Michigan Department of Transportation
Accelerated Innovation Deployment (AID) Grant
Final Report – May 22, 2015

The US-131 NB & SB bridges over 3 Mile Road superstructure replacements utilizing the Slide-in Bridge Construction (SIBC) method received an Accelerated Innovation Deployment (AID) grant in 2014. The project utilized the Construction Manager/General Contractor (CMGC) process for design and delivery and was coordinated with a road crush & shape project along the same corridor. Construction began on April 23, 2014. The project was substantially completed on October 17, 2014.

The superstructures for the US-131 NB & SB bridges over 3 Mile Road were both rated in poor condition prior to construction. Superstructure replacements were the chosen scope of work to bring these bridges up to good condition and widen to meet current standards.

The SIBC method, which is included in the Every Day Counts (EDC) initiative, was selected to reduce traffic disruption and increase safety. There are documented benefits of building the new bridges off alignment and sliding them in place including: accelerated construction time frame, reduced traffic disruption, increased safety for motorists and construction workers, greater quality and constructability and reduced environmental impacts from vehicle and construction equipment emissions.

The performance goals for the innovation were to reduce traffic disruption and have a safe working environment for the construction workers and the motoring public.

These two bridges was Michigan's first project to utilize Slide-in Bridge Construction methods/technology. The CMGC process was utilized which allowed the Michigan Department of Transportation (Michigan DOT) to gain the contractor's input and perspective on this construction method. This proved useful in developing the details associated with the temporary works and slide since we were able to gain insight related to their means and methods. The CMGC process also allowed us to work together on a schedule and the details associated with the required closure to minimize traffic disruptions.

Using the SIBC method on this project was expected to greatly improve traffic flow on this major north/south route in Michigan, known for high weekend volumes average approximately 40% more than the average weekday traffic.

The improved traffic flow was evident through the analysis done during design which analyzed user delays and costs associated with different maintenance of traffic schemes. The maintenance of traffic schemes, cross-overs and part-width construction using single lane closures, were analyzed and compared to the SIBC method. Below is the original table included in the grant application which highlights the expected user delay hours and associated costs analyzed during the design phase.

	Cross-Over	Part-Width	Bridge Slide NB	Bridge Slide SB
User Delay	45,760 hours	87,600 hours	1,333 hours	1,491 hours
***User Delay Costs**	$1,620,000	$2,560,000	$126,660	$141,829

*2013 user delays, commercial and passenger vehicles combined

Using the SIBC method on this project performed better than expected. The actual measured user delay costs using actual field measurements is shown in the table below along with the calculated costs. This was based on implementing a 5 day detour for both the northbound and southbound bridges while the bridges were slid into place.

	Calculated Bridge Slide NB	Calculated Bridge Slide SB	Actual Bridge Slide NB	Actual Bridge Slide SB
User Delay	1,333 hours	1,491 hours	1,242 hours	1,267 hours
***User Delay Costs**	$126,660	$141,829	$119,111	$121,526

*2013 user delays, commercial and passenger vehicles combined

The estimated additional Construction cost associated with utilizing the SIBC methods and technology was $1.76M for both structures ($880,000 per structure). When you evaluate the additional construction costs compared to the user delay costs for part-width construction there is a significant savings to the motoring public. You cannot accurately compare the SIBC option to the cross-over option since there would have been additional road improvement costs associated with building the cross-overs and any associated widening. Based upon this evaluation the SIBC method was the most cost effective solution.

Increased quality of construction is also expected since, for example, the longitudinal construction joint which would be present with part-width construction would be eliminated.

Safety was another key goal of this project. Two safety components were expected to be analyzed and the results reported. The first being worker safety. There were no reported worksite accidents during the project. Worker safety was exceptional, in large part due to not being adjacent to active traffic for the majority of the project.

The second key goal was to reduce work zone crashes. The full crash analysis and results can be found in Attachment A. Attachment A contains the original baseline for past crashes, which was submitted with the grant application. This baseline analyzed two different past project work zones. The two different work zones analyzed included a work zone using cross-overs and another using single lane closures. Both of these work zones were within the same corridor and very near the existing project location. The project utilized two, five day, freeway closures and traffic was maintained using a detour route. As a result of this, it was decided to also include a baseline for the detour route. It proved to be very difficult to properly compare the different work zones due to the durations each were in place. Attachment A further explains the analysis and results.

The construction quality appeared to be very similar to a typical superstructure project. One difference noted was less deck cracking, most likely due to the fact the bridge was constructed without using part-width construction and no adjacent traffic. The Michigan DOT will continue to monitor the bridge and any deficiencies.

As part of this project and another bridge slide project (M-50 over I-96), a lessons learned document was created for consideration when using SIBC. This document can be found as Attachment B.

The overall project was deemed a success and the Michigan DOT has now added this innovative technology to its toolbox. Following is a comment that was sent to MDOT from an appreciative user, *"I am very impressed and pleased with the 131 bridge project in Mecosta County. Despite all of the work, my drive was never impacted. We drive up north every Friday and return the following Sunday. Last week we came home over the old southbound structure and upon returning this past Sunday, we crossed the brand new bridge! I've never experienced a bridge replacement project without a detour!! WELL DONE!!"*

US-131 over 3 Mile Road
Average Crashes Comparison

Freeway Baseline
In order to be able to effectively monitor work zone crashes during construction, a baseline for past crashes is needed. To determine this, three years of crash data (2010-2012) during possible construction dates (April 1 – November 15) was analyzed. The analysis revealed the following monthly average crashes by type. Crash types not shown in the table below had no reported crashes during the three year period.

CRASH TYPE	NUMBER OF CRASHES			MONTHLY AVERAGE
	2010	2011	2012	
Minor Injuries	0	0	8	0.33
Serious Inj / Fatalities	0	1	0	0.04
Misc Single Vehicle	3	0	3	0.25
Overturn	1	0	0	0.04
Parking	1	0	0	0.04
Fixed Object	2	5	4	0.46
Other Object	1	0	2	0.13
Rear End Straight	0	0	1	0.04
Side Swipe Same	0	0	1	0.04
Dual Left Turn	1	0	0	0.04
Total Crashes	9	6	19	
Average Number of Crashes per Construction Season				11.3

Single Lane Closure
For comparison purposes, crashes that occurred during the previous construction project on US-131 over 3 Mile Road were analyzed. This project utilized single lane closures and was completed between July 23, 2007 and September 29, 2007. As the exact dates of lane closures is unknown, it was assumed that lane closures were in place on both bounds from August 1 – September 30, 2007. Furthermore, as there were significant queues observed during construction, crashes were pulled for the stretch of roadway 2 miles in advance of the structure to a point approximately 0.25 mile beyond the structure (in the direction of traffic). The analysis revealed the following monthly average crashes by type. Crash types not shown in the table below had no reported crashes during the analysis period. It should also be noted, that a large diversion rate was observed during the lane closures. Vehicles were observed using the exits before the lane closure and using Federal Road as an alternate route. In an effort to ease congestion, this route was actually signed as an alternate route, which could have led to a decrease in the crash rates.

ATTACHMENT A

CRASH TYPE	NUMBER OF CRASHES	MONTHLY AVERAGE
Rear End Straight	3	1.5
Overturn	1	0.5
Total Crashes	4	

Crossovers (Maintaining 2 lanes one direction, 1 lane the other direction)

For comparison purposes, crashes that occurred during a previous construction project on US-131 between Pierson Road and Tamarack Road in Montcalm County were analyzed. This project utilized crossovers and occurred from April 16 – October 20, 2012. The analysis revealed the following monthly average crashes by type. Crash types not shown in the table below had no reported crashes during the analysis period.

CRASH TYPE	NUMBER OF CRASHES	MONTHLY AVERAGE
Minor Injuries	9	1.5
Rear End Straight	13	2.2
Fixed Object	8	1.3
Side Swipe Same	4	0.7
Overturn	3	0.5
Misc Single Vehicle	2	0.3
Other Object	1	0.2
Backing	1	0.2
Total Crashes	31	

Detour Route Baseline

In order to be able to effectively monitor work zone crashes during construction, a baseline for past crashes is needed. To determine this, three years of crash data (2010-2012) during possible construction dates (May 1 – November 1) was analyzed. The analysis revealed the following monthly average crash experience. Crash types not shown in the table below had no reported crashes during the three year period. It should be noted this baseline was not included in the original application. Due to the reporting requirements, it was determined additional information should be included in the analysis to accurately report the crashes and overall safety.

ATTACHMENT A

CRASH TYPE	NUMBER OF CRASHES			MONTHLY AVERAGE
	2011	2012	2013	
Minor Injuries	9	0	6	0.83
Serious Inj / Fatalities	0	0	1	0.06
Misc Single Vehicle	2	1	1	0.22
Overturn	0	0	1	0.06
Pedestrian	0	1	1	0.11
Fixed Object	1	3	4	0.44
Other Object	0	1	0	0.06
Head-on	1	0	1	0.11
Angle Straight	1	1	0	0.11
Rear End Straight	1	2	2	0.28
Angle Turn	1	1	0	0.11
Rear End Left Turn	0	1	1	0.11
Rear End Drive	1	0	0	0.06
Side Swipe Opposite	1	0	0	0.06
Total Crashes	18	11	18	
Average Number of Crashes per Construction Season				15.7

Project Construction - Freeway Closure using Detour Route

There were a total of 4 crashes that occurred during the project construction. These crashes occurred on the detour route. There was one angle straight, one backing, one rear-end left turn and one rear-end straight and none were considered serious injury crashes. The types of crashes that occurred could have been expected to occur along the detour route at any time, regardless of the additional detour traffic. They can all be attributed to driver error and inattentiveness.

Utilizing SIBC allowed MDOT to implement two 5-day duration detours for each bound of US-131, ultimately reducing user delays and minimize impacts to the motoring public during peak travel times.

After further consideration, MDOT feels the best way to evaluate the motorist safety is to compare the detour route crashes based upon vehicle trips. Because the traffic increased on the detour route substantially during the freeway closure, evaluating the crashes based upon vehicle trips should produce the best “apples to apples” comparison.

An analysis of the rate of crashes was performed for the different types of maintaining traffic schemes along with the baseline (no project) for the freeway and the detour route. As the table shows, the actual crash rate for the time when the detour route was in place was lower than would be expected.

The crash rate for crossovers was slightly lower and the crash rate for a single lane closure was close in line for the baseline freeway analysis. Because the duration of the closure was much longer the vehicle trips are more which correlates to a decreased crash rate. A longer duration closure also allows traffic time to get used to a new situation and the accidents tend to decrease as drivers become more comfortable with the new situation.

ATTACHMENT A

	Total Crashes	Days of analysis period	ADT	Total ADT for evaluation period	Crash Rate per million trips
Baseline Freeway	17	229	20,000	4,580,000	3.7
Single Lane Closure	6	61	20,000	1,220,000	4.9
Crossovers	49	189	20,000	3,780,000	13.0
Detour Route base	21	184	4,500	828,000	25.4
Actual Detour route	4	10	24,500	245,000	16.3

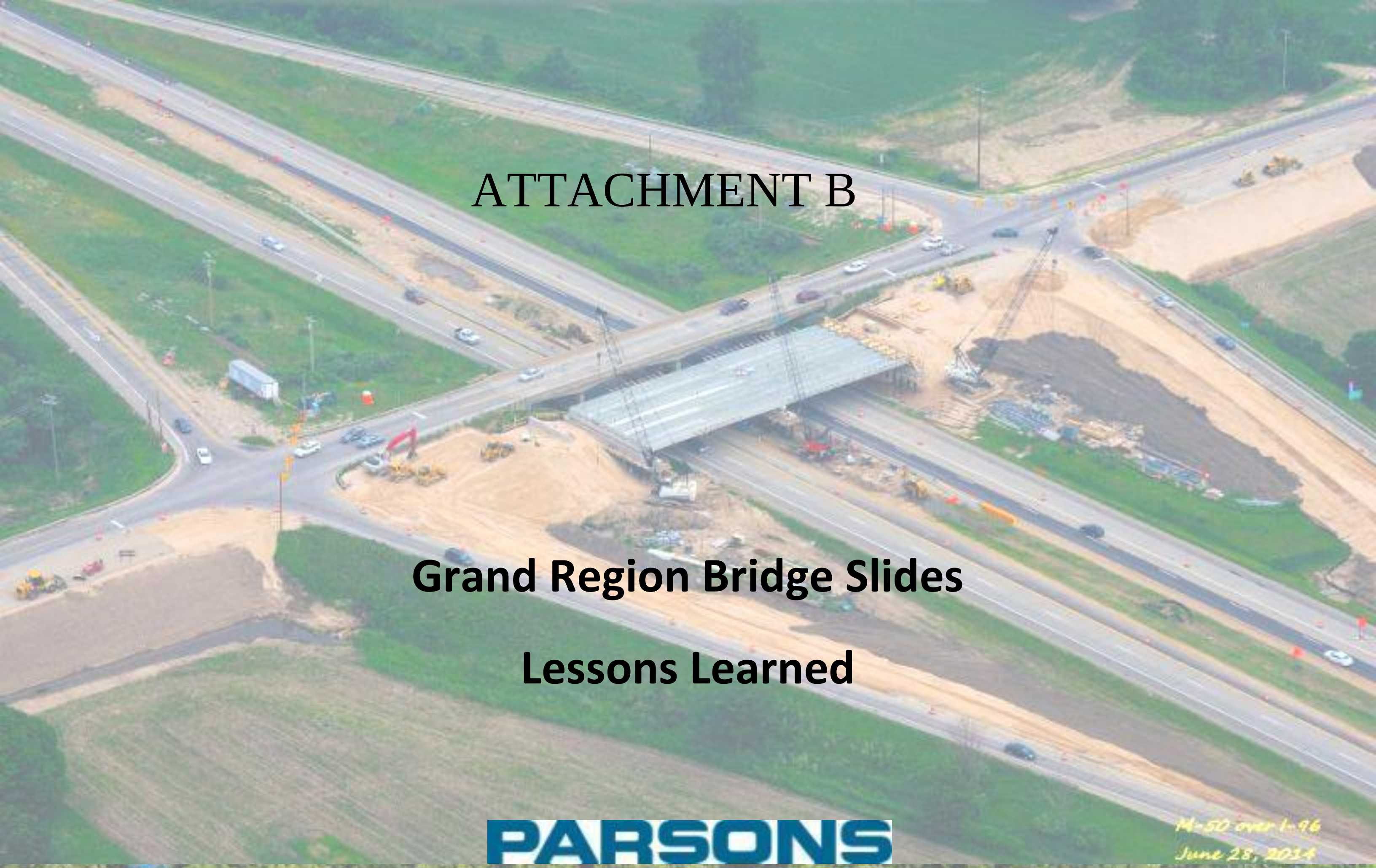
A cost comparison was also performed using the average economic cost per death, injury or crash as detailed by the National Safety Council. The costs used are outlined below:

Crash Type	**Cost**
Death	$1,500,000
Nonfatal Disabling Injury	$80,700
Property Damage Crash (including non-disabling injuries)	$9,300

These costs were used to determine a total cost for each type of project closure, as described in the tables shown above. The total costs are detailed in the table below. As shown, the actual crash costs for the project, were the lowest of all the different maintaining traffic options. The actual cost was approximately 66% of the cost of using single lane closures and approximately 10% of the cost of utilizing crossovers. Clearly, this shows that the crash costs were minimized using the SIBC methods and technology.

	Total Crash Cost		
	Single Lane Closure	Crossovers	Actual
Fatal	0	0	0
Nonfatal Disabling Injury	0	0	0
Property Damage Crash (including non-disabling injuries)	6	40	4
Total Project Crash Cost	$55,800	$372,000	$37,200

MDOT believes the overall safety of the project, both motorist and worker/job site, was increased as a result of utilizing SIBC methods and technology. Limiting the time workers were exposed to live traffic exposure and the limited duration detour was the major contributing factor for increased safety.



ATTACHMENT B

Grand Region Bridge Slides

Lessons Learned

PARSONS

For

MDOT
Michigan Department of Transportation

March 2015

1 Introduction

In 2013 the Michigan Department of Transportation, Grand Region undertook the design of two bridge slide projects:

- US-131 over 3 Mile Road, near Morley, Mecosta County
- M-50 over I-96, Lowell Township, Kent County

The two contracts were procured under a Construction Manager/General Contractor (CM/GC) procurement model. MDOT designed the bridges in-house as well as the M-50 interchange reconstruction. Parsons Transportation Group was selected to provide design services for the temporary works and slide as well as design assistance during construction. Parsons' team also provided maintenance of traffic and signal design for the M-50 project.

This paper will present a summary of the design process and the lessons learned during the construction and execution of the two bridge slides. The purpose is to present methods to improve the slide design details and methods. The development of the design, slide methodology and process are only briefly described for context.

2 US – 131 over 3 Mile Road

2.1 Project Description

US-131 over 3 Mile Road (S03 and S04 of 54013) has separate northbound and southbound structures. 3 Mile Road is a very low volume local road. The existing bridges are one span 86'-0" long by 45'-10 ½" wide structures comprised of side by side box beams on spread footing abutments.

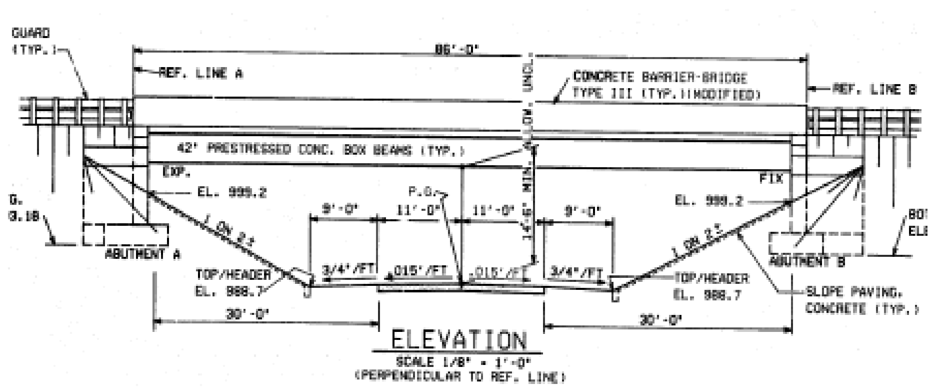


Figure 1 - Existing Bridge

The scope of work for the bridge project was a superstructure replacement. At the same time a corridor crush and shape project was taking place. The proposed superstructure was widened along with the existing substructure to provide standard shoulders. The replacement superstructure consisted of spread box beams with a dependent backwall. The existing abutment top surface was not modified but the abutments were widened to provide a wider shoulder on the structures.

Figure 2 - Extension of abutment with new wing wall

Figure 3 - Elevation view of the abutment extension

Figure 4 - Extended footing toe for overturning resistance

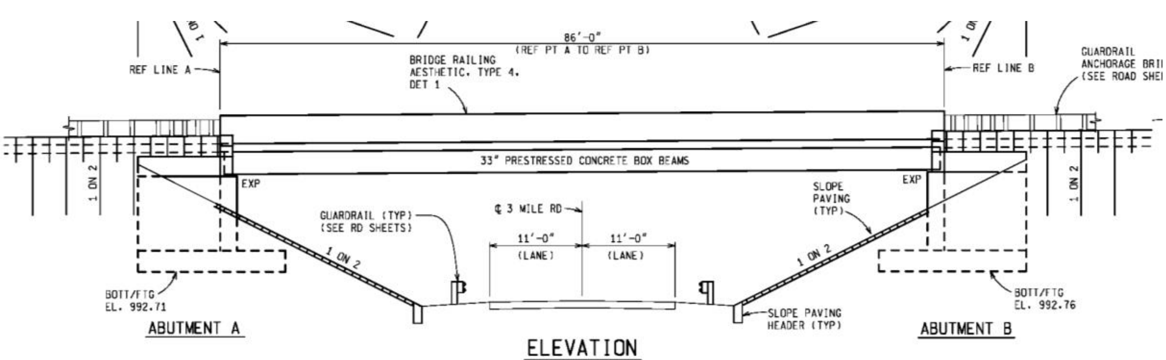


Figure 5 - Proposed Bridge

2.2 Bridge Slide Method

The new superstructure was constructed adjacent to the existing bridge on a temporary pile bent. After the superstructure was complete the existing superstructure was demolished and the new superstructure was pulled into place. After the superstructure was in place the abutments were backfilled and temporary HMA was placed. A concrete approach slab was then constructed part-width within the corridor roadway project.

The new bridge superstructure was constructed on a slide beam which had stainless steel sliding shoes, which sat on Teflon coated elastomeric bearings. The bridge was pulled into place using a hydraulic core jack with Dywidag bars connected to the slide beam. The length of the pull was 64’-9”. The superstructure unfactored dead load was approximately 1600 kips.

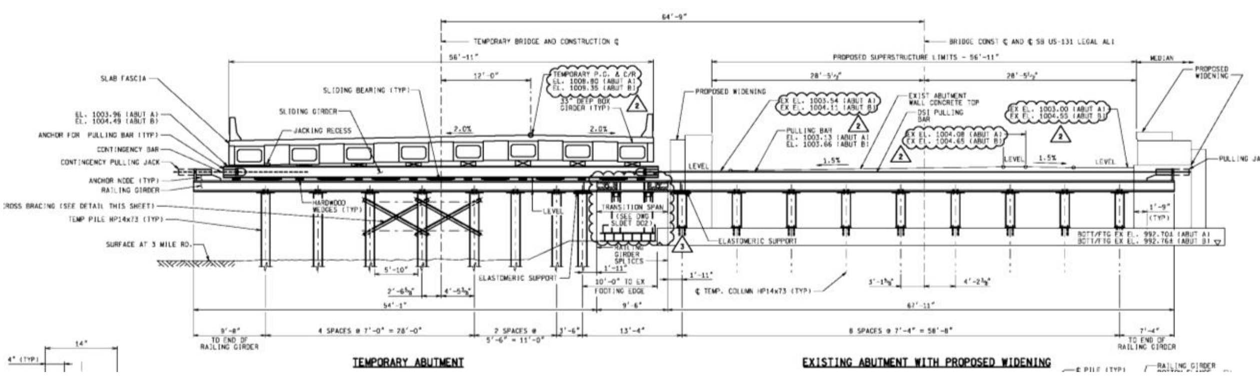


Figure 6 - Temporary/Proposed Cross Section

2.3 Lessons Learned

2.3.1 Temporary Loadings

Due to the soil conditions and the existing abutment being on a spread footing the pile driving was restricted. No pile driving could take place within 10’-0” of the existing footing. This resulted in a substantial distance to be bridged from the last temporary pile to the first temporary column supported on the abutment footing. Due to the constructability sequence this gap was bridged by a pinned transition beam. This transition beam also served as a potential rotation point for the substructure, with the temporary works on piles and final substructure on spread footings. There was some initial concern that settlement and rotation, due to the eccentric temporary loading, of the spread footing could be an issue during the slide even though the geotechnical report anticipated 0.1 inches of settlement from the added live and dead load of the new structure. No actual settlement was measured during the slide.

Lesson: The geotechnical analysis needs to have a higher degree of certainty with a more refined analysis. A traditionally conservative analysis may have unintended consequences.

The designers analyzed the dead load in the temporary alignment position but did not analyze the transition of the dead load in increments across the transition beam. When the construction engineers analyzed the loads an overstress was found in the last two temporary piles and the first column. A temporary support with a double column was placed on a spread footing to minimize the loads.

Lesson: Analyze dead load in all slide positions.

2.3.2 Jacks

For the first slide, northbound, the contractor used a single action 100 ton hollow core jack with a 2" stroke manifolded to a single pump. This jack was selected due to the specification requirement that the jack have 150% of the anticipated jacking load and the assumption of a worst case friction factor of 0.15. Due to the short stroke, single action, and long cycle time the first slide took 28 hours. The field experienced friction factor for breakout for the first slide was calculated to be approximately 0.10. For the southbound slide the contractor chose a double actuating, 60 ton hollow core jack with a 10" stroke running off two independent pumps. This resulted in the second slide taking only 6 hours. While there were other factors that lead to the decrease in slide time, some of which are in the following section, such as: more careful monitoring of the bearings so they don't roll, shift or move during the sliding operation; and learning curve of the crew and more confidence in the position checking after each jack stroke. The jack stroke, double action and cycle speed of the hydraulic pump and jacks were the biggest factor in reducing the slide time.

Lesson: Consider jack stroke and cycle time in determining the duration of the slide. The size of the jack should be carefully considered with respect to the anticipated jacking loads.

2.3.3 Temporary Bearings

Temporary bearings consisted of a 10x10x1 ¾" elastomeric bearing with a smooth PTFE surface. The bearings were designed for a 50 durometer hardness with internal steel plates. The temporary elastomeric bearings experience several major issues during the first slide:

- Shoving – this was where the slide shoe would push the elastomeric bearing along the railing girder. This was due in part to lubricant getting between the bearing and beam. In addition, the PTFE surface was not dimpled; so much of the lubricant was squeezed off the surface. This allowed the bearings to be shoved along the beam as shown in the photo below.

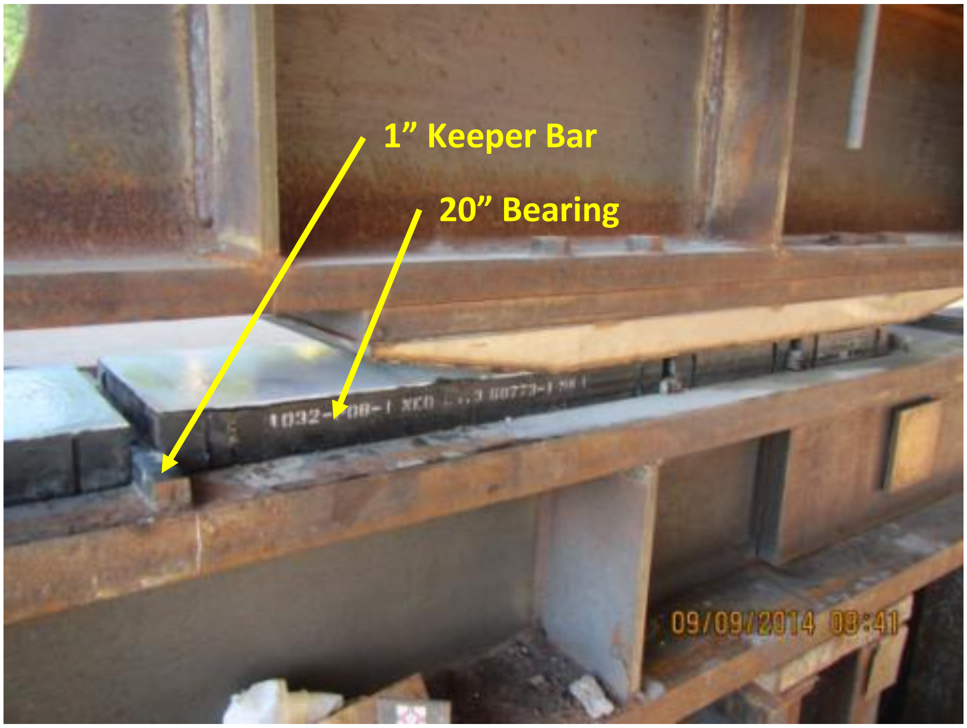
- Tipping – when the slide shoe began to load the temporary bearing the leading edge would tip up as shown in the photo below. This was caused by the bearing being compressed up to 1/8" under load, so as the tapered end of the shoe came in contact with the next bearing that was not compressed it tipped and pushed the bearing. The combined tipping and shoving action caused the bearing to ride up onto the 3/8" keeper bar.

- Expulsion – in one extreme instance the shoving and tipping combined to expel three temporary bearings out of the side of the slide shoe.

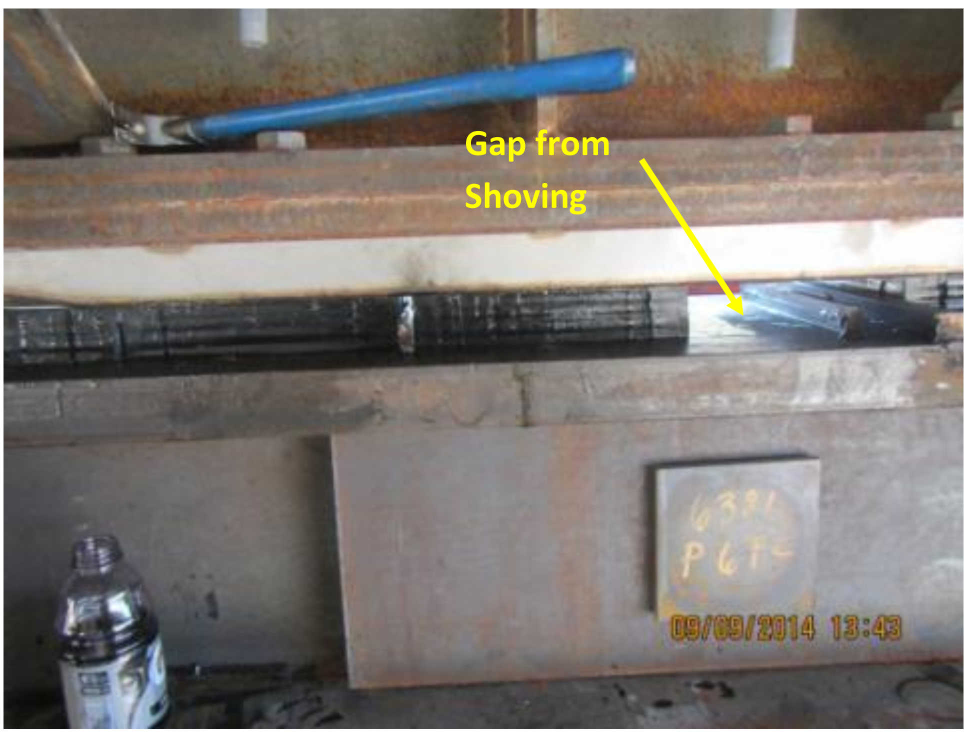
- Breaking the keeper bar welds – By reducing the friction between the bearing and the beam, more horizontal load was placed on the keeper bars and the keeper bar welds broke in some locations.

Retaining the bearings due to the shoving and tipping action took a substantial amount of manpower and effort including blocking and wedging as shown in the photo below. In addition the 10" dimension required that the contractor "leap frog" the temporary bearings, which also took a lot of manpower.

After the experience on the northbound structure the contractor increased the keeper bar size to 1" and bearing length to 20", but did not use a dimpled PTFE surface.



These modifications improved the bearing performance but some shoving and breaking of the welds still occurred.



Lessons: The issues experienced by the bearings are difficult to quantify empirically from a stress and force perspective. However, it is apparent that the bearings are resisting substantial shear forces. In addition elastomeric bearings are not traditionally designed or expected to function in this manner so the design and detailing must take into account the effects of the shear forces as well as the variances in structural steel fabrication tolerances. From our field observations, several modifications should improve the bearing performance:

- Reduce the bearing height as much as practical to reduce the dimensions of the shearing action and moment arm on resisting elements. Investigate the bearing shape factor and impacts of creep if the structure rests on the temporary bearings for an extended amount of time.
- Use at least a 70 durometer hardness to reduce the deflection of the bearing.
- Reduction in bearing heights and increase in hardness will reduce the ability of the bearings to absorb the differences in beam elevations due to normal milling tolerances, so consider modifications to the milling tolerances.
- Instead of keeper bars use two adjacent 1” pintles on the leading edge of the bearing. This will provide restrain in both longitudinal and transverse direction.
- Provide a longer temporary bearing. Note that the length of the temporary bearing should be less than ½ the clear distance between the sliding shoes. This is to allow replacement of damaged bearings during the slide.
- The angle of the leading edge of the sliding shoe should be as flat as with the gap between the bearing and leading edge of the sliding shoe double the estimated bearing deflection.

2.3.4 Lateral Drift

The roadway profile for the bridge was less than one percent grade from north to south, however, the northbound bridge began to drift north during the slide (against the slope). The drifting is likely due to slight imperfections in the flatness and/or straightness of the temporary works. There were no deviations out of specified construction tolerances in the slide specification (cross reference to the 2012 Standard Specifications for Construction section 706.03.M.4). For the northbound slide the contractor used pancake hand jacks to reposition the structure longitudinally, then steel and wood shims to keep the structure traveling within tolerances.

For the southbound structure the superstructure drifted down grade slightly. However, the contractor had developed a Hillman roller assembly on the temporary piles and a Teflon slide bar on the abutment to keep the superstructure in line. The system was constructed such that there was a 2” gap between the restraint and the slide system at each end of the bridge to prevent racking and binding. This system functioned very well.

Lesson: Provide a lateral drift restraint system. Note that it is critical that the lateral drift system be constructed such that there is a sufficient gap such that racking and binding of the structure cannot occur.

3 M-50 over I-96, Lowell Township, MI

3.1 Project Description

The existing M-50 over I-96 (S06 of 41024) was a four span concrete arch structure. The bridge was 227’ in length and 37’-5” wide carrying two lanes on spread footings.

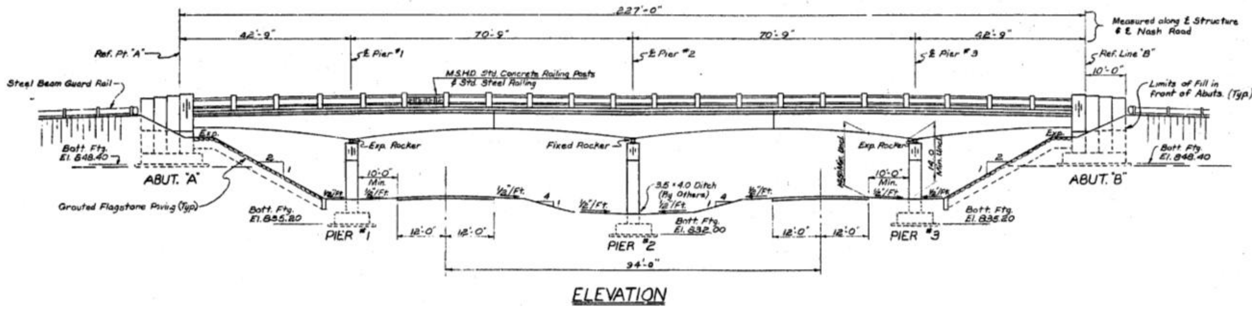


Figure 7 - Existing Bridge

The scope of work for the bridge project was a bridge replacement, with some realignment and raising of the ramp terminals to meet the revised structure grade. The proposed bridge was widened to provide four lanes and standard shoulders as well as raised to provide adequate underclearance. The replacement superstructure consisted of spread box beams with an independent backwall on a pile supported abutment with MSE wall. The center pier was founded on a spread footing.

Figure 8 - Elevation of Bridge after Slide

Two things that make this bridge slide unique are:

1. The bridge is a two span structure that will slide both spans at the same time from the temporary position to the final position.
2. The bridge was used as a temporary runaround and that ran normal traffic loading on a portion of the bridge until the existing bridge was removed and the new foundation constructed prior to the slide.

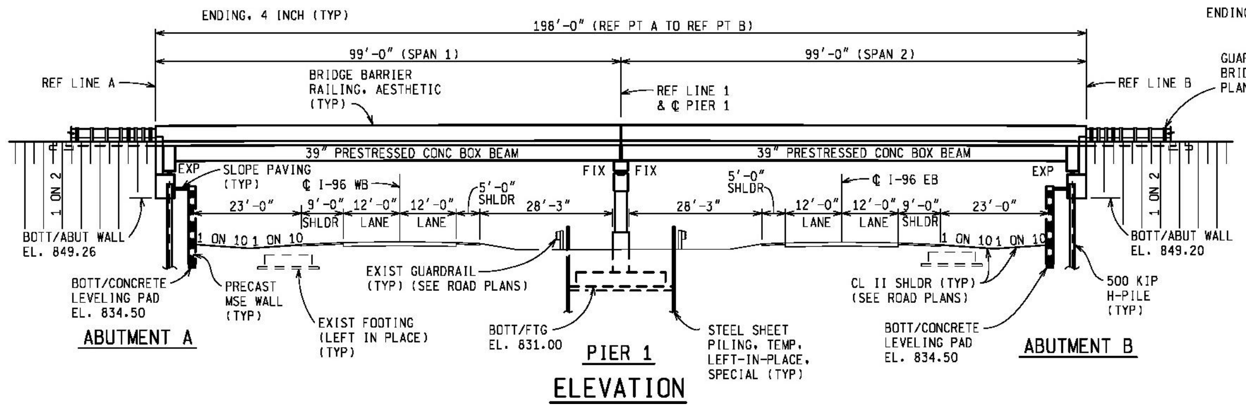


Figure 9 - Proposed Bridge

3.2 Bridge Slide Method

The new superstructure was constructed adjacent to the existing bridge on a temporary pile bent. At the abutments the new wingwall and MSE was constructed and used a part of the temporary support. A temporary runaround was constructed, including a 25' temporary approach span, and traffic was diverted to the runaround. After traffic was diverted the existing bridge was demolished and a new substructure was constructed. Once the substructure was complete the new superstructure was pushed into place. The concrete approach slab was then poured and after a 24 hour cure the bridge was opened to traffic.

By placing the temporary transverse beams at the pier and abutments, the concrete box beams were place on top of them and vertical lifting jacks were eliminated from the sliding and setting process. The temporary works that supported the bridge prior to the slide was intitially a steel beam set on the pile bent. This cap beam was to be welded to the piles and set in a notch at the abutments and pier. The new bridge superstructure was constructed with a deep diaphragm which had stainless steel sliding shoes, which sat on Teflon coated elastomeric bearings. The bridge was pushed into place using a

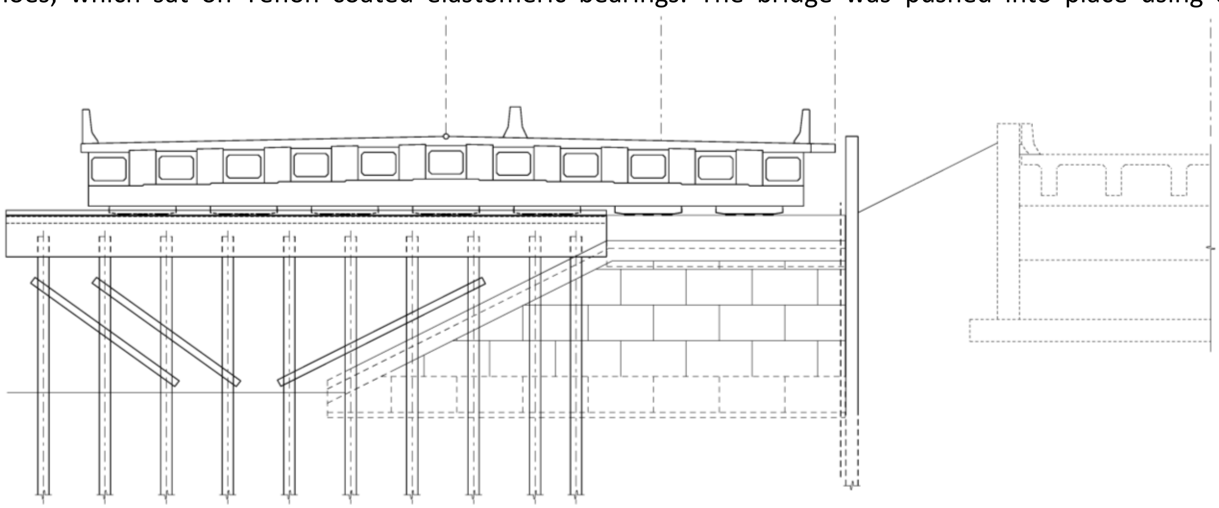


Figure 10 - Temporary/Proposed Cross Section

proprietary Mammoet hydraulic jack system. The length of the push was 74’-8”. The superstructure unfactored dead load was approximately 4549 kips with ½ of this dead load (2275 k) applied to the pier slide shoes and ¼ of the dead load (1137 k) applied to each abutment. The temporary concrete pile caps were temporarily tied into the new concrete cap to provide lateral resistance to the jacking forces.

3.3 Lessons Learned

3.3.1 Temporary Works

The Contractor elected to eliminate the steel cap beams at the pier and abutments due to the expense of steel and delivery time needed to get the beams. There was also a substantial amount of welding involved with the attachment of this beam to the piles. All of the welding would have to be tested since the temporary works were being used to support traffic during staging.

The Contractor utilized a concrete beam section over the temporary piles. This section had several advantages; first it was wider and provided them with more working surface to use for construction and sliding. Second, it was quicker to construct and demolish since no welding was necessary and there was no long lead time for the concrete.

3.3.2 Eccentric Loading

The Span 1 concrete box beams were erected on a precast transverse beam, which were later cast into the diaphragm with the deck pour. At the abutment the beams were set on the centerline of bearing, however, at the pier the beams create an eccentric load until the concrete box beams for span 2 are erected. The piers were checked for the eccentric load, but the method of temporary support was not specified, since it was within the contractor’s means and methods. The contractor attempted to counteract this eccentric load using plywood blocking, which was insufficient. The contractor then had to use bottle jacks to right the beam, and steel/plastic shims.

Lesson: Ensure that contractor provides detailed erection plan, review full erection plan, and provide oversight of contractors operations.

Another option to this issue would have been to provide vertical tie downs between the transverse beam and temporary pile cap. This would have restricted any movement or rotation of the transverse beam around the temporary bearings. These temporary tie downs could be left in place until the both beam spans are in place and the Contractor was ready to place the pier diaphragm tying everything together.

3.3.3 Temporary and Permanent Bearings

Temporary bearings consisted of a 10x12x1 ¾" elastomeric bearing with dimpled PTFE surface. Permanent bearings consisted of 11"x4'-9"x1 ¾" elastomeric bearing with PTFE surface. The bearings were designed for a 50 durometer hardness with internal steel plates. Permanent bearings were designed similarly. The Mammoet bearings sat inside the Mammoet sliding track and were restrained by a pintle. In order to eliminate vertical jacking, the bridge was constructed on top of the temporary bearings inside the Mammoet track.

The following issues were experienced during the bridge slide:

3.3.3.1 Temporary Bearing Thickness

Mammoet requested to use their standard depth 1 ½" temporary bearing, which was approved. Since this bearing was ¼" thinner than the permanent bearings wood and steel shims were used. However, the plywood shims used swelled in the wet weather prior to the move, increasing their depth, and the steel plate shims placed on top of the plywood provided a secondary slide plane. Finally, the addition of the secondary slide plane combined with the lateral forces increased the moment force on the temporary bearing restraints. The temporary bearing restraints used a ¾" diameter mild steel threaded rod screwed into an anchor. Some of these rods bent and broke.

Lesson Learned: Coordinate with contractor to ensure consistent temporary bearing thickness.

3.3.3.2 Temporary Bearing Damage

As noted previously the bridge was constructed on the temporary bearings. In addition, as can be seen in the photo above, the Mammoet track left very little room between the top of the track and the bottom of the transverse beam. Therefore, all of the temporary bearings were placed in the track. This combination prevented the temporary bearings from being adequately cleaned prior to the slide. As the bridge was slid you could hear the grinding of dirt on the PTFE surfaces. Many of the PTFE sheets separated from the elastomeric, or were otherwise damaged, and the additional friction forces may have contributed to pintle damage (see discussion later). Apparently the dirt became embedded in the stainless steel surface as the grinding continued through the whole slide. In addition, the selected lubricant (Royal Purple) began to congeal in the low temperatures and rainy conditions so the contractor switched to common dish soap.

Lesson Learned: If bridge is constructed on temporary bearings provide adequate space to place and clean the bearings. Alternately, do not construct the bridge on temporary bearings. Use dish soap for lubrication.

3.3.3.3 Pintle Damage

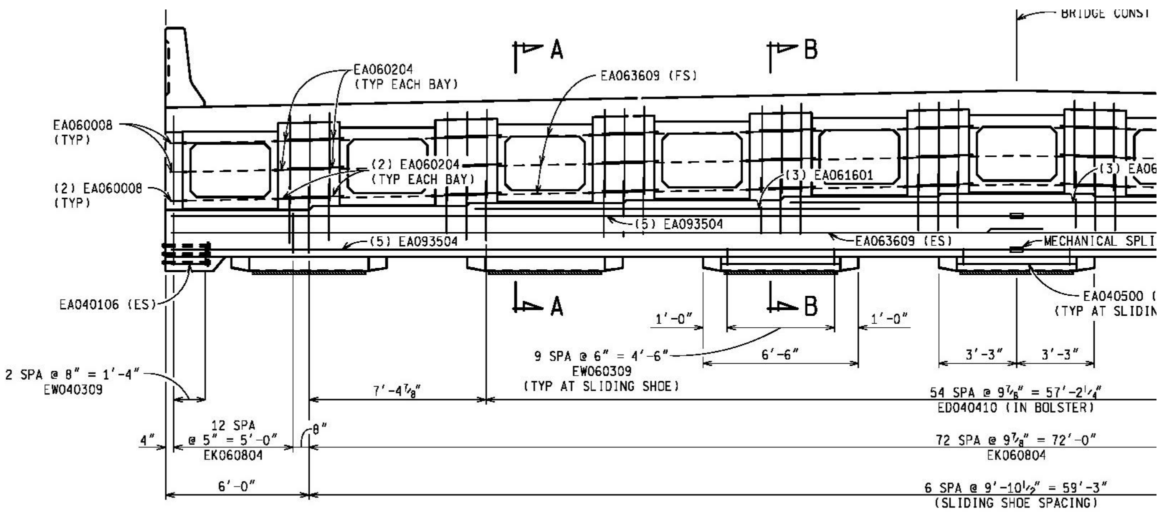
As noted previously, the temporary bearings were restrained using a pintle in the Mammoet track. The pintle was placed on the leading end of the bearing. This configuration eliminated the shoving experienced on the US-131 slide, as the bearing trailing edge would lift up but was restrained by the pintle as the slide shoe transitioned onto the bearing. The Mammoet track was not extended onto the permanent abutment/pier in order to eliminate the need to vertically jack the structure. The contractor elected to use a redhead insert with a ¾” diameter mild steel rod to restrain the temporary bearings. The permanent bearings were restrained by 1” diameter position dowel.

Several of the rods on the abutment were snapped off during the slide allowing the temporary bearings to shove. This likely occurred due to a combination of factors; the temporary bearing shims noted earlier; the dirt on the bearings; and, the weather conditions diluting the lubrication. All of these factors likely increased the coefficient of friction between the bearing and slide shoe creating a combination of shear and bending moment that broke the mild steel rods.

Lesson Learned: Temporary pintles diameter and material should match the permanent pintles. Keeping the sliding surface clean needs to be a priority to reduce friction during the slide.

3.3.3.4 Damage to Permanent Bearings

The permanent bearings were 4’-9” in length. The distance between the slide shoes was 3—4 ½”. As noted earlier with the temporary bearings, the permanent bearings also experienced some damage to the PTFE surface. In onecase a permanent bearing became wedged between the pintle and slide shoe damaging the bearing. Since the permanent bearings were longer than the space between the slide shoes it was not possible to replace a damaged bearing.



Lesson Learned: Always provide a method to replace both temporary and permanent bearings. Specify that additional temporary or permanent bearings be supplied to replace damaged bearings. Even though this slide was intended to be completed without vertically lifting the bridge, the design should always include provision for lifting the bridge to replace damaged bearings once the slide is complete.

3.3.3.5 Jack Failure

During the sliding operations, after breakout, the pier jack experienced mechanical problems. Since the sliding force necessary to move the structure after breakout was approximately half, the abutment jacks were sufficient to move the structure. It is also easier to coordinate two jacks instead of three.

Lesson Learned: Consider reducing the number of jacks in operation after initial breakout to improve coordination.

3.3.3.6 Communications

During the slide the contractor experience a failure in their radio communications system. This reduced the effectiveness of the communication between the pump operator and the crew monitoring the bridge movement.

Lesson Learned: Require the contractor to have a backup communications system available.

3.3.3.7 Drainage and Erosion Protection

No drainage was specified between the construction of the temporary MSE wall and the existing structure. In addition, no temporary erosion control measures were specified.

Lesson Learned: Particular came must be given to the drainage of the roadway and erosion control measures during the temporary conditions.