bicyclists over to the inbound detour.

In Phase 1, access to the hospital, high school, and emergency evacuation egress would be similar to Section 6.3.3 with minimal delay. All other phases would be similar to existing conditions where both lanes would be converted to facilitate outbound flow during an evacuation event.

Transit and school bus priority would be no different than current SH 82 conditions with minimal delays expected in normal peak period congestion.

6.4.3 Shifted—Two-lane Bridge During All Phases

To facilitate this option, a shift of the replacement bridge is required. Jacobs estimates maintaining two lanes of traffic through the construction for a bridge replacement would have a minimal impact on the current traffic condition. The Highway Capacity Manual (TRB 2010) indicates a single lane work zone can pass 1,400 vph. The capacity of the temporary lanes on the bridge is comparable or greater than the prevailing S-Curve capacity limitations. Construction conditions may constrain flow through the S-Curves for short durations (up to 15 minutes) but is not expected to increase daily queues and delays noticeably more than existing conditions.

Pedestrians can use the northern sidewalk until the final phase (the construction of the north side of the bridge). During this final phase, pedestrians will be rerouted to Power Plant Road. It is possible to keep the pedestrians on the bridge during construction of all phases; however, it requires an even wider overbuild.

In this option, access to the hospital, the high school, and emergency evacuation egress would operate similarly to existing conditions. Both lanes would be converted to facilitate outbound flow during an evacuation event.

7. Overall Project Costs

This section summarizes potential overall project costs associated with bridge reconstruction or rehabilitation and factoring other ancillary costs. A matrix of overall project costs for the different bridge rehabilitation and reconstruction are found in Appendix K.

7.1 Other Project Costs

The bridge costs detailed in previous sections have noted additional project costs. This section elaborates on those costs.

7.1.1 Unlisted Construction Items

Unlisted items include high-cost items associated with construction costs.

- Mobilization – cost associated with mobilization of construction equipment, estimated at 15% of bridge construction cost.
- Removal of Existing Castle Creek Bridge –cost to safely demolish and remove the existing bridge while the new bridge is constructed.
- Utilities – cost associated with relocation of City-owned utilities. Relocation of private utilities will be at the expense of the utility owner.
- Roadway Approaches – cost of all roadway improvements needed to tie the new bridge to the existing roadway.
- Temporary Detour Construction – cost associated with construction of the detour route. For the inbound detour, the cost assumes constructing an 11-foot lane detour road with 4-foot shoulders, a 10-foot paved trail that parallels the detour road, and a temporary bridge over castle creek.
- Traffic Control and Transit/School Bus Priority – cost of daily maintenance of traffic operations and temporary traffic control devices (traffic signs, cones or barrels and variable message signs) or staff such as flaggers utilized during construction.
- Other Contingencies – a contingency factor of 20% was applied to account for all other unlisted construction items that may be smaller in nature.

7.1.2 Planning (NEPA) and Design

Engineering design costs are estimated as 15 percent of the construction replacement cost and 10 percent for a rehabilitation project. This would include developing preliminary and final plan drawings, specifications and final construction estimate for the City to bid the project.

NEPA costs for the bridge reconstruction options assume that a new Environmental Impact Statement (EIS) and Record of Decision (ROD) will be required. The estimated cost for the two-lane bridge scenario is $2M, the three-lane bridge is estimated at $3M.

7.1.3 Right-of-Way and Easements

Right-of-way acquisitions are anticipated for certain bridge alternatives as noted in Section 5.4. The cost associated with ROW was assumed to be $8,000 per square foot as directed by the City based on recent projects.

Construction easements may also be required for construction activities or staging of materials outside of the right-of-way. For estimating purposes, it is assumed that the bridge rehabilitation and two-lane replace will require a 10-foot wide easement along the south side of the bridge across two properties (~300-feet). The bridge reconstruction assumes a range between 10 and 20-foot wide easements through the same two properties on the south side of the bridge at similar lengths. Temporary easement costs are estimated at $1,500/square foot.

Table 11. Estimated Right-of-Way and Easement Costs

Material	Alternative	Easement Costs	ROW Cost
N/A	Rehabilitation	$4,500,000	$0
Cast-in-place Concrete	Two-lane Replace	$4,500,000	$0
	Three-lane Centered	$4,500,000	$0
	Three-lane Faster	$10,500,000	$4,600,000
	Three-lane Shifted*	$15,750,000	$5,400,000
Steel	Two-lane Replace	$4,500,000	$0
	Three-lane Centered	$4,500,000	$0
	Three-lane Faster	$10,500,000	$4,600,000
	Three-lane Shifted*	$15,750,000	$5,400,000

*Additional ROW and Easement costs could be needed along SH 82 (Hallam St) depending on final roadway alignment shift.

7.1.4 Public Involvement

Public involvement costs are those associated with public communication and stakeholder engagement during construction, such as informing people about lane closures, business access impacts, work hours and work zones, and detours. Public communication methods can include project information meetings/open houses, mass mailing of project information flyer/brochures, and project website/social media sites. Public involvement is estimated as $1,200/day for construction duration.

7.1.5 Construction Engineering and Indirects (CE&I)

Construction engineering costs include the supervision, inspection and quality control of materials during construction activities. Indirect costs are costs that are incurred for the benefit of the project (City staff and CDOT time) that are not project specific. The CE&I cost has been estimated as 26 percent of the construction cost. CDOT construction projects also factor the same 26 percent.

7.2 Overall Project Costs

The 2024 total project costs are a summation of the bridge costs and other project costs listed in Section 7.1. A year over year inflation rate of 4% was applied to project costs for calendar year 2028.

Table 12. Summary of Overall Costs for Options

Project Year	Rehabilitation	Two-Lane Replace (CIP)	Three-Lane Centered (CIP)	Three-Lane Faster (CIP)	Three-Lane Shifted (CIP)
Overall Project (2024)	$43.61M	$68.58M	$72.95M	$81.85M	$69.28M
Overall Project (2028)	$51.01M	$80.23M	$85.34M	$95.75M	$81.05M

Three-lane Shifted costs come in favorable to the Two-Lane Replace and Three-Lane Centered because the shifted option does not require a detour.

7.3 Economic and User Costs

Construction related delays would result in severe congestion or traffic halts that will increase user costs for residents and visitors. Travelers might be required to wait in lengthy queues or use detours to reach their destinations or might opt to postpone or cancel their trip. The construction related delay would add additional VMT and VHT during the construction period. While construction of the project would only occur over designated months, the roadway impacts are expected to be year-round. Depending on the alternative selected, construction is expected to last between two and eight construction seasons and the delay impacts are expected to last one to four years.

Delay costs can be estimated using input values provided by the U.S. Department of Transportation's (USDOT's) Benefit-Cost Analysis Guidance for Discretionary Grant Programs (USDOT 2024). The assumptions and inputs used in the user costs calculations used standard inputs and values from USDOT's BCA guidance such as vehicle occupancy rates, value of time, and vehicle operating costs. Mode split between personal vehicles and commercial vehicles and the estimated travel delay in the network was estimated by Jacobs traffic engineers.

Construction related travel delays are expected with the Alternating Single Lane scenario and the West End Power Plant Road detour scenario. Applying the appropriate traffic inputs and USDOT values we estimated both of these scenarios would exceed $100 Million in annual user costs. Granted some folks when stuck in construction traffic would begin to make behavior changes and switch to mass transit or stay home, but that level of change may only amount to 30 percent savings on user costs. In Subsection 6.3.1 and 6.3.2 we ruled out these options as untenable due to travel delay and the associated annual user costs (> $100M) reinforce that decision.

Both the Inbound temporary detour across the Marolt-Thomas (with outbound lane across the CCB) and the phased two-lane bridge construction (Three Lane Shifted) is estimated to function most closely to existing conditions and have negligible user costs when compared to a West End Power Plant Road detour and an Alternating Single Lane scenario.

8. References

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Appendix A
2022 Structure Inspection and Inventory Report by CDOT

Routine Inspection
Colorado Department of Transportation
Structure Inspection and Inventory Report (English Units)

Highway Number (ON) 5D: 082A _
Mile Post (ON) 11: 40.181 mi
Linear Ref. Sys. MP: 40.198 mi

Bridge Key: H-09-B | Inspection Date: 09/07/2022 | Suff Rating: 50.3 FO | G/F/P Condition: Fair

NBI Reporting ID:	H-09-B	Main Mat/Desgn 43A/B:	4	3	Bridge Cost 94:	5,598,087.00		
District (Region/Sect):	Reg 3 MSec 2	Appr Mat/Desgn 44A/B:	0	0	Roadway Cost 95:	559,808.00		
Tran Region 2T:	11	Main Spans Unit 45:	5		Total Cost 96:	8,397,131.00		
County Code 3:	097	Approach Spans 46:	0		Year of Cost Estimate 97:	2016		
097 PITKIN		Horiz Clr 47:	30.00 ft		Brdr Brdg Code/% 98A/B:	-2		
Place Code 4:	03620	Max Span 48:	90.0 ft		Border Bridge Number 99:			
ASPEN		Str Length 49:	423.6 ft		Defense Highway 100:	0		
Rte.(On/Under) 5A:	1	Curb Wdth L/R 50A/B:	8.0 ft	5.0 ft	Parallel Structure 101:	N		
Signing Prefix 5B:	3	Width Curb to Curb 51:	27.00 ft		Direction of Traffic 102:	2		
Level of Service 5C:	1	Width Out to Out 52:	40.0 ft		Temporary Structure 103:	_		
Direction Suffix 5E:	0	Deck Area:	16964		Highway Systems 104:	1		
Feature Intersected 6:		Min Clr Ovr Brdg 53:	99.99		Fed Lands Hiway 105:	0		
CO RD, CASTLE CREEK		Min Undrclr Ref 54A:	H		Year Reconstructed 106:			
Facility Carried 7:		Min Underclr 54B:	45.9 ft		Deck Type 107:	1		
SH 82 ML		Min Lat Clrnce Ref R 55A:	H		Wearing Surface 108A:	6		
Alias Str No.8A:		Min Lat Undrclr R 55B:	1.0 ft		Membrane 108B:	0		
		Min Lat Undrclr L 56:	0.0 ft		Deck Protection 108C:	0		
Prll Str No. 8P:		Deck 58:	6		Truck ADT 109:	3.00 %		
N/A		Super 59:	5		Trk Net 110:	1		
Location 9:		Sub 60:	6		Pier Protection 111:	!		
ASPEN		Channel/Protection 61:	9		NBIS Length 112:	Y		
Max Clr 10:	99.99	Culvert 62:	N		Scour Critical 113:	8		
BaseHiway Net12:	1	Oprtng Rtg Method 63:	1 LF Load Factc		Scour Watch 113M:	N		
IrsinvRout 13A:	082A	Operating Rating 64:	41.00		Future ADT 114:	26,750		
IrssubRout No13B:	00	Operating Factor 64:	-		Year of Future ADT 115:	2040		
Latitude 16:	39d 11' 43.50"	Inv Rtng Method 65:	1 LF Load Factc		CDOT Str Type 120A:	RGC		
Longitude 17:	106d 50' 3.08"	Inventory Rating 66:	24.60		CDOT Constr Type 120B:	00		
Detour Length 19:	1 mi	Inventory Factor 66:	-		Expansion Dev/Type 124:	A		
Toll Facility 20:	3	Asph/Fill Thick 66T:	4.0 in		Brdg Rail Type/Mod 125A/B:	K	2	
Custodian 21:	01	Str. Evaluation 67:	5		Posting Trucks 129A/B/C:	47.2	64.4	62.1
Owner 22:	01	Deck Geometry 68:	3		Str Rating Date 130:	11/14/2018		
Functional Class 26:	14	Undrclr Vert/Hor 69:	3		Within 1 Mile:	NO		
Year Built 27:	1961	Posting 70:	5 At/Above Lega		Special Equip 133:	88.00		
Lanes On 28A:	2	Waterway Adequacy 71:	9		Vert Clr N/E 134A/B/C:	X	99.99	0.00
Lanes Under 28B:	4	Approach Alignment 72:	8		Vert Clr S/W 135A/B/C:	X	99.99	0.00
ADT 29:	25,000	Type Of Work 75A:	-2		Vertical Clr Date:	09/14/2016		
Year of ADT 30:	2020	Work Done By 75B:	!		Weight Limit Color 139:	0, White		
Design Load 31:	5 MS 18 (HS 20)	Length of Improvment 76:	424		Userkey 1, Insp System:	ONSYS		
Apr Rdwy Width 32:	44.00 ft	Insp Team Indicator 90B:	WHITE TEAM		Userkey 4, Insp Sched:	ODD SEP C20		
Median 33:	0	Inspector Name 90C:	MARSTELLERE		Userkey 5, UW Sched:			
Skew 34:	0 °	Frequency 91:	24 months		Userkey 6, Pin Sched:			
Structure Flared 35:	0	FC Frequency 92A:			FHWA Bridge Risk:	LOW		
Sfty Rail 36a/b/c/d:	0 0 0 0	UW Frequency 92B:			FHWA UW Risk:	NA		
Rail ht36h:	55.0 in	SI Frequency (Pin) 92C:			FHWA Load Rating Risk:	LOW		
Hist Signif 37:	5	FC Inspection Date 93A:			CBTE:	Eligible		
Posting status 41:	A	UW Inspection Date 93B:			Inspection Key:	QSQG		
Service on/un 42A/B:	5 6	SI Date (Pin) 93C:			Date Entered:	9/8/2022 12:00:0		
					Entered By:	MARSTELLERE		

Inspection Type:	Regular NBI
EOR:	Unknown

Data Responsibility: Asset Management | Inspection | Rating

Element Inspection Report

Elm/Env	Description	Unit	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4
12/1	**Re Concrete Deck**	**sq.ft**	16945	92%	15600	4%	695	4%	650	0%	0

Covered with asphalt.

Underside:
Edge of the deck is broken on the left side at Abutment 1, see the 2006 photo.
30 square feet of scaling in the underside of the right sidewalk, above Pier 4.

Several light transverse cracks throughout, most have efflorescence, and some show rust staining.
A couple areas of map cracking with efflorescence. Roughly 35 square feet of map cracking with efflorescence in Bay B, forward of Pier 4.
Cracks with efflorescence and cracks with rust, respectively as follows: (square feet)
Bay 1A (8)(60), 1B (72)(0), 1C (82)(3), 1D (48)(2), 1E (40)(14),
Bay 2A (6)(36), 2B (20)(52), 2C (16)(8), 2D (32)(20), 2E (0)(60),
Bay 3A (12)(60), 3B (24)(8), 3C (24)(0), 3D (32)(8) 3E (6)(84),
Bay 4A (5)(66), 4B (46)(16), 4C (24)(4), 4D (30)(10), 4E (18)(36),
Bay 5A (12)(48), 5B (38)(6), 5C (45)(5), 5D (24)(0), 5E (0)(36).

Bay 5B has 1 square foot spall approximately 10 feet from Abutment 6 and delamination on the rear side of Cross-Frame 3.
6 square foot area of delamination cracking and minor spalls in Bay 5B, within 24 feet of Abutment 6.

Bay 5E has a spall with exposed rebar approximately 20 feet from Abutment 6.

Elm/Env	Description	Unit	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4
510/1	Wearing Surfaces	sq.ft	12708	100%	12708	0%	0	0%	0	0%	0

3.5 inches of asphalt. New overlay prior to 2020 inspection.

2022: Construction in right lane and too many cars on left side to view wearing surface.

Elm/Env	Description	Unit	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4
1080/1	Delamination/Spall/Patc	sq.ft	8	0%	0	13%	1	88%	7	0%	0

See Element 12 notes.

Elm/Env	Description	Unit	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4
1090/1	Exposed Rebar	sq.ft	1	0%	0	0%	0	100%	1	0%	0

See Element 12 notes.

Elm/Env	Description	Unit	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4
1120/1	Efflorescence/Rust Stai	sq.ft	1306	0%	0	51%	664	49%	642	0%	0

See Element 12 notes.

Elm/Env	Description	Unit	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4
107/1	**Steel Opn Girder/Beam**	**ft**	2532	0%	0	87%	2212	13%	320	0%	0

The sidewalk girders are also framed into the main girders via the diaphragms, see the 2006 photo.

Exterior (I-beams with thicker webs) Girders A and F (smaller shorter sidewalk girders) have R1 - R2 corrosion for the full length on the flanges, and R3 - R4 corrosion on the bottom flanges and lower portion of the webs, below the rail posts, see photos.
Interior (Riveted) Girders B - E have light R1 - R1 corrosion throughout, and spots of R2 corrosion below the joints, see the Tally Sheet.
Girder 4E, Bay 4D, Cross-Frame 4 has a hole through the top right corner.
Spot of R4 corrosion at the vertical web bearing stiffener for Girder F at Abutment 6, see Photo. R3 corrosion with section loss for the last 15 feet of Girder 5F at Abutment 6.
Actual quantity is 2212 linear feet in condition state 2 used 2062 to avoid double counting.

There was a 2011 project that ground out / removed some of the cracked tack welds made prior to riveting in 1961.
About 150 cracks through the tack welds were removed from the top and bottom connections of the vertical web stiffeners and diaphragms, see photos.
These cracks have been reported since 2000 and were ground out in 2011, see the 9-18-14 photo. Not all cracks were ground out.

Elm/Env	Description	Unit	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4
515/1	Steel Protective Coatin	sq.ft	2532	0%	0	0%	0	0%	0	100%	2532

Failed in areas of corrosion.

1000/1	Corrosion	ft	2382	0%	0	87%	2062	13%	320	0%	0

See Element 107 notes.

1010/1	Cracking	ft	150	0%	0	100%	150	0%	0	0%	0

See Element 107 notes.

205/1	**Re Conc Column**	**each**	8	75%	6	25%	2	0%	0	0%	0

Tall and slender columns with struts between them. All are water stained.
A couple of spalls on Column 2B, (1) with exposed rebar, about 15 feet above groundline at the forward right corner, has been covered with a thin layer of grout.
Columns 2B and 3B have some spots of delamination and some small patched spalls.
Patch on Column 3B is only lightly covered, not a full depth column patch.

215/1	**Re Conc Abutment**	**ft**	80	88%	70	13%	10	0%	0	0%	0

Top of Abutment 1 behind the girders was replaced the summer of 2011, and the rockers were reset.
Abutment 1 is covered with debris due to construction on the deck.
Abutment 6 has some light scale and (2) minor delaminations with exposed rebar on the left side.

234/1	**Re Conc Pier Cap**	**ft**	164	78%	128	17%	28	5%	8	0%	0

Moderate - heavy water staining on all.
Light scale and delamination on the faces of the caps under the curb lines due to water leaking through.
Some hairline - light vertical and diagonal cracks.
Pier 2 Cap rear face under Girder E has a 4 foot x 4 foot spall with exposed, corroded rebar.
Pier 3 Cap rear face has delamination and cracks with efflorescence in the rear face under Girder 2B and there is delamination above Column 3B and for 8 feet along the top on the right side. The forward side of the pier cap is similar. Some delamination at Pier 3, Bay D and below Girder 3E.
Pier 4 Cap rear side has delamination, shallow spalls, and up to 0.020 inch wide horizontal and diagonal cracking below Bay 3C. The forward side of the Pier 4 Cap is delaminating below Girder B and E.
Pier 5 Cap right side under Bay 4D is starting to delaminate. On the left side of the forward face at Pier 5 there is delamination with exposed rebar in Bay 5A (9 sq. ft.).

260/1	**Slope Prot/Berms**	**(EA)**	2	50%	1	50%	1	0%	0	0%	0

(2) sections of metal wall (cribbing) holding the slope back along the roadway, and a timber retaining wall along the bike path at Abutment 1.
A section of steel cribbing has lost its backfill because the slope is so steep, see the 2006 photo.
Per Aspen Maintenance personnel - the timber wall (for the bike path) has to be pushed back up / vertically every year.
Minor washing of the soft dirt and cobble slope at Abutment 6. Some erosion trenches in the top of the berm.

301/1	**Pourable Joint Seal**	**ft**	180	100%	180	0%	0	0%	0	0%	0

Joint work at Abutment 1 during 2022 inspections, see photos. Previously noted: Abutment 1 has polymer mortar epoxy ends dams, was silicon sealed during the 2020 inspection, see 2020 photo.
Saw cut and sealed at Pier 2, Pier 3, Pier 4, Pier 5, and Abutment 6. Roadway joints are in good condition.
The joints in the left sidewalk are in expectable condition, but the right sidewalk joints are not sealed well (sidewalk joints don't appear to be accounted for in total quantity so all left in Condition State 1).

311/1	**Moveable Bearing**	**each**	30	0%	0	100%	30	0%	0	0%	0

Main girder bearings are slightly in the expansion position, but not excessively.
There are loose anchor bolt nuts at all of the bearings.
R1 corrosion on all, and spots of R2 corrosion on those at Abutment 1. Currently are surrounded by debris.
Pin bolts backing out at Bearings 1C, 1D, 3E and 3F.
Missing pins at 2D and 3E were replaced in 2012.

515/1	Steel Protective Coatin	sq.ft	30	0%	0	0%	0	0%	0	100%	30

Failed in areas of corrosion.

313/1	**Fixed Bearing**	**each**	6	0%	0	67%	4	33%	2	0%	0

Only found at Abutment 6 (steel risers appear as rockers but they are flat on the bottom).
Some R1 corrosion on all. R2 corrosion forming on Bearing 6E.
The grout pad is broken / missing (2" x 6" area missing), with about 12 percent bearing loss, under Girder 6B, see photos.
Grout pad breaking up under Bearings 6C - 6E.
Grout pad below Bearing F was regrouted in 2011, but the grout is broken out and missing under the rear side. There is roughly 15 percent bearing loss with 1 inch of the pad missing for the full width on the rear side, see photos. Bearing is starting to tip towards the rear of the structure, continue to monitor.

515/1	Steel Protective Coatin	sq.ft	6	0%	0	0%	0	0%	0	100%	6

Failed in areas of corrosion.

326/1	**Bridge Wingwalls**	**(EA)**	4	100%	4	0%	0	0%	0	0%	0

Stub extensions of the abutments.
Some minor delaminations in Abutment 6 Left Wingwall.

329/1	**Sidewalk/Median/Curb**	**(LF)**	848	0%	0	97%	823	3%	25	0%	0

Several moderate transverse cracks throughout.
Light - moderate scale on the tops and faces; worst spot on right at Pier 4; 8 feet of heavy scale.
There are a few spots of exposed rebar in the faces of the gutters due to insufficient concrete cover. The face of the right sidewalk has moderate scale throughout with exposed rebar in various places.
Minor spall at the forward left edge of the sidewalk at Abutment 6, see photo.
A new widened sidewalk was installed on the left side prior to the 2020 inspection.

330/1	**Metal Bridge Railing**	**ft**	848	83%	707	17%	141	0%	0	0%	0

Painted Type K bridge rail.
The rails are on the exterior edge of the sidewalks, and the left pedestrian rail is significantly taller than the right.
There is automotive rail on the inside of the left sidewalk.
R1 corrosion on all posts.

515/1	Steel Protective Coatin	sq.ft	848	83%	707	0%	0	0%	0	17%	141

Failed at areas of corrosion.

501/1	**Channel/Bank**	**(EA)**	1	100%	1	0%	0	0%	0	0%	0

Castle Creek.
Good alignment way below the bridge, but Pier 4 is right in the center of the swift flowing channel; no scour though.
Rocky bed.
Mountainous drainage.

9504/1	**BankCond**	**(EA)**	1	100%	1	0%	0	0%	0	0%	0

Rocky. Lined with trees and bushes.

Inspection References and Definitions:

Crack Width Descriptions for Reinforced Concrete:
Insignificant cracking (in.) = Less than 0.012 wide
Moderate cracking (in.) = 0.012 to 0.05 wide
Wide cracking (in.) = Greater than 0.05 wide

Crack Width Descriptions for Prestressed Concrete:
Insignificant cracking (in.) = Less than 0.004 wide
Moderate cracking (in.) = 0.004 to 0.009 wide
Wide cracking (in.) = Greater than 0.009 wide

Rust Codes (R Codes):
R1 = Peeling of the paint, pitting, surface rust, etc., no measurable section loss.
R2 = Flaking, minor section loss (< 10% thickness loss).
R3 = Flaking, swelling, mod section loss (10% < thickness loss <30%).
R4 = Heavy section loss (> 30% thickness loss), may have holes through base metal.

Concrete Scaling Codes (S Codes):
S1 = Light scale up to 1/4" deep.
S2 = Moderate scale up to 1/2" deep with agg. exposed.
S3 = Heavy scale up to 1" deep with some agg. loose or missing.
S4 = Critical scale > 1" deep with reinforcing bars exposed and general disintegration of the concrete.

Maintenance Activity Summary

MMS Activity	Description	Recommended	Status	Target Year	Priority
352.02	Substruct-Cln Abutment/Pier Seat	9/25/2018	_	2026	Medium

Insure that the construction debris has been removed from Abutment 1.

355.01	Bearings-Clean Assemblies/Paint	9/14/2016	_	2026	Medium

Clean and paint the girders and bearings, especially the exterior girders because they are starting to lose section at most of the bridge rail post locations.

357.04	Bearings-Rehabiliatation	9/25/2018	_	2024	High

Repair the grout pads under Bearings 6B and 6F so there is no loss of bearing. The pads at 6C - 6E are beginning to crack and could also be repaired.

Bridge Notes (Inspection > Inventory > Admin)

Item 59 was changed to 5 and Item 60 was changed to 6 due to extensive work including eliminating tack weld cracks and rehabbing at Abutment 1.

With the new widened sidewalk on the left side, the A-40 might not have the reach to deploy from this side. There is roughly 9 feet to span from the inside of the traffic rail to the exterior of the utility conduit on the left side. The sidewalk is about 7.5 feet wide and there is a 55" high (from sidewalk) pedestrian rail on the exterior side.
Used the A-40 off the right side (South) during the 2010 inspection, but it was not used in 2012, immediately following the repairs. Used on the right side in 2014, 2016 and 2018. A40 was unable to be used due to (unannounced) construction on the deck during the 2022 inspection.

Utilities: (4) 3 inch diameter alloy conduits; (2) 4 inch diameter steel pipes, and (1) 2.5 inch diameter pipe in Bay A (broken at Abutment 6), below the left walk.
There is also an 8 inch diameter natural gas line and a 2.5 inch diameter PVC in front of Abutment 1.
Drain extensions were placed in Span 4, at both overhangs, in 2012 due to a land owner's complaint of water and icicles falling on her driveway, see the 9-18-14 photo.

Inspection Notes (Inspection > Condition)

TIME: 12:45 p.m.
TEMPERATURE: 83 degrees F
WEATHER: Clear skies but hazy

Scour Item 113 Documentation (Inspection > CDOT Bridge)

Bat Present At Bridge (Inspection > Inventory > Agency Items > userkey9)

-1

Inspection Access Requirements (Inspection > CDOT Bridge)

Uses A40. Unable to use A40 in 22, want to use A40 in 24.

Scheduling Notes (Inspection > Schedule)

Scope:

- [x] NBI
- [x] Element
- [ ] Underwater
- [ ] Fracture Critical
- [ ] Other

Type: Regular NBI

Team Leader Inspection Check-off:

- [ ] FCM's
- [ ] Vertical Clearance
- [ ] Posting Signs
- [ ] Stream Bed Profile
- [ ] Essential Repair Verification

Inspection Team: WHITE TEAM

Inspection Date: 09/07/2022

Inspector: Unknown

Inspector (Team Leader): ERIK MARSTELLER

Structure No. H-09-B
Inspection Date: 9/7/2022
Team: Gold

Active Construction at Abutment 1 on Joint

Construction Debris at Abutment 1

R3 - R4 Corrosion of Girder 5F near Abutment 6

Bearing Loss Due to Missing Grout Pad Under Bearing 6B

Structure No. H-09-B
Inspection Date: 9/7/2022
Team: Gold

Broken Bearing Pad under Bearing F at Abutment 6 15
Percent Bearing Loss

Bearing 6F isTilting to the Rear

Appendix B
H-09-B (Castle Creek Bridge), In-depth Superstructure Investigation Report by Engineering Operations, LLC (eO)

H-09-B (CASTLE CREEK BRIDGE)
IN-DEPTH SUPERSTRUCTURE INVESTIGATION REPORT

Prepared for:
JACOBS ENGINEERING, CITY OF ASPEN

Jacobs

CITY OF ASPEN

Prepared by:
Engineering Operations, LLC

Engineering Operations, LLC.

November 28th-30th, 2023

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LIST OF FIGURES

BRIDGE INFORMATION

Table 1: Bridge Information

Structure ID	H-0-9-B
Structure Alias	Castle Creek Bridge
Place code (Item 4)	03620 - Aspen
Feature Intersected (Item 6)	Castle Creek, Power Plant Rd
Facility Carried (Item 7)	SH 82
Detour Length (Item 19)	1 mi.
Year Built (Item 27)	1961
Lanes On (Item 28A)	2
ADT/Year (Item 30/31)	25,000/2020
Posting Status (Item 41)	A (not posted)
Main Spans Unit (Item 45)	5
Structure Length (Item 49)	423.6 ft.
Width Curb to Curb (Item 51)	27.00 ft.
Width Out to Out (Item 52)	40.00 ft.
Superstructure Rating (Item 59)	5
Structure Type (Item 120A)	RGC (Riveted Girder Continuous)

INSPECTION TEAM

Table 2: Inspection Team

Role	Personnel	Signature
Team Leader	Nate Proffitt	
Assistant Inspector	Jonathan Ivey	
QC Reviewer	Remy Stern, PE	

1.0 EXECUTIVE SUMMARY

The intent of this inspection was to collect comprehensive data on the current condition of the superstructure elements of the Castle Creek Bridge (H-09-B) – predominantly pertaining to the (6) girders that run the full length of the structure. This report details the location, severity, and quantities of a multitude of defects found throughout the superstructure elements. Inspection of the deck and the substructure was not included as part of the scope of this investigation and has not been documented within this report. For details on the condition of the deck and substructure elements, see the most recent routine inspection report data as shown in the Structure Inventory & Assessment (SIA), from the CDOT inspection dated 09/07/2022.

Castle Creek Bridge is a 5-span steel multi-girder structure originally built in 1961. The girders are riveted, built-up sections with vertical stiffeners spaced every 3 foot 9 inches on both sides of the girders. The girders are continuous over the piers and thus carry negative and positive moments. In order to gain close proximity and hands on access throughout the lengths of the girders, inspectors utilized an Aspen Aerial UB60 UBIT truck. Inspection was primarily visual, with NDT equipment used as deemed necessary from visual inspection.

Special attention was given to areas where vertical stiffeners were tack welded to either the girder web (steel plate 3/8” x 54”) or girder top/bottom flanges (double angles 6” x 4” x 5/8”). Many tack welds are still present on the structure from original construction fit-up and are in-tact. However, many have been found to be cracked and self-arrested (SA) – this condition refers to a completely cracked through tack weld where stress has been relieved and there is no longer potential for crack propagation into the base metal. The remaining tack welds that were cracked but not self-arrested (NSA) were found to be confined to the welds and not propagating into the base metal of the girders. Although there were no locations where cracking was found to enter the girder base metal at NSA tack welds, since the areas are not self-arrested there is a possibility that the crack could make its way into the base metal in the future as it grows. Although it is more likely the crack would make its way through the tack weld and become a self-arrested location, there is always the potential for the crack to migrate into the girder itself.

Surface corrosion was present in the interior girders (Girders B-E) primarily in 10-15 foot long segments under the deck joints (at piers) and also at the girder ends. No section loss was observed in the interior girders. The exterior sidewalk girders (Girders A & F) had more significant corrosion and section loss. Laminar corrosion was widespread in the lower 6 inches of the web and in the top of the bottom flange resulting in significant section loss to the member in these areas. Some isolated areas of less severe laminar corrosion were found in the tops of the exterior girder webs, typically within 6 inches of the top flange.

The superstructure is in overall fair condition. Prior to the inspection, the NBI rating for Item 59 (Superstructure) was a "5". Per the in-depth investigation, it was found that this rating is accurate and appropriate for the superstructure in its current state. It is recommended to program a cleaning and painting of the exterior girders in order to prevent against further corrosion and maintain current section thicknesses found in the girders. It is also recommended to clean and paint the interior girders within close proximity to the joints in order to arrest current corrosion and prevent future section loss that may occur. Finally, it is recommended to attempt to grind out tack welds that have been identified as "NSA" in order to prevent cracks from forming in the base metal of girders. A more comprehensive (and costly) solution would be to eliminate all tack welds on the structure.

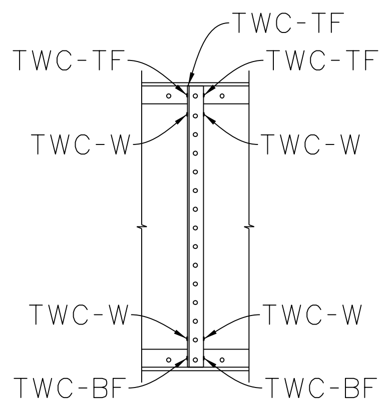
2.0 PROCEDURE/SCOPE

A team of (2) inspectors were on site for a detailed inspection of the superstructure elements between November 28th, 2023 and November 30th, 2023. An Under Bridge Inspection Truck (UBIT) was used to visually inspect all steel girders hands on. The UBIT bucket was deployed from the south lane of the structure to avoid interference within the sidewalk at the north. Traffic control was utilized to fully close the south lane of the structure during inspection times. Fatigue prone details, cracking, areas of corrosion/section loss were closely inspected and documented.

The structure and associated stiffeners have been inventoried from West to East; with substructure numbering as follows: Abutment 1, Pier 2, Pier 3, Pier 4, Pier 5, and Abutment 6. Each girder line included 113 stiffeners on each face of the girder – these have been numbered from west to east in accordance with the bridge inventory direction. Girders have been labeled A-F from left to right when looking in the direction of inventory.

3.0 SUMMARY OF FINDINGS

There are numerous tack welds at each stiffener location throughout the lengths of the interior girders. Tack welds were made during construction to fit-up stiffeners to the girders while rivets were being installed. Typically, there are (8) tack welds per stiffener; (2) at the top flange, (4) at the web, and (2) at the bottom flange. In about half of the stiffeners, there is an additional tack weld at the top of the stiffener to the horizontal leg of the top flange angle, and very rarely another tack weld between the vertical leg of the top flange angle and the girder web. See the image to the right for an illustration of tack weld locations, accompanied by a code legend below. See also Figure 9 for an annotated photograph showing a typical stiffener and associated tack welds. The codes that have been used in the legend and graphic shown here have been used throughout the report and drawings in order to more quickly identify the location of a tack weld within the cross section of the girder.



GIRDER FACE VIEW

These tack welds are of poor quality as they were only intended to be used for ease of construction and are prone to cracking. At the time of original construction, it was typical for contractors to leave tack welds like the ones found on the Castle Creek Bridge in place after project completion. However, in more recent construction, standards have become more stringent and typically require tack welds used for fit-up to be ground out or incorporated into a full structural weld prior to acceptance.

CODE LEGEND
TWC-W: TACK WELD CRACK - WEB TO STIFFENER
TWC-BF: TACK WELD CRACK - BOTTOM FLANGE TO STIFFENER
TWC-TF: TACK WELD CRACK - TOP FLANGE TO STIFFENER

ADD-ON CODES:
*: NOT SELF ARRESTED
-R: EFFECTIVE REPAIR PERFORMED
-P: POSSIBLE CRACK
(#): NUMBER OF CRACKS OF SAME TYPE

Although many locations where tack welds were made have cracked through (self-arrested and considered benign), the concern is focused on welds that have cracked but have not self-arrested (NSA). At these locations, cracks cannot definitively be seen to make their way through the entire tack weld and are assumed to still be growing. If too much fusion was obtained during tack-welding between the filler metal and the base metal the potential is increased for these cracks to propagate into girder cross section elements. As seen throughout the structure at self-arrested locations, this scenario was not found to be present – but nonetheless is still possible.

3.1 Interior Girder Finding Details

There are (452) stiffener locations throughout the four interior girders, each with (8-9) tack welds, totaling an estimated (3850) tack welds. The majority of the tack welds are still in place and have not cracked. Approximately (415) of the tack welds are cracked but have self-arrested (SA) or have been ground out during efforts in 2011 and are considered to be a benign condition. Approximately (36) tack welds were found to be cracked and not self-arrested. It is imperative to monitor these cracks for propagation during routine inspections if they are not repaired. All tack weld cracks that were identified during our inspection are visually displayed in Appendix 6.2 and tallied in table form in Appendix 6.3.

The lower stiffener weld of stiffener 16 at the north face of Girder C is fractured. The stiffener weld terminates at the end of the vertical angle leg and the crack does not extend upward into the stiffener at this location. See Figure 21 and Figure 22 for photographs of this condition. Additionally, there is (1) sheared rivet head at this location, see Figure 23.

Surface corrosion was found in the interior girders (Girders B-E) primarily at the piers under the joints, see Figure 24. Surface corrosion and minor pitting (negligible section loss) was observed in the bottom of the bottom flanges of Girders B and E at the piers, see Figure 25. Some areas in the girder ends at the abutments exhibited laminar corrosion with negligible section loss, see Figure 26.

Bearing stiffeners at the abutments and piers were made of double angles and had pack rust between the faying surfaces up to 1/2 inch thick, bowing the stiffener legs in some places, see Figure 27 and Figure 28. Section loss was negligible in these areas but the pack rust over time will work the angles apart from each other, further distorting the angle shape.

Stiffener 62 at the north face of Girder B is deflected up to 1 inch out of plane over a 6 inch height, near the bottom 1/3 point of the stiffener, see Figure 29.

3.2 Exterior Girder Finding Details

Significant corrosion was found in the exterior faces of the exterior girders (Girders A & F). These girders predominantly serve to support the sidewalk and do not see significant live loading from the travel lanes. The corrosion was laminar in form and concentrated primarily in the bottom 6 inches of the web and on the top of the exterior bottom flange. Several isolated locations have less severe laminar corrosion within the top 6 inches of the web. The laminar corrosion was present in the full length of the north exterior girder (Girder A) and primarily under the rail posts in the south exterior girder (Girder F). Upon cleaning of several of the worst areas of laminar corrosion in the webs, section loss was determined to be up to 40% in very localized areas. More typically, section loss is in the 10-30% range within the areas of corrosion.

- Original web thickness = 0.49 inch
- Minimum remaining web thickness = 0.29 inch (40% Section Loss)
- Average remaining web thickness = 0.39 inch (20% Section Loss)

The vertical web bearing stiffener of Girder F at Abutment 6 had a 2 inch diameter corrosion hole at the bottom of the stiffener adjacent to the bottom flange (100% section loss), see Figure 33.

Both exterior girders had visible load sag with 3+/- inches of downward displacement near mid span locations as seen when looking along the length of the girder, see Figure 34.

3.3 Girder Bearing Finding Details

Bearings were in fair condition with surface corrosion throughout, see Figure 35. Some bearings had anchor bolt nuts that were backing out and/or missing lock nuts, see Figure 36. The bearing pedestal for the Girder A fixed bearing at Abutment 6 had an impending spall causing loss of bearing area, see Figure 37. The fixed bearing for Girder F at Abutment 6 was tipped away from the abutment, bending the anchor bolt, see Figure 38. Several of the grout pads at the piers had minor deterioration with no reduction to bearing area.

3.4 Diaphragms

Diaphragms between exterior girders and interior girders consist of rolled C-Channels. Diaphragms between interior girders were made up of a series of steel angles and plates in the form of cross bracing. Diaphragms located away from piers were in overall good condition. Cross bracing between interior girders at pier locations typically have areas of surface corrosion. The C-Channel diaphragms between the exterior and interior girders at piers had laminar corrosion with 10-30% section loss of the webs.

4.0 MAINTENANCE RECOMMENDATIONS

The following recommendations have been made with the intent to extend the life of the steel girders and reduce the likelihood of tack weld crack propagation into the structural steel members. Completing the tack weld removal recommendations would limit the possibility of fatigue cracking in the steel girders. This could be a viable avenue to reduce the frequency of hands-on inspection of the girders in the future.

- Clean and paint exterior girders.
- Clean and paint portions of interior girders where surface corrosion has initiated through paint.
- Remove cracked tack welds that are considered to not be self-arrested (see Appendix 6.2 and Appendix 6.3 for location details).
- Remove all tack welds from structure if funds allow to avoid future close-proximity monitoring at higher frequencies.
- Continue to monitor tack welds during future routine bridge inspections until they can be removed.

5.0 CONCLUSION

The superstructure of H-09-B (Castle Creek Bridge) was confirmed to be in fair condition with NBI Item 59 = 5 after completion of the in-depth superstructure investigation. Many tack weld cracks were found to have been cracked through and self-arrested. Several locations were found to have incomplete tack weld cracks where future propagation/extension of the crack may occur. None of these locations were found to have cracks propagating into the base metal of the girder section. The structure should continue to be accessed via under bridge inspection truck during routine inspections in order to closely monitor tack welds that are cracked but not self-arrested as well as the development of new tack weld cracks.

6.0 APPENDICES

6.1 PHOTO LOG

Figure 1: South Elevation

Figure 2: General Superstructure of Spans 1 & 2

Figure 3: General Superstructure of Spans 3, 4, & 5

Figure 4: Span 1 Underdeck, Looking West

Figure 5: Span 2 Underdeck, Looking West

Figure 6: Span 3 Underdeck, Looking West

Figure 7: Span 4 Underdeck, Looking West

Figure 8: Span 5 Underdeck, Looking East

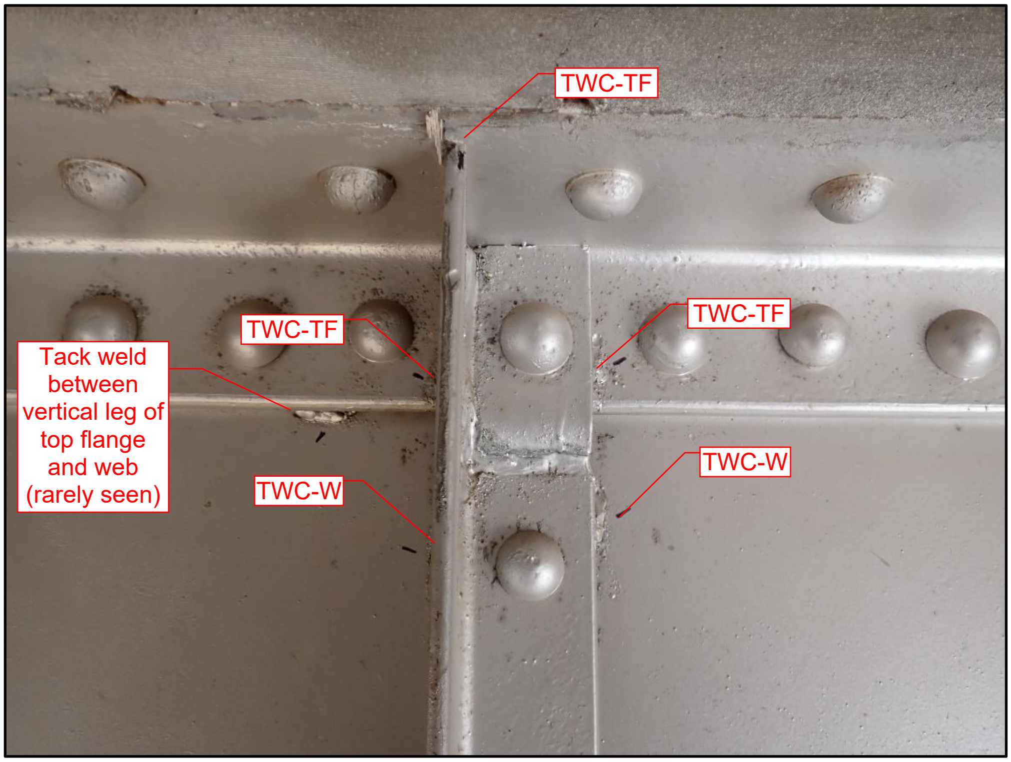


Figure 9 :Typical Stiffener Tack Weld Configuration with Crack Code Call-Outs

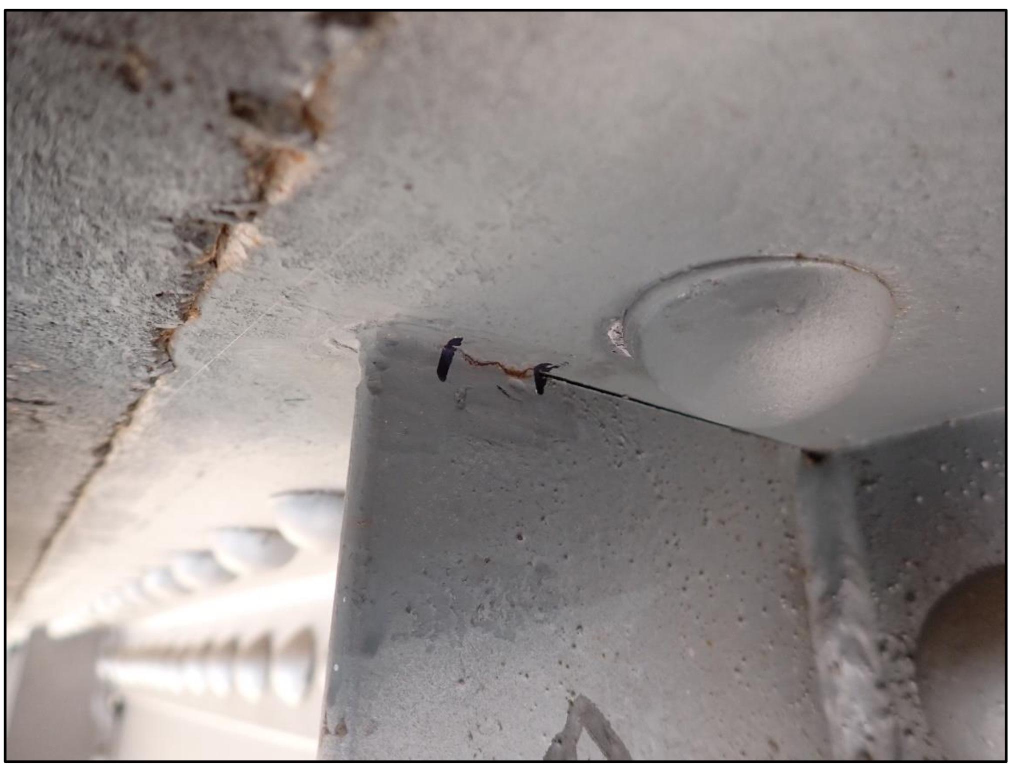


Figure 10: NSA Cracked Tack Weld – Girder B South Face – Stiffener 19 to Horizontal Leg of Top Flange Angle

Figure 11: SA Cracked Tack Weld – Girder B South Face – Stiffener 17 to Horizontal Leg of Top Flange Angle

Figure 12: NSA Cracked Tack Weld – Girder C North Face – Stiffener 105 to Vertical Leg of Top Flange Angle

Figure 13: NSA Cracked Tack Weld – Girder D North Face – Stiffener 97 to Vertical Leg of Top Flange Angle

Figure 14: NSA Cracked Tack Weld – Girder E North Face – Stiffener 107 to Vertical Leg of Top Flange Angle

Figure 15: SA Cracked Tack Weld – Girder C South Face – Stiffener 16 to Vertical Leg of Top Flange Angle

Figure 16: SA Cracked Tack Welds – Girder B South Face – Stiffener 41 to Vertical Leg of TF Angle and Web

Figure 17: NSA Cracked Tack Weld – Girder E South Face – Stiffener 55 to Web

Figure 18: NSA Cracked Tack Weld – Girder C North Face – Stiffener 109 to Web

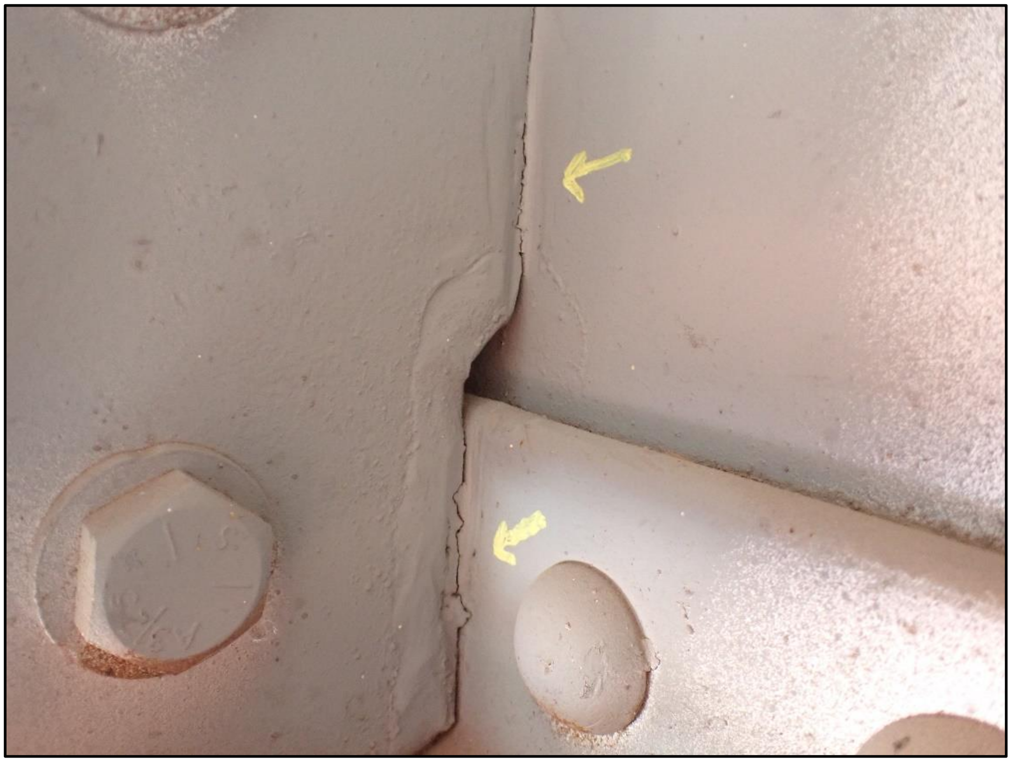


Figure 19: SA Cracked Tack Welds – Girder C North Face – Stiffener 26 to Vertical Leg of Bottom Flange and Web

Figure 20: Partially Ground Out Tack Weld – Girder C South Face – Girder Top Flange to Web At Stiffener 16

Figure 21: Fractured Stiffener 16 Weld – Girder C North Face at Bottom - Left

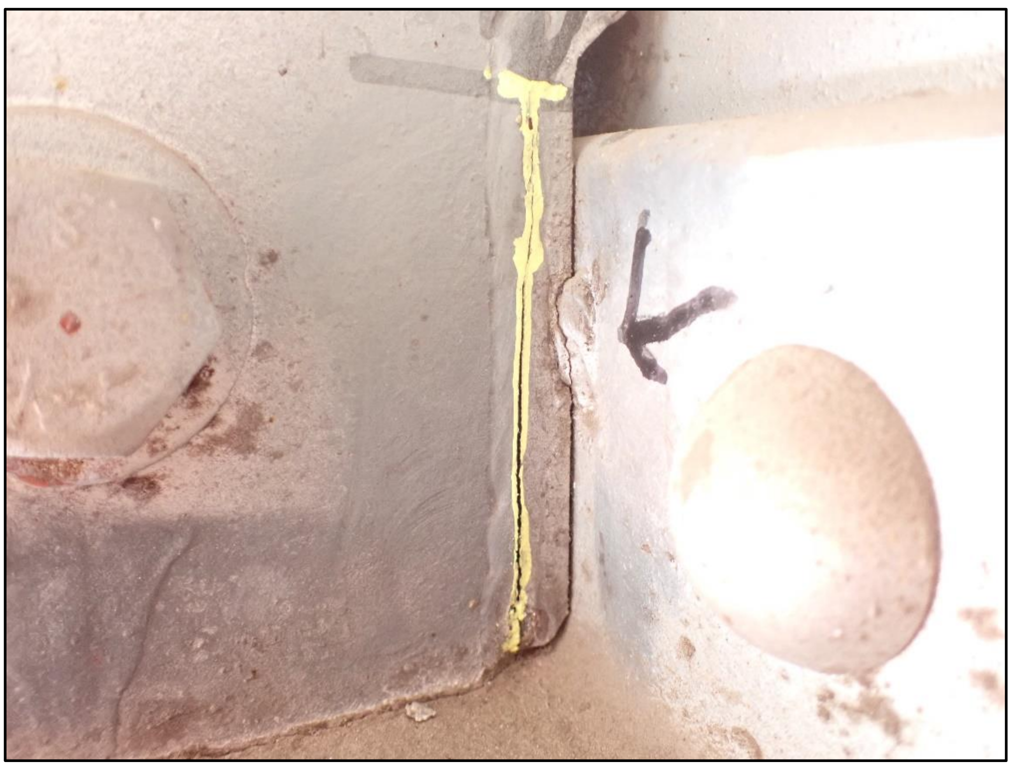


Figure 22: Fractured Stiffener 16 Weld – Girder C North Face at Bottom – Right

Figure 23: Sheared Rivet Head – Girder C North Face at Stiffener 16 Near Bottom

Figure 24: Typical Surface Corrosion of Interior Girder at Pier – Girder E South Face at Pier 4

Figure 25: Surface Corrosion and Minor Pitting in Bottom of Bottom Flange of Girder E at Pier 4

Figure 26: Laminar Corrosion of Girder E End at Abutment 6

Figure 27: Typ. Pack Rust between Bearing Stiffeners of Int. Girders at Pier – Girder E S. Face at P4 – Stiffener 69

Figure 28: Typ. Pack Rust between Bearing Stiffeners of Int. Girders at Abut. – Girder B N. Face at A1 – Stiffener 1

Figure 29: Deflection of Stiffener 62 at North Face of Girder B


Figure 30: Laminar Corrosion in Web and Top of Bottom Flange of North Exterior Girder A

Figure 31: Laminar Corrosion in Top of Web of North Exterior Girder A

Figure 32: Laminar Corrosion in Web of South Exterior Girder F

Figure 33: Corrosion Hole in Base of Bearing Stiffener – Girder F at Abutment 6

Figure 34: Load Sag of North Exterior Girder A

Figure 35: Typical Surface Corrosion of Bearing at Pier – Girder B at Pier 1

Figure 36: Typical Anchor Bolt Nut Backed Out and Missing Lock Nut – Girder B at Pier 2

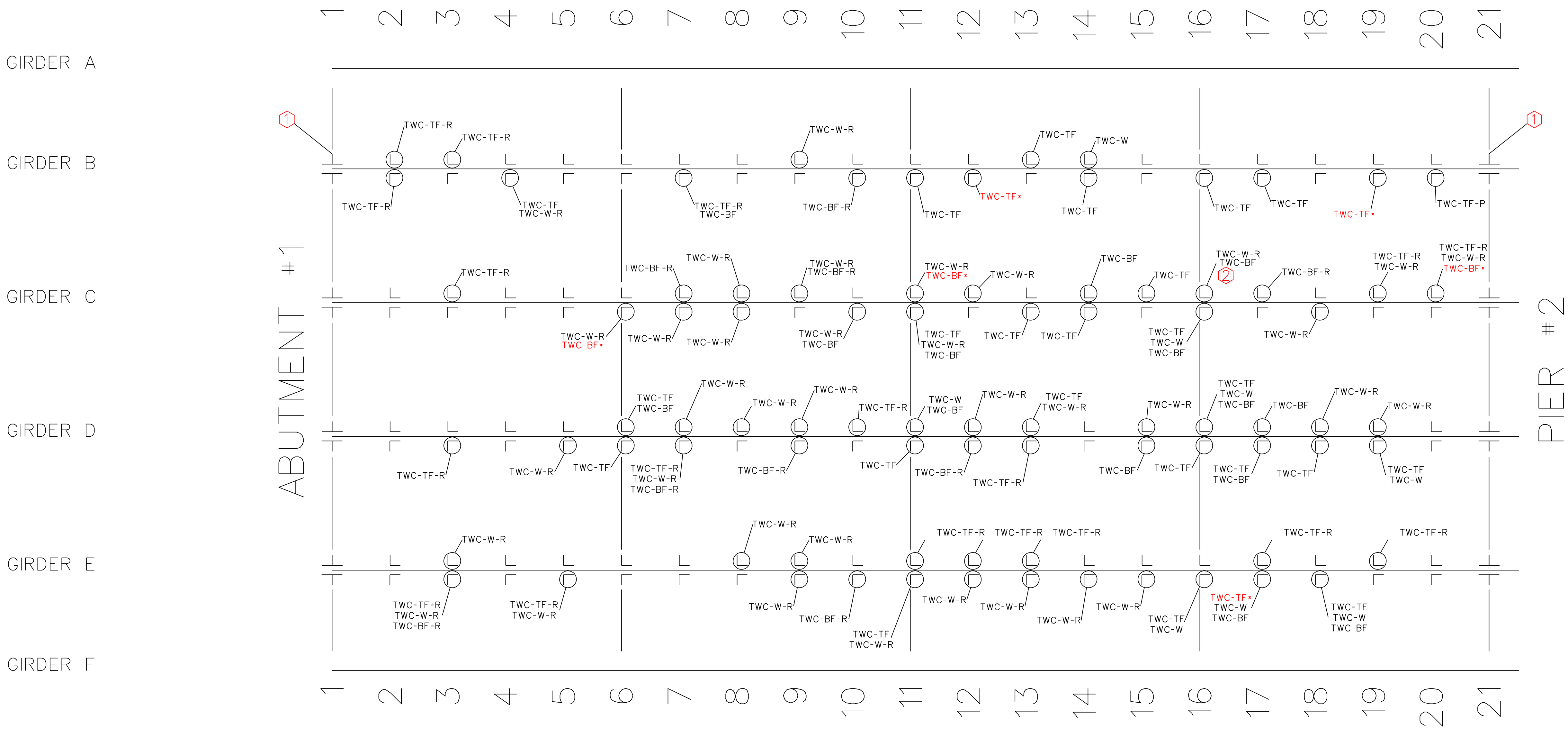
Figure 37: Impending Spall of Bearing Pedestal of Girder A at Abutment 6

Figure 38: Tilted Bearing of Girder F at Abutment 6

6.2 DEFECT DRAWINGS

H-09-B GIRDER DEFECT SKETCH

SPAN 1 (75'-0")



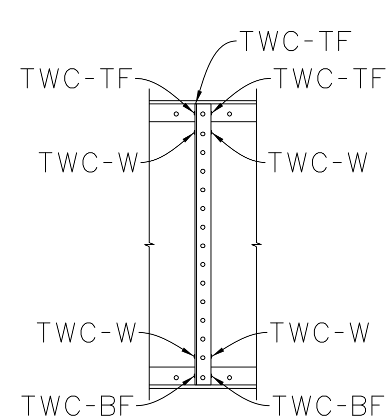
KEYNOTES:

1. PACK RUST BETWEEN FAYING SURFACES OF ANGLE LEGS, UP TO 1/2" THICK, TYP. AT BEARING STIFFENER LOCATIONS.
2. CRACKED STIFFENER WELD AT BOTTOM

CODE LEGEND

TWC-W: TACK WELD CRACK - WEB TO STIFFENER
TWC-BF: TACK WELD CRACK - BOTTOM FLANGE TO STIFFENER
TWC-TF: TACK WELD CRACK - TOP FLANGE TO STIFFENER

ADD-ON CODES:
*: NOT SELF ARRESTED
-R: EFFECTIVE REPAIR PERFORMED
-P: POSSIBLE CRACK
(#): NUMBER OF CRACKS OF SAME TYPE



GIRDER FACE VIEW

NOTES:

1) The structure and associated stiffeners has been inventoried from West to East. Girders are labeled A-F from left to right when looking in the direction of inventory.

2) Drawing is not to scale and serves only for illustration and locating purposes.

3) Defects included in the drawing are limited to superstructure defects pertinent to stiffener attachment locations.

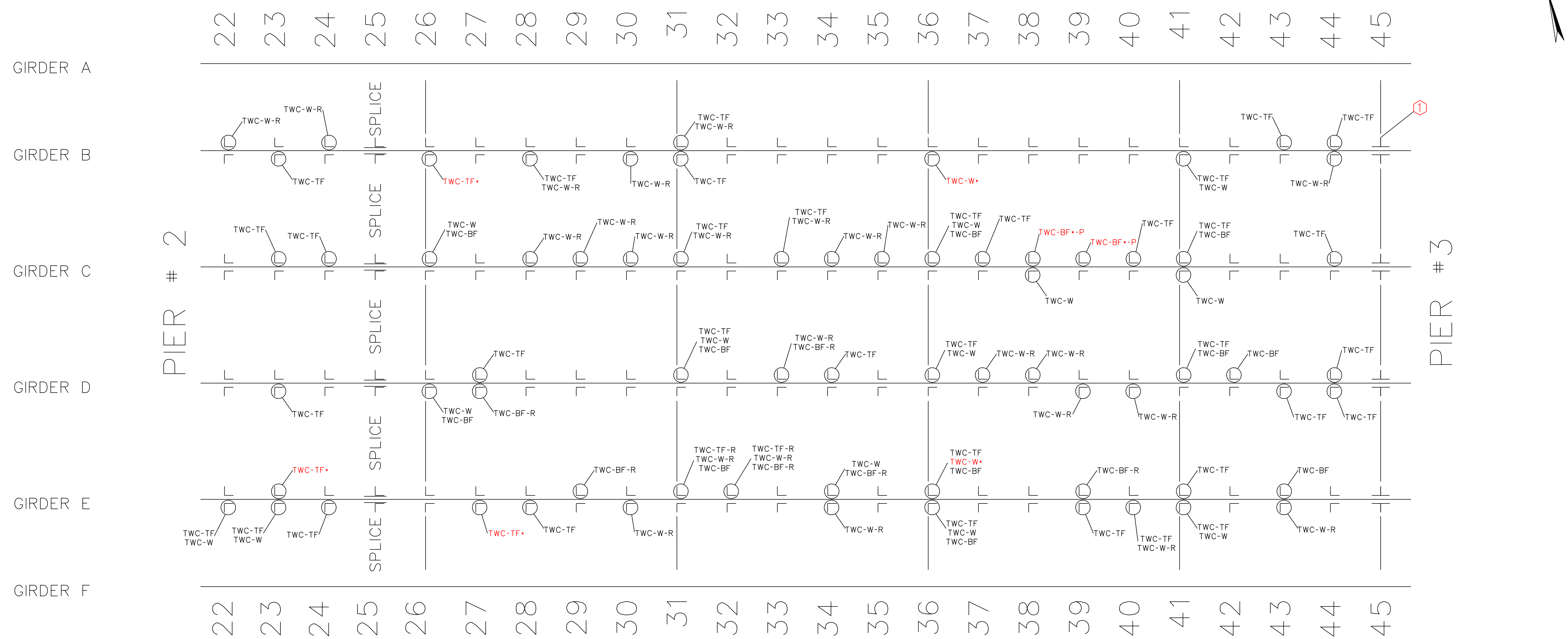
4) Defects that are more broad in nature (i.e. widespread corrosion) have been omitted for clarity and can be found within the narrative report.

5) See accompanying tally sheet for each defect location in tabular form as well as additional notes where unusual circumstances are present.

H-09-B SH 82 OVER CASTLE CREEK	
Drawn By: N. PROFFITT	Date of Inspection: 11/30/2023
Checked By: R. STERN	Sheet: 1 of 5

H-09-B GIRDER DEFECT SKETCH

SPAN 2 (90'-0")



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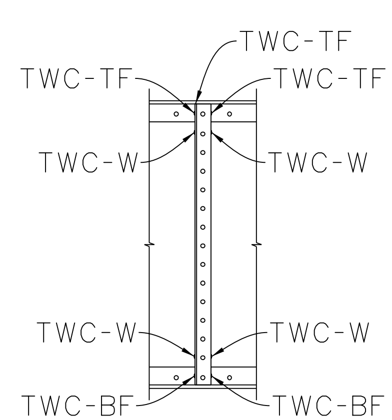

GIRDER FACE VIEW

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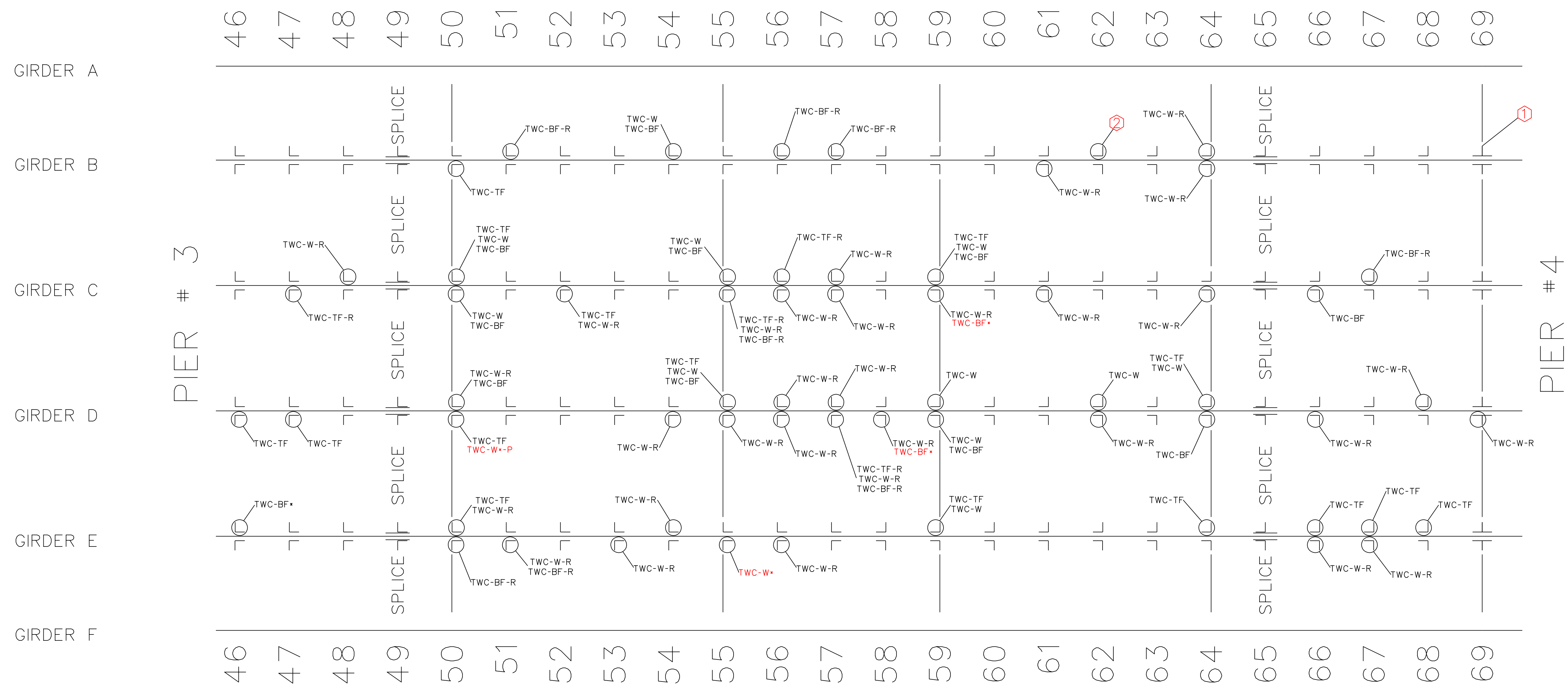
5) See accompanying tally sheet for each defect location in tabular form as well as additional notes where unusual circumstances are present.

H-09-B SH 82 OVER CASTLE CREEK		
Drawn By:	N. PROFFITT	Date of Inspection: 11/30/2023
Checked By:	R. STERN	Sheet: 2 of 5

H-09-B GIRDER DEFECT SKETCH

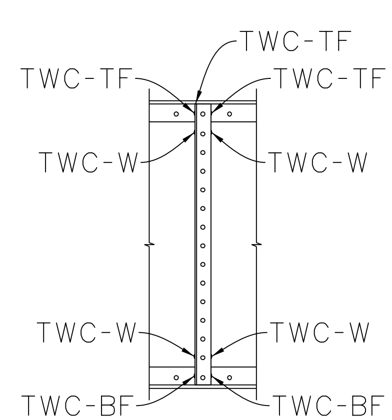
SPAN 3 (90'-0")



KEYNOTES:
1. PACK RUST BETWEEN FAYING SURFACES OF ANGLE LEGS, UP TO 1/2" THICK, TYP. AT BEARING STIFFENER LOCATIONS.
2. STIFFENER DEFLECTED, UP TO 1" OUT OF PLANE OVER 6" HEIGHT, NEAR BOTTOM 1/3 POINT

CODE LEGEND
TWC-W: TACK WELD CRACK - WEB TO STIFFENER
TWC-BF: TACK WELD CRACK - BOTTOM FLANGE TO STIFFENER
TWC-TF: TACK WELD CRACK - TOP FLANGE TO STIFFENER

ADD-ON CODES:
*: NOT SELF ARRESTED
-R: EFFECTIVE REPAIR PERFORMED
-P: POSSIBLE CRACK
(#): NUMBER OF CRACKS OF SAME TYPE



GIRDER FACE VIEW

NOTES:
1) The structure and associated stiffeners has been inventoried from West to East. Girders are labeled A-F from left to right when looking in the direction of inventory.

2) Drawing is not to scale and serves only for illustration and locating purposes.

3) Defects included in the drawing are limited to superstructure defects pertinent to stiffener attachment locations.

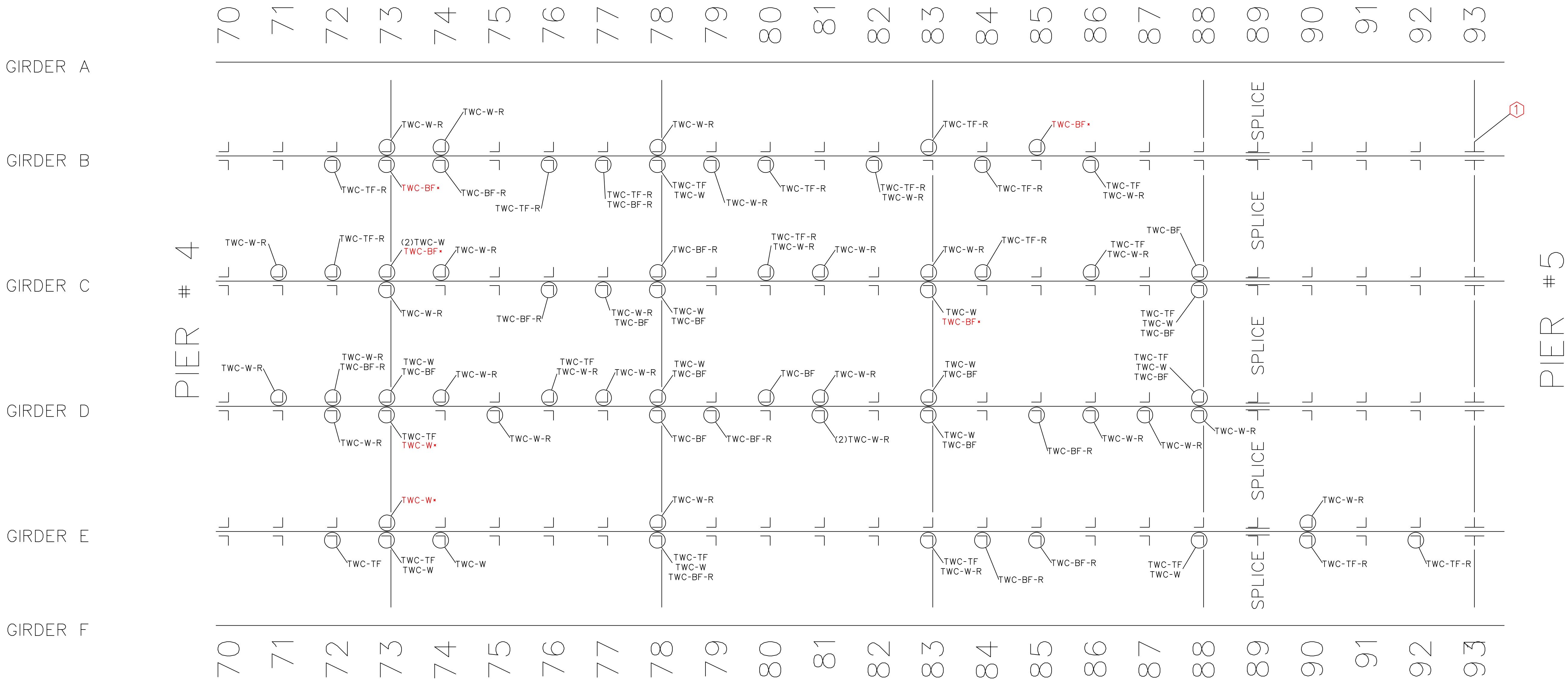
4) Defects that are more broad in nature (i.e. widespread corrosion) have been omitted for clarity and can be found within the narrative report.

5) See accompanying tally sheet for each defect location in tabular form as well as additional notes where unusual circumstances are present.

H-09-B SH 82 OVER CASTLE CREEK		
Drawn By: N. PROFFITT		Date of Inspection: 11/30/2023
Checked By: R. STERN		Sheet: 3 of 5

H-09-B GIRDER DEFECT SKETCH

SPAN 4 (90'-0")



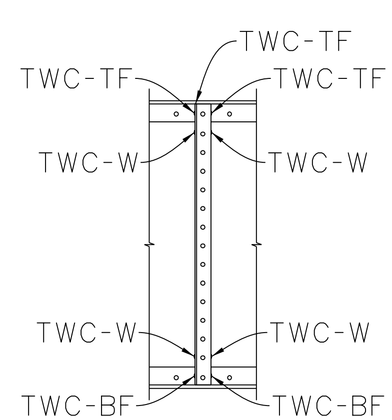
KEYNOTES:

1 PACK RUST BETWEEN FAYING SURFACES OF ANGLE LEGS, UP TO 1/2" THICK, TYP. AT BEARING STIFFENER LOCATIONS.

CODE LEGEND

TWC-W: TACK WELD CRACK - WEB TO STIFFENER
TWC-BF: TACK WELD CRACK - BOTTOM FLANGE TO STIFFENER
TWC-TF: TACK WELD CRACK - TOP FLANGE TO STIFFENER

ADD-ON CODES:
*: NOT SELF ARRESTED
-R: EFFECTIVE REPAIR PERFORMED
-P: POSSIBLE CRACK
(#): NUMBER OF CRACKS OF SAME TYPE



GIRDER FACE VIEW

NOTES:

1) The structure and associated stiffeners has been inventoried from West to East. Girders are labeled A-F from left to right when looking in the direction of inventory.

2) Drawing is not to scale and serves only for illustration and locating purposes.

3) Defects included in the drawing are limited to superstructure defects pertinent to stiffener attachment locations.

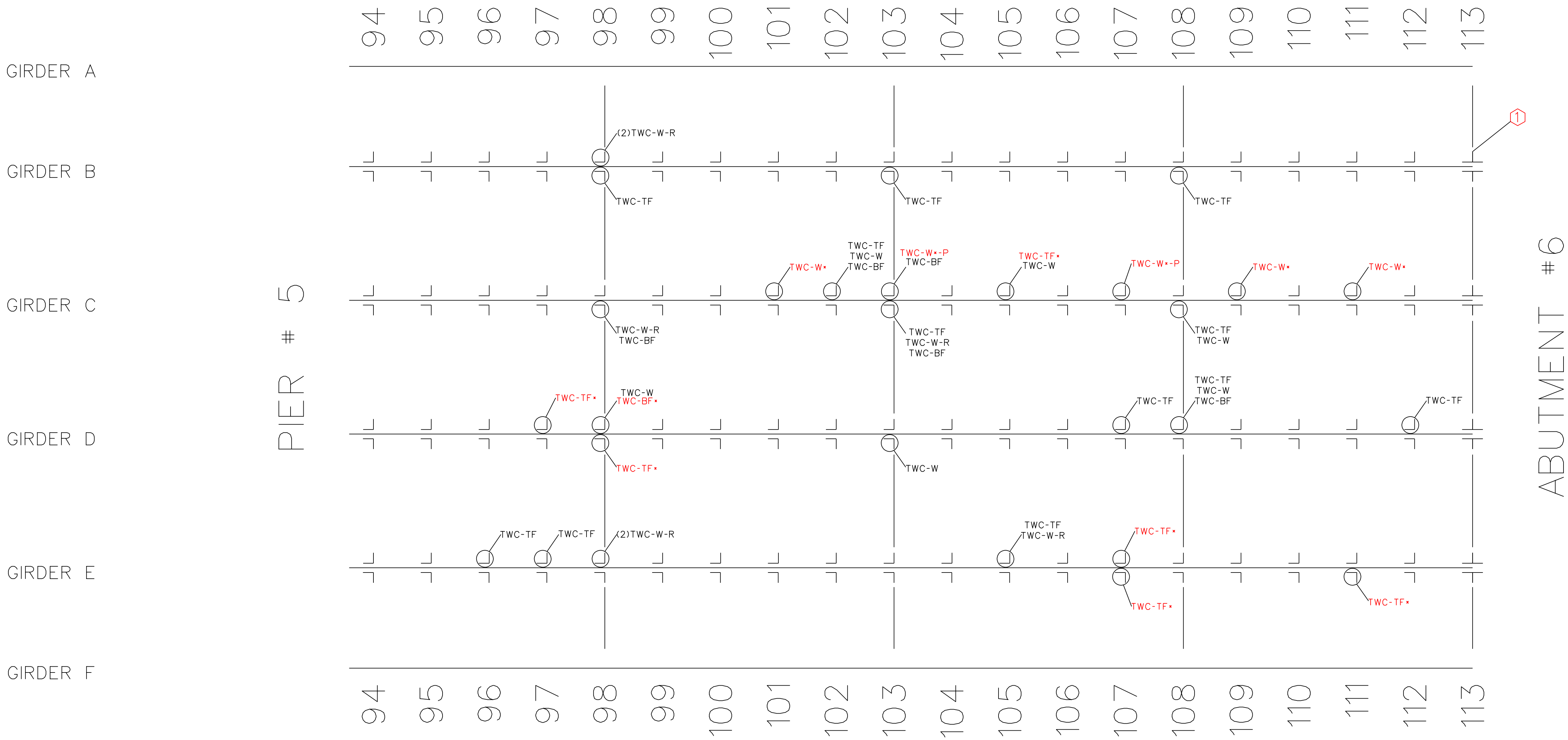
4) Defects that are more broad in nature (i.e. widespread corrosion) have been omitted for clarity and can be found within the narrative report.

5) See accompanying tally sheet for each defect location in tabular form as well as additional notes where unusual circumstances are present.

H-09-B SH 82 OVER CASTLE CREEK			
Drawn By:	N. PROFFITT	Date of Inspection: 11/30/2023	
Checked By:	R. STERN	Sheet:	4 of 5

H-09-B GIRDER DEFECT SKETCH

SPAN 5 (75'-0")



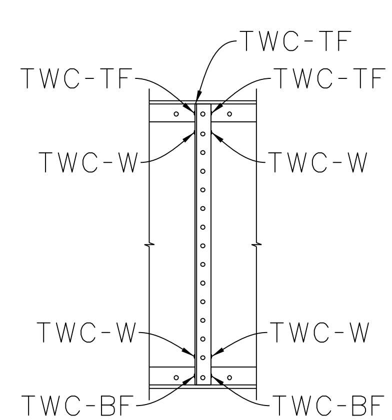
KEYNOTES:

1 PACK RUST BETWEEN FAYING SURFACES OF ANGLE LEGS, UP TO 1/2" THICK, TYP. AT BEARING STIFFENER LOCATIONS.

CODE LEGEND

TWC-W: TACK WELD CRACK - WEB TO STIFFENER
TWC-BF: TACK WELD CRACK - BOTTOM FLANGE TO STIFFENER
TWC-TF: TACK WELD CRACK - TOP FLANGE TO STIFFENER

ADD-ON CODES:
×: NOT SELF ARRESTED
-R: EFFECTIVE REPAIR PERFORMED
-P: POSSIBLE CRACK
(#): NUMBER OF CRACKS OF SAME TYPE



GIRDER FACE VIEW

NOTES:

1) The structure and associated stiffeners has been inventoried from West to East. Girders are labeled A-F from left to right when looking in the direction of inventory.

2) Drawing is not to scale and serves only for illustration and locating purposes.

3) Defects included in the drawing are limited to superstructure defects pertinent to stiffener attachment locations.

4) Defects that are more broad in nature (i.e. widespread corrosion) have been omitted for clarity and can be found within the narrative report.

5) See accompanying tally sheet for each defect location in tabular form as well as additional notes where unusual circumstances are present.

H-09-B SH 82 OVER CASTLE CREEK	
Drawn By: N. PROFFITT	Date of Inspection: 11/30/2023
Checked By: R. STERN	Sheet: 5 of 5

6.3 TALLY SHEET

Code Legend	
TWC-W	Tack Weld Crack - Web to Stiffener
TWC-BF	Tack Weld Crack - Bottom Flange to Stiffener
TWC-TF	Tack Weld Crack - Top Flange to Stiffener
-R	Add on -R in order to indicate a repaired TWC
*	Add on asterisk in order to indicate a TWC that has not self-arrested
See Notes	See notes for atypical situations and locations where multiple defects are present

		Defect															
	Stiffener #	Girder B				Girder C				Girder D				Girder E			
		North Face	Notes	South Face	Notes	North Face	Notes	South Face	Notes	North Face	Notes	South Face	Notes	North Face	Notes	South Face	Notes
SPAN 1	1 (Abut 1)	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener
	2	TWC-TF-R		TWC-TF-R													
	3	TWC-TF-R				TWC-TF-R						TWC-TF-R		TWC-W-R		See Notes	TWC-TF-R, TWC-W-R, TWC-BF-R
	4			See Notes	TWC-TF, TWC-W-R												
	5											TWC-W-R				See Notes	TWC-TF-R, TWC-W-R
	6							See Notes	TWC-W-R, TWC-BF*	See Notes	TWC-TF, TWC-BF	TWC-TF					
	7			See Notes	TWC-TF-R, TWC-BF	TWC-BF-R		TWC-W-R		TWC-W-R		See Notes	TWC-TF-R. TWC-W-R. TWC-BF-R				
	8					TWC-W-R		TWC-W-R		TWC-W-R				TWC-W-R			
	9	TWC-W-R				See Notes	TWC-W-R, TWC-BF-R			TWC-W-R		TWC-BF-R		TWC-W-R		TWC-W-R	
	10			TWC-BF-R				See Notes	TWC-W-R, TWC-BF	TWC-TF-R						TWC-BF-R	
	11			TWC-TF		See Notes	TWC-W-R, TWC-BF*	See Notes	TWC-TF, TWC-W-R, TWC-BF	See Notes	TWC-W, TWC-BF	TWC-TF		See Notes	TWC-TF, TWC-W, TWC-BF	See Notes	TWC-TF, TWC-W-R
	12			TWC-TF*		TWC-W-R				TWC-W-R		TWC-BF-R		TWC-TF		TWC-W-R	
	13	TWC-TF						TWC-TF		See Notes	TWC-TF, TWC-W-R	TWC-TF-R		TWC-W-R		TWC-W-R	
	14	TWC-W		TWC-TF		TWC-BF		TWC-TF								TWC-W-R	
	15					TWC-TF				TWC-W-R		TWC-BF				TWC-W-R	
	16			TWC-TF		See Notes	TWC-W-R, TWC-BF, Cracked stiffener weld at bottom	See Notes	TWC-TF, TWC-W, TWC-BF	See Notes	TWC-TF, TWC-W, TWC-BF	TWC-TF				See Notes	TWC-TF, TWC-W
	17			TWC-TF		TWC-BF-R				TWC-BF		See Notes	TWC-TF, TWC-BF	See Notes	TWC-TF, TWC-W, TWC-BF	See Notes	TWC-TF*, TWC-W, TWC-BF
	18							TWC-W-R		TWC-W-R		TWC-TF				See Notes	TWC-TF, TWC-W, TWC-BF
	19			TWC-TF*		See Notes	TWC-TF-R, TWC-W-R			TWC-W-R		See Notes	TWC-TF, TWC-W	TWC-TF			
	20			See Notes	Possible TWC-TF	See Notes	TWC-TF-R, TWC-W-R, TWC-BF*										
	21 (Pier 2)	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener
SPAN 2	22	TWC-W-R														See Notes	TWC-TF, TWC-W
	23			TWC-TF		TWC-TF						TWC-TF		TWC-TF*		See Notes	TWC-TF, TWC-W
	24	TWC-W-R				TWC-TF										TWC-TF	
	25																
	26			TWC-TF*		See Notes	TWC-W, TWC-BF					See Notes	TWC-W, TWC-BF				
	27									TWC-TF		TWC-BF-R				TWC-TF*	
	28			See Notes	TWC-TF, TWC-W-R	TWC-W-R										TWC-TF	
	29					TWC-W-R								TWC-BF-R			
	30			TWC-W-R		TWC-W-R										TWC-W-R	
	31	See Notes	TWC-TF, TWC-W-R	TWC-TF		See Notes	TWC-TF, TWC-W-R			See Notes	TWC-TF, TWC-W, TWC-BF			See Notes	TWC-TF-R, TWC-W-R, TWC-BF		
	32													See Notes	TWC-TF-R, TWC-W-R, TWC-BF-R		
	33					See Notes	TWC-TF, TWC-W-R			See Notes	TWC-W-R, TWC-BF-R						
	34					TWC-W-R				TWC-TF				See Notes	TWC-W, TWC-BF-R	TWC-W-R	
	35					TWC-W-R											
	36			TWC-W*		See Notes	TWC-TF, TWC-W, TWC-BF			See Notes	TWC-TF, TWC-W			See Notes	TWC-TF, TWC-W*, TWC-BF	See Notes	TWC-TF, TWC-W, TWC-BF
	37					TWC-TF				TWC-W-R							
	38					See Notes	Possible TWC-BF*	TWC-W-R		TWC-W-R							
	39					See Notes	Possible TWC-BF*					TWC-W-R		TWC-BF-R		TWC-TF	
	40					TWC-TF						TWC-W-R				See Notes	TWC-TF, TWC-W-R
	41			See Notes	TWC-TF, TWC-W	See Notes	TWC-TF, TWC-BF	TWC-W-R		See Notes	TWC-TF, TWC-BF			TWC-TF		See Notes	TWC-TF, TWC-W
	42									TWC-BF							
	43	TWC-TF										TWC-TF		TWC-BF		TWC-W-R	
	44	TWC-TF		TWC-W-R		TWC-TF				TWC-TF		TWC-TF					
	45 (Pier 3)	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener

		Defect															
	Stiffener #	Girder B				Girder C				Girder D				Girder E			
		North Face	Notes	South Face	Notes	North Face	Notes	South Face	Notes	North Face	Notes	South Face	Notes	North Face	Notes	South Face	Notes
SPAN 3	46											TWC-TF		TWC-BF*			
	47							TWC-TF-R				TWC-TF					
	48					TWC-W-R											
	49																
	50			TWC-TF		See Notes	TWC-TF, TWC-W, TWC-BF	See Notes	TWC-W, TWC-BF	See Notes	TWC-W-R, TWC-BF	See Notes	TWC-TF, Possible TWC-W*	See Notes	TWC-TF, TWC-W-R	TWC-BF-R	
	51	TWC-BF-R														See Notes	TWC-W-R, TWC-BF-R
	52							See Notes	TWC-TF, TWC-W-R								
	53															TWC-W-R	
	54	See Notes	TWC-W, TWC-BF									TWC-W-R		TWC-W-R			
	55					See Notes	TWC-W, TWC-BF	See Notes	TWC-TF-R, TWC-W-R, TWC-BF-R	See Notes	TWC-TF, TWC-W, TWC-BF	TWC-W-R				TWC-W*	
	56	TWC-BF-R				TWC-TF-R		TWC-W-R		TWC-W-R		TWC-W-R				See Notes	(2) TWC-W-R
	57	TWC-BF-R				TWC-TF-R		TWC-W-R		TWC-W-R		See Notes	TWC-TF-R, TWC-W-R, TWC-BF-R				
	58											See Notes	TWC-W-R, TWC-BF*				
	59					See Notes	TWC-TF, TWC-W, TWC-BF	See Notes	TWC-W-R, TWC-BF*	TWC-W		See Notes	TWC-W, TWC-BF	See Notes	TWC-TF, TWC-W		
	60																
	61			TWC-W-R				TWC-W-R									
	62	See Notes	Deflected up to 1" out of plane over 6" height							TWC-W		TWC-W-R					
	63																
	64	TWC-W-R		TWC-W-R				TWC-W-R		See Notes	TWC-TF, TWC-W	TWC-BF		TWC-TF			
	65																
	66							TWC-BF				TWC-W-R		TWC-TF		TWC-W-R	
	67					TWC-BF-R								TWC-TF		TWC-W-R	
	68									TWC-W-R				TWC-TF			
	69 (Pier 4)	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	TWC-W-R, Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener
SPAN 4	70																
	71					TWC-W-R				TWC-W-R							
	72			TWC-TF-R		TWC-TF-R				See Notes	TWC-W-R, TWC-BF-R	TWC-W-R				TWC-TF	
	73	TWC-W-R		TWC-BF*		See Notes	(2) TWC-W, TWC-BF*	TWC-W-R		See Notes	TWC-W, TWC-BF	See Notes	TWC-TF, TWC-W*	TWC-W*		See Notes	TWC-TF, TWC-W
	74	TWC-W-R		TWC-BF-R		TWC-W-R				TWC-W-R						TWC-W	
	75											TWC-W-R					
	76			TWC-TF-R				TWC-BF-R		See Notes	TWC-TF, TWC-W-R						
	77			See Notes	TWC-TF-R, TWC-BF-R			See Notes	TWC-W-R, TWC-BF	TWC-W-R							
	78	TWC-W-R		See Notes	TWC-TF, TWC-W	TWC-BF-R		See Notes	TWC-W, TWC-BF	See Notes	TWC-W, TWC-BF	TWC-BF		TWC-W-R		See Notes	TWC-TF, TWC-W, TWC-BF-R
	79			TWC-W-R								TWC-BF-R					
	80			TWC-TF-R		See Notes	TWC-TF-R, TWC-W-R			TWC-BF							
	81					TWC-W-R				TWC-W-R		See Notes	(2) TWC-W-R				
	82			See Notes	TWC-TF-R, TWC-W-R												
	83	TWC-W-R				TWC-W-R		See Notes	TWC-W, TWC-BF*	See Notes	TWC-W, TWC-BF	See Notes	TWC-W, TWC-BF			See Notes	TWC-TF, TWC-W-R
	84			TWC-TF-R		TWC-TF-R										TWC-BF-R	
	85	TWC-BF*										TWC-BF-R				TWC-BF-R	
	86			See Notes	TWC-TF, TWC-W-R	See Notes	TWC-TF, TWC-W-R					TWC-W-R					
	87											TWC-W-R					
	88					TWC-BF		See Notes	TWC-TF, TWC-W, TWC-BF	See Notes	TWC-TF, TWC-W, TWC-BF	TWC-W-R				See Notes	TWC-TF, TWC-W
	89																
	90													TWC-W-R		TWC-TF-R	
	91																
	92															TWC-TF-R	
	93 (Pier 5)	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener
SPAN 5	94																
	95																
	96													TWC-TF			
	97									TWC-TF*				TWC-TF			
	98	See Notes	(2) TWC-W-R	TWC-TF				See Notes	TWC-W-R, TWC-BF	See Notes	TWC-W, TWC-BF*	TWC-TF*		See Notes	(2) TWC-W-R		
	99																
	100																
	101					TWC-W*											
	102					See Notes	TWC-TF, TWC-W, TWC-BF										
	103			TWC-TF		See Notes	Possible TWC-W*, TWC-BF	See Notes	TWC-TF, TWC-W-R, TWC-BF			TWC-W					
	104																
	105					See Notes	TWC-TF*, TWC-W							See Notes	TWC-TF, TWC-W-R		
	106																
	107					See Notes	Possible TWC-W*			TWC-TF				TWC-TF*		TWC-TF*	
	108			TWC-TF				See Notes	TWC-TF, TWC-W	See Notes	TWC-TF, TWC-W, TWC-BF						
	109					TWC-W*											
	110																
	111					TWC-W*										TWC-TF*	
	112									TWC-TF							
	113 (Abut 6)	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener	See Notes	Pack rust between faying surfaces of bearing stiffener

Appendix C
Rehabilitation Sufficiency Rating Calculation

Bridge: H-09-B

Description: Rehabilitation of the existing two-lane bridge

Bridge Sufficiency Rating

Ref: 'Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges', Report No. FHWA-PD-96-001

SUFFICIENCY RATING = $S_1 + S_2 + S_3 - S_4$

Each Factor uses the following data items for the calculation, denoted by the Item Number used in the inspection report:

1. STRUCTURAL ADEQUACY AND SAFETY
   - 59 Superstructure
   - 60 Substructure
   - 62 Culvert
   - 66 Inventory Rating
   - S_1 = 55% Max

2. SERVICEABILITY AND FUNCTIONAL OBSOLESCENCE
   - 28 Lanes on Structure
   - 29 ADT
   - 32 Appr. Roadway Width
   - 43 Structure Type, Main
   - 51 Bridge Roadway Width
   - 53 VC over Deck
   - 58 Deck Condition
   - 67 Structural Evaluation
   - 68 Deck Geometry
   - 69 Underclearances
   - 71 Waterway Adequacy
   - 72 Appr. Rdwy. Aliignment
   - 100 STRAHNET Designation
   - S_2 = 30% Max

3. ESSENTIALITY FOR PUBLIC USE
   - 19 Detour Length 29 ADT
   - 29 ADT
   - 100 STRAHNET Designation
   - S_3 = 15% Max

4. SPECIAL REDUCTIONS
   - 19 Detour Length
   - 36 Traffic Safety Features
   - 43 Structure Type, Main
   - S_4 = 13% Max

Input from Structural Inventory and Appraisal (SI&A) Sheet

Item	Description	Input		SI	Units	Note
19	Detour Length	0.6	mi	0.97	km	
28A	Lanes on Structure	2				
29	ADT	25000				
32	Approach Roadway Width	44.00	ft	13.41	m	
36A	Bridge Railings	**1**				Assumed value after rehab
36B	Transitions	**1**				Assumed value after rehab
36C	Approach Guardrail	**1**				Assumed value after rehab
36D	Approach Guardrail Ends	**1**				Assumed value after rehab
42A	Type of Service On Bridge	5				
42B	Type of Service Under Bridge	6				
43A	Structure Type Main: Matl	4				
43B	Structure Type Main: Type	3				
51	Bridge Width Curb to Curb	27.00	ft	8.23	m	
53	Min Clr Over Bridge	99.99	ft	30.48	m	
58	Deck Condition	6				Assumed value after rehab
59	Superstructure Condition	**6**				Assumed value after rehab
60	Substructure Condition	6				Assumed value after rehab
66	Inventory Rating	24.60	T	22.3	t	
67	Structural Evaluation	**6**				Assumed value after rehab
68	Deck Geometry	3				
69	Underclearances Vert/Hor	3				
71	Waterway Adequacy	9				
72	Approach Road Alignment	8				
100	Defense (STRAHNET)	0				

Bridge: H-09-B

Description: Rehabilitation of the existing two-lane bridge

1. Structural Adequacy and Safety (55% maximum).					
a. Only the lowest rating code of Item 59 and 60 applies					
Item 59	(Superstructure Condition) =	6			
Item 60	(Substructure Condition) =	6			
Item 62	(Culvert Condition) =	99	if "N", use 99		
Controlling Condition Rating =		6			
				A =	0.0%
b. Reduction for Load Capacity:					
Item 66	(Inventory Rating) =	22.3			
				B =	10.4%
S_1 = 55 - (A + B)				**S1 =**	**44.6%**

2. Serviceability and Functional Obsolescence (30% maximum)						
a. Rating Reductions (13% maximum).						
	Item 58	(Deck Condition) =	6		A =	0%
	Item 67	(Structural Evaluation) =	6		B =	0%
	Item 68	(Deck Geometry) =	3		C =	4%
	Item 69	(Underclearances) =	3		D =	4%
	Item 71	(Waterway Adequacy) =	9		E =	0%
	Item 72	(Appr. Rd. Alignment) =	8		F =	0%
		J = A + B + C + D + E + F			J =	8%
b. Width of Roadway Insufficiency (15% maximum)						
	X = ADT / #Lanes =		12500			
	Y = Width / #Lanes =		4.11			
(1)	If Item 51 (Bridge Width) + 0.6 m < Item 31 (Approach Rdwy Width) then G = 5%					
	8.2296+0.6=	8.83	<	13.41	G =	5%
(2)	For 1-lane bridges only					
	Item 28A (Lanes on Structure) =		2		H =	NA
(3)	For 2 or more lane bridges; If these limits apply, do not continue to (4);					
	If Lanes = 2 and Y ≥ 4.9; H = 0%					
	If Lanes = 3 and Y ≥ 4.6; H = 0%					
	If Lanes = 4 and Y ≥ 4.3; H = 0%					
	If Lanes ≥ 5 and Y ≥ 3.7; H = 0%					
	Item 28A (Lanes on Structure) =		2		H =	NA
(4)	For all except 1-lane bridges					
	X =	12500	Y = 4.11			
	Case / Condition					
1.	Y < 2.7 and X > 50	H = 15% =		15.0%	H=	NA
2.	Y < 2.7 and X ≤ 50	H = 7.5% =		7.5%	H=	NA
3.	Y ≥ 2.7 and X ≤ 50	H = 0% =		0.0%	H=	NA
4.	50 < X ≤ 125 and Y < 3.0	H = 15% =		15.0%	H=	NA
5.	50 < X ≤ 125 and 3.0 ≤ Y < 4.0	H = 15(4 - Y)% =		-1.7%	H=	NA
6.	50 < X ≤ 125 and Y ≥ 4.0	H = 0% =		0.0%	H=	NA
7.	125 < X ≤ 375 and Y < 3.4	H = 15% =		15.0%	H=	NA
8.	125 < X ≤ 375 and 3.4 ≤ Y < 4.3	H = 15(4.3 - Y)% =		2.8%	H=	NA
9.	125 < X ≤ 375 and Y ≥ 4.3	H = 0% =		0.0%	H=	NA
10.	375 < X ≤ 1350 and Y < 3.7	H = 15% =		15.0%	H=	NA
11.	375 < X ≤ 1350 and 3.7 ≤ Y < 4.9	H = 15[(4.9 - Y) / 1.2]% =		9.8%	H=	NA
12.	375 < X ≤ 1350 and Y ≥ 4.9	H = 0% =		0.0%	H=	NA
13.	X > 1350 and Y < 4.6	H = 15% =		15.0%	H=	15.0%
14.	X > 1350 and 4.6 ≤ Y < 4.9	H = 15[(4.9 - Y) / 1.2]% =		9.8%	H=	NA
15.	X > 1350 and Y ≥ 4.9	H = 0% =		0.0%	H=	NA

Bridge: H-09-B

Description: Rehabilitation of the existing two-lane bridge

	From (2) through (4), Use	H=	15.0%
	G + H (15% maximum)	G+H =	15.0%
c. Vertical Clearance Insufficiency (2% maximum)			
If Item 100 (STRAHNET Highway) > 0 and			
Item 53 (VC over Deck) ≥ 4.87; I = 0%			
Item 53 (VC over Deck) < 4.87; I = 2%			
If Item 100 (STRAHNET Highway) = 0 and			
Item 53 (VC over Deck) ≥ 4.26; I = 0%			
Item 53 (VC over Deck) < 4.26; I = 2%			
Item 100 (STRAHNET Highway) =	0		
Item 53 (Lanes on Structure) =	30.48 m		
		I =	0.0%
S_2 = 30 - [J + (G + H) + I]		**S2 =**	**7.0%**

3. Essentiality for Public Use (15% maximum)			
a. Determine K = (S1 + S2) / 85		K =	0.606837482
b. Calculate A = 15 [(ADT (#29) x Detour Length (#19)) / (320,000 x K)]			
Item 29 (ADT) =	25000		
Item 19 (Detour Length) =	0.97		
		A =	1.9%
c. STRAHNET Highway Designation:			
If Item 100 (STRAHNET Highway) > 0, B = 2%			
If Item 100 (STRAHNET Highway) = 0, B = 0%			
Item 100 (STRAHNET Highway) =	0		
		B =	0.0%
S_3 = 15 - (A + B)		**S3 =**	**13.1%**

$S_1 + S_2 + S_3$ =	**64.7%**

4. Special Reductions (Use only with $S_1 + S_2 + S_3 \geq 50$) (13% maximum);			
a. Detour Length Reduction (maximum 5%) A = (#19)4 x (7.9x10-9)			
Item 19 (Detour Length) =	0.97 km		
		A =	0.0%
b. If the 2nd and 3rd digits of Item 43 (Structure Type, Main) are equal to 10, 12, 13 14, 15, 16 or 17, then B = 5%			
Item 43B Structure Type	03		
		B =	0.0%
c. If 2 digits of Item 36 (Traffic Safety Features) = 0, C = 1%			
If 3 digits of Item 36 (Traffic Safety Features) = 0, C = 2%			
If 4 digits of Item 36 (Traffic Safety Features) = 0, C = 3%			
Item 36A Bridge Railings	1		
Item 36B Transitions	1		
Item 36C Approach Guardrail	1		
Item 36D Approach Guardrail Ends	1		
Total 0's	0		
		C =	0.0%
S_4 = A + B + C		**S4=**	**0.0%**

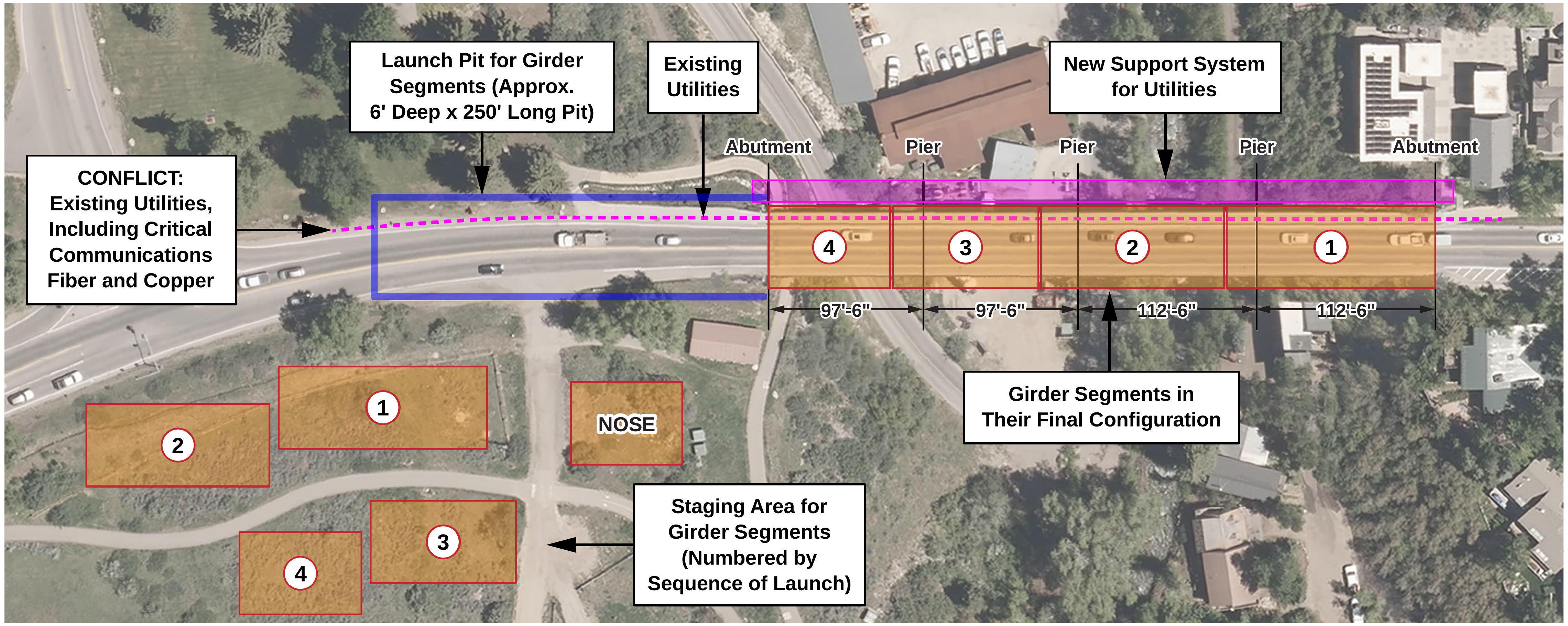
Sufficiency Rating =	**64.7%**

Appendix D
Rehabilitation Cost Estimate

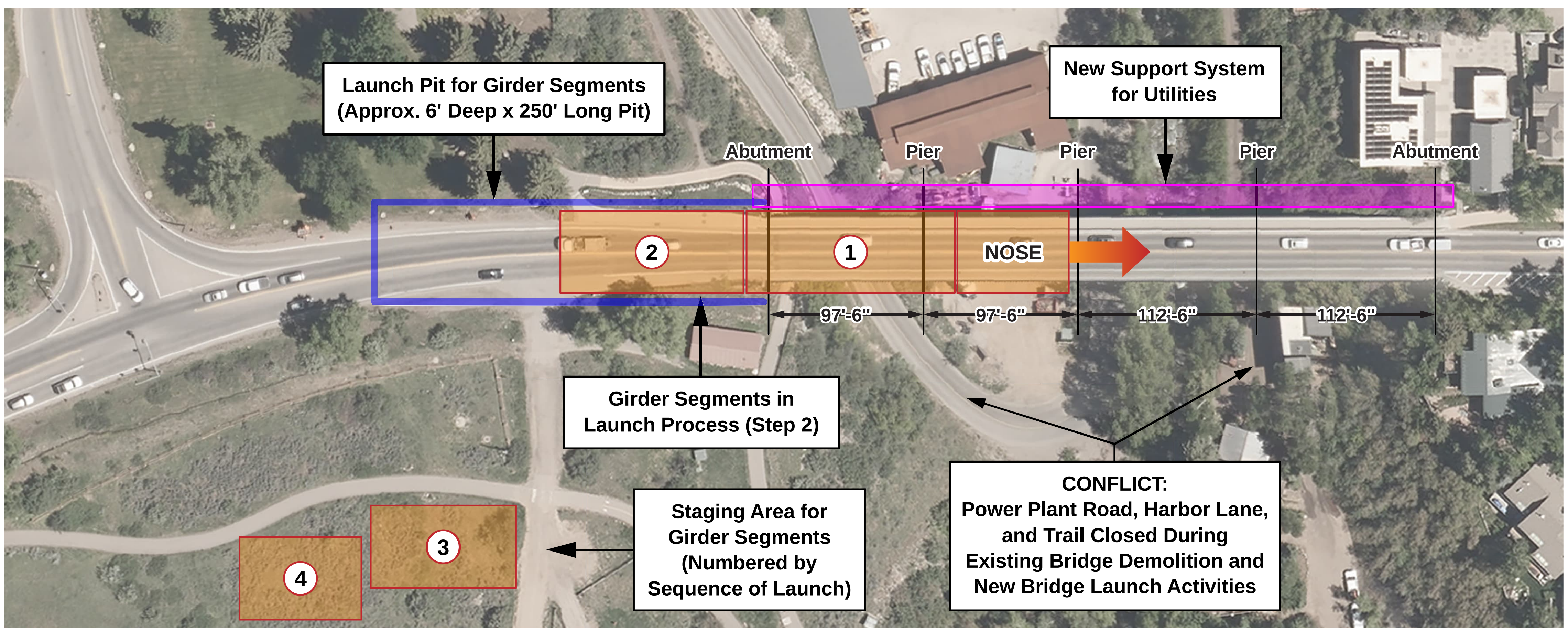
Cost Estimate Summary						
Bid Item	**Item Description**	**Units**	**Qty**	**Unit Cost**	**Total Cost**	**Note**
202-00055	Removal of Fiber Optic Cable	LF	551	$5.00	$2,755.00	Unit cost estimated from average bid price on 2022 cost book
202-00040	Removal of Electrical Conduit	LF	551	$14.00	$7,714.00	Unit cost estimated from Engineering Estimate price on 2021 cost book
202-XXXXX	Removal of Other Utilities	LF	551	$10.00	$5,510.00	Unit cost average of FO and electric
202-00220	Removal of Asphalt Mat	SY	1405	$12.00	$16,865.00	Asphalt overlay will be replaced with PPC overlay.
202-00505	Removal of Portions of Present Structure	SF	5507	$175.00	$963,746.88	Removal limit: 6.5 feet from both edges to remove sidewalk and exterior girder
202-05150	Sandblasting	SF	18432	$1.95	$35,942.40	All structural steel. Unit Cost estimated from 2020 cost book.
203-02330	Laborer	HR	61	$60.00	$3,630.00	For Removal of NSA Tack Welds
210-00530	Rebuild Portions of Present Structure	SF	5507	$150.00	$826,068.75	Rebuild limit: 6.5 feet from both edges to rebuild sidewalk and portion of deck. Only includes concrete work and pedestrian bridge rail reset.
XXX-XXXXX	Replace Fiber Optic Cable	LF	551	$10.00	$5,510.00	Assumed
XXX-XXXXX	Replace Electrical Conduit	LF	551	$28.00	$15,428.00	Assumed
XXX-XXXXX	Replace Other Utilities	LF	551	$20.00	$11,020.00	Assumed
250-00100	Environmental Health and Safety	LS	1	$1,000,000.00	$1,000,000.00	Assumed cost for lead paint removal and preparation for repainting
408-01100	Joint Sealant	LF	240	$50.00	$12,000.00	Unit cost based on 2023 cost book
509-00000	Structural Steel	LB	91600	$5.50	$503,800.00	Unit cost based on 2022 cost book
512-00101	Bearing Device	EA	12	$4,000.00	$48,000.00	Unit cost based on 2022 cost book
509-90003	Paint Structural Steel	LS	1	$350,000.00	$350,000.00	Unit cost based on 2021 cost book
519-03035	Place Thin Bonded Overlay (Polyester Concrete)	SY	1405	$135.00	$189,731.25	Based on previous BPM project work.
519-03055	Furnish Thin Bonded Overlay (Polyester Concrete)	CF	791	$205.00	$162,062.11	Based on previous BPM project work.
601-03000	Concrete Class D	CY	1	$3,500.00	$3,500.00	For bearing pedestal, Based on 2023 cost book data
601-06100	Concrete (Patching)	CY	25	$3,500.00	$86,345.00	Unit cost based on 2022 cost book
606-11035	Bridge Rail Type 10 MASH	LF	848	$340.00	$288,320.00	Assume rail installed on both sides of bridge.
Total					**$4,537,948.38**	
Contingency (30%)					**$1,361,384.52**	
Grand Total					**$5,900,000.00**	

CY = Cubic yard
EA = Each
FO = Fiber optic
HR = Hourly
LF = Linear foot
LS = Lump sum
NSA = Not self arrested
SF = square feet
SY = square yard

Appendix E
ABC Method: Incremental Bridge Launch



Staging Area and Final Configuration of Girder Segments After Incremental Launch



Incremental Launch – Typical Launch Sequence (Step 2 Shown)

Figure 43a. ABC Incremental Bridge Launch Layout
SH82 Over Castle Creek Bridge Feasibility Study

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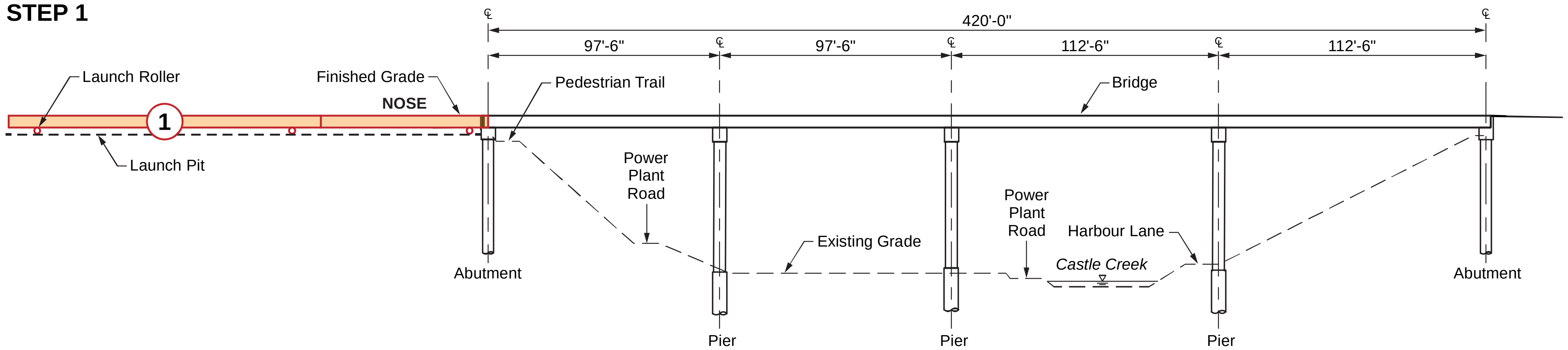
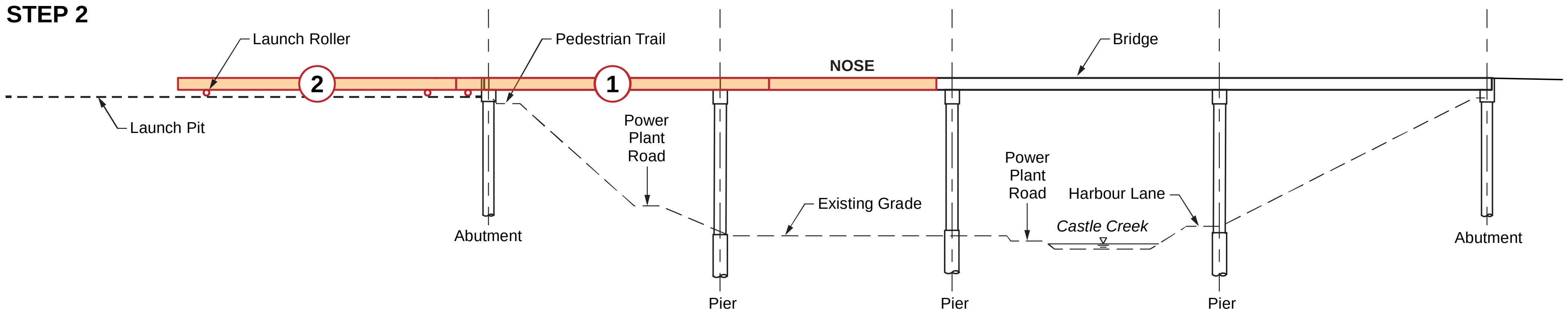
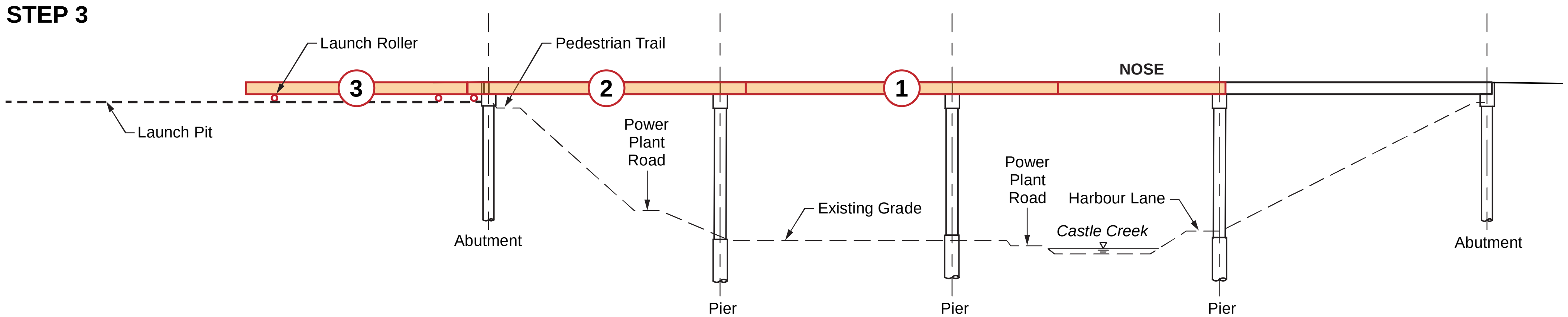
STEP 1



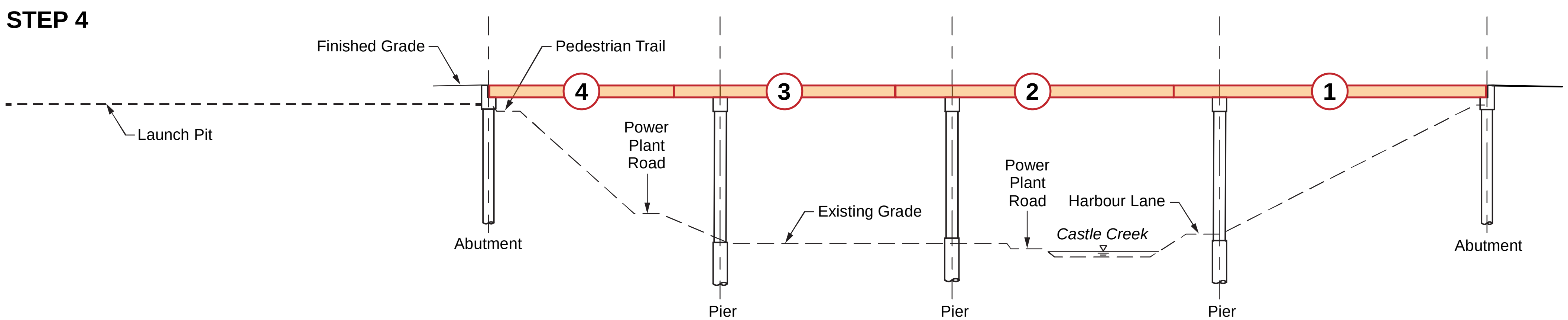
STEP 2



STEP 3



STEP 4



Step 0 (Not Shown):
Build the piers.
Close the existing bridge.
Demolish the existing bridge.
Build the abutments.
Dig the launch pit and set up launch rollers.

Step 1:
Set up and connect the launching nose and Segment 1 on the launch rollers.
Push and launch girders forward over the first pier.

Step 2:
Set up Segment 2 on the launch rollers and connect to Segment 1.
Push and launch girders forward over the next pier.

Step 3:
Set up Segment 3 on the launch rollers and connect to Segment 2.
Push and launch girders forward over the next pier.

Step 4:
Set up Segment 4 on the launch rollers and connect to Segment 3.
Push and launch girders forward to the abutment.
Disconnect the launching nose.
Replace temporary bearings with permanent bearings.
Foam/pour/cure the deck, diaphragms, approaches.

Figure 43b. ABC Incremental Bridge Launch Sequence
SH82 Over Castle Creek Bridge Feasibility Study

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Appendix F
ABC Method: Bridge Slide

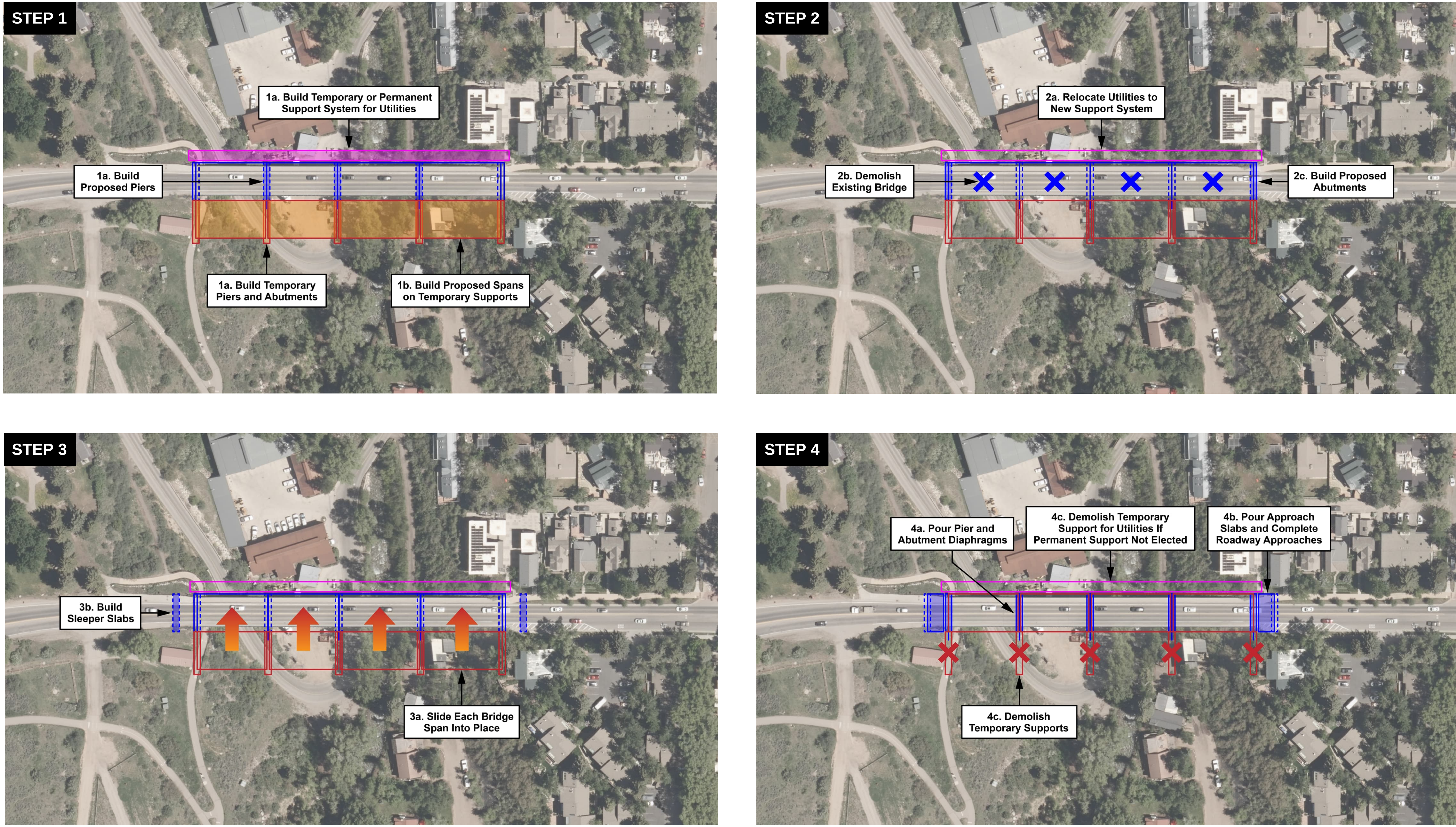


Figure 44a. Typical ABC Bridge Slide
SH82 Over Castle Creek Bridge Feasibility Study

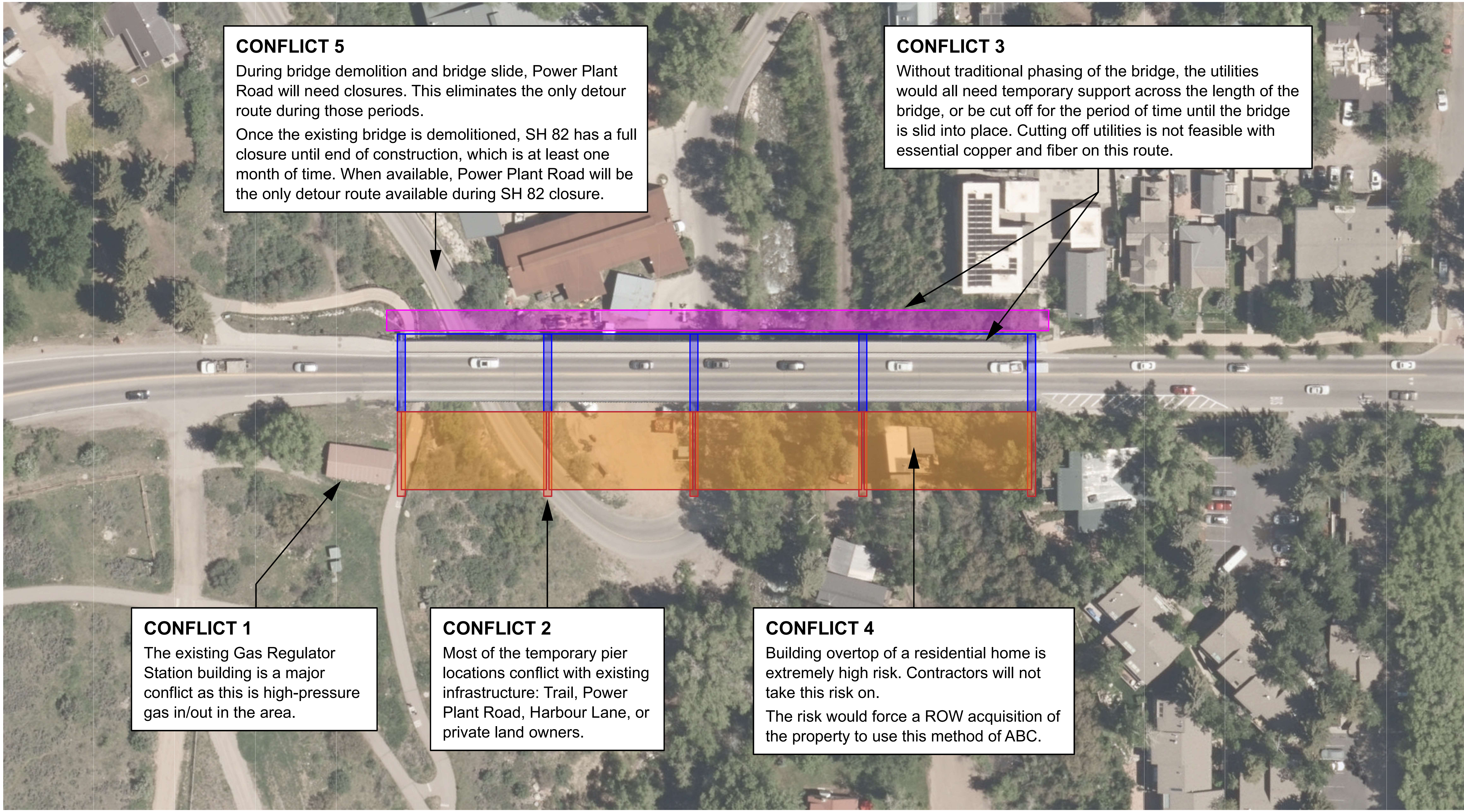


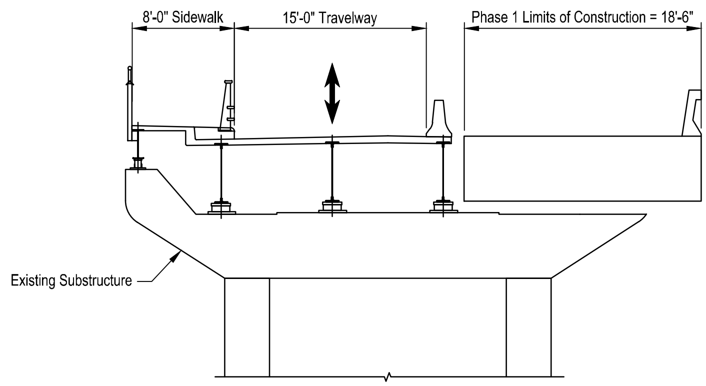
Figure 44b. Major Conflicts to an ABC Bridge Slide
SH82 Over Castle Creek Bridge Feasibility Study

Jacobs

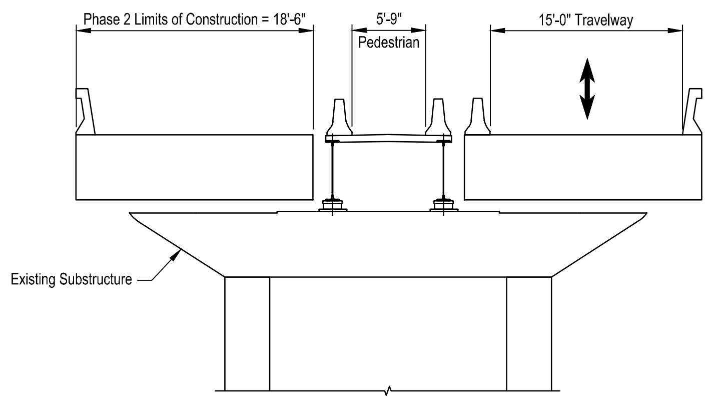
Appendix G
Replacement Phasing Options

TWO LANE

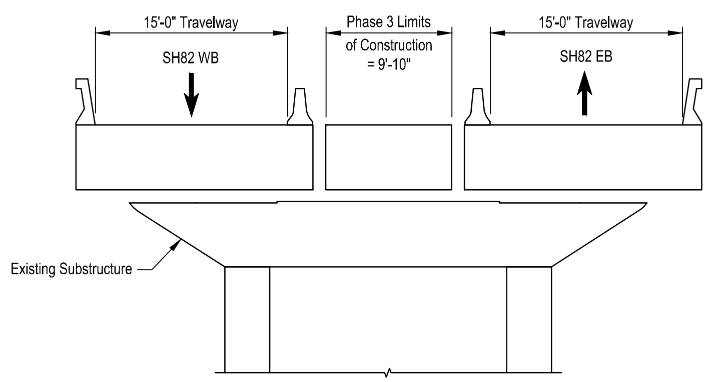
Rebuild the two lane + north trail layout that currently exists; eliminates the south sidewalk.



PHASE 1



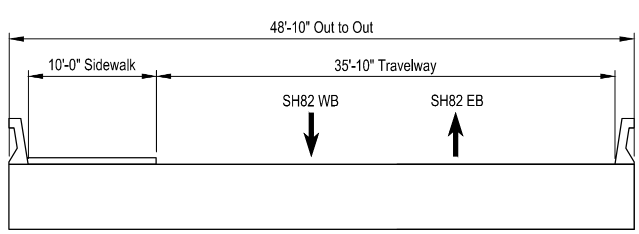
PHASE 2



PHASE 3

NOTE

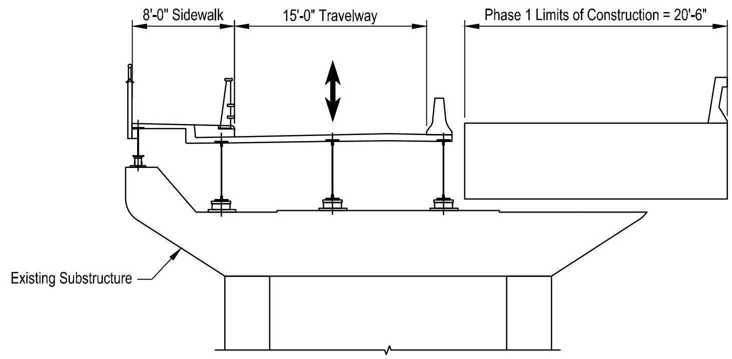
No pedestrian accommodation is currently provided in this phase.



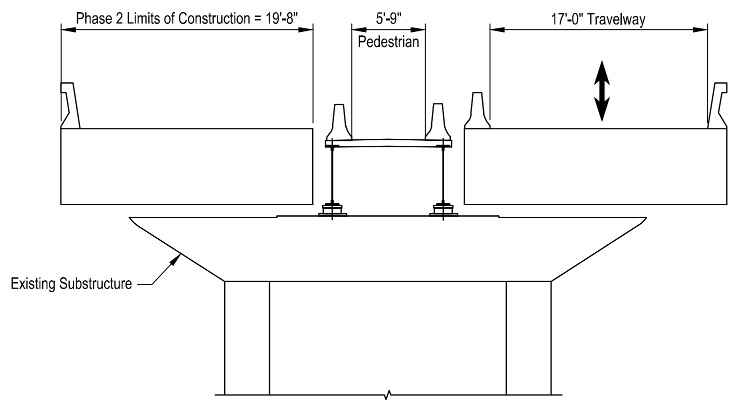
FINAL

THREE LANE OPTION 1

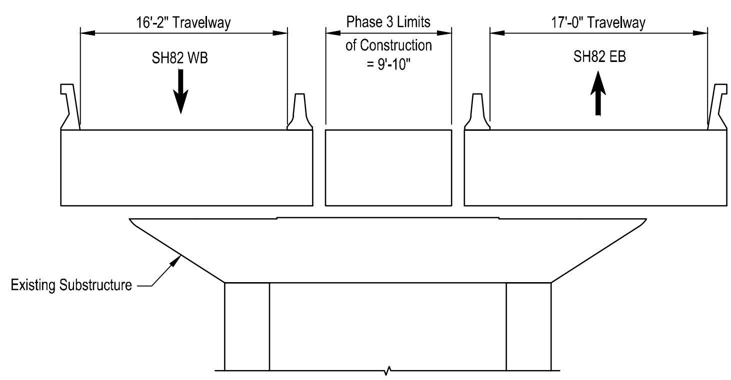
Replace with three lane + north trail layout that currently exists; eliminates the south sidewalk.



PHASE 1

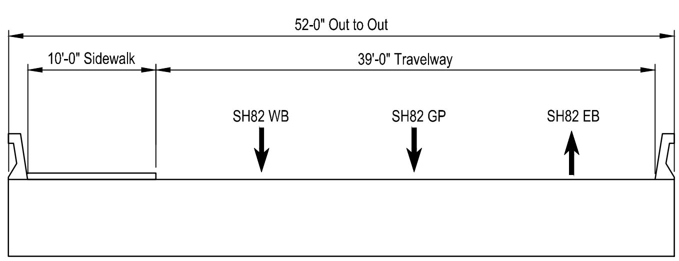


PHASE 2



NOTE
No pedestrian accommodation is currently provided in this phase.

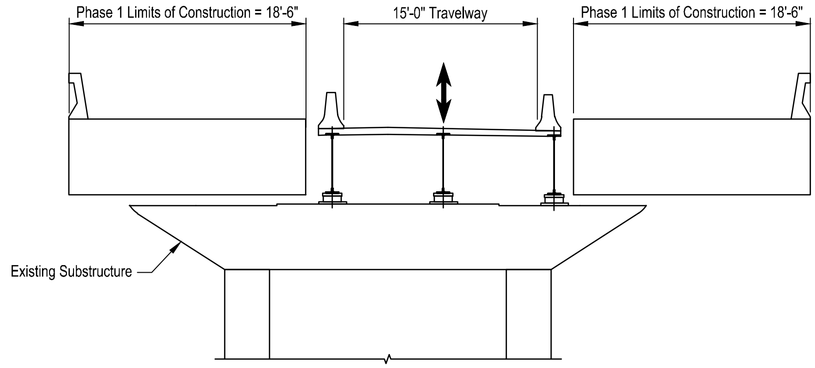
PHASE 3



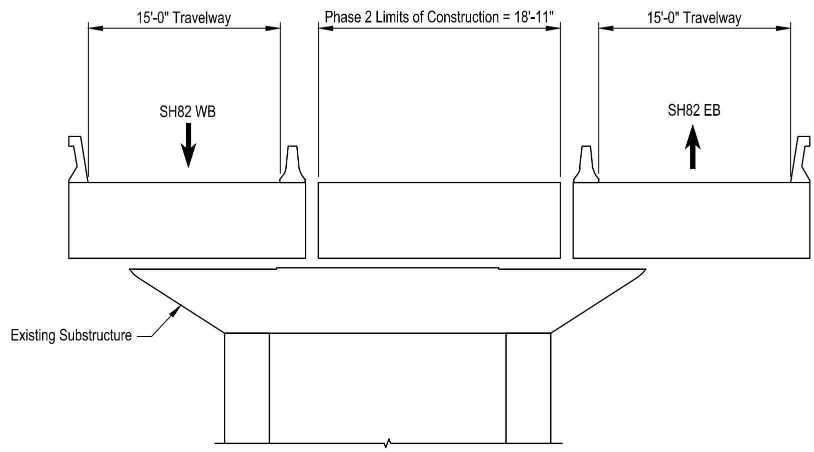
FINAL

THREE LANE OPTION 2

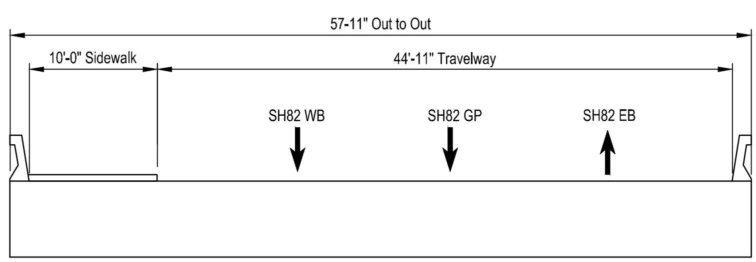
Replace with three lane + north trail layout that currently exists; eliminates the south sidewalk.



PHASE 1



PHASE 2



FINAL

NOTE

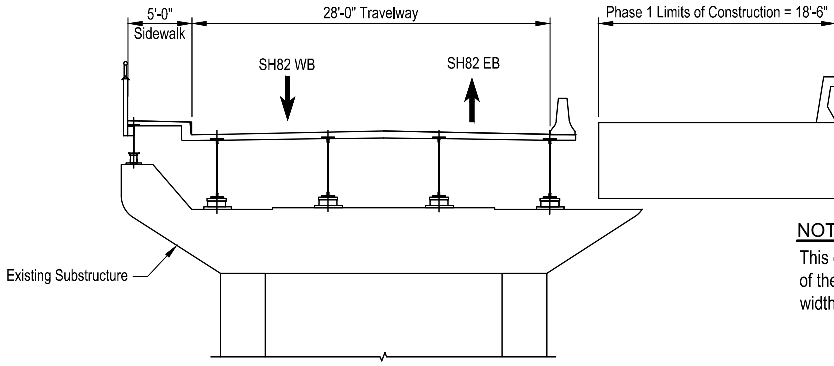
This option does not accommodate pedestrians in any phase of construction.

This option requires approximately 6 feet of overbuild on the bridge to minimize the time needed for using a single lane.

This option has 3.8 feet of ROW impact on the south side of the bridge.

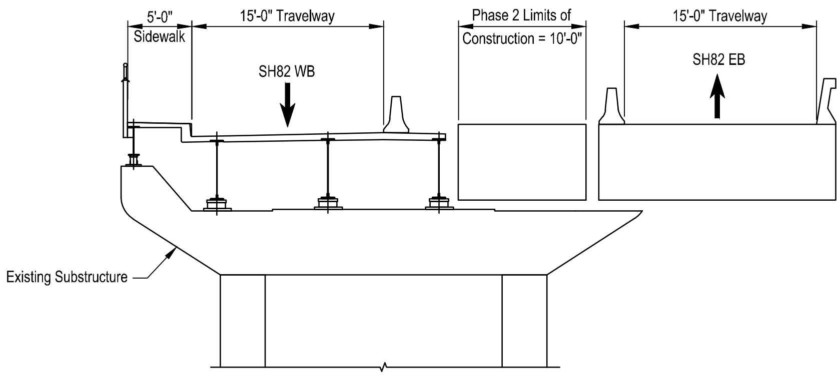
THREE LANE OPTION 3

Replace with three lane + north trail layout that currently exists; eliminates the south sidewalk.

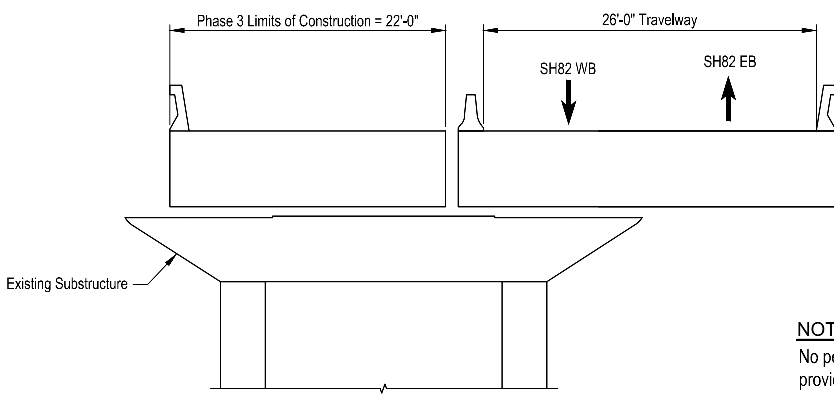


NOTE
This option requires removal of the "new" portion of the sidewalk to gain an additional 3 feet of width for construction phasing.

PHASE 1

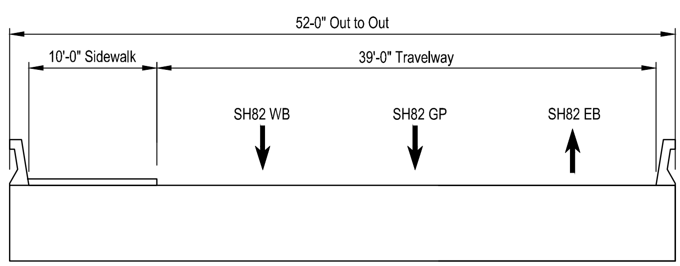


PHASE 2



NOTE
No pedestrian accommodation is currently provided in this phase.

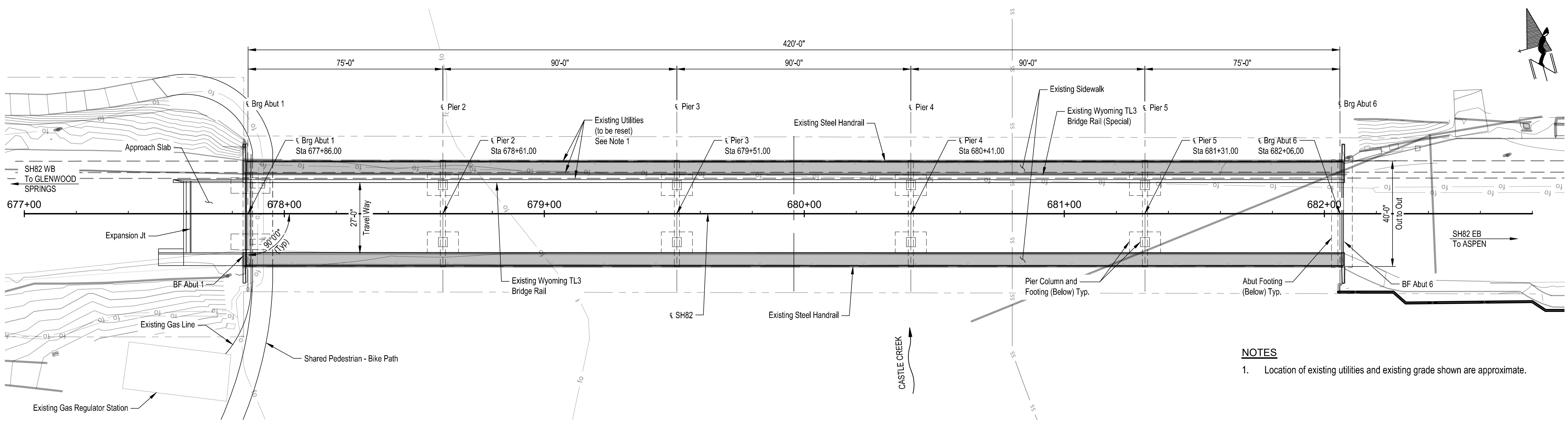
PHASE 3



FINAL

NOTE
This option has 4.6 feet of ROW impact on the south side of the bridge.

Appendix H
Conceptual Bridge Rehabilitation Plans



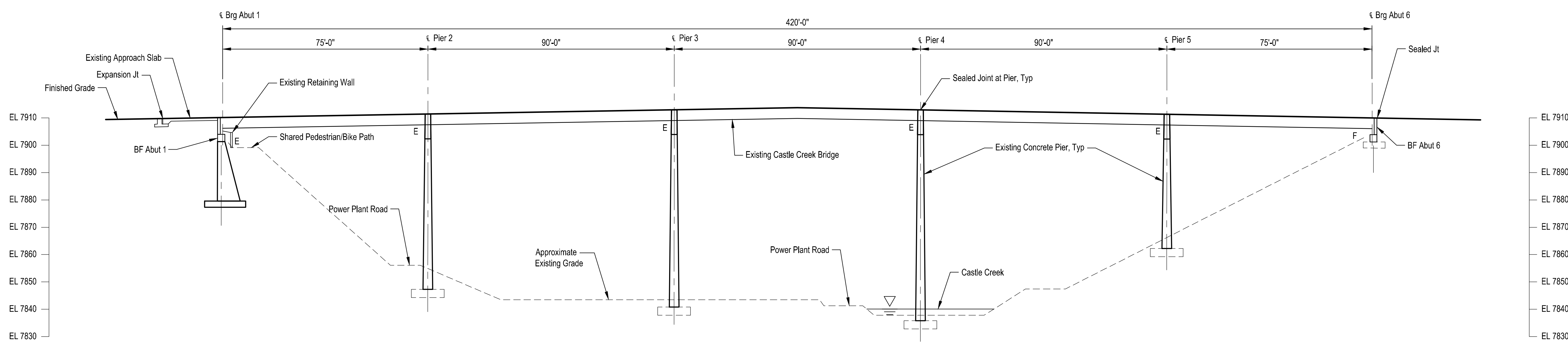
PLAN
SCALE: 1" = 40'

NOTES

1. Location of existing utilities and existing grade shown are approximate.

LEGEND

Limits of rebuild portions of present structure



ELEVATION
SCALE: 1" = 40'
(Existing railing not shown for clarity)

All seals for this set of drawings are applied to the cover page(s)

Print Date:	\$\$DATE\$\$
File Name:	\$\$FILE\$\$
Horiz. Scale:	AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed

No Revisions:

Revised:

Void:

SH82 over Castle Creek Bridge

REHABILITATION - ALTERNATIVE
GENERAL LAYOUT

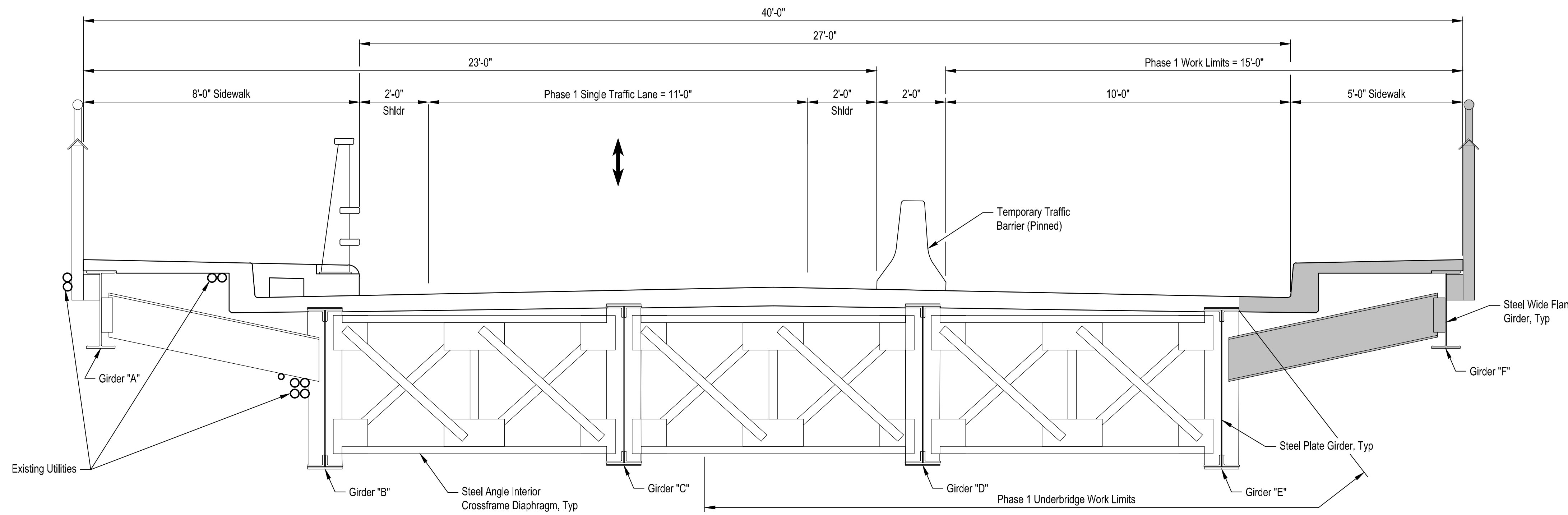
Designer:	A. BHANDARI	Structure Numbers	H-09-B
Detailer:	J. PROTHERO		
Sheet Subset:	BRIDGE	Subset Sheets:	B10 of B15

Project No./Code

2023-218

Sheet Number 2

\$\$PLOT_INFO\$\$

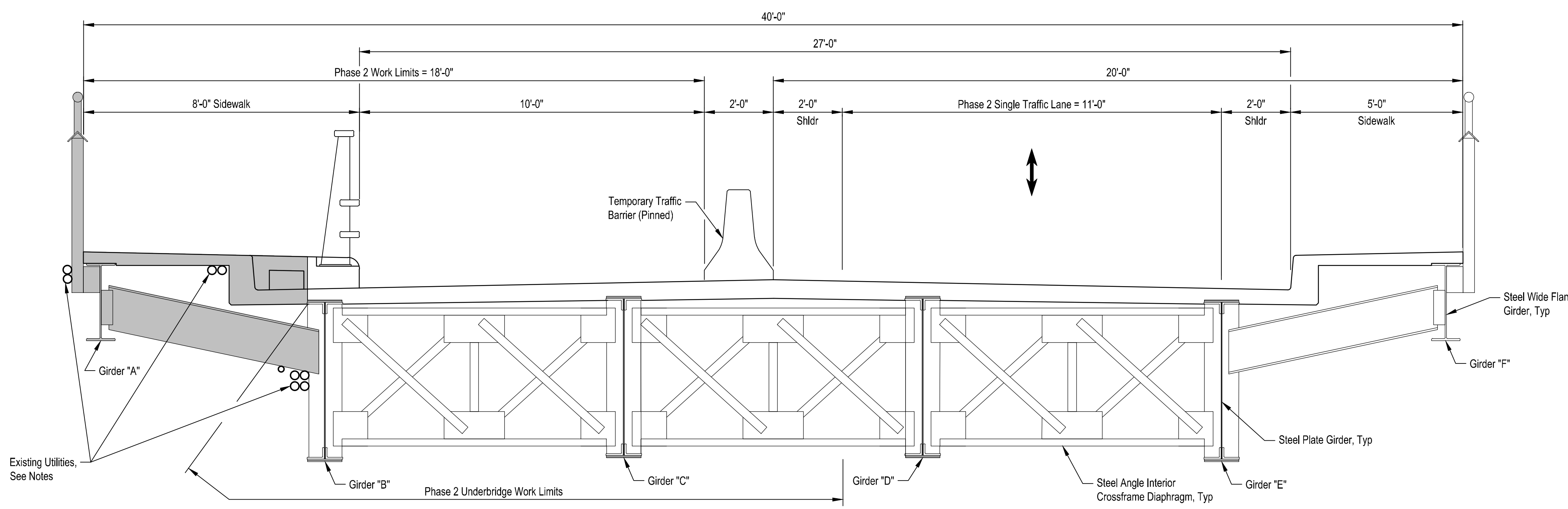


PHASE 1 SECTION AT BRIDGE

SCALE: 1/4" = 1'-0"
(Looking East)

PHASE 1:

- Remove and rebuild South sidewalk and framing



PHASE 2 SECTION AT BRIDGE

SCALE: 1/4" = 1'-0"
(Looking East)

LEGEND

Proposed removal and replacement

PHASE 2:

- Existing utilites to be removed and reset and/or relocated to South side of bridge.
- Remove and rebuild North sidewalk and framing

$$PLOT_INFO$$

All seals for this set of drawings are applied to the cover page(s)

Print Date: $$DATE$$
File Name: $$FILE$$
Horiz. Scale: AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed

No Revisions:
Revised:
Void:

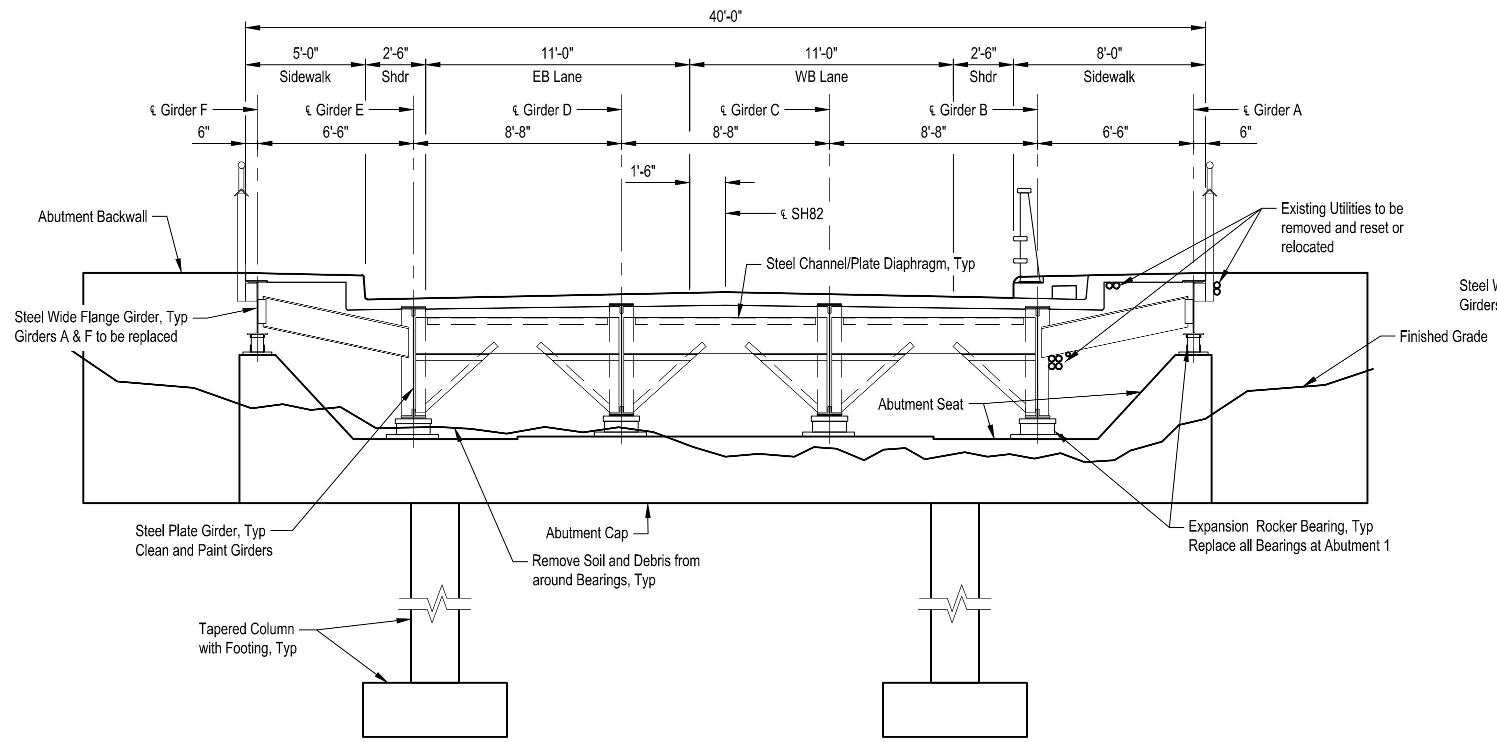
SH82 over Castle Creek Bridge
REHABILITATION - ALTERNATIVE CONSTRUCTION PHASING

Designer: A. BHANDARI	Structure Numbers	H-09-B
Detailer: J. PROTHERO		
Sheet Subset: BRIDGE	Subset Sheets:	B11 of B15

Project No./Code

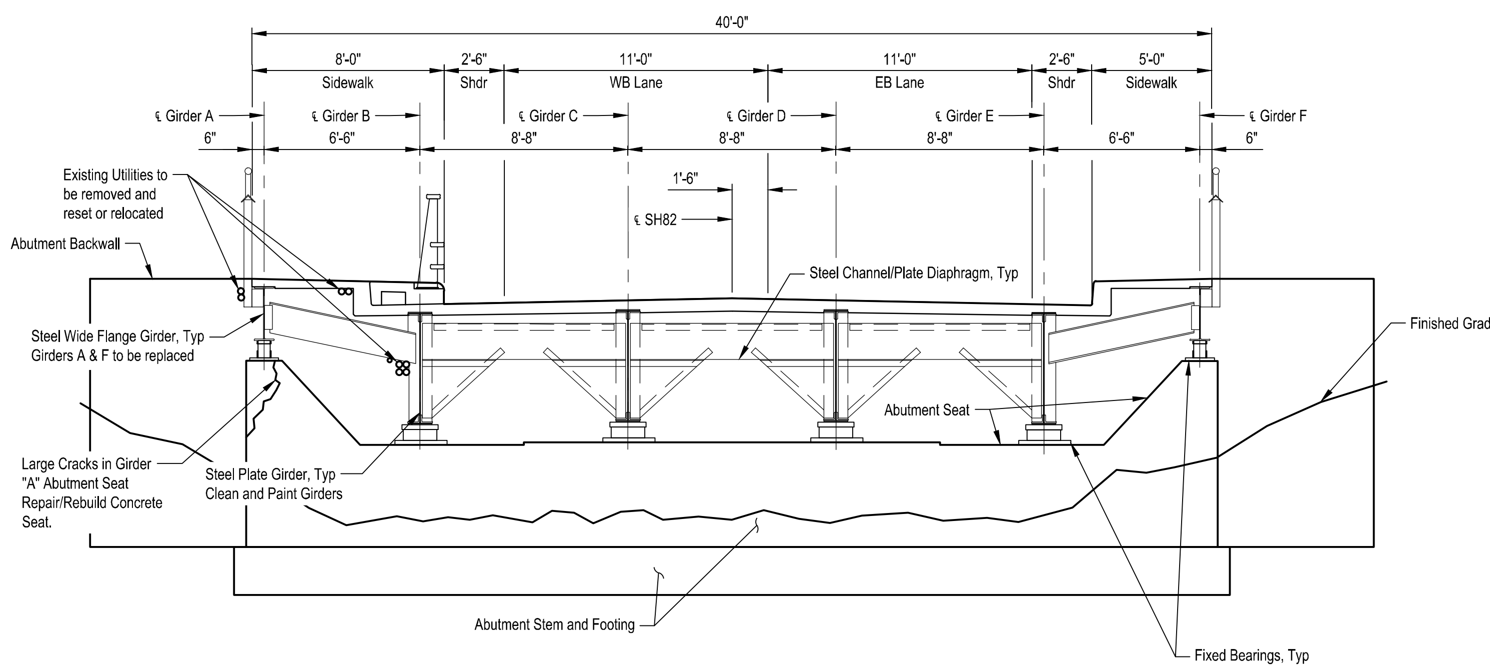
2023-218

Sheet Number 3



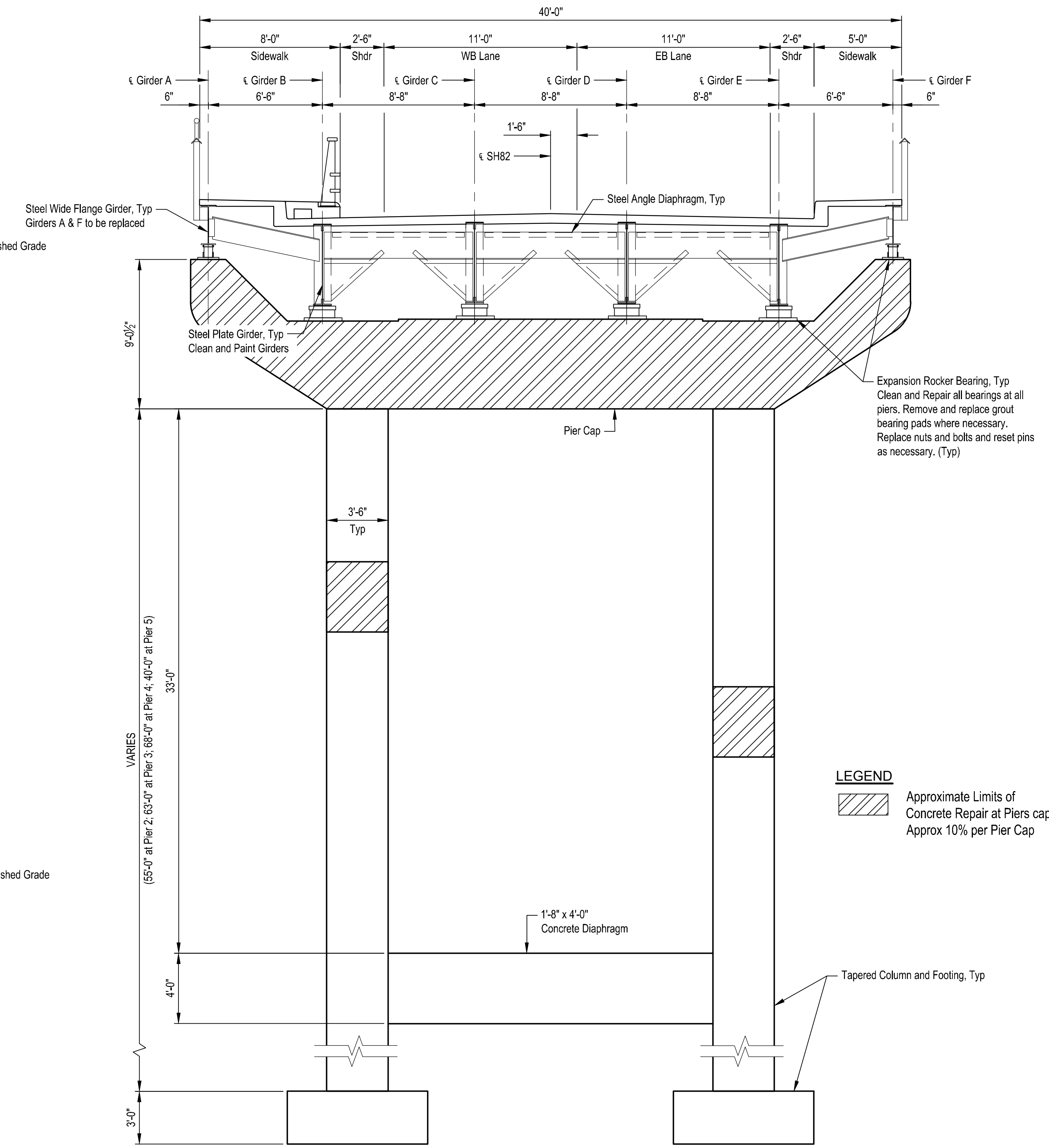
TYPICAL SECTION - ABUT 1

SCALE: 1/8" = 1'-0"
(LOOKING WEST)



TYPICAL SECTION - ABUT 6

SCALE: 1/8" = 1'-0"
(LOOKING EAST)



TYPICAL SECTION - PIERS 2, 3, 4 AND 5

SCALE: 1/8" = 1'-0"
(Looking East)

All seals for this set of drawings are applied to the cover page(s)

Print Date:	$SDATE$S
File Name:	$SFILE$S
Horiz. Scale:	AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed

No Revisions:

Revised:

Void:

SH82 over Castle Creek Bridge

REHABILITATION - ALTERNATIVE
TYPICAL SECTIONS

Designer:	A. BHANDARI	Structure Numbers	H-09-B
Detailer:	J. PROTHERO		
Sheet Subset:	BRIDGE	Subset Sheets:	B12 of B15

Project No./Code

2023-218

Sheet Number 4

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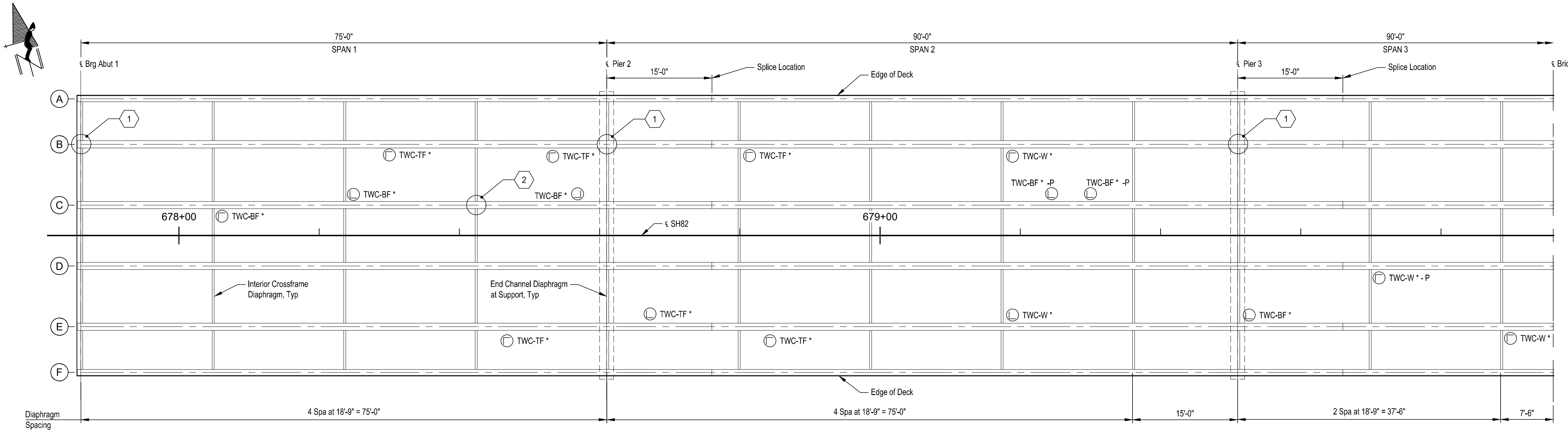
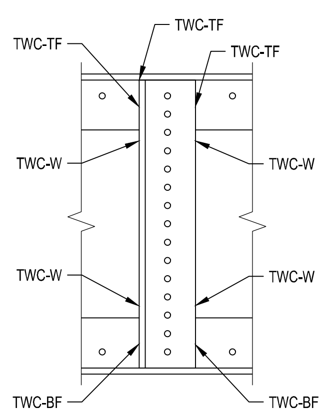


PARTIAL FRAMING PLAN
SCALE: 1" = 15'

NOTE: Not all girder defects shown. Only tack weld cracks that are NOT self-arresting (NSA) are noted. All NSA welds shall be repaired at a minimum to improve the service life of the steel plate girders.

KEY NOTES

1. Pack rust between faying surfaces of angle legs, up to ½" thick, typical at bearing stiffener locations.
2. Cracked stiffener weld at bottom.



GIRDER FACE VIEW

CODE LEGEND

TWC-W	TACK WELD CRACK - WEB TO STIFFENER
TWC-BF	TACK WELD CRACK - BOTTOM FLANGE TO STIFFENER
TWC-TF	TACK WELD CRACK - TOP FLANGE TO STIFFENER

ADD-ON CODES:

*	NOT SELF ARRESTED
-P	POSSIBLE CRACK

All seals for this set of drawings are applied to the cover page(s)

Print Date: $$DATE$$
File Name: $$FILES$$
Horiz. Scale: AS NOTED

Jacobs

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Date	Comments	Init.

As Constructed
No Revisions:
Revised:
Void:

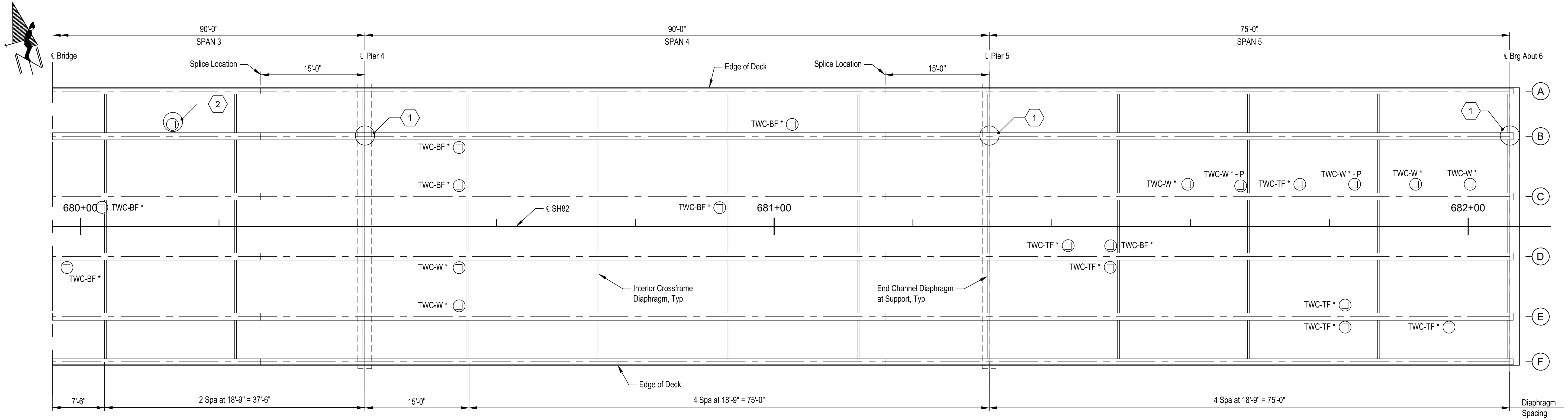
SH82 over Castle Creek Bridge
REHABILITATION - ALTERNATIVE
FRAMING PLAN 1 OF 2

Designer: A. BHANDARI
Detailer: J. PROTHERO
Structure Numbers: H-09-B
Sheet Subset: BRIDGE
Subset Sheets: B13 of B15

Project No./Code
2023-218

Sheet Number 5

$$PLOT_INFO$$



PARTIAL FRAMING PLAN

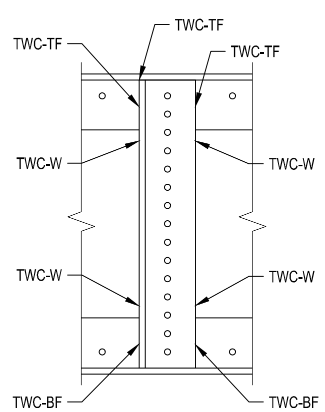
SCALE: 1" = 15'

NOTE: Not all girder defects shown. Only tack weld cracks that are NOT self-arresting (NSA) are noted. All NSA welds shall be repaired at a minimum to improve the service life of the steel plate girders.

KEY NOTES

1. Pack rust between faying surfaces of angle legs, up to ½ " thick, Typ at bearing stiffener locations.

2. Stiffener deflected, up to 1" out of plane over 6" height, near bottom ⅓ point.



GIRDER FACE VIEW

CODE LEGEND

TWC-W	TACK WELD CRACK - WEB TO STIFFENER
TWC-BF	TACK WELD CRACK - BOTTOM FLANGE TO STIFFENER
TWC-TF	TACK WELD CRACK - TOP FLANGE TO STIFFENER

ADD-ON CODES:

*	NOT SELF ARRESTED
-P	POSSIBLE CRACK

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File Name:	$SFILES$S
Horiz. Scale:	AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed

No Revisions:

Revised:

Void:

SH82 over Castle Creek Bridge

REHABILITATION - ALTERNATIVE FRAMING PLAN 2 OF 2

Designer:	A. BHANDARI	Structure Numbers	H-09-B
Detailer:	J. PROTHERO		
Sheet Subset:	BRIDGE	Subset Sheets:	B14 of B15

Project No./Code

2023-218

Sheet Number 6

$SPLOT_INFO$S

Clean, repaint, and regrout bearings at piers where necessary. Replace loose nuts and bolts where necessary. (Typical of all bearings at piers)

TYPICAL EXPANSION ROCKER BEARING AT PIER

Grind out all Non Self-Arresting (NSA) Tack Welds

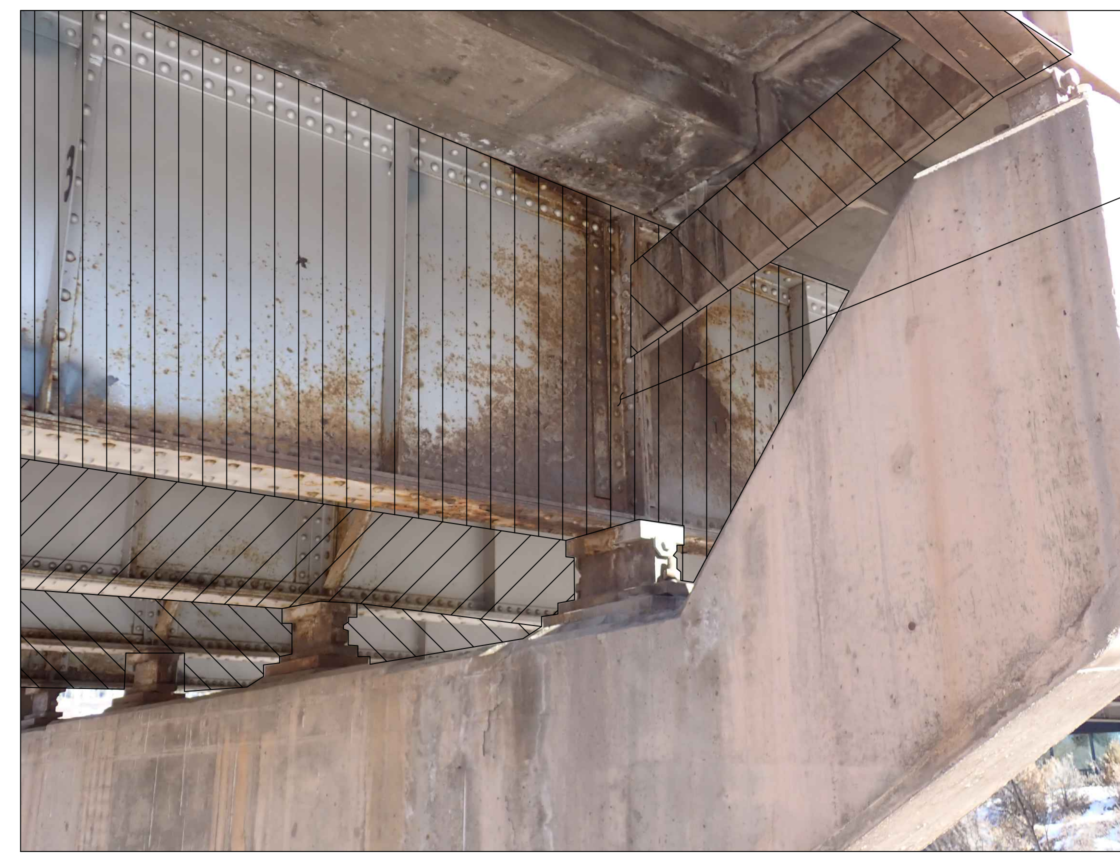
TYPICAL STIFFENER TACK WELD

Fixed Bearing Girder "A"

Girder "B" (Beyond)

Concrete Repair of Bearing pedestal at Abutment 6, Girder A
Partial removal and reconstruction of pedestal due to extents of defects

ABUTMENT 6-SUPPORT PEDESTAL AT GIRDER "A"



Clean and Repaint all Structural Steel

STEEL GIRDER SUPERSTRUCTURE

All seals for this set of drawings are applied to the cover page(s)

Print Date: $$DATE$$
File Name: $$FILES$$
Horiz. Scale: AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed
No Revisions:
Revised:
Void:

SH82 over Castle Creek Bridge
REHABILITATION - ALTERNATIVE
REPAIR DETAIL NOTES

Designer: A. BHANDARI
Detailer: J. PROTHERO
Sheet Subset: BRIDGE
Structure Numbers: H-09-B
Subset Sheets: B15 of B15

Project No./Code
2023-218
Sheet Number 7

$$PLOT_INFO$$

Appendix I
Conceptual Two-lane Bridge Replacement Plans

GENERAL NOTES

The existing bridge is a 5 span (75'-0", 90'-0", 90'-0', 90'-0", and 75'-0") continuous steel girder bridge over Power Plant Road, Castle Creek, and Harbour Lane. 8'-0" north sidewalk and 5'-0" south sidewalk, 27'-0" roadway, Wyoming TL3 Bridge Rail, and (2) Side-Mounted Bridge Rails.

Rehabilitation work considered includes, bearing replacement and repair, bearing pedestal replacement at Abut 6, exterior girder replacement, steel painting, tack weld removal, concrete repairs for deck and pier caps, joint sealant replacement, and bridge rail repair.

2-Lane replacement considers replacement of the existing bridge with a 4 span (97'-6", 97'-6", 112'-6", and 112'-6") continuous post-tensioned CIP box girder bridge. 10'-0" sidewalk, 36'-0" roadway, and (2) 1'-6" Type 9 Bridge Rails.

3-Lane replacement considers replacement of the existing bridge with a 4 span (97'-6", 97'-6", 112'-6", and 112'-6") continuous post-tensioned CIP box girder bridge. 10'-0" sidewalk, 39'-0" roadway, and (2) 1'-6" Type 9 Bridge Rails.

Phased construction assumed for all alternatives.

See "SH82 over Castle Creek Bridge Feasibility Study" for additional information.

DESIGN DATA

AASHTO, 9th Edition LRFD with current interims

Design Method: Load and Resistance Factor Design

Live Load: HL-93 (design truck or tandem, and design lane load)
Dead Load: Assumes 36 psf for bridge deck overlay
Assumes 5 psf for permanent deck forms

Reinforced Concrete:

Class D Concrete:	f'c	= 4,500 psi
Reinforcing Steel:	f'y	= 60,000 psi

Drilled Shaft Concrete:

Class BZ Concrete:	f'c	= 4,000 psi
Reinforcing Steel:	fy	= 60,000 psi

Structural Steel:

AASHTO M270 (ASTM A709) Grade 36 fy = 36,000 psi
AASHTO M270 (ASTM A709) Grade 50 fy = 50,000 psi

Post-Tensioned concrete:

Class S concrete	f'c	= (see details)
	f's	= 270,000 psi

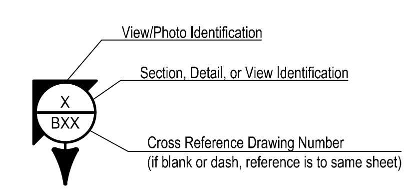
INDEX OF DRAWINGS

ABBREVIATIONS

(Per M-100-2 or as shown below)

Ea	= Each
BF	= Back Face
FF	= Front Face
FFBW	= Front Face Backwall
RC	= Reinforced Concrete
WSEL	= Water Surface Elevation
EB	= Eastbound
WB	= Westbound
GP	= General Purpose
Abut	= Abutment
Sta	= Station
Typ.	= Typical
E	= Expansion Bearing
F	= Fixed Bearing

811
Know what's below.
Call before you dig.



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	File Name: $$FILE$$	Date	Comments	Init.
	Horiz. Scale: AS NOTED		
	Jacobs		

As Constructed	SH82 over Castle Creek Bridge		Project No./Code
No Revisions:	GENERAL NOTES		2023-218
Revised:	Designer: S. SOWAL	Structure Numbers: H-09-B	
Void:	Detailer: A. PRICE		Sheet Number 1
	Sheet Subset: BRIDGE	Subset Sheets: B01 of	

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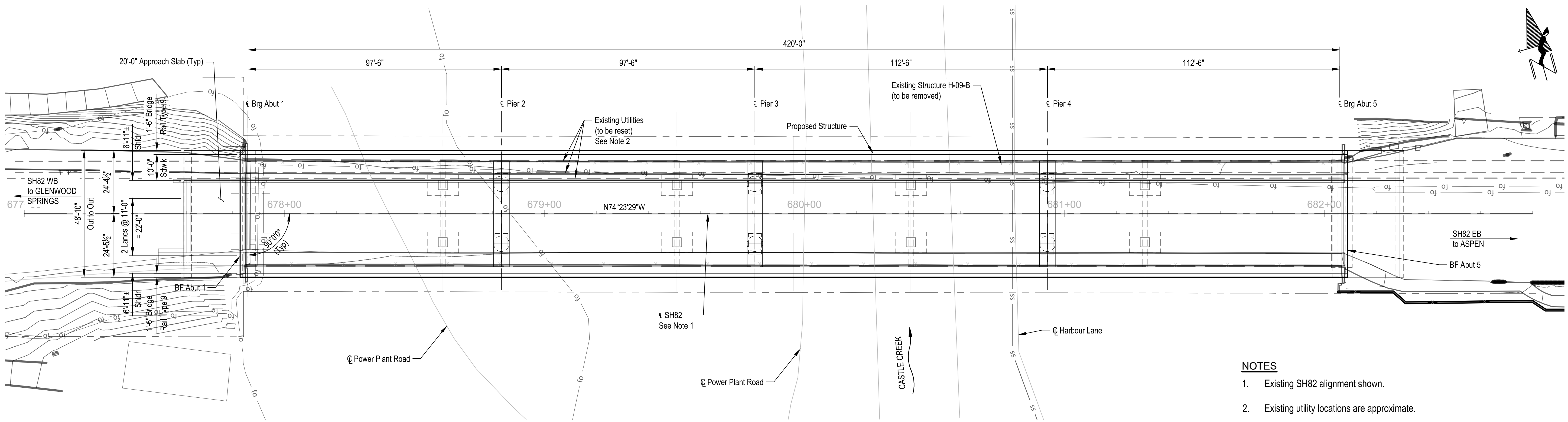
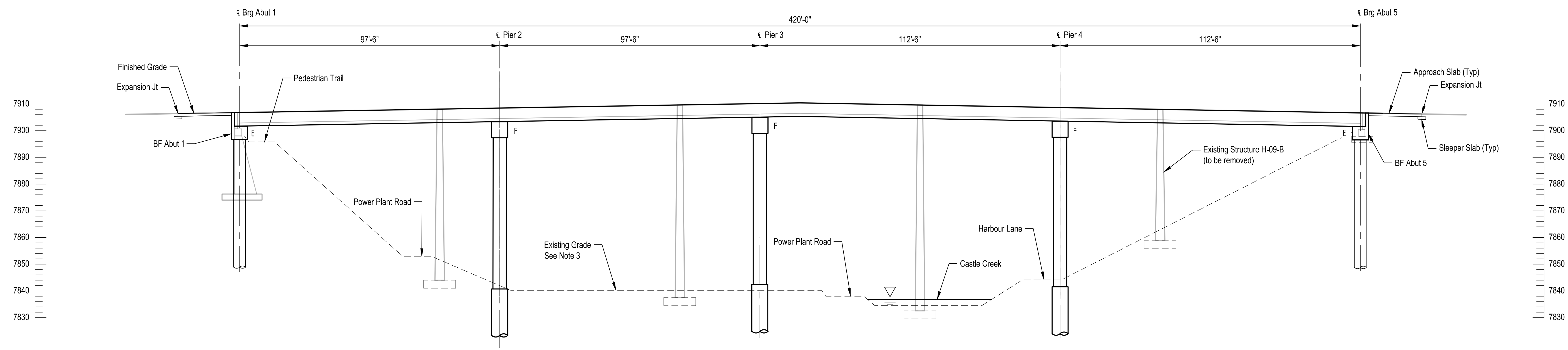


NOTES

1. Existing SH82 alignment shown.
2. Existing utility locations are approximate.
3. Existing grade is approximate.

PLAN
SCALE: 1" = 40'



ELEVATION
SCALE: 1" = 40'

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File Name:	$$FILE$$
Horiz. Scale:	AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed

No Revisions:

Revised:

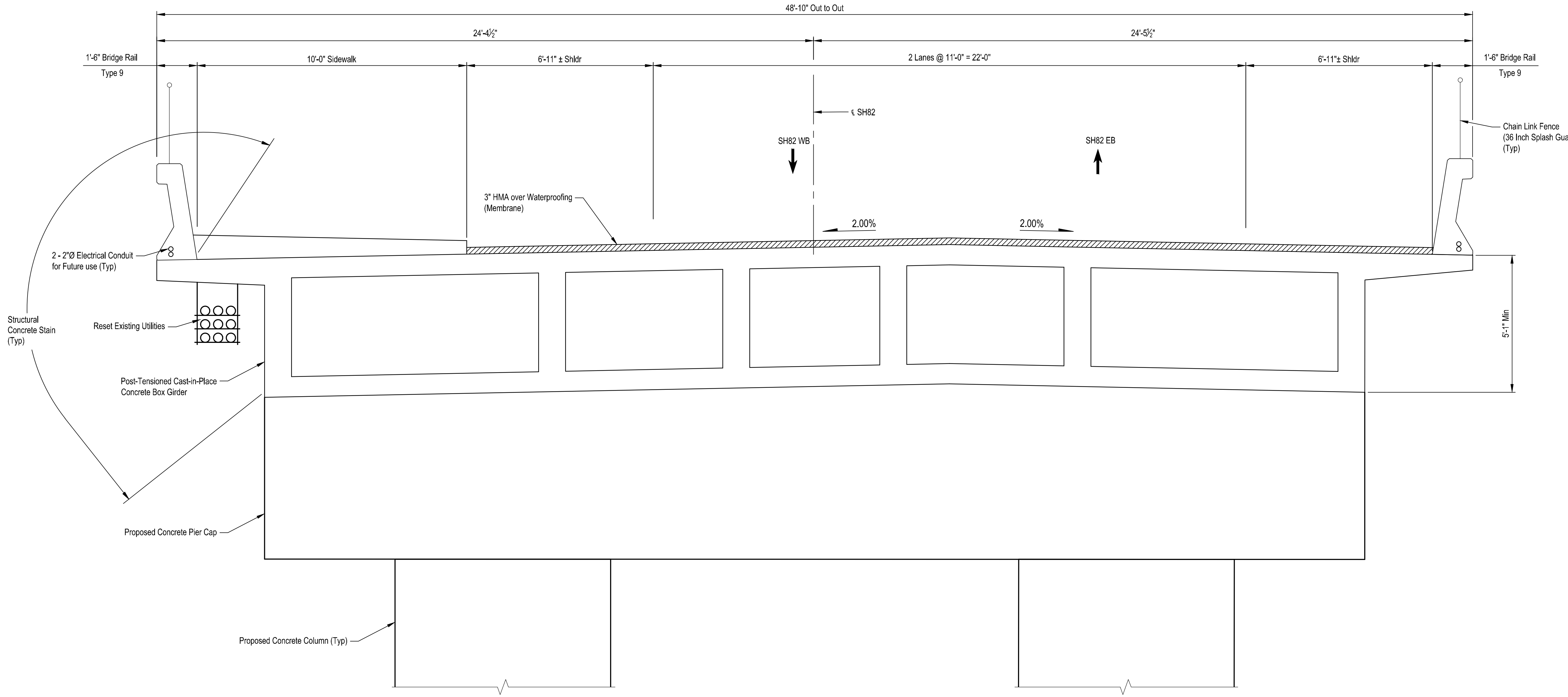
Void:

SH82 over Castle Creek Bridge
REPLACEMENT - 2 LANE ALTERNATIVE
GENERAL LAYOUT

Designer:	S. SOWAL	Structure Numbers	H-09-B
Detailer:	A. PRICE		
Sheet Subset:	BRIDGE	Subset Sheets:	B20 of B24

Project No./Code
2023-218

Sheet Number 8



TYPICAL SECTION
SCALE: 1/4" = 1'-0"
(Looking East, at Pier)

All seals for this set of drawings are applied to the cover page(s)

Print Date: $$DATE$$
File Name: $$FILE$$
Horiz. Scale: AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed
No Revisions:
Revised:
Void:

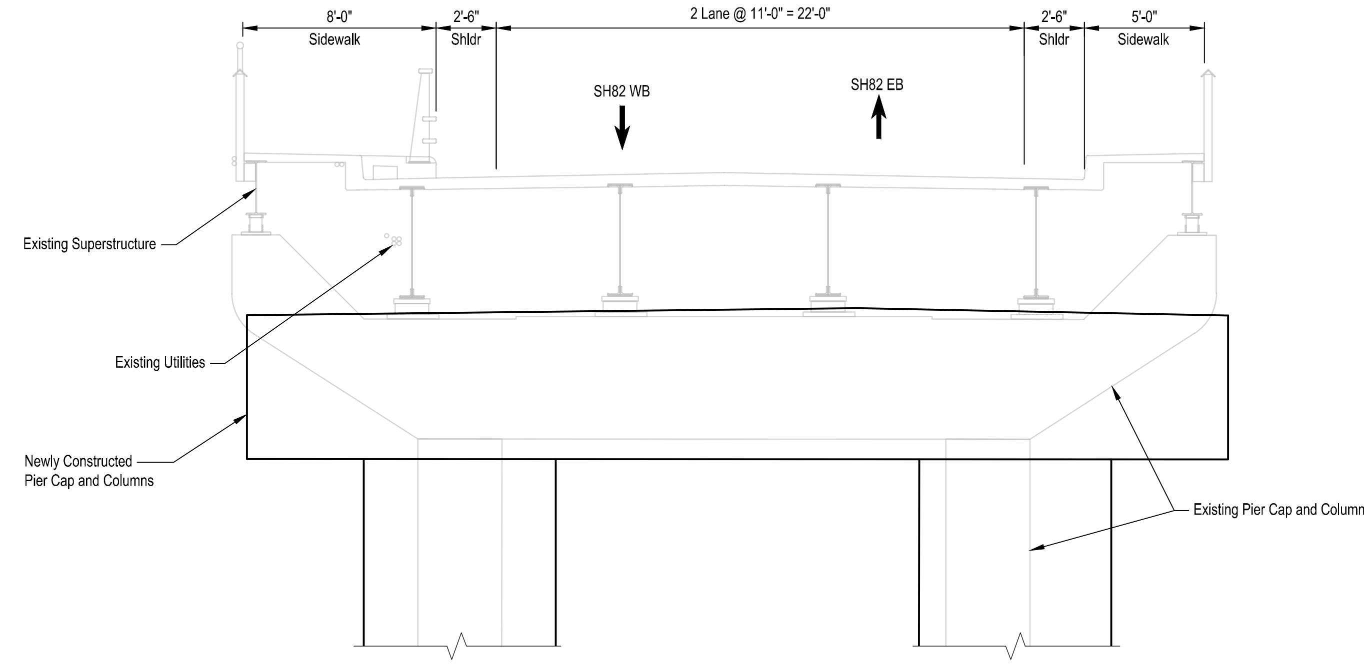
SH82 over Castle Creek Bridge
REPLACEMENT - 2 LANE ALTERNATIVE
TYPICAL SECTION

Designer:	S. SOWAL	Structure Numbers	H-09-B
Detailer:	A. PRICE		
Sheet Subset:	BRIDGE	Subset Sheets:	B21 of B24

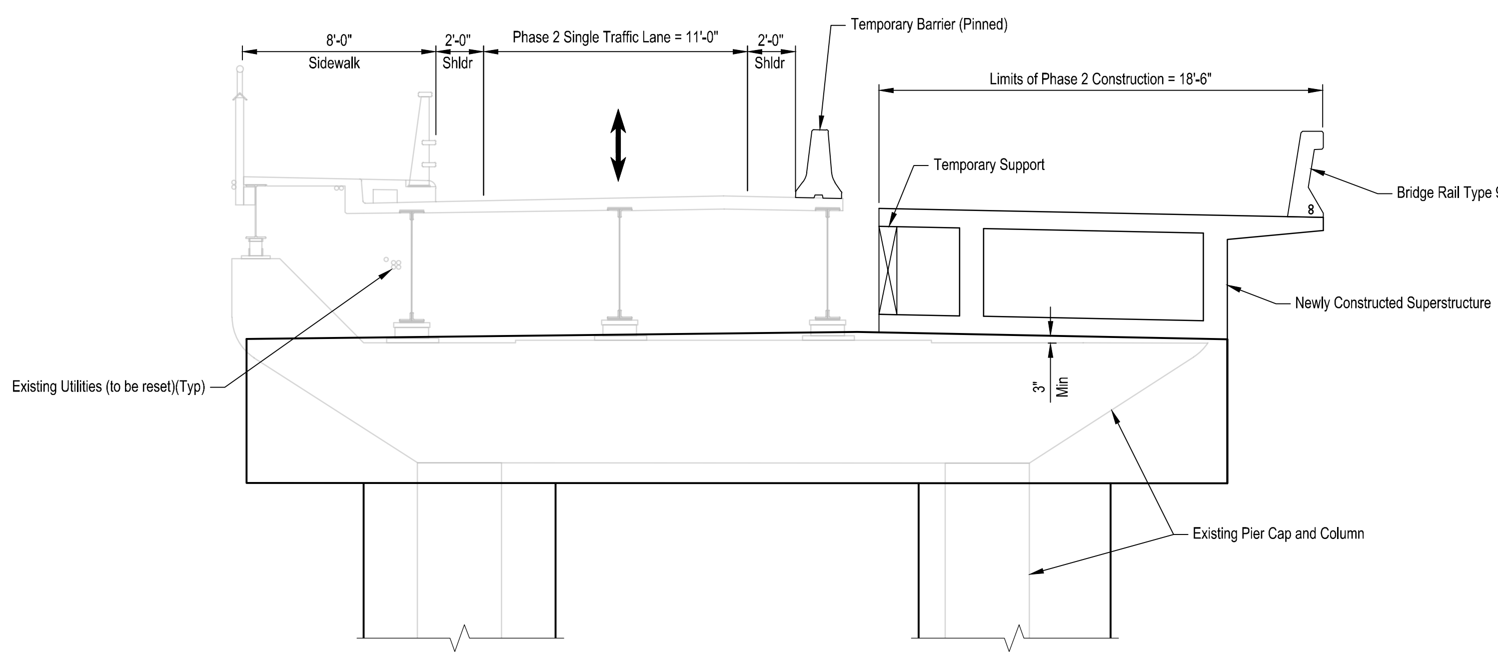
Project No./Code
2023-218

Sheet Number 10

$$PLOT_INFO$$



PHASE 1 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)



PHASE 2 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)

PHASE 1:

- Construct new piers under the existing superstructure.
- Traffic to remain on existing structure during this phase of construction.

PHASE 2:

- Install Temporary Barriers (pinned to existing bridge deck).
- Demolish southern portion of existing bridge. Remove exterior girder bearing seats from epiers and abutments.
- Build new section of bridge at the southern edge.
- EB and WB traffic shall remain on the existing bridge. A single lane shall be provided.
- Sidewalk shall remain on the existing structure.

All seals for this set of drawings are applied to the cover page(s)

Print Date: $$DATE$$
File Name: $$FILE$$
Horiz. Scale: AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed
No Revisions:
Revised:
Void:

SH82 over Castle Creek Bridge
REPLACEMENT - 2 LANE ALTERNATIVE
CONSTRUCTION PHASING (1 OF 3)

Designer: S. SOWAL
Detailer: A. PRICE
Sheet Subset: BRIDGE
Structure Numbers: H-09-B
Subset Sheets: B22 of B24

Project No./Code
2023-218
Sheet Number 10

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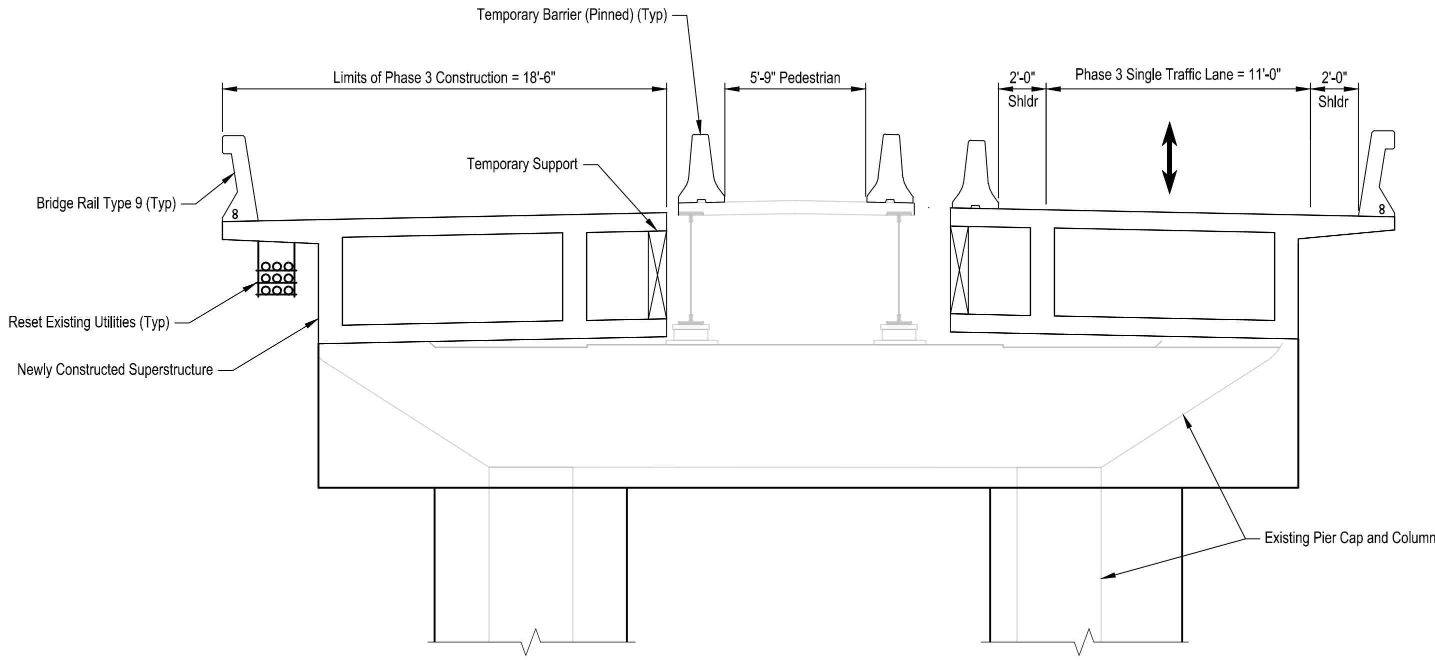
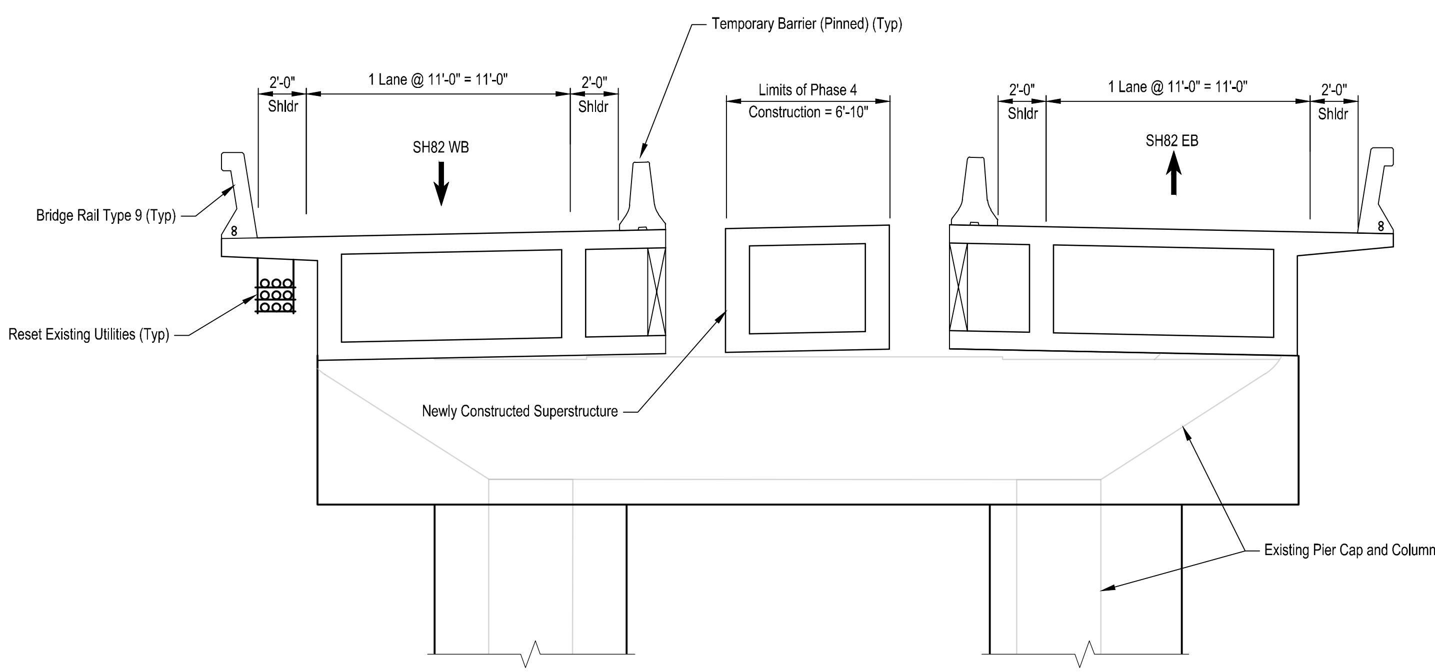


PHASE 3 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)



PHASE 4 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)

PHASE 3:

- Install Temporary Barriers (pinned to existing bridge and pinned to newly constructed deck).
- Demolish northern portion of existing bridge. Remove exterior girder bearing seats from piers and abutments.
- Build new section of bridge at northern edge.
- Reset existing utilities in hanger along the northern overhang.
- EB and WB traffic shall move to the southern portion of the newly constructed bridge. A single lane shall be provided.
- Pedestrian access shall be provided on the remaining portion of the existing bridge.

PHASE 4:

- Install Temporary Barriers (pinned to new deck).
- Demolish remaining portion of existing bridge.
- Build new section of bridge.
- EB and WB traffic shall be moved to newly constructed bridge segments.
- Pedestrian access is rerouted to under the bridge, along existing trail.

All seals for this set of drawings are applied to the cover page(s)

Print Date: $$DATE$$
File Name: $$FILE$$
Horiz. Scale: AS NOTED

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Sheet Revisions		
Date	Comments	Init.

As Constructed
No Revisions:
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Void:

SH82 over Castle Creek Bridge
REPLACEMENT - 2 LANE ALTERNATIVE
CONSTRUCTION PHASING (2 OF 3)

Designer: S. SOWAL
Detailer: A. PRICE
Sheet Subset: BRIDGE
Structure Numbers: H-09-B
Subset Sheets: B23 of B24

Project No./Code
2023-218

Sheet Number 11

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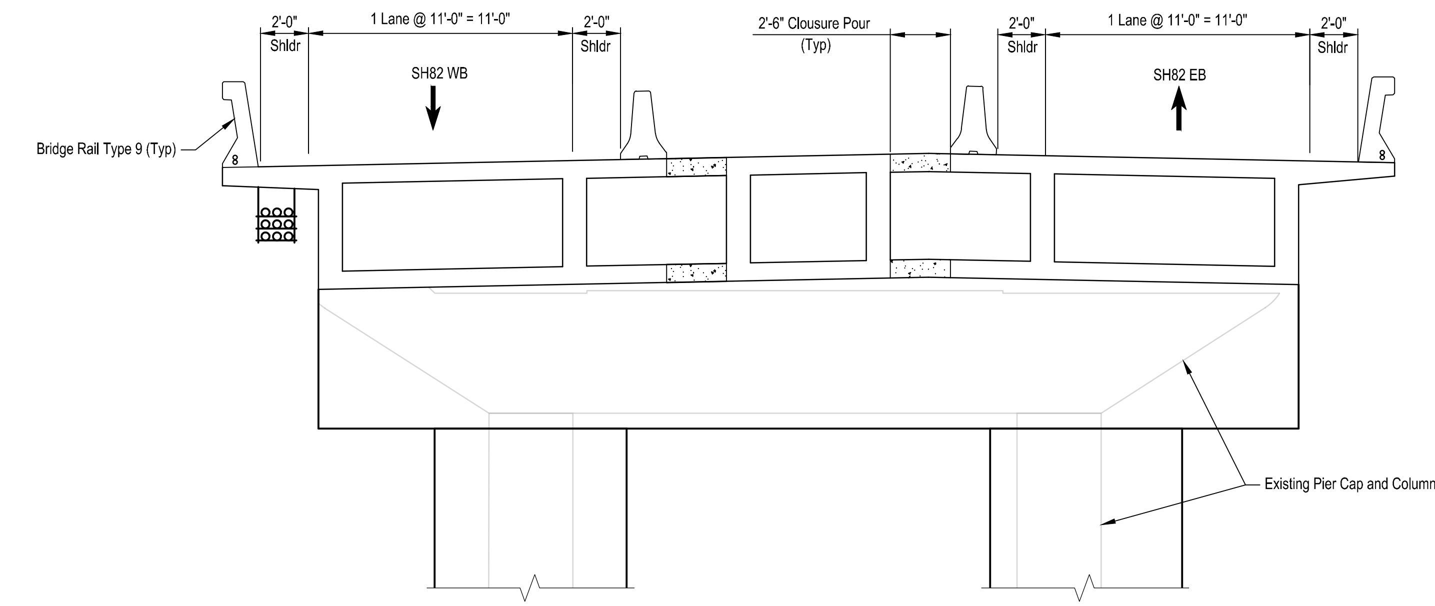
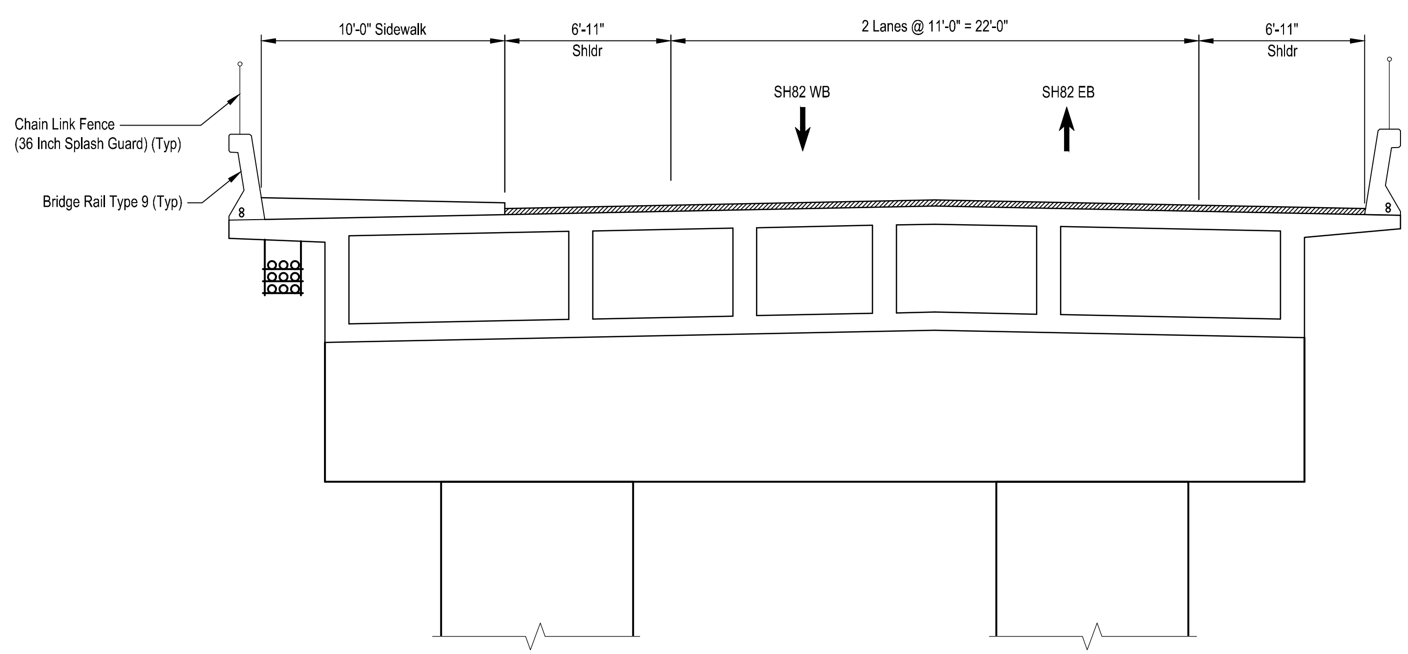


PHASE 5 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)



FINAL CONFIGURATION
SCALE: 1/8" = 1'-0"
(Looking East)

PHASE 5:

- Join new bridge segments with closure pour.

FINAL CONFIGURATION:

- Remove temporary Barriers.
- Construct sidewalk.
- Install HMA over Waterproofing (Membrane).
- Install Chain Link Fence (36 inch Splash Guard).
- Remove existing piers and abutments (all existing substructure)

All seals for this set of drawings are applied to the cover page(s)

Print Date:	$$DATE$$
File Name:	$$FILE$$
Horiz. Scale:	AS NOTED

Jacobs

Sheet Revisions

Date	Comments	Init.

As Constructed
No Revisions:
Revised:
Void:

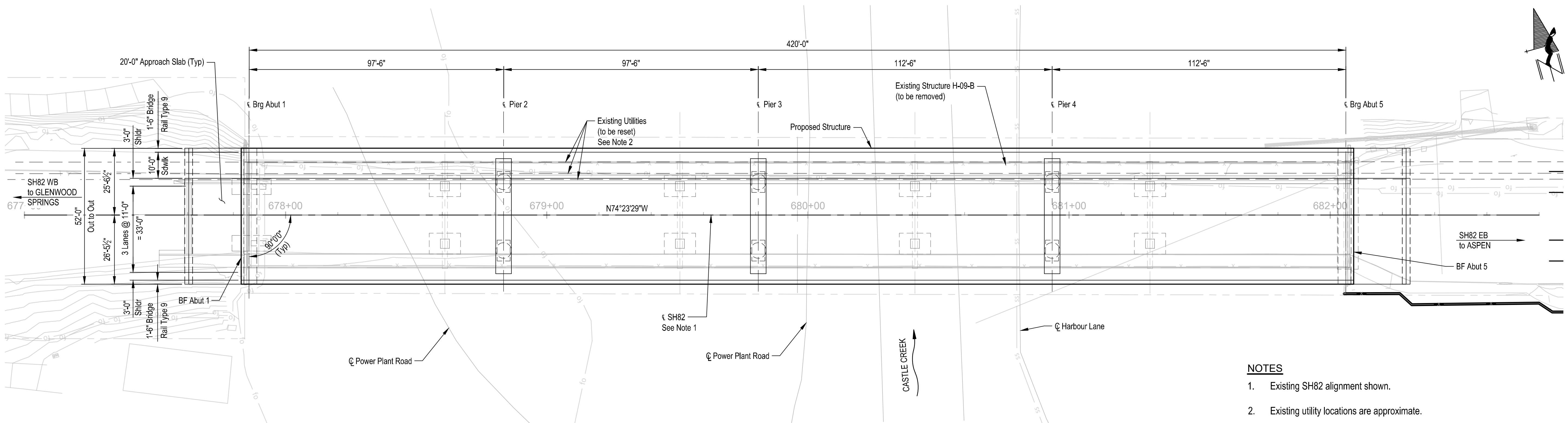
SH82 over Castle Creek Bridge
REPLACEMENT - 2 LANE ALTERNATIVE
CONSTRUCTION PHASING (3 OF 3)

Designer:	S. SOWAL	Structure Numbers	H-09-B
Detailer:	A. PRICE		
Sheet Subset:	BRIDGE	Subset Sheets:	B24 of B24

Project No./Code
2023-218
Sheet Number 13

$$PLOT_INFO$$

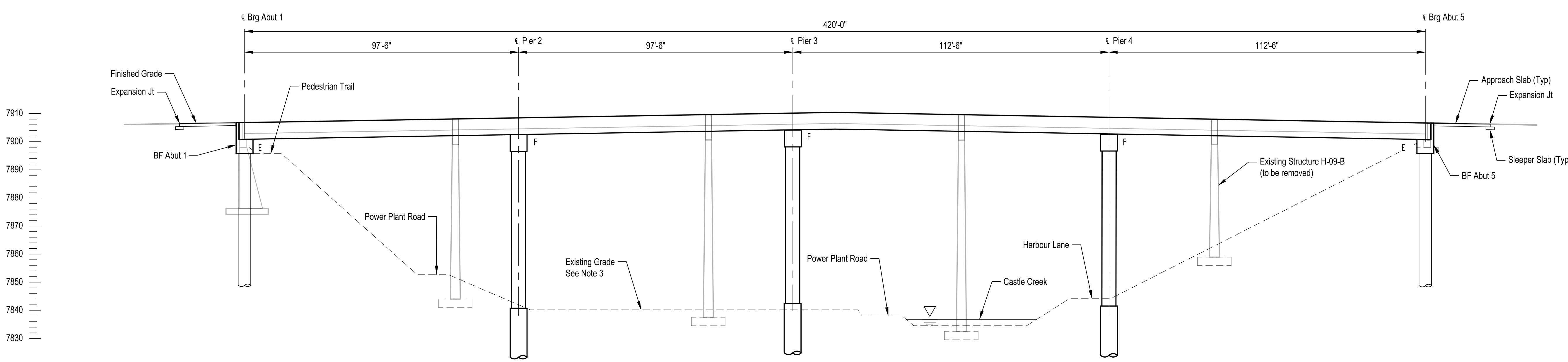
Appendix J
Conceptual Three-lane Bridge Replacement Plans



PLAN
SCALE: 1" = 40'

NOTES

1. Existing SH82 alignment shown.
2. Existing utility locations are approximate.
3. Existing grade is approximate.



ELEVATION
SCALE: 1" = 40'

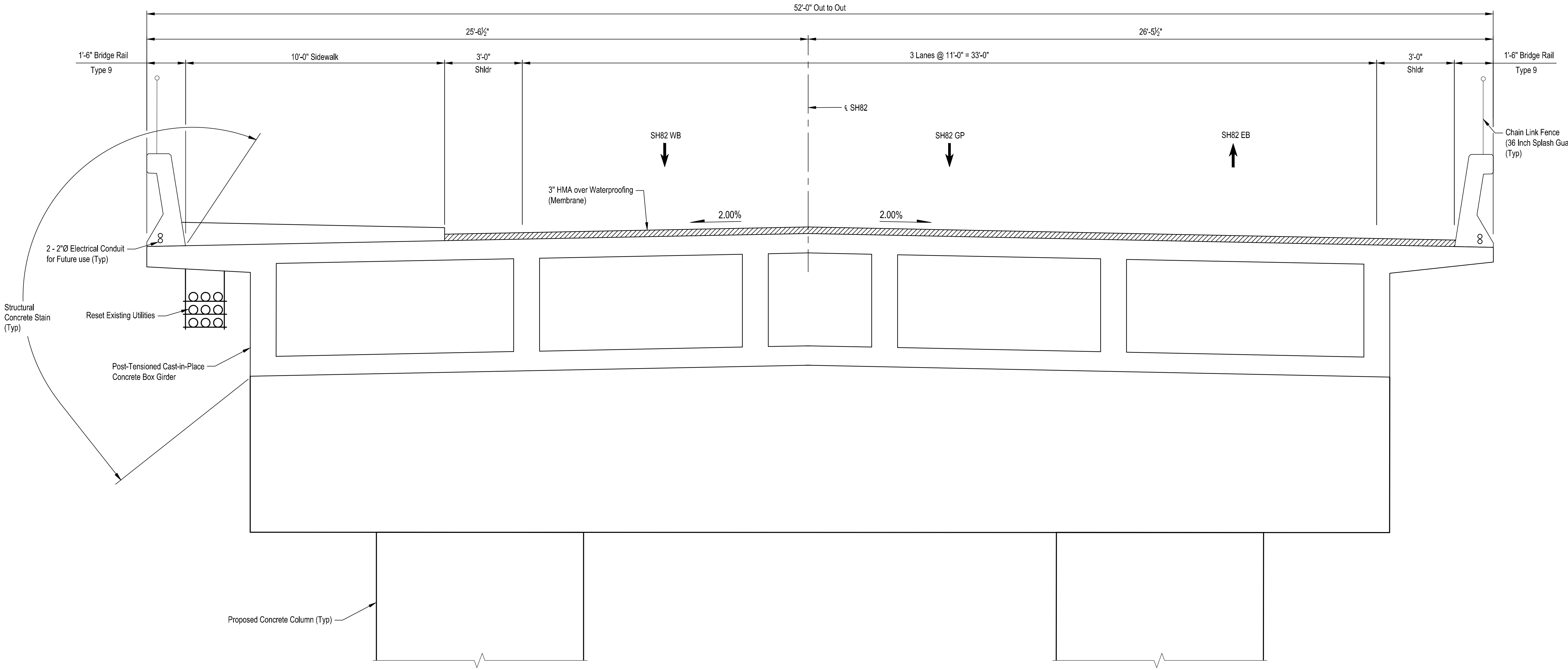
All seals for this set of drawings are applied to the cover page(s)	Print Date: $SDATE$$	
	File Name: $SFILE$$	
	Horiz. Scale: AS NOTED	
	Jacobs	

Sheet Revisions

Date	Comments	Init.

As Constructed	SH82 over Castle Creek Bridge REPLACEMENT - 3 LANE ALTERNATIVE GENERAL LAYOUT		**Project No./Code**
No Revisions:			2023-218
Revised:	Designer: S. SOWAL	Structure Numbers: H-09-B	
	Detailer: A. PRICE		
Void:	Sheet Subset: BRIDGE	Subset Sheets: B30 of B34	Sheet Number 13

$SPLOT_INFO$$



TYPICAL SECTION

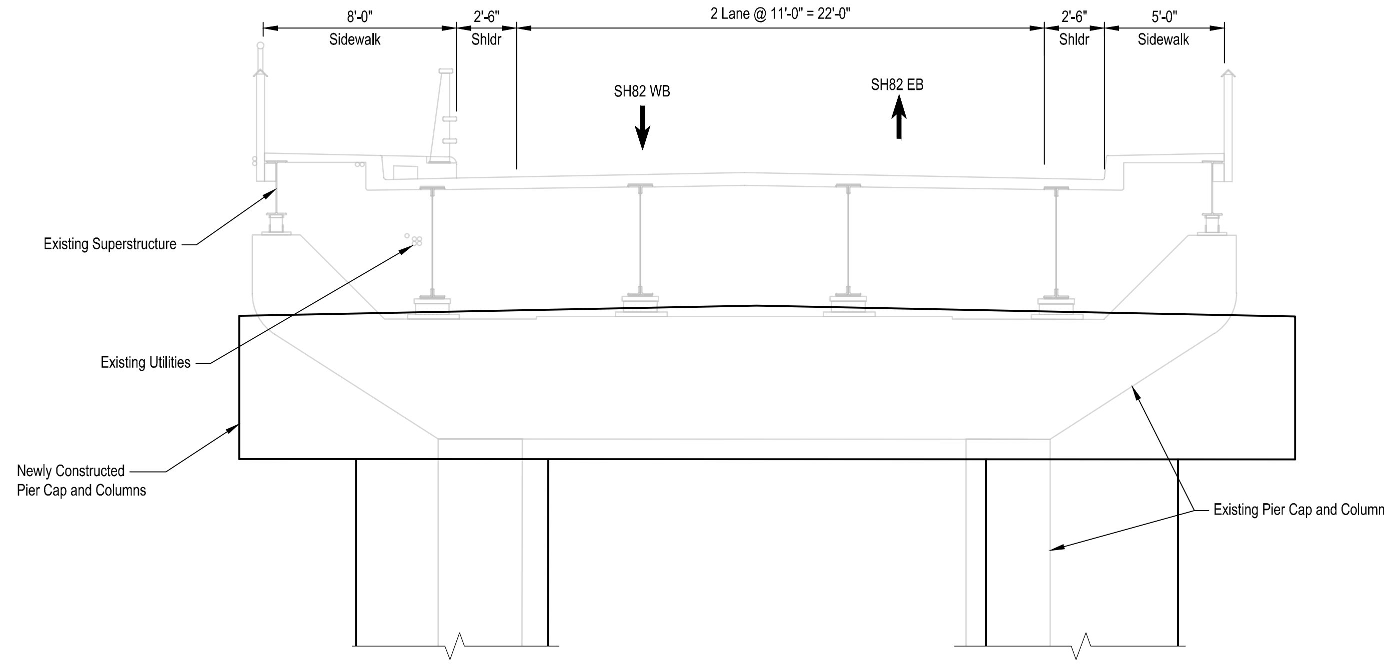
SCALE: 1/4" = 1'-0"
(Looking East, at Pier)

All seals for this set of drawings are applied to the cover page(s)	Print Date: $$DATE$$	As Constructed	SH82 over Castle Creek Bridge		Project No./Code
	File Name: $$FILE$$	No Revisions:	REPLACEMENT - 3 LANE ALTERNATIVE		2023-218
	Horiz. Scale: AS NOTED	Revised:	TYPICAL SECTION		
	Jacobs	Void:	Designer: S. SOWAL	Structure Numbers: H-09-B	
			Detailer: A. PRICE		Sheet Number 15
			Sheet Subset: BRIDGE	Subset Sheets: B31 of B34	

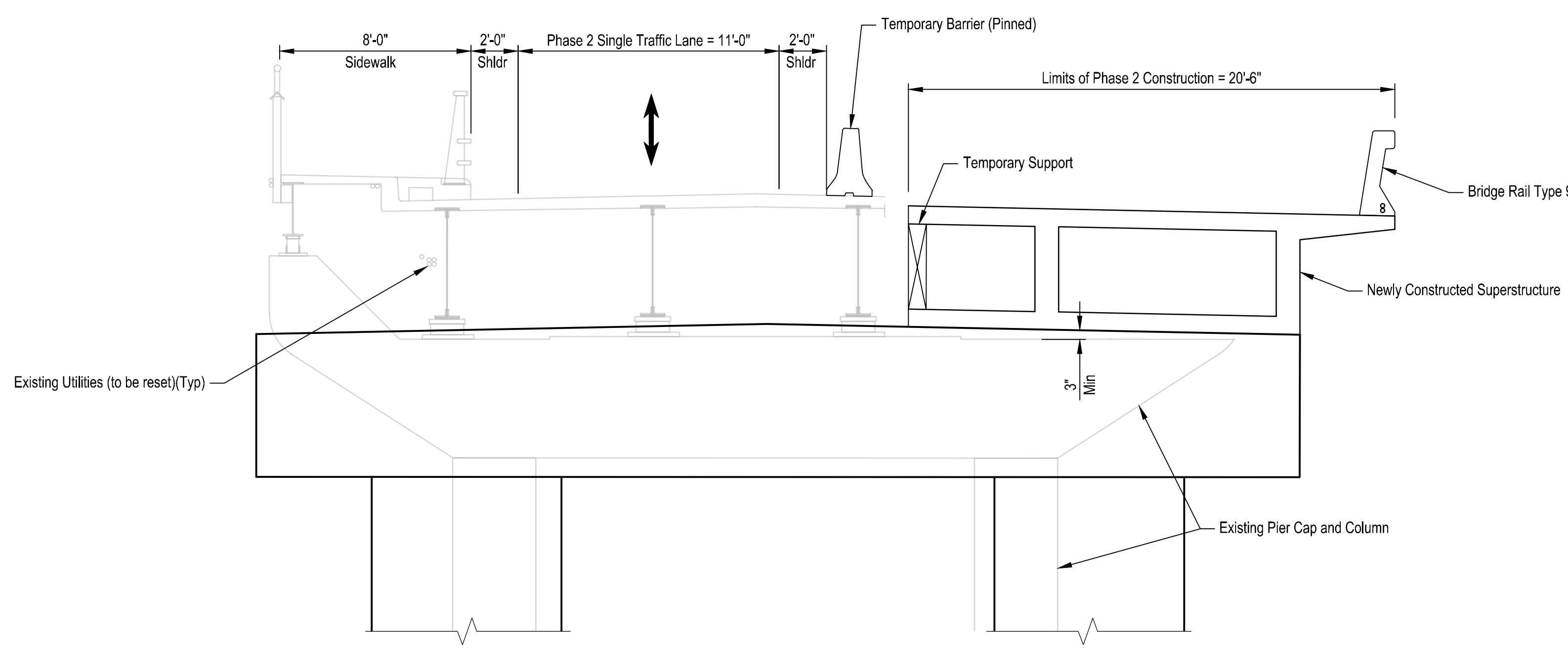
Sheet Revisions

Date	Comments	Init.

$$PLOT_INFO$$



PHASE 1 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)



PHASE 2 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)

PHASE 1:

- Construct new piers under the existing superstructure.
- Traffic to remain on existing structure during this phase of construction.

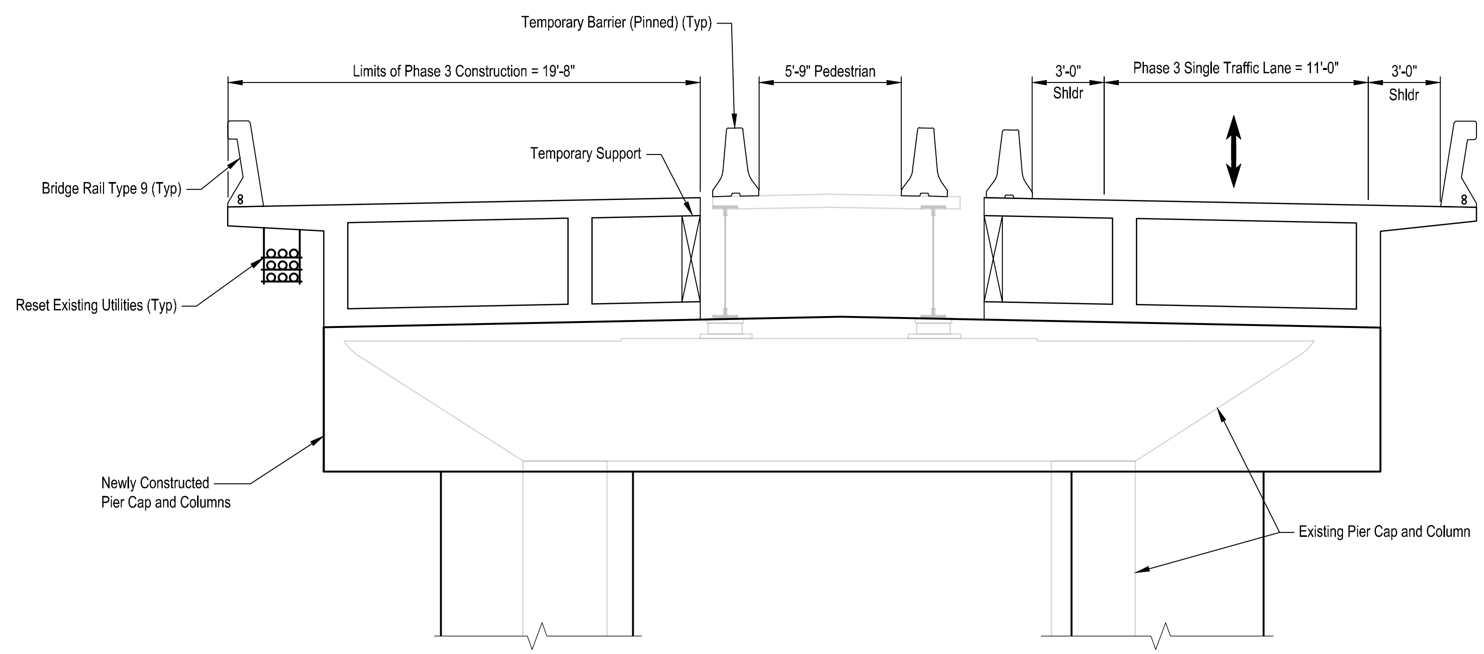
PHASE 2:

- Install Temporary Barriers (pinned to existing bridge deck).
- Demolish southern portion of existing bridge. Remove exterior girder bearing seats from piers and abutments.
- Build new section of bridge at the southern edge.
- EB and WB traffic shall remain on the existing bridge. A single lane shall be provided.
- Sidewalk shall remain on the existing structure.

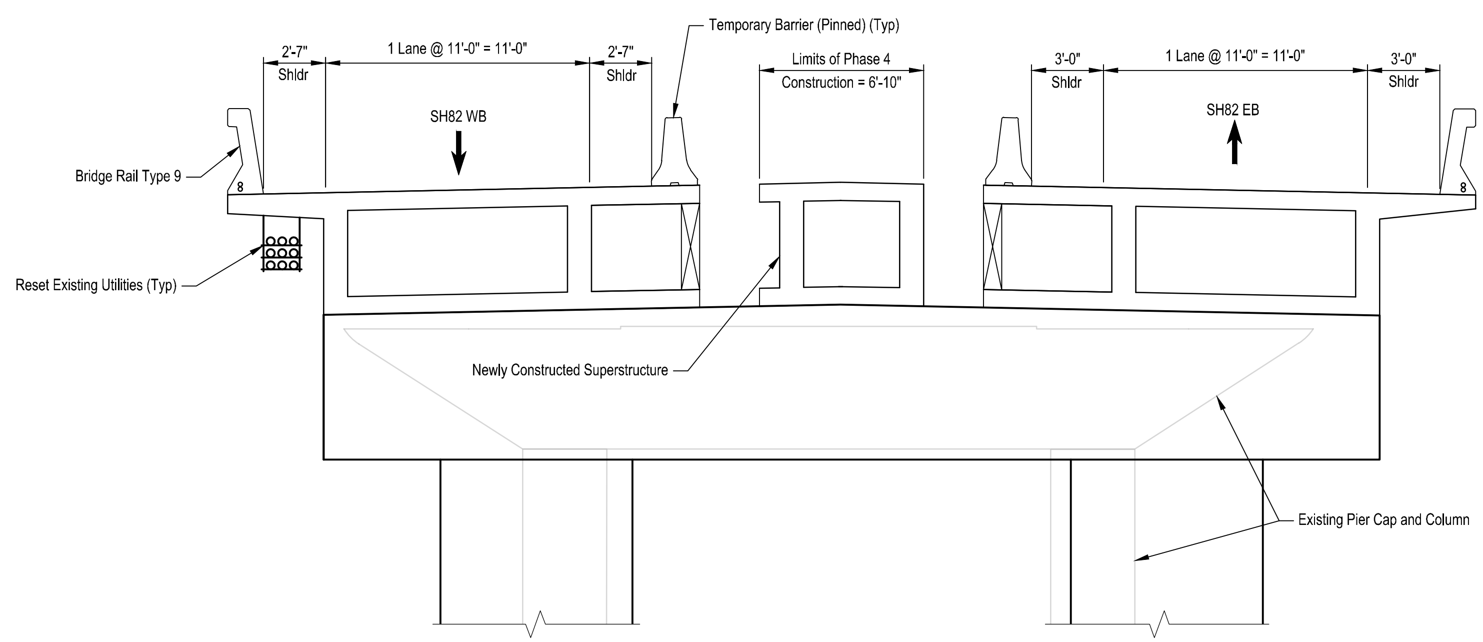
All seals for this set of drawings are applied to the cover page(s)	Print Date: $$DATE$$ File Name: $$FILES$$ Horiz. Scale: AS NOTED Jacobs	**Sheet Revisions** Date / Comments / Init.	**As Constructed** No Revisions: Revised: Void:	SH82 over Castle Creek Bridge REPLACEMENT - 3 LANE ALTERNATIVE CONSTRUCTION PHASING (1 OF 3) Designer: S. SOWAL Detailer: A. PRICE Sheet Subset: BRIDGE Structure Numbers: H-09-B Subset Sheets: B32 of B34

Project No./Code
2023-218
Sheet Number 15

$$PLOT_INFO$$



PHASE 3 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)



PHASE 4 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)

PHASE 3:

- Install Temporary Barriers (pinned to existing bridge and pinned to newly constructed deck).
- Demolish northern portion of existing bridge. Remove exterior girder bearing seats from piers and abutments.
- Build new section of bridge at northern edge.
- Reset existing utilities in hanger along the northern overhang.
- EB and WB traffic shall move to the southern portion of the newly constructed bridge. A single lane shall be provided.
- Pedestrian access shall be provided on the remaining portion of the existing bridge.

PHASE 4:

- Install Temporary Barriers (pinned to new deck).
- Demolish remaining portion of existing bridge.
- Build new section of bridge.
- EB and WB traffic shall be moved to newly constructed bridge segments.
- Pedestrian access is rerouted to under the bridge, along existing trail.

$$PLOT_INFO$$

All seals for this set of drawings are applied to the cover page(s)

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Jacobs

Sheet Revisions

Date	Comments	Init.

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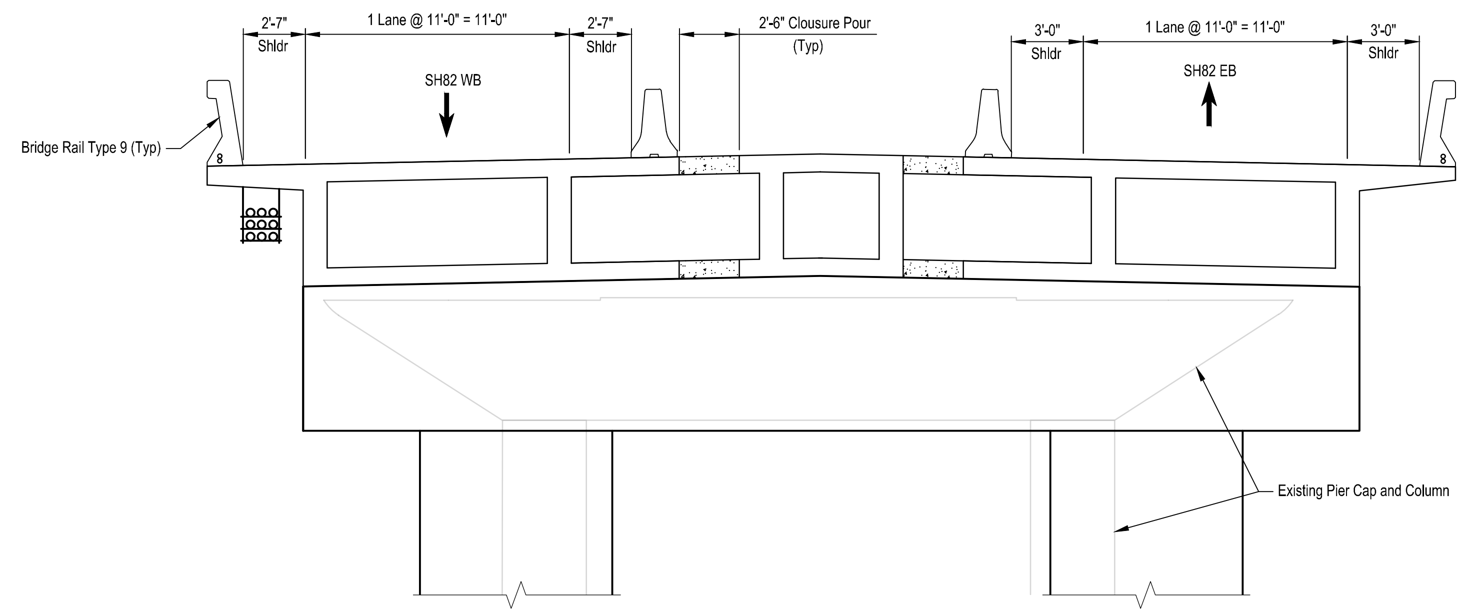
SH82 over Castle Creek Bridge
REPLACEMENT - 3 LANE ALTERNATIVE
CONSTRUCTION PHASING (2 OF 3)

Designer: S. SOWAL
Detailer: A. PRICE
Sheet Subset: BRIDGE
Structure Numbers: H-09-B
Subset Sheets: B33 of B34

Project No./Code
2023-218

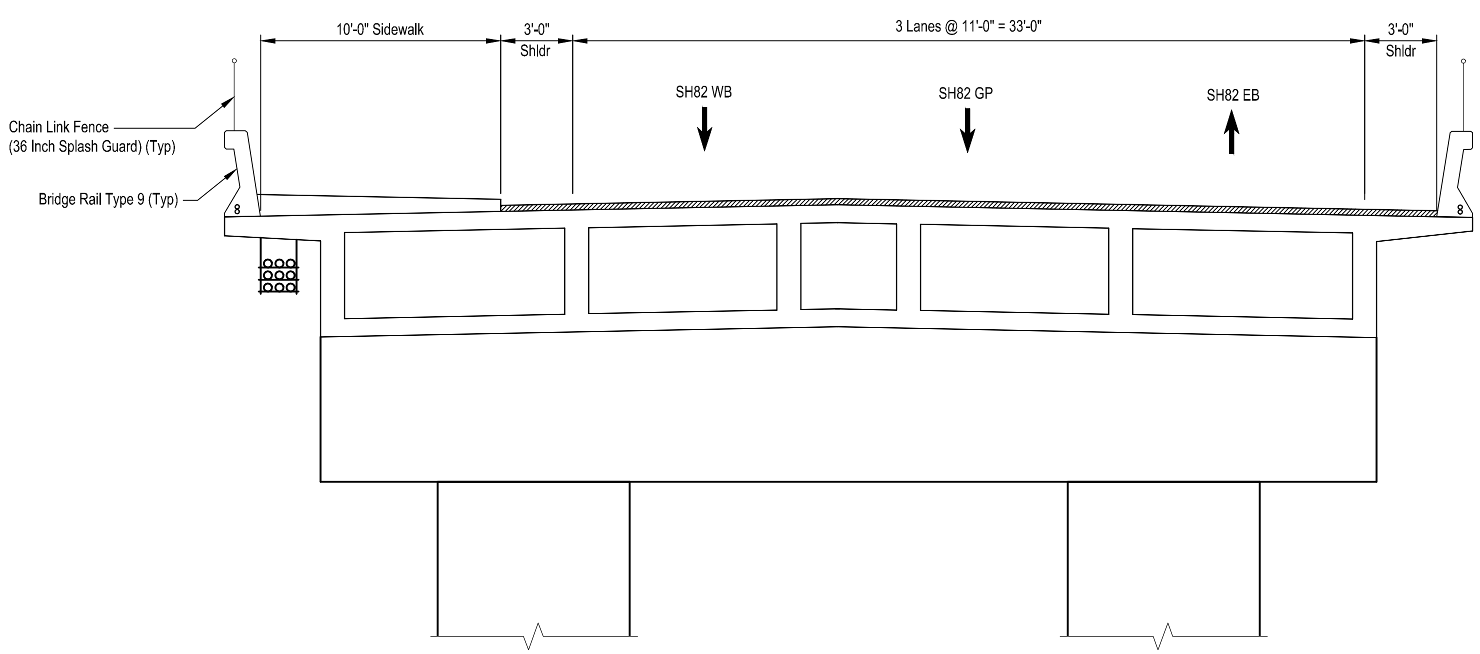
Sheet Number 16



PHASE 5 SECTION AT BRIDGE
SCALE: 1/8" = 1'-0"
(Looking East)

PHASE 5:

- Join new bridge segments with closure pour.



FINAL CONFIGURATION
SCALE: 1/8" = 1'-0"
(Looking East)

FINAL CONFIGURATION:

- Remove temporary Barriers.
- Construct sidewalk.
- Install HMA over Waterproofing (Membrane).
- Install Chain Link Fence (36 inch Splash Guard).
- Remove existing piers and abutments (all existing substructure).

Appendix K
Overall Project Cost Matrix – Bridge Rehabilitation and Replacement Options

Castle Creek Bridge
Overall Project Costs
Rehabilitation and Replacement

		Bridge Rehabilitation (functionally obsolete)	Two-Lane Bridge		Three-Lane Bridge					
			Replace		Centered		Fastest		Shifted	
			CIP Concrete	Steel	CIP Concrete	Steel	CIP Concrete	Steel	CIP Concrete	Steel
(A)	**Bridge Construction Items**	**$ 5,900,000**	**$ 9,500,000**	**$ 10,000,000**	**$ 10,000,000**	**$ 10,500,000**	**$ 11,100,000**	**$ 11,700,000**	**$ 10,000,000**	**$ 10,500,000**
	Unlisted Construction Items									
	Mobilization (15%)	$ 885,000	$ 1,425,000	$ 1,500,000	$ 1,500,000	$ 1,575,000	$ 1,665,000	$ 1,755,000	$ 1,500,000	$ 1,575,000
	Removal of Existing CC Bridge	$ -	$ 4,000,000	$ 4,000,000	$ 4,000,000	$ 4,000,000	$ 4,000,000	$ 4,000,000	$ 4,000,000	$ 4,000,000
	Utilities (relocation of City fiber)	$ 15,000	$ 15,000	$ 15,000	$ 15,000	$ 15,000	$ 15,000	$ 15,000	$ 15,000	$ 15,000
	Roadway Approaches/Improvements	$ 500,000	$ 2,000,000	$ 2,000,000	$ 3,500,000	$ 3,500,000	$ 3,500,000	$ 3,500,000	$ 4,500,000	$ 4,500,000
	Temporary Detour Construction (across Marolt-Thomas)	$ 13,000,000	$ 13,000,000	$ 13,000,000	$ 13,000,000	$ 13,000,000	$ 13,000,000	$ 13,000,000	$ -	$ -
	Traffic Control & Transit/Bus Priority	$ 3,650,000	$ 7,300,000	$ 7,300,000	$ 7,300,000	$ 7,300,000	$ 5,475,000	$ 5,475,000	$ 7,300,000	$ 7,300,000
	Subtotal Unlisted Construction Items	$ 18,050,000	$ 27,740,000	$ 27,815,000	$ 29,315,000	$ 29,390,000	$ 27,655,000	$ 27,745,000	$ 17,315,000	$ 17,390,000
	Other Contingency Items (20%)	$ 3,610,000	$ 5,548,000	$ 5,563,000	$ 5,863,000	$ 5,878,000	$ 5,531,000	$ 5,549,000	$ 3,463,000	$ 3,478,000
(B)	**Total Unlisted Construction Items**	**$ 21,660,000**	**$ 33,288,000**	**$ 33,378,000**	**$ 35,178,000**	**$ 35,268,000**	**$ 33,186,000**	**$ 33,294,000**	**$ 20,778,000**	**$ 20,868,000**
(C)	**Total of Construction Items Cost (A + B)**	**$ 27,560,000**	**$ 42,788,000**	**$ 43,378,000**	**$ 45,178,000**	**$ 45,768,000**	**$ 44,286,000**	**$ 44,994,000**	**$ 30,778,000**	**$ 31,368,000**
(D)	**NEPA**	$ 750,000	$ 2,000,000	$ 2,000,000	$ 3,000,000	$ 3,000,000	$ 3,000,000	$ 3,000,000	$ 3,000,000	$ 3,000,000
(E)	**Engineering Design - Rehab 10%/Replace 15% of (C)**	**$ 2,756,000**	**$ 6,418,200**	**$ 6,506,700**	**$ 6,776,700**	**$ 6,865,200**	**$ 6,642,900**	**$ 6,749,100**	**$ 4,616,700**	**$ 4,705,200**
(F)	**ROW and TCEs**	$ 4,500,000	$ 4,500,000	$ 4,500,000	$ 4,500,000	$ 4,500,000	$ 15,092,000	$ 15,092,000	$ 21,134,000	$ 21,134,000
(G)	**Public Involvement During Construction**	**$ 876,000**	**$ 1,752,000**	**$ 1,752,000**	**$ 1,752,000**	**$ 1,752,000**	**$ 1,314,000**	**$ 1,314,000**	**$ 1,752,000**	**$ 1,752,000**
(H)	**CE&I - 26% of (C)**	**$ 7,165,600**	**$ 11,124,880**	**$ 11,278,280**	**$ 11,746,280**	**$ 11,899,680**	**$ 11,514,360**	**$ 11,698,440**	**$ 8,002,280**	**$ 8,155,680**
(I)	**Overall Project Cost (2024)(C+D+E+F+G+H)**	**$ 43,607,600**	**$ 68,583,080**	**$ 69,414,980**	**$ 72,952,980**	**$ 73,784,880**	**$ 81,849,260**	**$ 82,847,540**	**$ 69,282,980**	**$ 70,114,880**
	Overall Project Cost Inflated to 2028, (I) - inflated 4% yoy	**$ 51,014,724**	**$ 80,232,503**	**$ 81,205,709**	**$ 85,344,668**	**$ 86,317,873**	**$ 95,752,057**	**$ 96,919,904**	**$ 81,051,287**	**$ 82,024,493**

Castle Creek Bridge
Overall Project Costs
Rehabilitation and Replacement

<u>Assumptions:</u>

1) Utility Relocations

Removal & replacement of existing conduits are included in bridge construction cost.

Relocation of City fiber (~700 lf) included in unlisted items.

Relocation of Comcast, Lumen/Ting fiber lines will be responsibility of utility owner - not included in project c

2) ROW & TCEs (Right-of-Way and Temp. Construction Easements)

ROW is estimated at $8,000/SF.

3-Lane Faster = 574 SF (bridge approach)	*$*	*4,592,000*
3-Lane Shifted = 673 SF (bridge approach)	*$*	*5,384,000*

TCE costs are estimated at $1,500/SF.

Rehabilitation - assume TCE 10' south side of bridge 300 LF	*$*	*4,500,000*
Bridge Reconstruction - assumes TCE on 2 properties		
2-Lane Replace - (10' South side of bridge 300 LF)	*$*	*4,500,000*
3-Lane Centered - (10' South side of bridge 300 LF)	*$*	*4,500,000*
3-Lane Faster - (20' South side of bridge 350 LF)	*$*	*10,500,000*
3-Lane Shifted - (30' South side of bridge 350 LF)*	*$*	*15,750,000*

**Does not include ROW costs for 3-Lane roadway realignment*

Inbound Detour assumes it is in Main St. easement east of Castle Creek

3) Detour Options & Traffic Control (TC)

Outbound CCB lane w/Inbound detour across the Marolt-Thomas - $13 million

Traffic Control and Transit Priority estimated at $5K/day

3-Lane Shifted bridge replacement does not need a detour

Traffic Control estimated at $5K per Day - includes establishing a priority for buses

4) Construction Duration

Bridge Rehabilitation ~ 2-years

2-Lane Replace ~ 4-years

3-Lane Centered ~ 4-years

3-Lane Faster ~ 3-years

3-Lane Shifted ~ 4-years

Public Involvment estimated at $1,200/day for construction duration.

5) CE&I (Construction Engineering & Indirects)

Current value for CDOT construction projects is 26%

<u>Options:</u>

Rehabilitation - Rehabilitate the existing bridge in-place - reamains Functionally Obsoltete

2-Lane Replace - Replace the existing bridge in kind (CIP Concrete or Steel)

3-Lane Centered - Replace the existing bridge with 3-Lanes centered on exsiting bridge (CIP Concrete or Steel)

3-Lane Faster - Replace the existing bridge with 3-Lanes slightly shifted but faster construction timeframe (CIP Concrete or Steel)

3-Lane Shifted - Replace the existing bridge with 3-Lanes shifted south to facilitate 2-way traffic during construction (CIP Concrete or Steel)