# Zink Bridge Rehabilitation Concept Report

City of Tulsa, Oklahoma

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HNTB

# Chapter 1 - Executive Summary

As part of the City's maintenance and inspection program, an inspection of the Zink Bridge was conducted in September of 2014. The findings of the inspection, including numerous structural deficiencies in need of repair, were documented in the *Zink Bridge Inspection Report.*

This report is presented as a response to the inspection findings, and outlines a concept for a rehabilitation program. The functional use of the bridge is to be expanded by the addition of an upper deck for grade separated bicycle access. The rehabilitation concept includes the construction of new approach structures at the west bridge terminus to connect the upper deck to the River Park West Trail. At the east terminus, a proposed short span ties the upper deck to the River Park East Trail. A new span is required to replace the span crossing Riverside Drive.

The bridge is well over 110-years old, which exceeds the statistical design life for a new bridge, typically taken to be 75-years (AASHTO LRFD 2012 Edition).

A series of retrofits and repair attempts have been made since the 1980's, but the repairs have deteriorated to the extent that they are no longer serving their intended purpose. Significant rehabilitation work would be required in order for the bridge to remain in service. The work consists of repairing deficiencies, expanding the functional service, and creating new tie-ins at each end of the bridge.

The estimated cost to rehabilitate the bridge is approximately $17.5 Million to $19.9 Million.

The cost of rehabilitation should be compared against the cost of replacement, and many bridge owners prefer to replace a bridge when the retrofit costs approach 70% to 80% of the cost of a new bridge.

Figure 1-1: Project Overview

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# Chapter 3 - Project Understanding

As part of the City's maintenance and inspection program, an inspection of the Zink Bridge was conducted in September of 2014. The findings of the inspection, including numerous structural deficiencies in need of repair, were documented in the *Zink Bridge Inspection Report*. This report is presented as a response to the inspection findings, and outlines a comprehensive rehabilitation program. The rehabilitation concept includes the construction of new approach structures at the west bridge terminus to connect the upper deck to the River Park West Trail. At the east terminus, a proposed short span ties the upper deck to the River Park East Trail, and a new span crosses Riverside Drive. Additionally, the rehabilitation expands the functional use of the bridge by adding a second deck for grade-separated bicycle access.

The Zink Bridge crosses the Arkansas River with 14 identical steel Warren truss spans, each approximately 102 feet long, supported on plain concrete piers. The structure was built around 1904 forming a rail link to the City of Tulsa. Following the retirement of the rail line, ownership was transferred to the City in the 1970's. The bridge was modified to provide pedestrian access at the lower chord level, abandoning the railroad ties in place at the top chord level. The bridge is enjoyed by pedestrians, joggers, and cyclists, and serves as an access point to the adjacent parks, fishing pier, and dam control house.

Significant modifications were made to the structural system during the life of the bridge, but no record drawings are available other than some non-structural components. The inspection team was not able to review the retrofit contract plans or discuss the retrofit design intent with the designer of the retrofit. On that basis, some interpretation and assumptions are necessary in assessing the behavior of the structure.

Although the retrofit work was successful in extending the useful life of the bridge for decades, most of the modifications are no longer considered consistent with best practices. Some of the retrofit work has accelerated the deterioration of the bridge.

The addition of a pedestrian deck at the lower chord level employed thousands of welded connections directly to the fracture critical lower chord. Numerous welds exhibit defects that could compromise the reliability of the structure over the long term. Several welds appear to have cracked, and pose a risk to the fracture critical lower chords, suggesting crack growth could precipitate collapse. The cracks do not appear to present an immediate hazard requiring closure of the bridge, however it is not clear whether the cracks will grow and become unstable. The effort to grind out these cracks and replace the connections would be quite extensive given the large number of welds (numbering in the thousands). One alternative may be regular inspection to monitor the growth of the cracks over time.

The original sway bracing system consisted of X-bracing through the bridge cross section, located on 20' centers. It was removed to make room for pedestrian access on the lower chord, and replaced with a rigid frame system at the end points of the bridge. Additionally, the original built-up vertical hanger members were retrofitted by welding steel plates along the length of the batten webs. The hanger retrofit does not appear to have a corrosion protection system and is exhibiting advanced corrosion. Numerous welds have cracked and there is a risk that the growth of the cracks over time may propagate into the substrate, triggering brittle fracture of the hanger members.

Retrofit repairs to the bridge bearings were conducted in a manner that is no longer consistent with code requirements or best practices. The bearings consist of stacked neoprene pads, which are progressively walking out from underneath the bridge. The lateral resistance of the bearings was eliminated by the retrofit, and the bridge is at risk of unseating in an extreme lateral loading event such as an earthquake, high wind event, or severe flood. The bearings should be modified or replaced.

Floating debris during a major flood event can pose a significant hazard to the structure, and there are signs of damage corresponding to photos taken during the flood of 1986. A tanker truck was swept into the Arkansas River and impacted the lower chord, distorting it several inches out of plane. The retrofit work done to the bearings left the bridge without lateral restraint, and the span was pushed laterally out of position. It remains askew relative to the adjacent spans, but could have easily been pushed over.

Preliminary calculations indicate the upper lateral bracing, lower lateral bracing, and lower lateral strut members are significantly undersized. The lower lateral bracing system has been compromised throughout the 14 spans, taking on a buckled shape. Likewise, lower lateral strut members have buckled at several locations, indicative of inadequate performance of the sway frame system. Systematic replacement of these members would be required.

The bridge fencing system is deficient and will need to be replaced. The railing is connected by a combination of small bolts and screws, many of which are missing or loose. In some locations the wood on the top rail exhibits significant deterioration.

The bridge lacks any corrosion protection system, with the exception of some of the retrofit elements. The retrofit stringers were galvanized; retrofit portal frames were primed and are now corroding; tubular steel members on Span 1 were painted. Other elements received no corrosion protection at all, and are moderately to severely corroded. Material coupons can be taken and tested to assess the corrosion resistance of the original material. Some modifications to the retrofit hangers are warranted to eliminate corrosion-promoting details. It is likely that blast cleaning and painting the steel structures will be warranted, and this activity may require encapsulation.

The gusset plates exhibit extensive pack rust throughout the spans, ranging from minor to severe. Pack rust is an electrochemical reaction that causes rust formation within the joints between steel plates, causing the plates to bulge. If the problem is not resolved, pack rust creates serious structural problems and can seriously affect the load capacity and structural stability of bridges. Pack rust can be removed, and chemically treated with Reacted Alkaline Viscoelastic Calcium Sulfonate (RAVCS) coating system. Removing pack rust slows further damage from occurring but does not undo the damage that has been done. The out of plane deformations will remain and in some cases it may be necessary to replace these plates.

The condition of Span 14 is significantly more severe than Spans 1 through 13. During one of the retrofit phases, a concrete slab was placed on the east end of the span to allow pedestrians to access a nearby stairway. The concrete slab traps water against the upper chord and promotes corrosion. This has led to the most severe section loss and deterioration in any location on the bridge. The flange thickness has been reduced by up to 50% in some areas accompanied by 50% to 90% rivet head loss. Replacement of Span 14 is likely to be more economical than rehabilitation.

The piers appear to be composed of unreinforced, plain concrete. The piers exhibit severe spalling, cracking, and latent delamination. Plain concrete piers with large structural cracks behave similar to a stack of dry laid (unmortared) stone, and their behavior cannot be reliably assessed. A series of repairs were employed over the years, which have helped to extend the life of the structure until now. Repairs such as parging and epoxy injection are ineffective on unreinforced, plain concrete. Confinement of the pier caps by external bar tendons was conducted, and likely slowed the formation of cracks and spalls. Nevertheless, signs of serious cracking and spalling are evident. The piers are likely approaching the end of their useful life. The cost of repair should be compared against the cost of replacement.

The original wood trestle on the west end of the bridge is in poor condition and exhibits significant deterioration. It will need to be replaced if access is to be allowed on the top of the structure. The wood ramp leading to span 1 and the gazebo roof both exhibit moderate to severe wood deterioration and are nearing the end of their service life. The fishing pier was found to be in overall good condition and with some repairs its service life can be extended.

# Chapter 4 - Service Life

## Section 1 - Overview of Service Life of Bridges

The design life of a bridge system is a target life in years, set at the initial design stage and specified by the bridge owner. The AASHTO *LRFD Bridge Design Specifications* provide design guidance based on 75 year load expectancy and fatigue performance. Some projects require longer design lives, up to 100 years or more, and require special design methods and construction materials to achieve this level of performance. The Zink Bridge is over 110 years old, and the bridge is remarkable for having remained useful for such an unusually long life.

The end of service life for a component of a bridge does not necessarily signify the end of the bridge system service life, as long as the component could be replaced or its function resumed with a retrofit measure. If a component could be replaced or retrofitted, it may be possible for the bridge to continue providing the desired function.

The service life of a bridge component ends when it is no longer economical or feasible to undergo repairs or retrofits, and replacement is the only remaining option. The service life of the bridge ends when it is not possible to replace or retrofit one or more of its components economically or because of other considerations. The service life of a bridge system is governed by the service life of its critical components. The critical components are defined as those needed for the bridge as a system to provide its intended function.

In the rehabilitation of a truss bridge, repair work is typically undertaken so that the bridge system can live out its intended design life. Rehabilitation is not ordinarily undertaken to extend the design life because of the extensive analysis required and the difficulty associated with ascertaining the complete load history of the bridge.

There are different methods of enhancing the service life of existing and new bridges. Examples include: a) using improved, more durable materials and systems during original construction that will require minimal maintenance; and b) improving techniques and optimizing the timing of interventions, such as preventive maintenance actions. Interventions can be planned and carried out based on the assessment of individual bridge conditions and needs, or based on a program of preventive maintenance actions planned for similar components on a group of bridges. A simple example of a preventive, planned maintenance program might include the following activities:

- Spot painting steel structures
- Sealing decks or superstructures in marine environments
- Sealing substructures on overpasses
- Cleaning debris from bridge deck expansion joints
- Cleaning debris from bearings and truss joints
- Cleaning drainage outlets

Service life can be extended either by using more durable, deterioration-resistant materials or by planned intervention. A cost comparison can be made to determine the most cost-effective approach for a given bridge based on its exposure to the elements and the level of available maintenance and preservation actions.

# Chapter 5 - Rehabilitation Concept

A rehabilitation concept was developed in response to the deficiencies documented in the inspection report.

## Section 1 - Overview of Structural Systems and Rehab Strategies

The Inspection Report enumerated a broad range of types of deficiencies. Some deficiencies are minor and require no repair at this time. Of the deficiencies requiring repair, most are systemic problems requiring systemic repairs to each span that is to continue service. The proposed rehabilitation concept is organized by groups of bridge components that act together as subsystems of the bridge.

### 1.1 - Bridge Subsystems and Construction Activities

The bridge subsystems and construction activities include:

- *Mobilization, Site Access, and Demolition*
- *Upper and Lower Deck Systems*: New upper deck system, lateral bracing and sway framing, and existing lower deck rehabilitation
- *Bearings and Piers*
- *Necessary Structural Repairs*: Pedestrian railing, corrosion protection system, gusset plates repairs, bottom chord prestressing and structural steel repairs
- *Auxiliary Structures*: Approach structures and span over Riverside Drive

### 1.2 - Overview of Rehabilitation Strategies

Conventional approaches to repairing deficient truss bridges can be prohibitively costly, and it is important to approach this type of work minimally and surgically.

Many of the deficiencies documented in the Inspection Report require invasive measures to mitigate the condition. For instance, cracks located at the retrofit welded connections of the bridge members can necessitate the complete removal of the adjoining member in order to replace the welded connection with a bolted connection. The addition of bolted connections to a built-up riveted structure presents further difficulties of accommodating the rivets that are to remain in place. The removal of a load bearing member typically requires the addition of temporary struts or prestressing bars. Deficient members are typically either strengthened by the addition of plates and stiffeners, or removed and replaced completely.

The proposed rehab measures can be classified as either repair to structural deficiencies (paint, deterioration, inadequate strength, etc) or measures required to convert the structure for continued use (tying in to the new River Park East Trail, new span over Riverside Drive, addition of an upper deck, tying in at the west end, etc).

## Section 2 - Rehabilitation Items

### 2.1 - Mobilization, Site Access, and Demolition

*Mobilization and Site Access*

In order to perform the rehabilitation and necessary demolition of the structure, access is required across the riverbed. While barges were considered, the variable water depths and limited access would render this option ineffective. The most cost effective solution is to create a gravel road or causeway across the river parallel and adjacent to the bridge that can be used to continuously access the construction site.

The gravel road will aid in demolition of components that are to be replaced, and will facilitate crane access, concrete pumping, repair to the piers, and other construction activities.

Temporary platforms should be installed underneath the spans in order to provide safe access for performing structural repairs and surface painting that may be necessary. These temporary platforms can be supported from the bridge using wire rope, and can be shifted from span to span as the procedures are carried out.

*Demolition*

Some key areas of the Zink Bridge are sufficiently deteriorated that attempting repair would be ill advised. It is proposed that these areas be demolished and replaced with new structure.

It would be advantageous to demolish Span 1 (the first span at the west bank of the river). Numerous modifications were made to this span at a previous date that place welds on fracture critical members. Additionally, weld defects have been noted throughout the 14 spans, calling into question the welding techniques and the compatibility of the chemistry between the adjoining base metals.

Aside from structural deficiencies, by eliminating Span 1, the total length of the bridge can be shortened and the upper level could be brought to grade on the river side of the pathway, thereby eliminating property line issues. This would also eliminate the need to replace the existing abandoned trestle, which shows significant deterioration and should be demolished as soon as is practical.

Span 14 exhibited extensive, significant structural deficiencies which are described in the inspection report. Additionally, the design plans for the Gathering Place Project indicate modifications to the grade that would necessitate the removal of the span. It was hoped that the span could be salvaged and reused over Riverside Drive, but it is apparent that it would be most cost effective to replace this span instead of attempting to rehabilitate it.

The inspection report enumerates deficiencies in the wood pedestrian ramp connecting Span 1 to the trail. It would be more economical to replace the ramp, rather than attempt to repair damaged members, particularly given the advanced deterioration of the wood preservative treatment, and the excessively high moisture content of the core wood.

In Span 6, two wooden cantilever structures are attached to the bridge, and were closed to the public at the time of the inspection due to safety hazards posed by the structural deterioration. These structures exhibited numerous significant structural deficiencies which are described in the inspection report. It is likely not cost effective to rehabilitate them, and it is proposed that they should be demolished.

The pavilion (gazebo) north of Span 11 is composed of a concrete structure supporting a wooden roof. The concrete was found to be in overall good condition but the wooden structure was in poor condition. It is proposed that the pavilion roof be demolished and replaced.

## 2.2 - Upper and Lower Deck Systems

An upper deck is to be added to the structure to accommodate grade separated bicycle traffic. Given that a deck is to be added, it can be designed such that it behaves as part of the global structural system. If detailed properly, the deck can act like a rigid diaphragm under lateral loads. The diaphragm behavior can stiffen the structure to such an extent that the deck eliminates the need for any new upper lateral bracing or rigid frame members, which otherwise would require modification. Similarly, a reinforced concrete deck at the lower level could be implemented in a manner that resolves the deficiencies of the lower deck and lower lateral bracing system.

*New Upper Deck System*

The proposed upper deck system consists of a reinforced concrete deck slab bearing on and composite with structural steel stringers. The reinforced concrete slab is a 9” thick, and cast in place on stay-in-place forms. The structural steel stringers are W12x53 wide flanged rolled structural steel sections or similar. The concrete deck is made composite with the steel stringers through use of steel shear stud connectors, shop welded to the stringer flanges. Incorporating good details consistent with best practices, such as drip grooves on the outer edges, can delay further deterioration of the steel truss by directing moisture away from the steel-concrete interface.

By properly detailing the slab to be composite with the steel stringers and by creating a bolted moment-connection between the stringers and the top chords, the new upper deck system effectively renders the upper lateral bracing and sway framing systems unnecessary.

*Upper Lateral Bracing and Sway Framing*

The existing upper lateral bracing, lower lateral bracing and sway framing systems of the Zink Bridge exhibit many of the worst conditions documented in the inspection report. The defects included: widespread corrosion such as pack rust, rivet head loss and section loss (up to 100% in certain instances); deformed connection plates and buckled members (lower lateral bracing); poor welds and crack welds at the vertical hanger and top chord retrofit upper strut connections. Because of the nature and severity of these types of deficiencies, these systems cannot be relied upon to serve their intended purposes. Additionally, preliminary analysis indicates the original upper and lower lateral bracing members are undersized and do not have adequate load carrying capacity for the lateral loading requirements of the code. These members must be replaced or their required capacity must be restored by other means.

A Lusas model was developed to investigate the structural capacity with the upper lateral bracing and retrofit upper struts removed and replaced with an upper deck system composite with the upper chords. The results of the analysis demonstrated the upper deck, if properly detailed, acts as a rigid diaphragm between the top chords of the truss and effectively replaces the upper lateral bracing and sway framing systems. With the addition of this type of deck system, the original upper lateral bracing may be removed or remain in place without negatively affecting the structural performance. The retrofit upper struts and end frames should be removed due to the poor quality and cracked welded connections, which are not functioning as intended.

*Lower Lateral Bracing*

The lower lateral bracing consists of diagonal members (L3-12 x 3-1/2 x 1/2 angle members) spanning diagonally beneath each deck panel in the horizontal plane. At each panel point, a transverse member is composed of L3-1/2 x 3-1/2 x 1/2 angle members). These members are slender and undersized. They should be replaced with non-slender members. Also, due to the widespread rivet head loss and corrosion connecting these members, the existing riveted connections should be replaced with high strength bolted connections.

*Existing Lower Deck Rehabilitation*

The existing lower deck system is a steel framed system with a wooden deck. Wood boards are fastened to steel beams by self-tapping screws. The boards tend to uplift and separate from the steel beams, creating tripping hazards and requiring frequent (weekly) maintenance by the City. The steel beams consist of W4 rolled steel sections, bolted on each end to a connection tab plate. Each connection tab plate is welded to the primary tension member, the lower chord of the truss.

This type of connection is fatigue sensitive. Most of the connection tab welds exhibit apparent defects, and many may be cracked. The welds require mitigation to prevent the further propagation of cracks or the formation of additional cracks. The welded connection tabs should be abandoned and replaced by bolted connections between the steel beams and the lower chord.

Repairing cracked welds is costly work and there are thousands of welds requiring mitigation throughout the lower chord level. Therefore, it is recommended to remove the existing lower deck system (wooden deck boards and W4 stringers) and replace them with a composite concrete deck slab system.  A concrete deck system can be detailed in a manner that its rigid diaphragm behavior eliminates the need to repair the lower lateral bracing system. The concrete deck would have the additional advantage of durability and reduced maintenance as compared to the ongoing maintenance needs of the existing wood plank deck.

## 2.3 - Bearings and Piers

The bearings are the metal pins and synthetic rubber pads that support the bridge at the tops of the piers. The bearings have undergone some deterioration over the years, but more critically their function has been compromised by the movement of the stacked neoprene pads over time. Crucially, the bearings do not provide adequate resistance to lateral loading from severe wind, earthquake, or flood events. The bearings must be replaced. Bearing replacement is typically accomplished in four steps: install jacking beams, raise the bridge vertically on jacks, swap out the bearings, and lower the bridge back onto the piers.

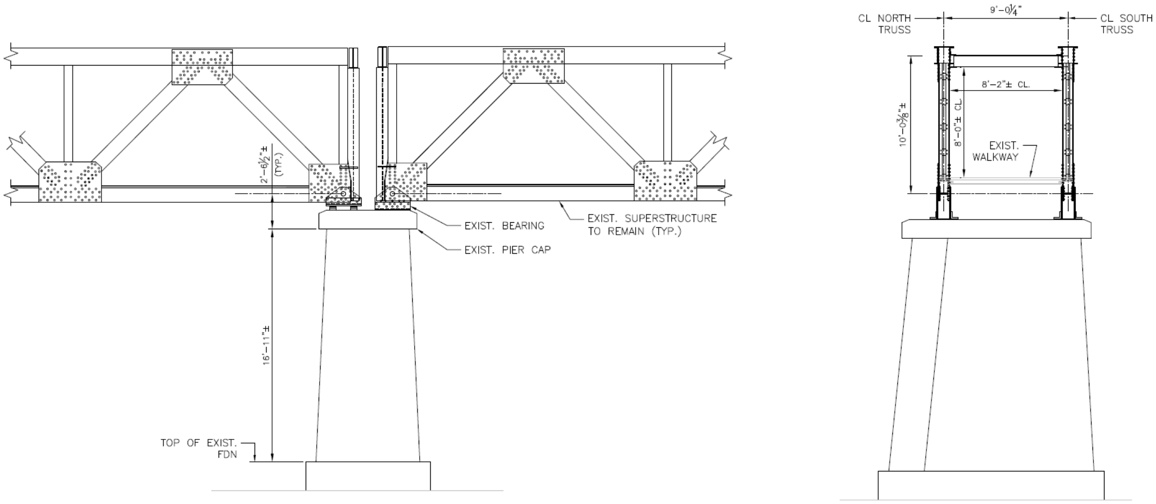


Figure 5-1: Raising a bridge on jacks for bearing replacement

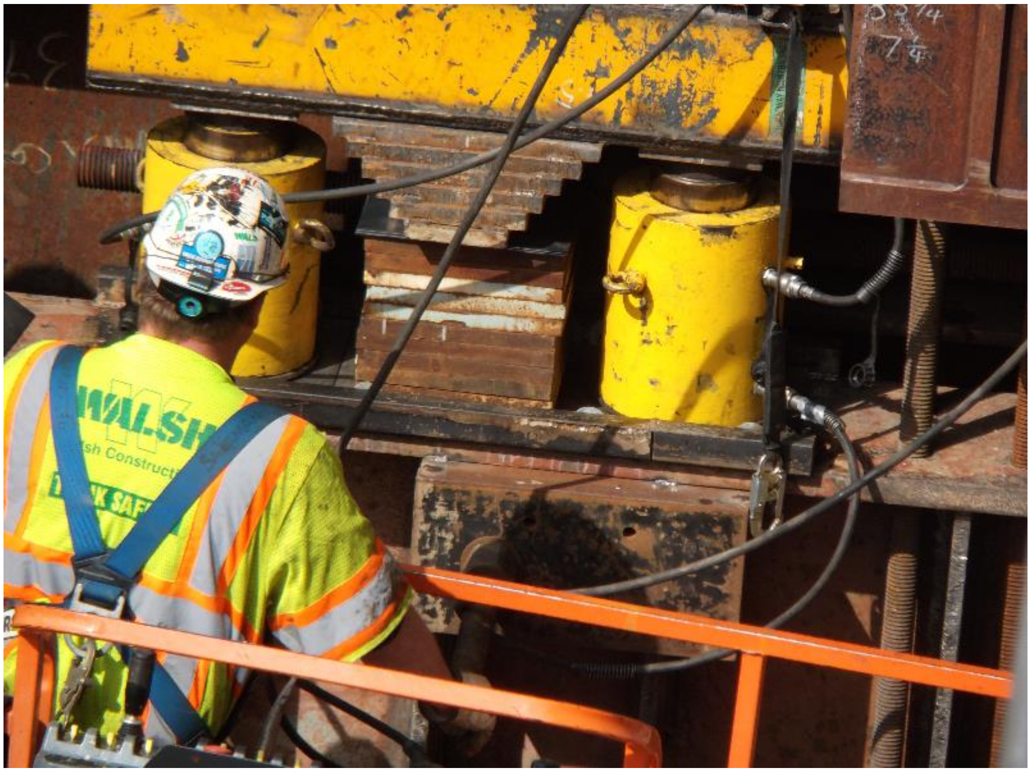


Figure 5-2: Raising a bridge on jacks for bearing replacement

The piers are composed of unreinforced, plain concrete. They have cracked and spalled extensively in their 110 year life. The absence of reinforcing steel means conventional repair methods, such as epoxy injection and sparging, are purely cosmetic, and would not restore the structural capacity of the piers. Other repair methods can be implemented in unreinforced, plain concrete to confine the concrete and delay the growth of existing cracks. These repair methods include externally prestressing the pier caps, and wrapping the piers in composite fiber material (FRP or CFRP). External posttensioning was installed on the caps some time before the 1986 flood, and this measure has successfully extended the life of the piers up until now. Unfortunately, the structural (shear) cracks at the caps have continued to grow, and in some circumstances the cracks appear to have propagated across the full depth of the cap. At one location, the continued crack growth has precipitated the spall of a large block of concrete, exposing an underlying through-section shear crack that, if left to grow, could fail the pier and drop the spans. The effect of the external prestressing appears to have been to slow the deterioration of the piers, but it appears that the deterioration has reached the point that additional measures are required to permit the continued safe use of the piers.

Fiber wrapping has in the past been an effective means of restoring strength and ductility to inadequately reinforced or unreinforced plain concrete columns on some projects. Fiber wrapping tends to be economical when access to the column is straightforward and unobstructed. The installation of composite fiber wrapping on Zink Bridge would necessitate the use of cofferdams and dewatering to create a safe working environment for excavating / de-mucking around the bases of the piers. These operations are anticipated to be costly and would require the installation of a temporary access road in the riverbed for the duration of construction. Given the challenging constraints associated with dewatering and working within the Arkansas River, other solutions are anticipated to be cost competitive, while offering greater durability than composite fiber wrapping.

Given the challenges of suitable and economically repairing the existing piers, three alternative concepts were explored that would essentially replace the existing piers:

1. *Micropiles*: Coring through the piers from above to install small diameter micropiles, which would take over the function of the piers. The existing piers would effectively be abandoned in place.
2. *Reinforced concrete shell*: Placing a reinforced concrete jacket around the existing piers. The existing piers would be abandoned in place, within the shell.
3. *Lateral Slide*: Install new drilled shaft / monoshaft piers directly adjacent to the existing bridge alignment, and then move the bridge to the new alignment using jacks and temporary tracks.

*Micropiles:*

Micropiles are small diameter foundation elements consisting of a steel pipe casing (nominally ranging in size from 6” to 12” diameter), drilled into the ground and grouted in place to form a rock socket. Micropiles have successfully been implemented as a means to strengthen aging concrete structures, including dams and bridge piers. In this type of application, a small drill rig would be placed on the bridge above the piers, and holes would be cored down through the piers into rock. The steel casing would remain in place and serve both to form the grout and to confine and stiffen the resulting pile element. After placing the micropiles, the bridge would be raised on jacks and the tops of the piles would be encapsulated in a new reinforced concrete pier cap. Once the bridge is lowered, the weight of the structure would be borne by the new micropile foundation system, and the original piers would no longer carry any load from the structure. The old piers would effectively be abandoned in place.

Although this type of system has been implemented elsewhere, it has been shown to be economical only in particularly challenging sites with deep river piers, and with clear access for a drill rig atop the piers. In the case of Zink Bridge, the existing piers are apparently founded on relatively shallow rock, and accessibility for a drill rig would be complicated by costly accommodations. For these reasons, the micropile pier replacement concept was eliminated.

*Reinforced Concrete Shell:*

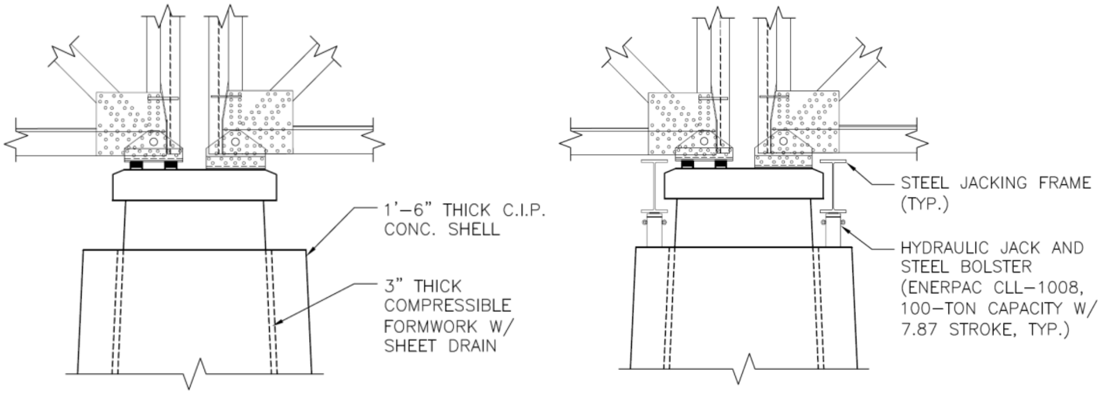


Figure 5-3: Stage 1 Pier Retrofit
*a)* Shell is built around existing pier, and truss gusset is reinforced to allow for jacking joints. *b)* The truss is jacked from the new shell, and raised off of the existing pier cap.

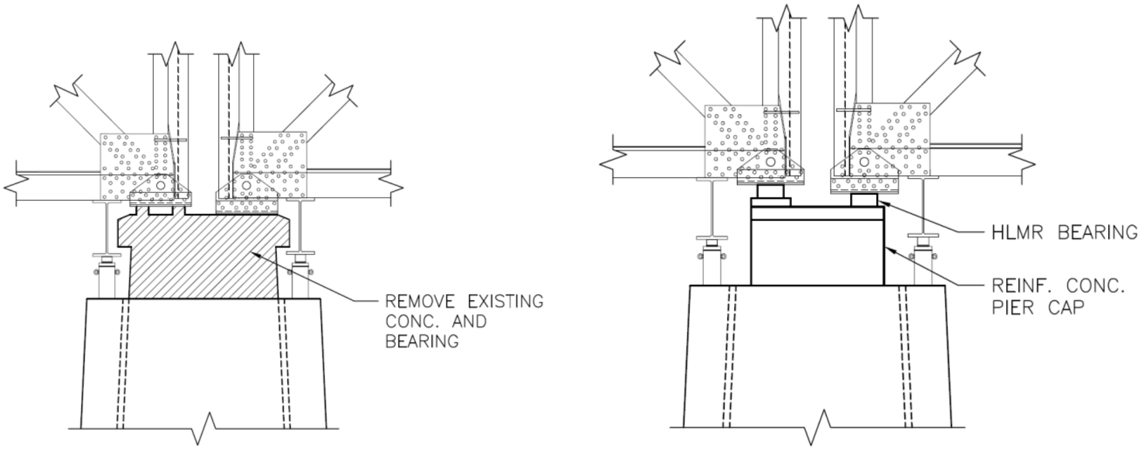


Figure 5-4: Stage 2 Pier Retrofit

*a)* The old pier top and the iron bearing are removed down to the level of the new shell. *b)* A new concrete pier top and new bearings are installed.

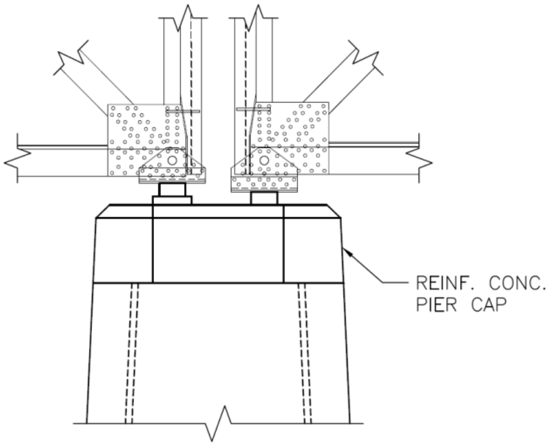


Figure 5-5: Stage 3 Pier Retrofit
The jacks are removed and the jack points are filled with reinforced concrete.

*Lateral Slide Construction*:

Slide-in Bridge Construction was originally developed as a cost-effective technique to rapidly replace an existing bridge while reducing impacts to mobility and safety. It is a technology that reduces the on-site construction time associated with building bridges. The technique is proposed for Zink Bridge because it is cost competitive with the other explored alternatives, while achieving a superior level of service and durability.

Slide-in Bridge Construction allows for construction of new bridge piers while maintaining traffic on the existing bridge. The new piers are built directly adjacent to the existing bridge alignment. Once construction is complete, the bridge is closed, the existing bridge structure is slid into its new location atop the new piers, and the old piers are demolished.

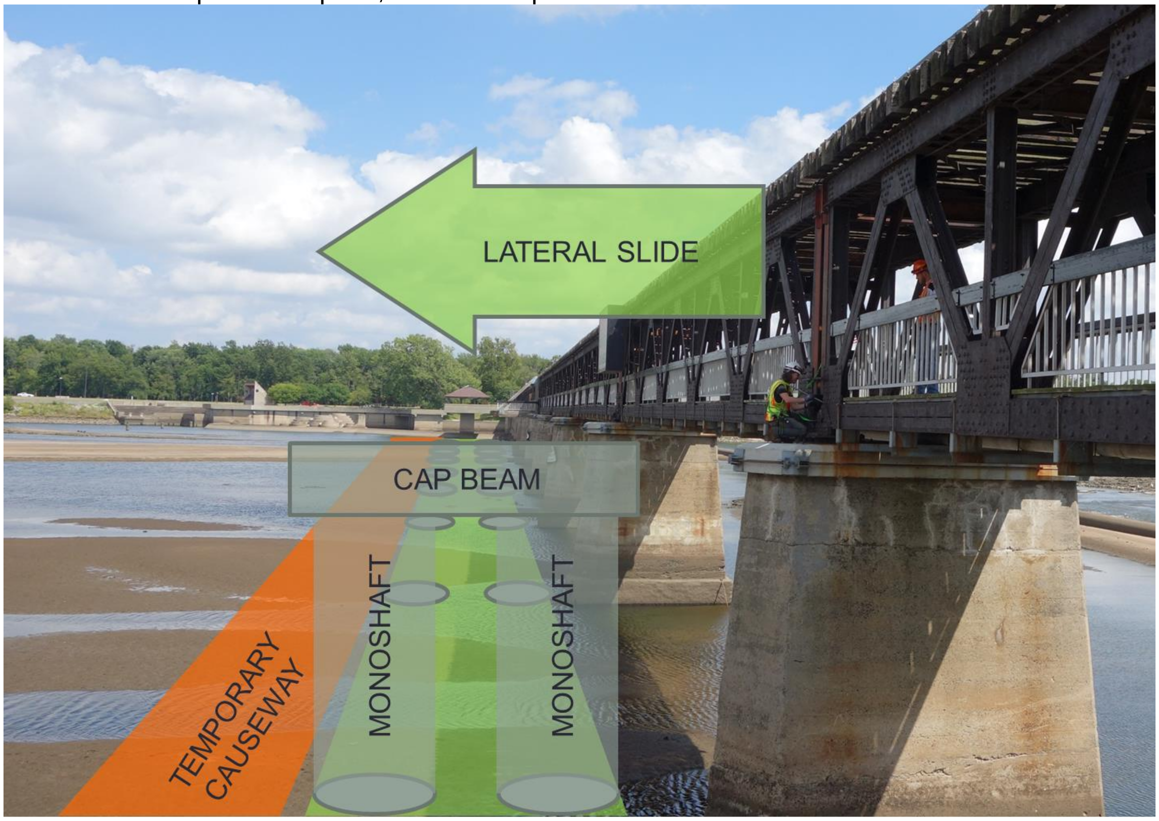


Figure 5-6: Lateral Slide Pier Construction Alternate

The Lateral Slide – the new piers consist of drilled shafts that extend above the surface, forming “monoshaft” columns. Using this technique, the foundation and columns are installed in a single step, eliminating the need for coffer dams, dewatering, and excavation.

Sliding a bridge is not a new concept and has been successfully implemented in many projects nationwide. Most often, these projects have been large bridges with high volumes of traffic and limited construction options. The technique has been successfully employed by state agencies and FHWA with small bridge replacements as an innovative option to minimize impacts to the traveling public.

There are several fundamental benefits to using Slide-in Bridge Construction, as compared with phased construction, including:

- Enhanced safety
- Shortened on-site construction time
- Reduced mobility impacts
- Potentially reduced project costs
- Improved quality
- Improved constructability

## 2.4 - Necessary Bridge Repairs

*Pedestrian Railing*

The inspection report noted that the pedestrian railing on the bridge is in poor condition. The attachment of the railing to the bridge is not adequate as designed and there is significant deterioration that undermines the rails load carrying ability. It is recommended that the hand rails be replaced in full.

The addition of an upper deck will necessitate railing at the upper level.

*Corrosion Protection System*

The existing base metal is uncoated and there is widespread evidence of corrosion and deterioration, in such forms as surface rust and scaling, pack rust and section loss. The deterioration is perhaps moderate enough to assume the possibility the base metal may perhaps be a form of weather steel; however, this would require some form of chemical testing to establish the existing base metal chemistry. This analysis was not performed as part of this concept phase, therefore a corrosion protection system should be applied to prevent further deterioration.

In order to ensure the corrosion protection system is effective, the base metal must blast cleaned to remove any surface latency that may inhibit the effectiveness of the system. Blast cleaning involves shooting steel shot under high pressure against the existing steel to remove surface latency. The span or area that is to be blast cleaned is temporarily encapsulated to prevent the steel shot from entering the environment. The blast clean surface shall conform to SSPC (Summary of Surface Preparation Specifications) – SP 10 (near white metal) surface prior to applying the primer coat.

In steel construction, paint is most often used as the corrosion protection system for structural members. Typical paint systems involve three or more coat system using an organic zinc-rich epoxy primer, a fast curing epoxy second coat and a final polyurethane acrylic paint.

*Gusset Plate Repairs*

As can be seen in the inspection report, pack rust is one of the main deficiencies that affect gusset plates. The deformations that take place could lead to further deformation due to structural forces. The pack rust can be removed using mechanical methods but if proper steps are not taken, the open space will again fill up with pack rust. There are proprietary systems that can be implemented in order to reduce or eliminate pack rust in the future. These methods involve removing rivets in the area of pack rust, cleaning out the affected area, heating the steel and applying the proprietary sealant. This work should be performed in conjunction with the corrosion protection that is selected for the bridge at large.

Several riveted gusset plate connections are overloaded, as determined in the analysis performed as part of this investigation. As outlined in the structural steel repair section, overloaded rivets should be replaced with high strength structural steel bolts.

Additionally, there are two locations where the gusset plates show significant defects requiring mitigation. It is proposed that retrofit plates can be attached to the connections.

*Bottom Chord Prestressing*

Bottom chord truss members are the primary tension members and are considered to be fracture critical members, meaning if the bottom chord members were to fracture a collapse mechanism would form. In a previous effort to retrofit the Zink Bridge, thousands of fatigue sensitive welds and potential fracture paths were added to bottom chord. An effective way to reduce the effects the sensitive welds is the use of prestressing. Prestressing may both eliminate vulnerability to fatigue, by reducing the tension stress range, and fracture, by creating a redundant load path through the prestressing tendon.

The original bottom chord section is comprised of 4 back to back angle sections riveted to batten plates. This type of construction offers some internal redundancy since crack propagation is limited to a single section constituent; however, multiple components of the bottom chord have been welded too. To account for this, the amount of prestressing was determined to be roughly equal to the equivalent force of two angle sections fracturing, with an impact factor of two. Refer to the figures below.

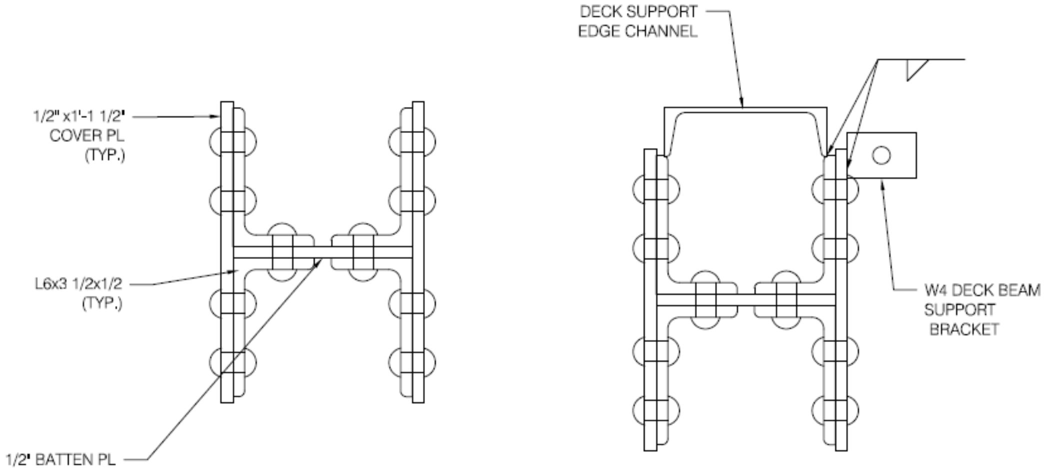


Figure 5-7: Chord Section

*a)* Original As-Built Bottom Chord Section. *b)* 1980’s Retrofitted Bottom Chord Section

The original bottom chord section is composed of 4 back to back L6x3-1/2x1/2 angles riveted to batten plates.

The 1980’s retrofitted bottom chord section added the deck edge channel and W4 deck support bracket (connection tab) and introduced fatigue / fracture sensitive welds to two angle components.

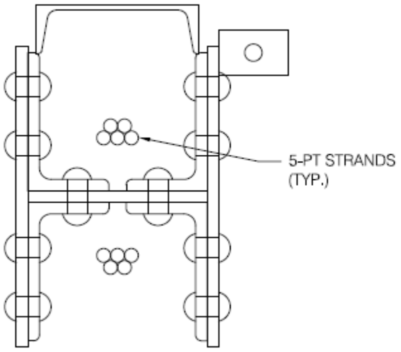


Figure 5-8: Proposed Post Tensioned Bottom Chord Section

The proposed post tensioned bottom chord section adds 10 post tensioning strands to provide a redundant load path in the event the potentially compromised bottom chord angle members fracture.

*Structural Steel Repairs*

There are several structural connections on the bridge that require rehabilitation, in order to bring the bridge up to a state of good repair. The beams that support the wooden deck on the lower level are currently bolted to a tab that is welded to the bottom chord. These welded tabs would have to be removed and have bolted elements that connect to the deck beams.

The retrofit struts and end frames exhibit ineffective weld details and have already shown signs of deterioration and failure. It will be necessary to replace these elements in order to restore the structural capacity that they are expected to deliver.

The lower struts (transverse members in the plane of the lower chord) have been shown to be inadequate by structural analysis and appear to have buckled in many cases. These members will need to be replaced with more robust elements, as previously discussed (See 2.2 Upper and Lower Deck Systems).

The vertical members were previously retrofitted with steel plates that were welded along the length of the members. These welds not only jeopardize the integrity of the existing base metal, but have also served to trap moisture and accelerate deterioration. Significant pack rust was noted at numerous locations in the *Inspection Report*. It will be necessary to remove these plates, grind off the welds and clean the areas that have pack rust or corrosion. This should be carried out in conjunction with the corrosion protection system that is implemented.

Replacing riveted connections will be necessary to perform some of the steel retrofits on the structure. In order to do this, the contractor must use mechanical methods to remove the rivet and replace it with a high strength bolt, tightened using industry standards.

## 2.5 - Auxiliary Structures

Two new structures are required for accessing the rehabilitated structure: West terminus connecting to River Park West Trail, East terminus connecting to River Park East Trail, and the span over Riverside Drive.

*Approach Structures*

The rehab concept assumes that Span 1 can be safely rehabilitated without additional measures beyond the repairs envisioned for Spans 2 through 13. Additional material sampling and lab testing will be required in order to demonstrate this assumption’s feasibility. Additionally, it may be possible to achieve a reduction to the overall cost of rehabilitation by demolishing Span 1. This approach may enable the total structure length to be reduced by eliminating the need for the structure to cross over the River Park West Trail. The following series of figures are alternative west approach alignments.

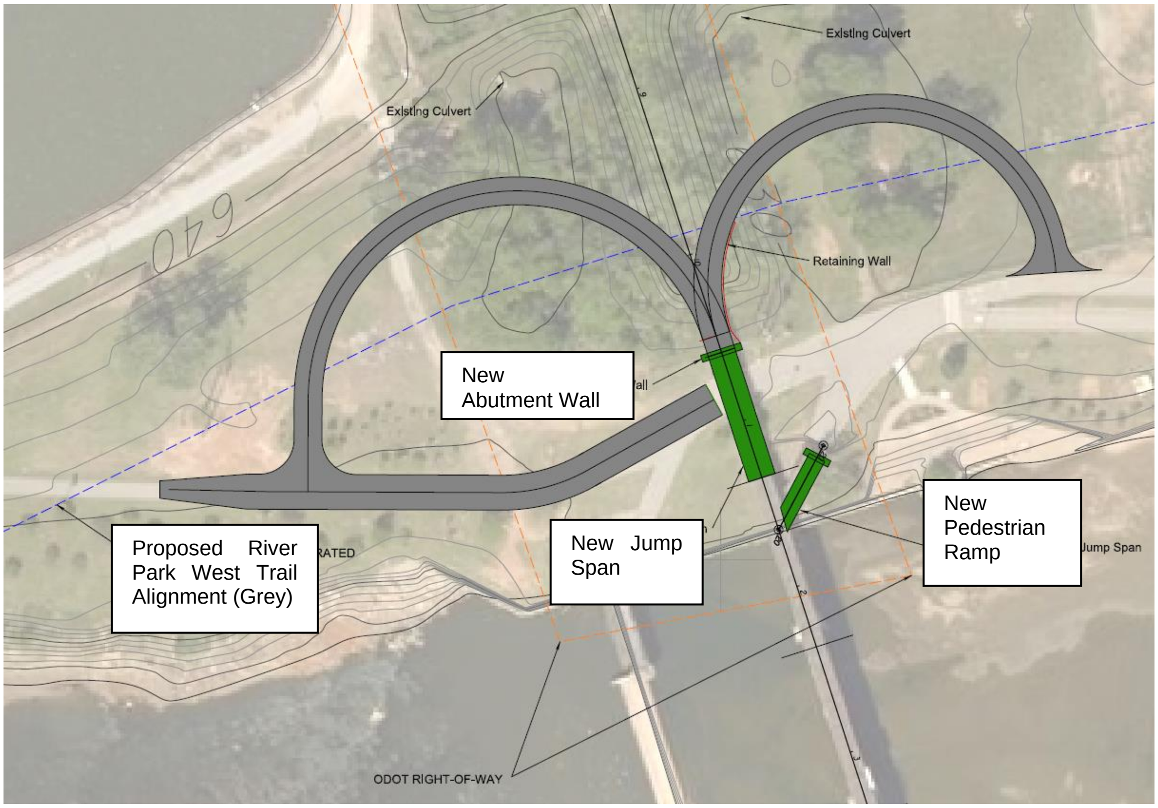


Figure 5-9: West approach alignment alternative one

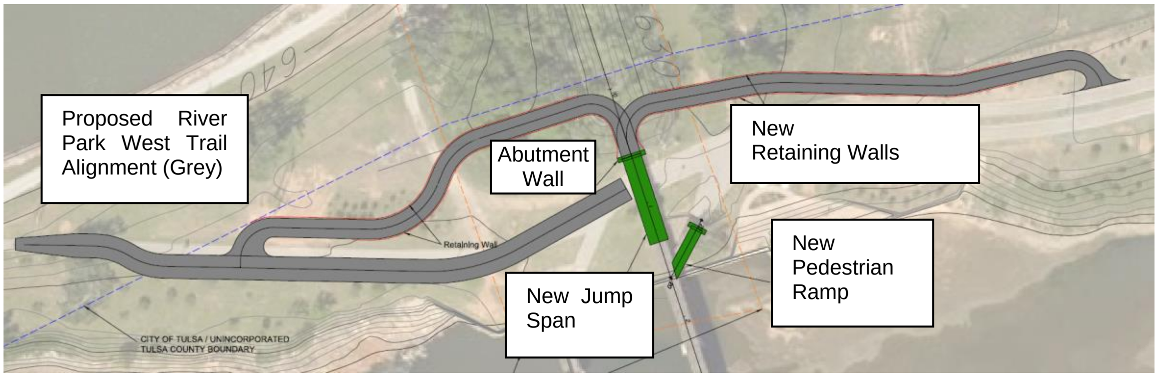


Figure 5-10: West approach alternative two

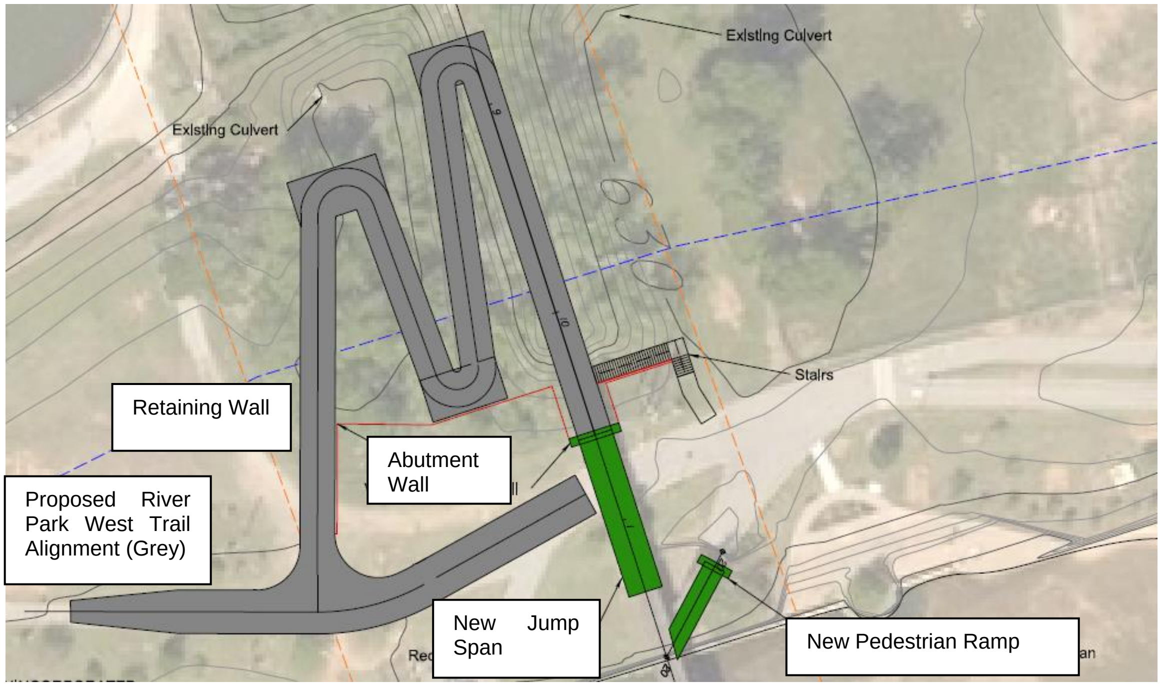


Figure 5-11: West approach alternative three

*Span Over Riverside Drive*

The existing span over Riverside Drive is no longer in service and has been demolished. To tie the River Side East Trail and the new Riverside Park to the River Side West Trail, a new span over Riverside Drive is required.

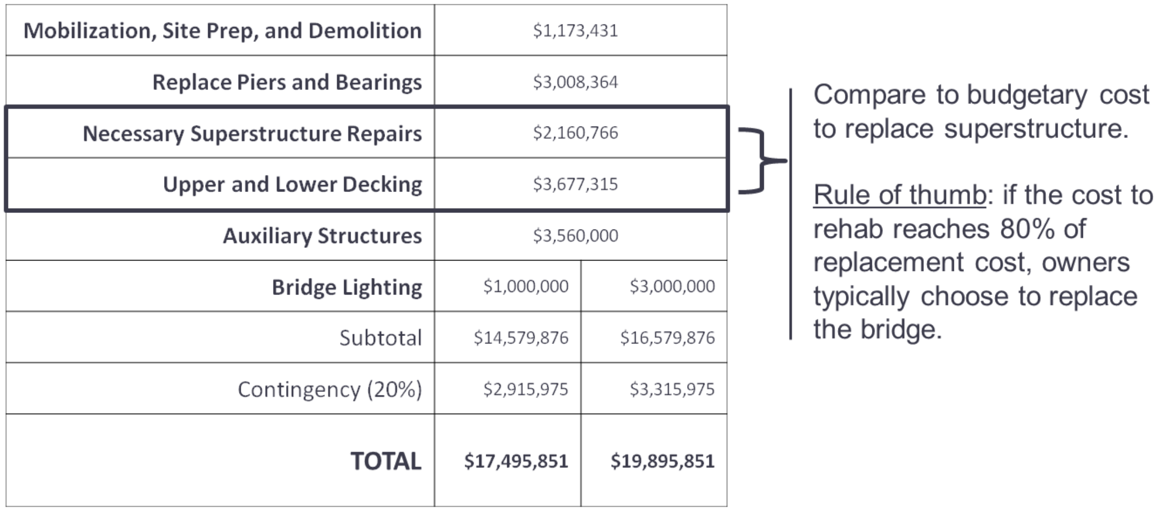
## 2.6 - Bridge Lighting

The criteria for lighting the bridge has not been established, and there is a wide range of possible levels of service. A minimal level of lighting is required for pedestrian and cyclist safety. Beyond the safety requirement, it may be desirable to position lighting fixtures in a manner that illuminates the structure to create a pleasing appearance. At the highest level of service, the bridge lighting could incorporate programmable, dimmable, LED lights that can be coordinated as a decorative feature display.

Although a cost estimate cannot be prepared without establishing the criteria for the lighting system, low-end and high-end values have been provided for budgetary purposes.

## Section 3 - Cost

A preliminary estimate of probable cost was developed in order to assist with developing a project budget. Where possible, the estimate utilizes regional unit cost data published by the Oklahoma Department of Transportation for local projects constructed during the past year. Many of the rehabilitation measures do not correspond to standard ODOT bid items. In some circumstances, the estimate is developed using R.S. Means construction cost data, based on presumed equipment and level of effort for each task. For some non-standard items the estimate was developed based on projects in other regions of the country, with prices adjusted to local market conditions using average geographical cost indexes.



| | | |
|---|---|---|
| **Mobilization, Site Prep, and Demolition** | $1,173,431 | |
| **Replace Piers and Bearings** | $3,008,364 | |
| **Necessary Superstructure Repairs** | $2,160,766 | |
| **Upper and Lower Decking** | $3,677,315 | |
| **Auxiliary Structures** | $3,560,000 | |
| **Bridge Lighting** | $1,000,000 | $3,000,000 |
| Subtotal | $14,579,876 | $16,579,876 |
| Contingency (20%) | $2,915,975 | $3,315,975 |
| **TOTAL** | **$17,495,851** | **$19,895,851** |

Compare to budgetary cost to replace superstructure.

Rule of thumb: if the cost to rehab reaches 80% of replacement cost, owners typically choose to replace the bridge.

Figure 5-12: Draft preliminary estimate of probable cost

The cost of rehabilitation should be compared against the replacement cost. Many bridge owners prefer to replace a bridge if the cost for rehabilitation approaches 70% to 80% of the cost of replacement.

*Duplicate Budget Items*

Several aspects of the proposed work would need to be conducted regardless of the decision to rehabilitate or replace the structure. These duplicate items can be separated out of the cost estimate in order to isolate the items that differentiate between rehab and replacement. The **duplicate items** include:

- *Mobilization, Site Access, and Demolition*
- *Bearings and Piers*
- *Auxiliary Structures*
- *Bridge Lighting*

The estimated cost of the duplicate items is $8.7 Million to $10.7 Million. This amount would be required regardless of whether the bridge is rehabilitated or replaced entirely.

*Superstructure Budget Items*

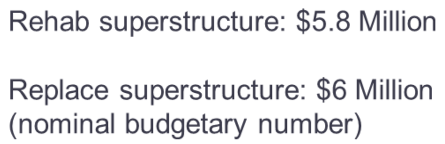
The two remaining budget items correspond entirely to **rehabilitation of the superstructure**, including:

- *Upper and Lower Deck Systems*
- *Necessary Structural Repairs*

The estimated cost to rehabilitate the superstructure is $5.8 Million. This amount is in addition to the cost to address the redundant items.

The cost to rehabilitate the superstructure should be compared against the cost to replace the superstructure.

For budgetary purposes, the estimated cost to replace the superstructure is $6 Million.



Rehab superstructure: $5.8 Million

Replace superstructure: $6 Million
(nominal budgetary number)

# Chapter 6 - Basis for Conceptual Design

## Section 1 - Preliminary Design Criteria

### 1.1 - Objective

The purpose of the preliminary design criteria is to document the design approach, design methodology, design assumptions and establish a preliminary governing criteria for the design of the Zink Bridge truss rehabilitation. The design of the substructure and superstructure shall conform to AASHTO LRFD Bridge Design Specifications, Seventh Edition, 2014, US Customary Units, as amended by AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, December 2009, with 2015 interim revisions.

The preliminary design criteria stipulated, herein, shall apply only to the design of the Zink Bridge truss rehabilitation as described in the subsequent sections of this document.

### 1.2 - Investigated Concepts

Several concepts were investigated as part of the conceptual design phase. These included the addition of a second deck above the top chord of the truss structures. The top chord deck is supported by W12 beams at panel points along the truss. The existing structure lateral bracing systems are severely compromised with widespread deterioration throughout; therefore, the structure was analyzed with no upper and lower lateral bracing systems. The proposed top deck effectively replaces the upper lateral bracing system and replacement of the bottom timber deck system with a rigid concrete deck would act similarly.

An effective rehabilitation strategy for truss type bridges is post tensioning. Several post tensioning schemes where investigated as follows:

1. Post tensioning along the bottom chord
2. Post tensioning along outer tension diagonal
3. Post tensioning along the inner tension diagonal
4. Draped post tensions path from PP0U to PP5L.

The objective of the post tensioning patterns was to reduce tensile forces in the fatigue sensitive members, such as the lower chords, which may have been compromised from previously implemented retrofits. Ultimately, post tensioning along the bottom chord of the truss was determined to be the most effective pattern.

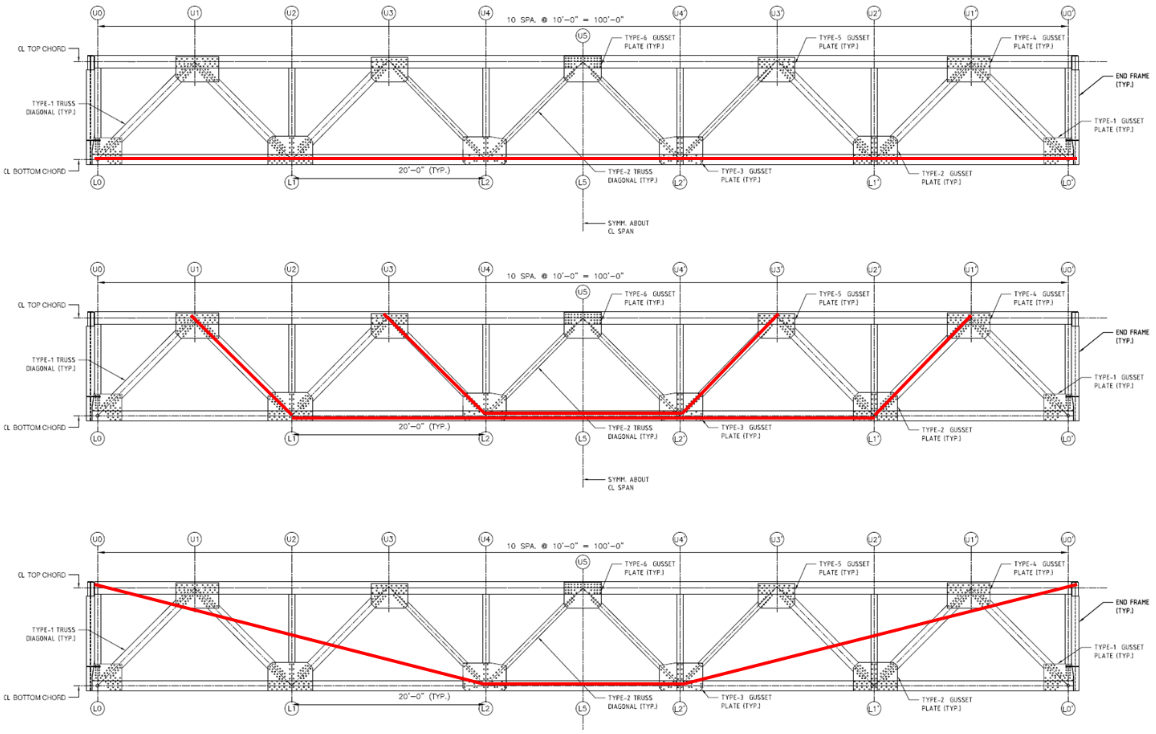


Figure 6-1: Post-tensioning Configurations

Several post-tensioning configurations were studied as a means to reduce fracture sensitivity.

## 1.3 - Standards and Codes

1. AASHTO LRFD Bridge Design Specifications, US Customary Units, 7th Edition, 2014 (referred to hereinafter as AASHTO LRFD).
2. AASHTO LRFD Guide Specifications for the Design of pedestrian Bridges, December 2009, with 2015 interim revisions (referred to hereinafter as AASHTO Pedestrian).
3. AASHTO Standard Specifications for Structural Supports for Highway, Signs, Luminaires, and Traffic Signals (referred to hereinafter as AASHTO Signs).
4. AASHTO Manual for Bridge Evaluation, 2nd Edition, 2014 (referred to hereafter as AASHTO MBE).
5. American Institute of Steel Construction, Steel Construction Manual, 13th Edition, 2005 (referred to hereinafter as AISC).
6. American Standards for Testing Materials (referred to hereafter as ASTM).
7. AISC Rehabilitation and Retrofit Guide (Steel Design Guide 15)
8. Units shall be US Customary units.

## 1.4 - Design Approach

In general, the design philosophy used shall be Load Resistance and Factor Design (LRFD). The design shall conform to the specifications of AASHTO LRFD and as modified by AASHTO Pedestrian and AASHTO Signs where appropriate. Material properties for the original base metal shall be determined using AASHTO MBE; similarly, the evaluation of the original gusset plates shall be in accordance with AAHSTO MBE. The extent of the design considered herein shall be the strength and service limit states under dead, live, wind and thermal loads as outlined in Section 1.6 Design Loads.

## 1.5 - Material and Material Properties

*Structural Steel*

*Existing Structural Steel*

The existing Zink Bridge truss members are assumed to be constructed from steel around 1904. In accordance with ASSHTO MBE, the follow material properties shall be used:

Fy = 26 ksi, yield stress of existing steel
Fu = 52 ksi, fracture stress of existing steel
Fr = 18, ksi, factored stress for existing rivets

*Rehabilitation Steel (Previous Rehabilitation)*

Structural steel used for the previous rehabilitation of the Zink Bridge is taken to be Grade 36 Steel and the following properties shall be used for analysis:

Fy = 36 ksi, minimum yield stress for new steel
Fu = 58 ksi, minimum facture stress for new steel

*Rehabilitation Steel*

Structural steel used for the rehabilitation of the Zink Bridge shall conform to AASHTO M270 Grade 50 Steel and the following properties shall be used:

Fy = 50 ksi, minimum yield stress for new steel
Fu = 65 ksi, minimum facture stress for new steel

Structural fasteners used for the rehabilitation of the Zink Bridge shall conform to ASTM A325 High Strength Bolts. The minimum tensile strength of high strength bolts shall be 120 ksi. Nuts for high strength bolts shall conform to ASTM A563 and washers shall conform to ASTM F436.

*Prestressing Steel*

Prestressing steel used for the rehabilitation of the Zink Bridge shall conform to ASTM A421 Low Relaxation Wire, 0.600 in diameter and minimum yield stress, fy = 270 ksi.

*Shear Connectors*

Shear connectors shall be provided between structural steel and structural concrete. Shear connections shall conform to ASTM A108, Fu = 65 ksi.

*Structural Concrete*

New structural concrete strength shall be a minimum of 4,000 psi concrete. Structural concrete shall be reinforced with reinforcing steel conforming to ASTM A615 and be Grade 60 steel (fy = 60 ksi).

## 1.6 - Design Loads

*Dead Loads (DC and DW)*

Unit dead loads are taken to be following:

| **Material** | **Unit Weight** |
|---|---|
| Structural Steel | 490 pcf |
| Structural Concrete (Reinforced) | 150 pcf |
| Structure Concrete (non-reinforced) | 145 pcf |
| Timber Ties | 50 pcf |
| Stay in Place Forms (SIP) | 10 psf |
| Timber Decking | 10 psf |

(Where pcf is defined as pounds per cubic foot and psf is defined as pounds per square foot) Existing truss member dead loads shall have a minimum contingency factor of 1.10 to account for batten plate, lacing bar and rivet dead load in the structural analysis.

Structural steel, structural concrete, timber ties and SIP forms shall be treated as DC dead loads in AASHTO LRFD Load Combinations. The existing timber decking, existing handrails and proposed handrails shall be treated as DW dead loads in AASHTO LRFD Load Combinations, to account for variability of the design.

*Live Loads (LL)*

*Pedestrian Loads*

Pedestrian live load shall be taken as 90 psf as defined in AASHTO Pedestrian. The following live load scenarios shall be considered:

1. Existing deck, fully loaded
2. Existing deck, right half loaded
3. Existing deck, left half loaded
4. Existing deck, front half loaded
5. Existing deck, back half loaded
6. New deck, fully loaded
7. New deck, right half loaded
8. New deck, left half loaded
9. New deck, front half loaded
10. New deck, back half loaded

The scenarios listed above were checked in conjunction with the applicable permutations of the other scenarios.

*Wind Loads (WS)*

*Design Wind Pressure*

Wind loads shall be computed in accordance with AASHTO Signs Articles 3.8.1 to 3.8.6. The basic wind speed shall be taken as 90 mph in the computation of the design wind pressure. Wind loads shall be computed on a per element basis and shall be applied in the transverse direction (perpendicular to traffic) in the structural analysis.

In accordance with AASHTO Pedestrian Article 3.4, an uplift (vertical) force of 0.020 ksf (20 psf) shall be applied to the full width of the proposed deck at the 1/4 points of deck, in the structural analysis.

*Thermal Loads (TU)*

*Coefficients of expansion*

1. Thermal expansion coefficient Structural Steel: 6.0 x $10^{-6}$ (in/in/$^{o}$F)
2. Thermal expansion coefficient Structural Concrete: 6.5 x $10^{-6.5}$(in/in/$^{o}$F)

The expansion coefficients listed above are used to determine deformations associated with uniform temperature changes.

*Design Temperature Changes*

The thermal loads shall be computed in accordance with Procedure A of AASHTO LRFD for “cold” climate. The temperature ranges for “Cold” Climate are given in AASHTO LRFD Table 3.12.2.1-1 and reiterated as follows:

Structural Steel Temperature Change:
1. 70$^{o}$F Temperature rise
2. 100$^{o}$F Temperature fall

Structural Concrete Temperature Change:
1. 30$^{o}$F Temperature rise
2. 70$^{o}$F Temperature fall

The assumed ambient temperature for Zink Bridge shall be taken as 60.8$^{o}$F, the average yearly temperature for the city of Tulsa, OK, as published by the National Oceanic and Atmosphere Administration.

*Load Combinations*

The load combinations for strength and service limit states shall be in accordance with AASHTO LRFD Table 3.4.1. The load combinations considered in the structural analysis are as follows:

1. Strength 1: 1.25 (0.9) DC + 1.5 (0.65) DW + 1.75 LL + 0.5 TU
2. Strength 3: 1.25 (0.9) DC + 1.5 (0.65) DW + 1.4 WS + 0.5 TU
3. Service 1: 1.0 DC + 1.0 DW + 1.0 LL + 0.3 WS + 1.2 TU

Where DC, DW, LL, WS and TU are previously defined.

The load combinations were investigated in two cases, existing and proposed. The proposed case included the addition of a second deck and attachments, above the top chord of the trusses. The results of these analyses are summarized in next section2 of this report.

## Section 2 - Structural Analysis

### 2.1 - Section Properties

*Built-up Members*

In general, the existing truss members of the Zink Bridge are built-up members consisting of back to back (B-B) channel or angle pieces, batten plates or lacing bars and cover plates. The following table summarizes the truss component built-up member pieces:

Table 6-1: Summary of Built-up Truss Members

| Truss Member | Components |
|---|---|
| Bottom Chord (PP0L – PP1L, PP4L – PP5L) | 4 B-B L6x3-1/2x1/2, 1/2 Batten PL |
| Bottom Chord (PP1 L – PP4L) | 4 B-B L6x3-1/2x1/2, 1/2 Batten PL, 1/2 x 13 Web Cover plate |
| Top Chord (PP0U – P1U, P5U – PP5U) | 2 B-B U15x39.8, 1/2 Lacing Bars, 1/2 x 17 Top Cover Plate |
| Top Chord (P1U – P5U) | 2 B-B U15x39.8, 1/2 Lacing Bars, 1/2 x 17 Top Cover Plate, 2 1/2 x 12 Web Cover Plates |
| Diagonal (PP0L – P2U, P4U – PP5L) | 4 B-B L6 x 3-1/2 x 1/2, 3/8 Batten PL |
| Diagonal (P2U-P4U) | 4 B-B L3-1/2 x 3-1/2 x 1/2, 3/8 Batten PL |
| Vertical (All PP) | 4 B-B L3-1/2 x 3-1/2 x 1/2, 3/8 Batten PL |

The bottom chord, diagonals and vertical are all of similar construction, 4 back to back angles with riveted batten plates; for example section see sketch below. The top chords consist of 2 back to back U sections with riveted top cover plates, bottom lacing bars and web cover plates (at the locations listed above); for example section sketch below.

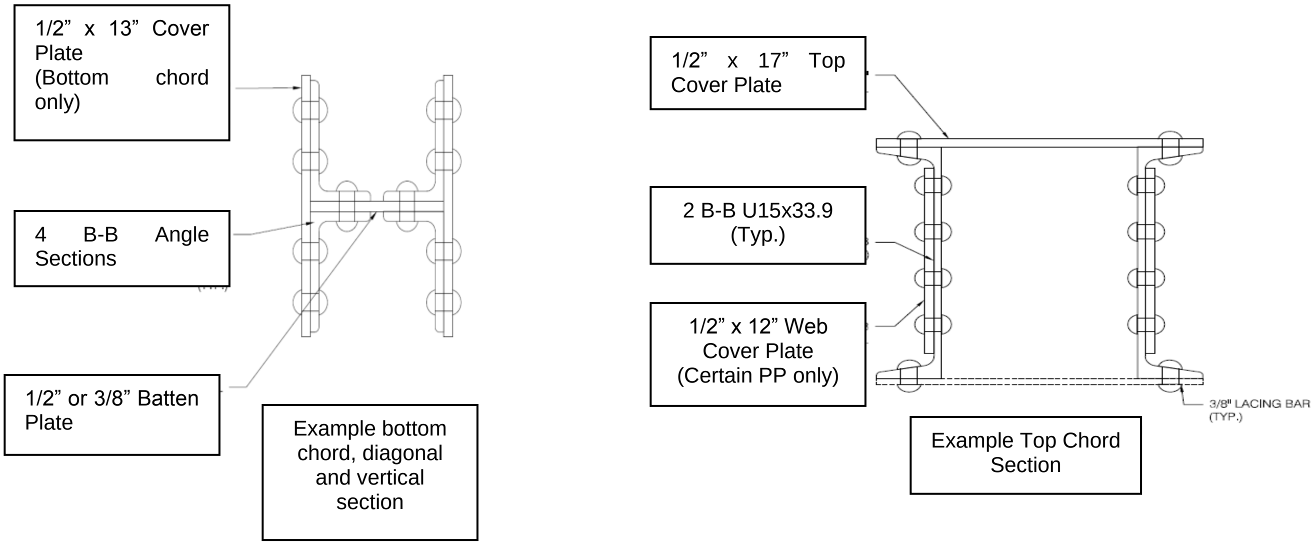


Figure 6-2: Example Built-Up Truss Member Sections

*a)* Bottom Chord, diagonal and vertical section *b)* top chord section

For built-up type member sections with batten plates and lacing bars, the section properties are not readily available for the composite section and therefore must be computed.

*Section Property Computations*

Based on Mechanics of Material theory the following section properties are required for the analysis and design of structural members:

A, Cross sectional area
Iz, Second moment of area about a horizontal axis through the CG of the section
Iy, Second moment of area about a vertical axis through the CG of the section
J, torsional constant of the section
Az, Shear area about a horizontal axis through the CG of the section
Ay, Shear area about a vertical axis through the CG of the section
Izy, Product of Iz and Iy, taken as 0 for the doubly symmetric sections

The cross section area, A, is defined as the total area of the section constituents of the built-up members. For plate type elements, the cross sectional area is defined as the product of the plate thickness and width. For angle and channel type sections, the cross sectional area may be obtained from the AISC historic database.

In order to determine Iz and Iy, the locations of the neutral axis is required. In Mechanics of Material theory, for a prismatic linear section, the location of the neutral axis coincides with the centroid under the assumption that plan sections remain plain. The centroid may located by computing the weighted average of the perpendicular distance away from an axis parallel to the neutral axis; where the weight is the sectional area of the constituent. Formulated mathematically,

Zbar = Sum(z*A) / Sum(A) and Ybar = Sum(y*A) / Sum(A)

Once the location of the natural axis is determined, the parallel axis theorem may be used to determine the Iz and Iy for the section. Recalling the parallel axis theorem mathematically,

$$I = Sum(I + Ad^2)$$

The value d is defined as the distance away from the axis under consideration and I is the second moment of inertia about an axis parallel to axis under consideration of the section constituent, in the equation above. The computations outlined above have been computed for the each built-up truss section.

*Equivalent Thickness of Batten Plates and Lacing Bars*

The effects of batten plates and lacing bars are quite complex. The method used to account for these effects considered herein consists of the following:

1. Compute an equivalent thickness of the batten plate and lacing bars as outlined in Torsion in Structures: An Engineering Approach by Kollbrunner, et al.
2. Compute gross section properties assuming the batten plates or lacing bars are plate elements of the thickness computed in 1.
3. Determine strength values by considering the stability of the built-up members with batten plates or lacing bars as outlined in Theory of Elastic Stability by Timenshenko, et al.

*Deterioration*

Zink Bridge main truss member exhibited areas of localized section loss. In order to account for the section loss, the section properties included areas of section loss as “negative” area and gross section properties were computed. These section properties were then used to generate capacity values and checked against member output forces from the Lusas Model as described in the next section. However, the section loss is considered minor enough to not explicitly be modeled in Lusas.

## 2.2 - Computer Modeling

*Linear Analysis Model*

A 3D finite element model of the Zink Bridge was created using Lusas*,* a 3D finite element structural analysis software. The geometry of the model was initially created using AUTOCAD drafting software and imported into Lusas. A typical 100 ft span of the Zink Bridge was analyzed in the both existing and proposed conditions. The structural members were modeled using 3D Thick Beam Element Meshes and were discretized into four (4) or more sections and each section was checked against the member capacity (see Section 2.3 for details). Cross sectional properties were computed, as outlined in Section 2.1, using a *Microsoft Excel* spreadsheet program and imported into Lusas. The proposed top deck was modeled using thick shell elements. Thick beam elements were assigned steel material properties and the top deck thick shell element was assigned concrete properties. See the figures below.

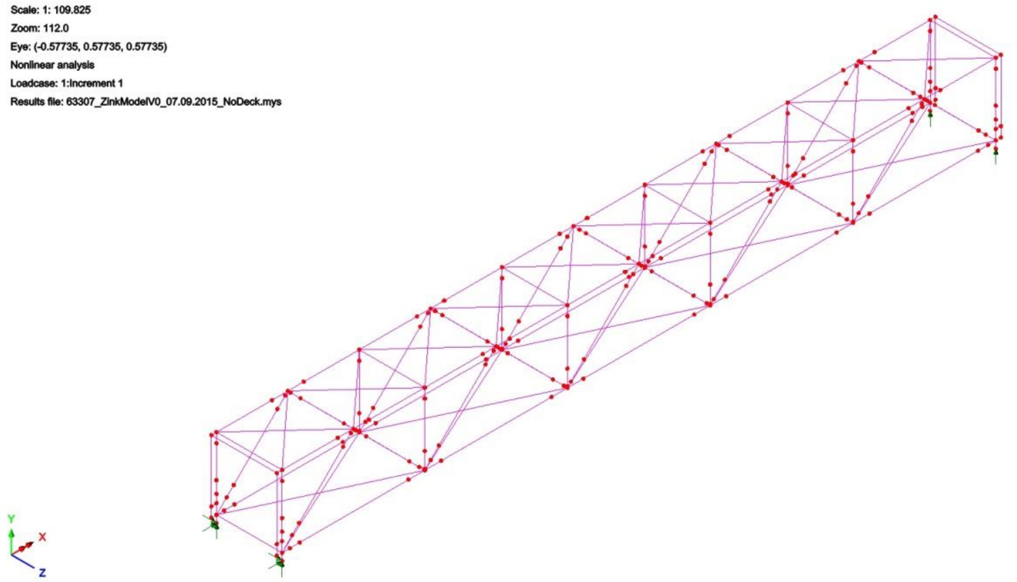


Figure 6-3: Existing Zink Bridge Model

The figure above depicts the existing Zink Bridge model. The features of this model include, main truss members (top chord, bottom chord, diagonals and verticals), upper and lower lateral bracing systems, retrofit upper struts and end portal frames. Structural members, as previously described, are modeled using 3D thick beam element meshes. A key feature of this model is the end portal frame, as shown in the figure below.

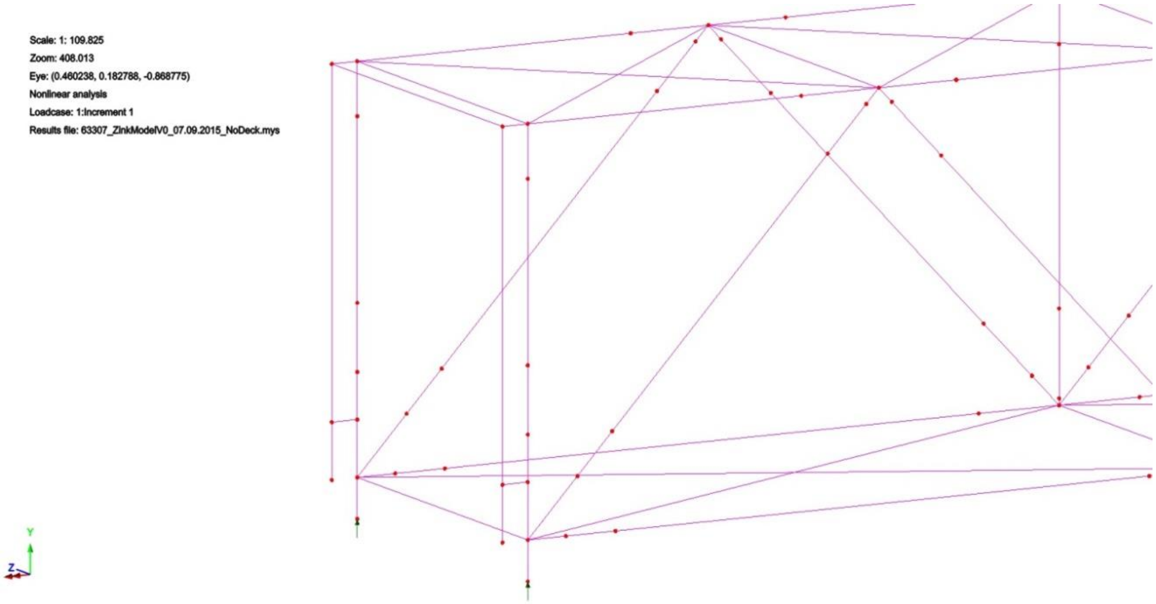


Figure 6-4: Detail View of End Portal Frame

The end portal frames were installed as part of an earlier retrofit strategy. End portal fames, referred to as end frames hereafter, consisted of a W18x50 strut (horizontal member) and two W10x49 columns. The strut was rigidly connected, through several welds, to the top chord members at the ends of the truss; whereas, the end frame columns were connected to the end truss verticals by stitch welds and a shear bar. To adequately model the existing conditions of the end frame, four stub connection elements were used. The top two stub connection elements retained the section properties of the top chord, to reflect the rigid connection between the end frame strut and top chord. The bottom two stub connection elements were give section properties equivalent to the shear plate element connecting the end fame columns to the end vertical. The stitch welds were ignored due to concerns of the existing truss metal weldability.

The proposed top deck configuration was modeled using 3D thick beam elements for the deck support stringers and a thick shell element for the concrete deck. A key feature of this model is the upper lateral bracing, retrofit strut and end frames have been removed. The figure below depicts the Zink Bridge proposed rehabilitation.

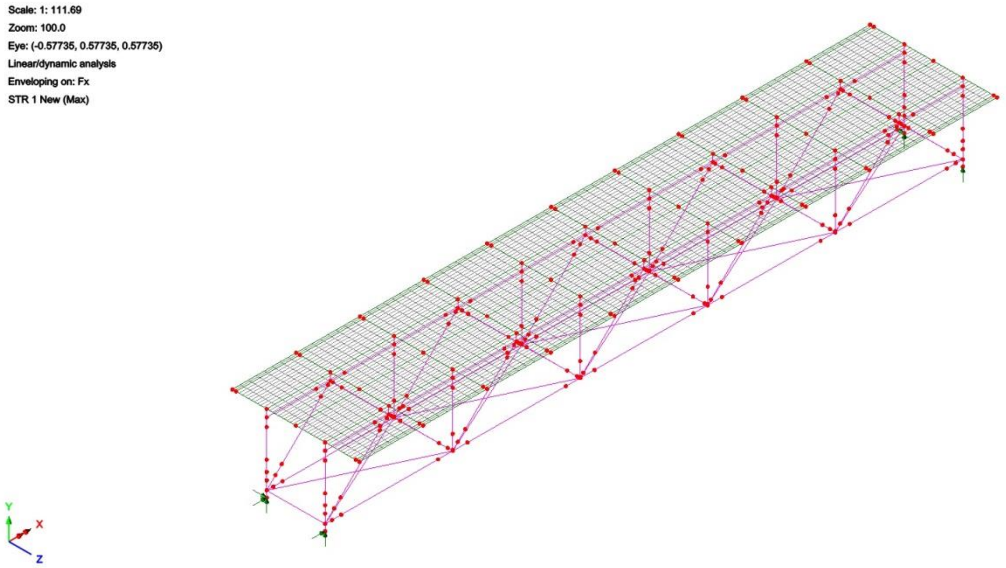


Figure 6-5: Proposed Zink Bridge Model

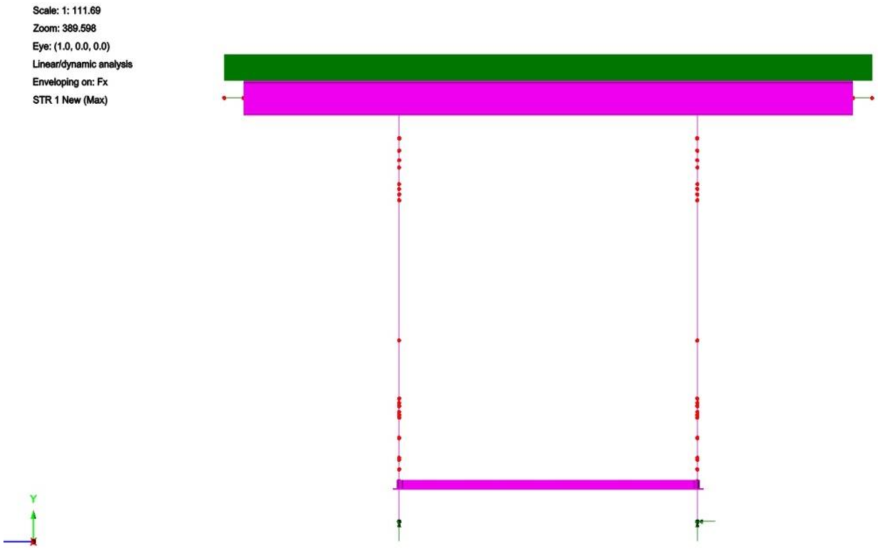


Figure 6-6: Detail View of Modeled Deck System

As shown in the figure above, the deck system is comprised of W12 support beams and a 9” thick concrete deck. The support beams were modeled using 3D thick beam elements and the concrete deck was modeled as a thick shell element.

A second order non-linear structural analysis of the proposed model was investigated to check second order effects, as described in the next section.

*Non-linear Analysis Model*

A second order non-linear structural analysis was investigated to check for structural stability and other second order effects, such as large displacements and member force magnification. The 3D thick beam element meshes were modified in Lusas to be 3D thick non-linear beam element meshes and the analysis was run, during which, the model converged to a solution. A non-stable structural system would fail to converge to a solution; therefore, the proposed top deck configuration is a stable structural system. The member capacities were checked at each discretized element section.

## 2.3 - Analysis Results

*Member Capacity Computations*

Member capacities for tension, compression, shear and bending forces were calculated in accordance with AASHTO LRFD. The member capacities were checked against *Lusas* output at each discretized section using a *Microsoft Excel* Spreadsheet program. Gusset plates were capacities were calculated by the procedures outlined in AASHTO MBE and check against applicable member forces. The following tables summarize the results of the member capacity checks.

Table 6-2: Capacity Summary 1

| | Existing | | | | Proposed | | | |
|---|---|---|---|---|---|---|---|---|
| Member | Strength | | Service | | Strength | | Service | |
| | (T) | (C) | (T) | (C) | (T) | (C) | (T) | (C) |
| BC1 | 0.291 | 0.358 | 0.321 | 0.358 | 0.133 | 0.000 | 0.400 | 0.000 |
| BC2 | 0.379 | 0.000 | 0.291 | 0.000 | 0.291 | 0.000 | 0.523 | 0.000 |
| DG1 | 0.389 | 0.285 | 0.306 | 0.141 | 0.675 | 0.768 | 0.568 | 0.535 |
| DG2 | 0.065 | 0.462 | 0.243 | 0.046 | 0.240 | 0.462 | 0.499 | 0.115 |
| TC1 | 0.410 | 0.654 | 0.435 | 0.473 | 0.435 | 0.473 | 0.387 | 0.650 |
| TC2 | 0.070 | 0.599 | 0.000 | 0.428 | 0.070 | 0.694 | 0.000 | 0.567 |
| VT1 | 0.243 | 0.592 | 0.000 | 0.339 | 0.243 | 0.592 | 0.000 | 0.484 |
| LLB | 0.000 | **9.961** | 0.000 | **8.071** | 0.376 | **10.055** | 0.000 | **8.071** |
| LLS | **2.713** | 0.000 | **1.222** | 0.000 | **3.743** | 0.000 | **1.323** | 0.000 |
| ULB | 0.656 | 0.999 | 0.000 | **1.854** | **1.083** | **1.260** | 0.000 | **1.937** |
| RFS | 0.000 | 0.293 | 0.061 | 0.075 | 0.000 | 0.394 | 0.101 | 0.115 |
| VT1_SL | 0.243 | 0.592 | 0.000 | 0.339 | 0.243 | 0.592 | 0.000 | 0.484 |
| DG1_SL | 0.039 | 0.302 | 0.028 | 0.090 | 0.028 | 0.885 | 0.000 | 0.540 |

Table 3-2 summarizes the capacities under the following conditions:

1. Dead, Live, Wind and Temperature loads are considered (see Chapter 3, Section 1).
2. Upper Lateral Bracing, Lower Lateral Bracing, Lower Lateral Struts, Retrofit Struts and end frames are included.
3. Non-linear effects are not considered
4. (T) denotes tensile axial force (C) denotes compressive axial force.

The entries in Table 3-2 are the maximum demand to capacity ratios for Strength 1 and Strength 3 and Service 1 limit states. The existing condition refers to the model results with only one deck on the bottom chord. The proposed condition includes the new top deck. A demand to capacity ratio greater than 1 indicates the member is overloaded and must be rehabilitated or replaced.

Members are as defined as below:

BC1 - Bottom chord from PP0 to PP1, PP4 to PP5
BC2 - All other Bottom Chord Members
DG1 - Diagonal members from PP0L - P1U, P1U - PP1L, PP1L - P2U, P4U - PP4L, PP4L - P5U, P5U - PP5L
DG2 - All other Diagonal members
VT1 - Vertical members
LLB - Lower later Bracing members (X members)
LLS - Lower lateral Struts
ULB - Upper later bracing members (X and Struts)

RFS - Retrofit Strut Members
VT1_SL - Vertical with section loss considered
DG1_SL - Diagonal with SL Considered

Table 6-3: Capacity Summary 2

| Member | Proposed | | | |
|---|---|---|---|---|
| | Strength | | Service | |
| | (T) | (C) | (T) | (C) |
| BC1 | 0.323 | 0.000 | 0.684 | 0.000 |
| BC2 | 0.750 | 0.000 | 0.939 | 0.000 |
| DG1 | 0.599 | 0.838 | 0.401 | 0.527 |
| DG2 | 0.064 | 0.159 | 0.325 | 0.090 |
| TC1 | 0.006 | 0.000 | 0.179 | 0.485 |
| TC2 | 0.688 | 0.456 | 0.000 | 0.633 |
| VT1 | 0.243 | 0.592 | 0.000 | 0.484 |
| LLB | 0.884 | **3.497** | 0.709 | 0.000 |
| LLS | **3.743** | 0.000 | **1.323** | 0.000 |
| VT1_SL | 0.039 | 0.302 | 0.028 | 0.090 |
| DG1_SL | 0.243 | 0.592 | 0.000 | 0.339 |

Table 3-3 summarizes the capacities under the following conditions:

1. Dead, Live, Wind and Temperature loads are considered (see Chapter 3, Section 1).
2. Upper Lateral Bracing, Lower Lateral Bracing, Lower Lateral Struts, Retrofit Struts and end frames are removed.
3. Non-linear effects are not considered
4. (T) denotes tensile axial force (C) denotes compressive axial force.

The entries in Table 3-2 are the maximum demand to capacity ratios for Strength 1 and Strength 3 and Service 1 limit states. The proposed condition includes the new top deck. A demand to capacity ratio greater than 1 indicates the member is overloaded and must be rehabilitated or replaced.

Members are as defined as below:

BC1 - Bottom chord from PP0 to PP1, PP4 to PP5
BC2 - All other Bottom Chord Members
DG1 - Diagonal members from PP0L - P1U, P1U - PP1L, PP1L - P2U, P4U - PP4L, PP4L - P5U, P5U - PP5L
DG2 - All other Diagonal members
VT1 - Vertical members
LLB - Lower later Bracing members (X members)
LLS - Lower lateral Struts
ULB - Upper later bracing members (X and Struts)
RFS - Retrofit Strut Members
VT1_SL - Vertical with section loss considered
DG1_SL - Diagonal with SL Considered

Table 6-4: Capacity Summary 3

| Member | Proposed Strength | |
|---|---|---|
| | (T) | (C) |
| BC1 | 0.000 | 0.000 |
| BC2 | 0.806 | 0.000 |
| DG1 | 0.579 | 0.903 |
| DG2 | 0.943 | 0.000 |
| TC1 | 0.479 | 0.000 |
| TC2 | 0.383 | 0.353 |
| VT1 | 0.243 | 0.592 |
| LLB | 0.770 | 0.000 |
| LLS | 0.000 | 0.444 |
| VT1_SL | 0.243 | 0.592 |
| DG1_SL | 0.039 | 0.302 |

Table 3-3 summarizes the capacities under the following conditions:

1. Dead, Live, Wind and Temperature loads are considered (see Chapter 3, Section 1).
2. Upper Lateral Bracing, Lower Lateral Bracing, Lower Lateral Struts, Retrofit Struts and end frames are removed.
3. Non-linear effects are considered
4. (T) denotes tensile axial force (C) denotes compressive axial force.

The entries in Table 3-2 are the maximum demand to capacity ratios for Strength 1 limit state. The proposed condition includes the new top deck. A demand to capacity ratio greater than 1 indicates the member is overloaded and must be rehabilitated or replaced.

Members are as defined as below:

BC1 - Bottom chord from PP0 to PP1, PP4 to PP5
BC2 - All other Bottom Chord Members
DG1 - Diagonal members from PP0L - P1U, P1U - PP1L, PP1L - P2U, P4U - PP4L, PP4L - P5U, P5U - PP5L
DG2 - All other Diagonal members
VT1 - Vertical members
LLB - Lower later Bracing members (X members)
LLS - Lower lateral Struts
ULB - Upper later bracing members (X and Struts)
RFS - Retrofit Strut Members
VT1_SL - Vertical with section loss considered
DG1_SL - Diagonal with SL Considered

Table 6-5: Capacity Summary 4

| | Proposed | |
|---|---|---|
| Member | Strength | |
| | (T) | (C) |
| BC1 | 0.000 | 0.000 |
| BC2 | 0.286 | 0.000 |
| DG1 | 0.301 | 0.633 |
| DG2 | 0.943 | 0.000 |
| TC1 | 0.136 | 0.000 |
| TC2 | 0.383 | 0.353 |
| VT1 | 0.243 | 0.592 |
| LLB | 0.689 | **2.790** |
| LLS | **1.714** | 0.516 |
| VT1_SL | 0.243 | 0.592 |
| DG1_SL | 0.039 | 0.302 |

Table 3-3 summarizes the capacities under the following conditions:

1. Dead, Live, Wind and Temperature loads are considered (see Chapter 3, Section 1).
2. Upper Lateral Bracing, Lower Lateral Bracing, Lower Lateral Struts, Retrofit Struts and end frames are removed.
3. Non-linear effects are considered
4. (T) denotes tensile axial force (C) denotes compressive axial force.

The entries in Table 3-2 are the maximum demand to capacity ratios for Strength 3 limit state. The proposed condition includes the new top deck. A demand to capacity ratio greater than 1 indicates the member is overloaded and must be rehabilitated or replaced.

Members are as defined as below:

BC1 - Bottom chord from PP0 to PP1, PP4 to PP5
BC2 - All other Bottom Chord Members
DG1 - Diagonal members from PP0L - P1U, P1U - PP1L, PP1L - P2U, P4U - PP4L, PP4L - P5U, P5U - PP5L
DG2 - All other Diagonal members
VT1 - Vertical members
LLB - Lower later Bracing members (X members)
LLS - Lower lateral Struts
ULB - Upper later bracing members (X and Struts)
RFS - Retrofit Strut Members
VT1_SL - Vertical with section loss considered
DG1_SL - Diagonal with SL Considered

### *Conclusions*

### *Member Capacity Evaluation*

From the analysis and capacity results presented herein, it is demonstrated that the Upper and Lower lateral bracing members are not adequate and should therefore be replaced or removed. The main truss members, top and bottom chords, diagonals and verticals remain adequate in the proposed condition.

### *End Frames and Retrofit Struts*

The analysis of the existing structure is relying on the adequate performance of the end fames and the retrofit struts. Unfortunately, these members are not performing adequately. The end frames and retrofit struts have been comprised through systemic flaws, such as cracked welds and poor welded connections, as documented in the *Zink Bridge Inspection Report*. The rehabilitation concepts provide means to address these flawed systems.

### *Existing Substructure*

There are no reasonable ways to properly ascertain the strength of unreinforced cracked concrete, because substructure exhibits several severe systemic flaws, such as widespread spalling; vertical, horizontal and diagonal type cracking and delamination, an analysis was not performed. It is recommended the substructure be replaced in its entirety as discussed in the rehabilitation concepts.

### *Bearings*

Existing Zink Bridge bearings offer no lateral restraint and represent a serious structural liability. Evidence of bearing “walking” is widespread throughout the structure as documented in *Zink Bridge Inspection Report.* The condition may be further exacerbated in a seismic type analysis, although not performed as part of this conceptual design report, recurring plastic deformation during a seismic event could potentially unseat the bridge, effectively forming a collapse mechanism. It is recommended the bearings be replaced as discussed in the rehabilitation concepts.

### *Gusset Plate Evaluation*

Gusset plate strength values were computed in accordance with AASHTO MBE and were checked with the appropriate member loads from the analyses shown above. Also, rivet connections were analyzed using the elastic vector method for eccentrically loaded bolt groups as outlined in AISC. Section loss and other gusset plate deterioration was considered. From the analysis investigated herein, the gusset plate steel is adequate. However, many riveted connections are not adequate and require rehabilitation.

# Chapter 7 - Subsurface Investigation Report (By Others)

A subsurface investigation was conducted by others as part of a project at the adjacent Zink Dam, and not in conjunction with the Zink Bridge inspection. The report is provided for informational purposes only.

# Geotechnical Drilling and Laboratory Testing

## Proposed Zink Dam Modifications

## Tulsa, Oklahoma

August 30, 2012
Terracon Project No. 04125129

**Prepared for:**
Tulsa County
Tulsa, Oklahoma

**Prepared by:**
Terracon Consultants, Inc.
Tulsa, Oklahoma

August 30, 2012

Tulsa County
500 South Denver Avenue, Room 322
Tulsa, Oklahoma 74103-3832

Attn: Ms. Linda Dorrell, Director of Purchasing
P: 918.596.5022
ldorrell@tulsacounty.org

Re: Results of Geotechnical Drilling and Laboratory Testing
Zink Dam Modifications
Tulsa, Oklahoma
Terracon Project No. 04125129

Dear Ms. Dorrell:

This letter presents a summary of our geotechnical exploration for the above project. Our services were performed in general accordance with Terracon Proposal No. P04120169R dated May 1, 2012. Boring logs, rock core photographs, test pit photographs, and laboratory test results are attached. This submittal also includes compact discs with downhole camera video.

## Subsurface Exploration Procedures

We drilled three borings (designated B-1, B-2, and B-3) to depths of approximately 35 to 40 feet below the existing ground surface on May 29, 2012. We used track-mounted, all-terrain, rotary drill rigs equipped with continuous flight augers and rotary cutting bits to advance the boreholes. We collected samples of the overburden soils using a split-barrel sampler, and we cored the bedrock using a NQ-size, diamond-bit core barrel. After the core samples were retrieved, the cores were placed in a box and logged. The rock was visually classified, and the "percent recovery" and rock quality designation (RQD) was determined for each run.

The "percent recovery" is the ratio of the recovered sample length to the cored length, expressed as a percent. An indication of the actual in-situ rock quality is provided by calculating the core's RQD. The RQD is the percentage of the length of broken cores retrieved which have core segments at least 4 inches in length compared to each core run length.

After drilling the borings, we used a downhole camera to collect video of the subsurface conditions within each borehole. We grouted the borings after downhole camera operations were complete.

In addition to the borings, we excavated eleven test pits (designated TP-1 through TP-11) using a track-mounted excavator (trackhoe) to depths of approximately 1.5 to 9 feet. We collected disturbed samples directly from the trackhoe bucket.

Terracon geotechnical engineers logged the borings and test pits in the field. A representative from the Tulsa County survey department provided ground surface elevations and horizontal locations.

## Laboratory Testing

Samples retrieved during the field exploration were taken to the laboratory for further observation by the project geotechnical engineer and were classified in accordance with the Unified Soil Classification System (USCS). Select samples were tested for moisture content and grain size distribution.

Bedrock materials were classified according to the General Notes and described using commonly accepted geotechnical terminology.

The laboratory test results are presented on the boring logs next to the respective samples and attached grain size distribution curves. Laboratory tests were performed in general accordance with the applicable ASTM, local or other accepted standards.

## General Comments

The scope of services for this project does not include either specifically or by implication any environmental assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This letter has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made.

We trust this provides you with the information you require at this time. If you have any questions regarding the contents of this letter, please do not hesitate to contact us.

Sincerely,
**Terracon Consultants, Inc.**
*Cert. of Auth. #CA-4531 exp. 6/30/13*

8/30/12
CONRAD S.
KOEHLER
20784

AFath

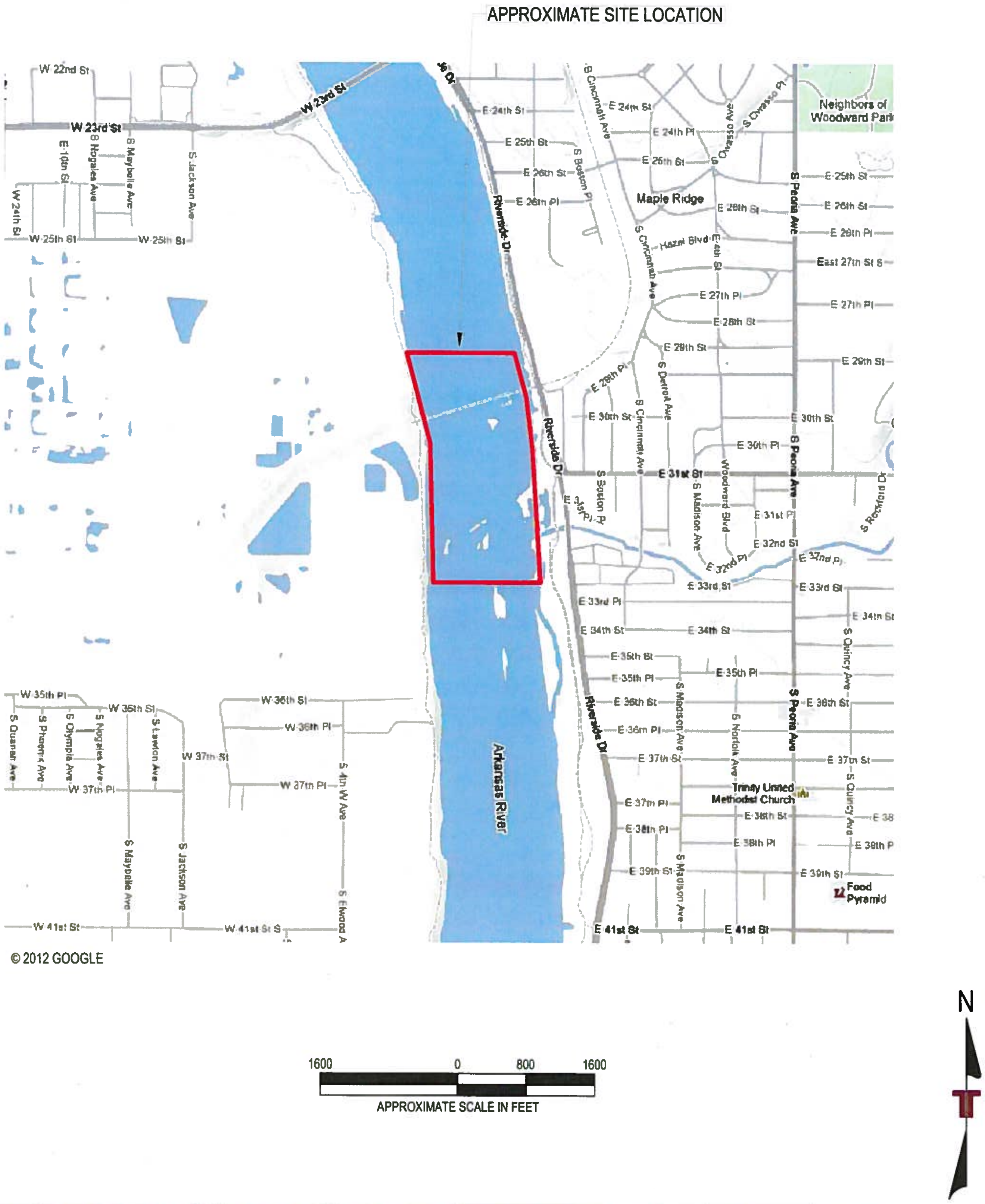
Atefeh Fathi, E.I.
Project Manager

Conrad S. Koehler, P.E.
Oklahoma No. 20784

AF:CSK:tm

Enclosures
- Figure A-1: Site Location Map
- Figure A-2: Boring Location Plan
- Boring Logs
- Grain Size Distribution Curves
- Rock Core Photographs
- Test Pit Photographs
- General Notes
- Compact Discs with Downhole Camera Video

Copies to: Addressee (3 via US Mail and 1 via email)



© 2012 GOOGLE

Project Mngr: AF
Drawn By: DC
Checked By: AF
Approved By: CSK

Project No. 04125129
Scale: SEE BAR SCALE
File No. 04125129
Date: AUGUST 2012

Terracon
Consulting Engineers and Scientists
10930 EAST 56th STREET TULSA, OKLAHOMA 74146
PH. (918) 250-0461 FAX. (918) 250-4570

SITE LOCATION MAP
GEOTECHNICAL EXPLORATION
ZINK DAM MODIFICATIONS
TULSA, OKLAHOMA

EXHIBIT NO.
A-1

AERIAL PHOTOGRAPH PROVIDED BY INCOG
SURVEY DATA PROVIDED BY TULSA COUNTY

N

**LEGEND**

- ● BORING LOCATION
- ☒ TEST PIT LOCATION

400 0 200 400
APPROXIMATE SCALE IN FEET

DIAGRAM IS FOR GENERAL LOCATION ONLY,
AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

| | | | |
|---|---|---|---|
| Project Mngr: AF | Project No. 04125129 | | |
| Drawn By: DC | Scale: SEE BAR SCALE | | |
| Checked By: AF | File No. 04125129 | | |
| Approved By: CSK | Date: AUGUST 2012 | | |

Terracon
Consulting Engineers and Scientists
10930 EAST 56th STREET TULSA, OKLAHOMA 74146
PH. (918) 250-0461 FAX. (918) 250-4570

# BORING LOCATION PLAN

GEOTECHNICAL EXPLORATION
## ZINK DAM MODIFICATIONS
TULSA, OKLAHOMA

EXHIBIT NO.
A-2

# LOG OF BORING NO. B-1

Page 1 of 2

| | |
|---|---|
| CLIENT<br>Tulsa County | |
| SITE Zink Dam<br>Tulsa, Oklahoma | PROJECT<br>Proposed Zink Dam Modifications |

| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 612.5 ft | DEPTH, ft. | USCS SYMBOL | SAMPLES NUMBER | SAMPLES TYPE | SAMPLES RECOVERY, in. | SAMPLES SPT-N BLOWS / ft. | TESTS WATER CONTENT, % | TESTS DRY UNIT WT pcf | TESTS UNCONFINED COMPRESSION, psf | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | **WELL GRADED SAND**<br>brown ▽ | | | | PA | | | | | | |
| 3 | 609.5 | | SW | 1 | AS | | | 11 | | | S-1<br>-#200=2% |
| 5 | **SANDSTONE++**<br>with thin interbedded shale seams, light gray, cemented<br>607.5 | | | | PA | | | | | | |
| 10 | light gray, fine-grained, unweathered, cemented<br>-joint at 5.1', 0°-5°, rough natural fractures<br>-joint at 5.4', 0°-5°, rough natural fractures<br>-joint at 5.8', 0°-5°, rough natural fractures<br>-joint at 6.5', 0°-5°, rough natural fractures<br>-joint at 8.5', 0°-5°, rough natural fractures<br>602.5 | 5 | | 2 | DB | 97% | RQD 90% | | | | |
| 15 | light gray, fine-grained, unweathered, cemented<br>-joint at 12.2', 0°-5°, rough natural fractures<br>-joint at 14.4', 0°-5°, rough natural fractures<br>597.5 | 10 | | 3 | DB | 100% | RQD 100% | | | | |
| 20 | light gray, fine-grained, unweathered, cemented<br>-joint at 16.4', 0°-5°, rough natural fractures<br>-joint at 16.5', 0°-5°, rough natural fractures<br>-joint at 19.3', 0°-5°, rough natural fractures<br>592.5 | 15 | | 4 | DB | 99% | RQD 98% | | | | |
| 25 | light gray, fine-grained, unweathered, cemented<br>-joint at 20.8', 0°-5°, rough natural fractures<br>-joint at 20.9', 0°-5°, rough natural fractures<br>-joint at 22.7', 0°-5°, rough natural fractures<br>-joint at 22.9', 0°-5°, rough natural fractures<br>-joint at 23.3', 0°-5°, rough natural fractures<br>-joint at 24.7', 0°-5°, rough natural fractures<br>587.5 | 20 | | 5 | DB | 100% | RQD 99% | | | | |
| 30 | light gray, fine-grained, unweathered, cemented<br>-joint at 26.4', 0°-5°, rough natural fractures<br>-joint at 27.5', 0°-5°, rough natural fractures<br>-joint at 27.8', 0°-5°, rough natural fractures<br>582.5 | 25 | | 6 | DB | 99% | RQD 99% | | | | |
| | **Continued Next Page** | 30 | | 7 | DB | 97% | RQD 97% | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | | |
|---|---|---|---|
| WL | ▽ 2 | WD | ▼ |
| WL | ▼ | | ▼ |
| WL | | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | ATV | FOREMAN | VER |
| APPROVED | CSK | JOB # | 04125129 |

BOREHOLE BORING LOGS.GPJ 2011 TULSA.GDT 8/24/12

# LOG OF BORING NO. B-1

Page 2 of 2

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| GRAPHIC LOG | DESCRIPTION | | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf |
| 34 | light gray, fine-grained, unweathered, cemented | 578.5 | | | | | | | | | |
| 35 | -joint at 30.5', 0°-5°, rough natural fractures<br>-joint at 30.8', 0°-5°, rough natural fractures<br>-joint at 31.1', 0°-5°, rough natural fractures<br>-joint at 31.5', 0°-5°, rough natural fractures<br>-joint at 32.0', 0°-5°, rough natural fractures<br>-joint at 32.1', 0°-5°, rough natural fractures<br>-joint at 32.5', 0°-5°, rough natural fractures<br>-joint at 32.7', 0°-5°, rough natural fractures<br>-joint at 33.5', 0°-5°, rough natural fractures<br>-joint at 33.9', 0°-5°, rough natural fractures<br>**SHALE++**<br>dark gray, fine-grained, moderately weathered, moderately hard<br>-joint at 34.2', 0°-5°, rough natural fractures<br>BOTTOM OF BORING<br><br>++Classification estimated from core samples. Petrographic analysis may reveal other rock types | 577.5 | 35 | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | | |
|---|---|---|---|
| WL | ▽ 2 | WD | ▼ |
| WL | ▼ | | ▼ |
| WL | | | |

**Terracon**

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | ATV | FOREMAN | VER |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. B-2

Page 1 of 3

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

GRAPHIC LOG | DESCRIPTION | DEPTH, ft.

Approx. Surface Elev.: 610.2 ft

**SANDSTONE++**
with thin interbedded shale seams, light gray, cemented

3 607

light gray, fine-grained, unweathered, cemented
-joint at 3.8', 0°-5°, rough natural fractures
-joint at 4.0', 0°-5°, rough natural fractures
-joint at 4.2', 0°-5°, rough natural fractures
-joint at 4.25', 0°-5°, rough natural fractures
8 602
-joint at 4.3', 10°-20°, rough natural fractures
-joint at 4.5', 0°-5°, rough natural fractures
-joint at 4.7', 10°-20°, rough natural fractures
-joint at 4.85', 0°-5°, rough natural fractures
-joint at 4.9', 0°-5°, rough natural fractures
13 597
-joint at 5.2', 0°-5°, rough natural fractures
-joint at 5.3', 0°-5°, rough natural fractures
-joint at 5.33', 0°-5°, rough natural fractures
-joint at 5.8', 0°-5°, rough natural fractures
-joint at 6.4', 0°-5°, rough natural fractures
-joint at 6.7', 0°-5°, rough natural fractures
-joint at 7.2', 0°-5°, rough natural fractures
18 592
-joint at 7.5', 0°-5°, rough natural fractures

light gray, fine-grained, unweathered, cemented
-joint at 8.05', 0°-5°, rough natural fractures
-joint at 10', 0°-5°, rough natural fractures
-joint at 11.7', 0°-5°, rough natural fractures
23 587
-joint at 12.3', 0°-5°, rough natural fractures

light gray, fine-grained, unweathered, cemented
-joint at 13.5', 0°-5°, rough natural fractures
-joint at 16', 0°-5°, rough natural fractures
-joint at 16.7', 0°-5°, rough natural fractures
-joint at 17.4', 0°-5°, rough natural fractures
28 582
-joint at 17.7', 0°-5°, rough natural fractures

Depth scale: 5, 10, 15, 20, 25, 30

**Continued Next Page**

| USCS SYMBOL | SAMPLES NUMBER | SAMPLES TYPE | SAMPLES RECOVERY, in. | SAMPLES SPT-N BLOWS / ft. | TESTS WATER CONTENT, % | TESTS DRY UNIT WT pcf | TESTS UNCONFINED COMPRESSION, psf |
|---|---|---|---|---|---|---|---|
| | | PA | | | | | |
| | 1 | SS | 0 | 50/3" | | | |
| | | PA | | | | | |
| | 2 | SS | 3 | 50/3" | | | |
| | | DB | 97% | RQD 70% | | | |
| | | DB | 99% | RQD 98% | | | |
| | | DB | 100% | RQD 99% | | | |
| | | DB | 100% | RQD 91% | | | |
| | | DB | 100% | RQD 92% | | | |
| | | DB | 97% | RQD 77% | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | | |
|---|---|---|---|
| WL | ▽ 0 | WD | ▼ |
| WL | ▼ | | ▼ |
| WL | | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | ATV | FOREMAN | ZMT |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. B-2

Page 2 of 3

CLIENT
**Tulsa County**

SITE
**Zink Dam**
**Tulsa, Oklahoma**

PROJECT
**Proposed Zink Dam Modifications**

| GRAPHIC LOG | | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf |
| 33 | 577 | | | | DB | 87% | RQD 63% | | | |
| 35 | 575 | 35 | | | | | | | | |
| 38 | 572 | | | | | | | | | |

DESCRIPTION

light gray, fine-grained, unweathered, cemented
-joint at 19.4', 0°-5°, rough natural fractures
-joint at 20.8', 0°-5°, rough natural fractures
-joint at 20.85', 0°-5°, rough natural fractures
-joint at 20.9', 0°-5°, rough natural fractures
-joint at 20.95', 0°-5°, rough natural fractures
-joint at 21.15', 0°-5°, rough natural fractures
-joint at 21.5', 0°-5°, rough natural fractures
-joint at 22.7', 0°-5°, rough natural fractures

light gray, fine-grained, unweathered, cemented
-joint at 24.7', 0°-5°, rough natural fractures
-joint at 25.4', 0°-5°, rough natural fractures
-joint at 25.5', 0°-5°, rough natural fractures
-joint at 25.6', 0°-5°, rough natural fractures
-joint at 27.5', 0°-5°, rough natural fractures

light gray, fine-grained, unweathered, cemented
-joint at 28.5', 5°-10°, rough natural fractures
-joint at 28.6', 5°-10°, rough natural fractures
-joint at 29.4', 0°-5°, rough natural fractures
-joint at 30', 0°-5°, rough natural fractures
-joint at 30.02', 0°-5°, rough natural fractures
-joint at 30.5', 0°-5°, rough natural fractures
-joint at 30.7', 0°-5°, rough natural fractures
-joint at 31', 0°-5°, rough natural fractures
-joint at 31.3', 0°-5°, rough natural fractures
-joint at 32', 0°-5°, rough natural fractures
-joint at 32.85', 0°-5°, rough natural fractures
-joint at 32.95', 0°-5°, rough natural fractures

light gray, fine-grained, unweathered, cemented
no joint

**Continued Next Page**

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | | |
|---|---|---|---|
| WL | ▽ 0 | WD | ▼ |
| WL | ▼ | | ▽ |
| WL | | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | ATV | FOREMAN | ZMT |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. B-2

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| GRAPHIC LOG | DESCRIPTION | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| | **SHALE++**<br>dark gray, fine-grained, moderately weathered, moderately hard<br>-joint at 35.1', 0°-5°, rough natural fractures<br>-joint at 35.3', 0°-5°, rough natural fractures<br>-joint at 35.54', 0°-5°, rough natural fractures<br>-joint at 36', 0°-5°, rough natural fractures<br>-joint at 36.2', 0°-5°, rough natural fractures<br>-joint at 37.95', 0°-5°, rough natural fractures<br>BOTTOM OF BORING<br>• ++Classification estimated from core samples. Petrographic analysis may reveal other rock types | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | | |
|---|---|---|---|
| WL | ▽ 0 | WD | ▼ |
| WL | ▼ | | ▽ |
| WL | | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | ATV | FOREMAN | ZMT |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. B-3

Page 1 of 2

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

DESCRIPTION — Approx. Surface Elev.: 606.9 ft

| Depth, ft | DESCRIPTION | Elev. |
|---|---|---|
| 2 | **WELL GRADED SAND**<br>brown, loose | 605 |
| 4 | **SANDSTONE++**<br>with thin interbedded shale seams, light gray, cemented | 603 |
| 5 | light gray, fine-grained, unweathered, cemented<br>no joint | 602 |
| 10 | light gray, fine-grained, unweathered, cemented<br>-joint at 5.85', 0°-5°, rough natural fractures<br>-joint at 7', 10°-15°, rough natural fractures<br>-joint at 7.05', 0°-5°, rough natural fractures | 597 |
| 15 | light gray, fine-grained, unweathered, cemented<br>-joint at 10.4', 0°-5°, rough natural fractures<br>-joint at 11.2', 0°-5°, rough natural fractures<br>-joint at 11.5', 0°-5°, rough natural fractures<br>-joint at 11.8', 0°-5°, rough natural fractures | 592 |
| 20 | -joint at 13.3', 0°-5°, rough natural fractures<br>-joint at 14.3', 0°-5°, rough natural fractures<br>-joint at 14.35', 0°-5°, rough natural fractures<br>-joint at 14.4', 5°-10°, very rough natural fractures<br>-joint at 14.5', 5°-10°, rough natural fractures | 587 |
| 25 | light gray, fine-grained, unweathered, cemented<br>-joint at 15.9', 0°-5°, rough natural fractures<br>-joint at 16.2', 0°-5°, rough natural fractures<br>-joint at 16.3', 0°-5°, rough natural fractures<br>-joint at 17.9', 0°-5°, rough natural fractures | 582 |
| 30 | light gray, fine-grained, unweathered, cemented<br>-joint at 21.1', 0°-5°, rough natural fractures<br>-joint at 22.85', 0°-5°, rough natural fractures<br>-joint at 24.8', 0°-5°, rough natural fractures | 577 |

| DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| | SW | 1 | PA<br>SS | 2 | 5 | 8 | | | S-1<br>-#200=6% |
| | | 2 | SS<br>PA | 5 | 50/5" | 16 | | | |
| | | 3 | DB | 100% | RQD<br>100% | | | | |
| 5 | | 4 | DB | 98% | RQD<br>98% | | | | |
| 10 | | 5 | DB | 100% | RQD<br>97% | | | | |
| 15 | | 6 | DB | 100% | RQD<br>100% | | | | |
| 20 | | 7 | DB | 100% | RQD<br>97% | | | | |
| 25 | | 8 | DB | 100% | RQD<br>100% | | | | |
| 30 | | 9 | DB | 98% | RQD<br>97% | | | | |

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | | |
|---|---|---|---|
| WL | ∇ 0 | WD | ▼ |
| WL | ▼ | | ∇ |
| WL | | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | ATV | FOREMAN | VER |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. B-3

CLIENT
**Tulsa County**

SITE **Zink Dam**
**Tulsa, Oklahoma**

PROJECT
**Proposed Zink Dam Modifications**

| GRAPHIC LOG | DESCRIPTION | | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf |
| 35 | | 572 | 35 | | 10 | DB | 93% | RQD<br>87% | | | |
| 37 | | 570 | | | | | | | | | |
| 40 | | 567 | 40 | | | | | | | | |

light gray, fine-grained, unweathered, cemented
-joint at 26.3', 0°-5°, rough natural fractures
-joint at 27.1', 0°-5°, rough natural fractures
-joint at 29.2', 0°-5°, rough natural fractures

light gray, fine-grained, unweathered, cemented
-joint at 30.7', 0°-5°, rough natural fractures
-joint at 31.9', 0°-5°, rough natural fractures
-joint at 32.9', 0°-5°, rough natural fractures
-joint at 33.4', 0°-5°, rough natural fractures

light gray, fine-grained, unweathered, cemented
-joint at 35.05', 10°-05°, rough natural fractures
-joint at 35.1', 10°-15°, rough natural fractures
-joint at 35.15', 10°-15°, rough natural fractures
-joint at 36.3', 0°-5°, rough natural fractures
-joint at 36.35', 0°-5°, rough natural fractures
-joint at 36.5', 0°-5°, rough natural fractures
-joint at 36.8', 0°-5°, rough natural fractures

**SHALE++**
dark gray, unweathered, moderately hard
-joint at 37.3', 0°-5°, rough natural fractures
-joint at 37.6', 0°-5°, rough natural fractures
-joint at 37.7', 0°-5°, rough natural fractures
-joint at 37.8', 0°-5°, rough natural fractures
-joint at 37.9', 0°-5°, rough natural fractures
-joint at 39', 0°-5°, rough natural fractures
-joint at 39.5', 0°-5°, rough natural fractures

BOTTOM OF BORING

++Classification estimated from core samples. Petrographic analysis may reveal other rock types

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | | |
|---|---|---|---|
| WL | ▽ 0 | WD | ▼ |
| WL | ▼ | | ▼ |
| WL | | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | ATV | FOREMAN | VER |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. TP-1

Page 1 of 1

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| GRAPHIC LOG | | DESCRIPTION<br>Approx. Surface Elev.: 607.5 ft | | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| | 1 | **POORLY GRADED SAND**<br>with gravel, brown | 606.5 | | SP | 1 | BS | | | 7 | | | S-1<br>-#200=0% |
| | 2 | **SANDSTONE**<br>gray, cemented | 605.5 | | | | | | | | | | |
| | | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ N/E | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | Trackhoe | FOREMAN | CSK |
| APPROVED | CSK | JOB # | 04125129 |

BOREHOLE BORING LOGS.GPJ 2011 TULSA.GDT 8/24/12

# LOG OF BORING NO. TP-2

Page 1 of 1

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 607.7 ft | DEPTH, ft. | USCS SYMBOL | SAMPLES NUMBER | SAMPLES TYPE | SAMPLES RECOVERY, in. | SAMPLES SPT-N BLOWS / ft. | TESTS WATER CONTENT, % | TESTS DRY UNIT WT pcf | TESTS UNCONFINED COMPRESSION, psf | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ▨ | **POORLY GRADED SAND**<br>with gravel, brown | | SP | 1 | BS | | | 7 | | | S-1<br>-#200=1% |
| | 2.5 — 605 | | | | | | | | | | |
| | 3.5 **SANDSTONE** ▽ 604<br>gray, cemented | | | | | | | | | | |
| | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 3.5 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | |
|---|---|
| BORING STARTED | 5-29-12 |
| BORING COMPLETED | 5-29-12 |
| RIG Trackhoe | FOREMAN CSK |
| APPROVED CSK | JOB # 04125129 |

# LOG OF BORING NO. TP-3

Page 1 of 1

|  |  |
|---|---|
| CLIENT<br>**Tulsa County** |  |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| GRAPHIC LOG | DESCRIPTION | DEPTH, ft. | USCS SYMBOL | SAMPLES |  |  |  | TESTS |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|---|
|  | Approx. Surface Elev.: 606.5 ft |  |  | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf |  |
|  | **WELL-GRADED SAND**<br>brown<br>(with gravel and boulders below 3 feet)<br>6 ▽ 600.5 | 5 | SW | 1 | BS |  |  | 1 |  |  | S-1<br>-#200=2% |
|  | 7 **SANDSTONE**<br>gray, cemented 599.5 |  |  |  |  |  |  |  |  |  |  |
|  | BOTTOM OF TEST PIT - TRACKHOE REFUSAL |  |  |  |  |  |  |  |  |  |  |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft |  |
|---|---|
| WL ▽ 6 | ▼ |
| WL ▼ | ▼ |
| WL |  |

Terracon

| | |
|---|---|
| BORING STARTED | 5-29-12 |
| BORING COMPLETED | 5-29-12 |
| RIG Trackhoe | FOREMAN CSK |
| APPROVED CSK | JOB # 04125129 |

# LOG OF BORING NO. TP-4

Page 1 of 1

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| | | | | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 610.2 ft | DEPTH, ft. | USCS SYMBOL | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| ░ | **WELL GRADED SAND**<br>with gravel, brown<br>2.5 ▽ 607.5 | | SW | 1 | BS | | | 3 | | | S-1<br>-#200=3% |
| | 3 **SANDSTONE** 607<br>gray, cemented<br>BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 2.5 | ▼ |
| WL | ▽ | ▼ |
| WL | | |

**Terracon**

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | Trackhoe | FOREMAN | CSK |
| APPROVED | CSK | JOB # | 04125129 |

BOREHOLE BORING LOGS.GPJ 2011 TULSA.GDT 8/24/12

# LOG OF BORING NO. TP-5

Page 1 of 1

CLIENT: **Tulsa County**

SITE: **Zink Dam**
**Tulsa, Oklahoma**

PROJECT: **Proposed Zink Dam Modifications**

| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 609.7 ft | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| 1 | **POORLY GRADED GRAVEL**<br>with silt and sand, brown ▽ 608.5 | | GP<br>GM | 1 | BS | | | 6 | | | S-1<br>-#200=10% |
| 1.5 | 608<br>**SANDSTONE**<br>gray, cemented | | | | | | | | | | |
| | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 1 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | |
|---|---|
| BORING STARTED | 5-29-12 |
| BORING COMPLETED | 5-29-12 |
| RIG Trackhoe | FOREMAN CSK |
| APPROVED CSK | JOB # 04125129 |

# LOG OF BORING NO. TP-6

**Page 1 of 1**

CLIENT
**Tulsa County**

SITE **Zink Dam**
**Tulsa, Oklahoma**

PROJECT
**Proposed Zink Dam Modifications**

| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 611.2 ft | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| 1.5 | **POORLY GRADED GRAVEL**<br>with sand, brown ▽ 609.5 | | GP | 1 | BS | | | 10 | | | S-1<br>-#200=3% |
| 2 | **SANDSTONE**<br>gray, cemented 609 | | | | | | | | | | |
| | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 1.5 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | |
|---|---|
| BORING STARTED | 5-29-12 |
| BORING COMPLETED | 5-29-12 |
| RIG Trackhoe | FOREMAN CSK |
| APPROVED CSK | JOB # 04125129 |

BOREHOLE BORING LOGS.GPJ 2011 TULSA.GDT 8/24/12

# LOG OF BORING NO. TP-7

Page 1 of 1

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 614.4 ft | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| | **WELL GRADED SAND**<br>brown | | SW | 1 | BS | | | 4 | | | S-1<br>-#200=2% |
| | 3 ▽ 611.5 | | | | | | | | | | |
| | 3.5 **SANDSTONE** 611<br>gray, cemented | | | | | | | | | | |
| | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 3 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | Trackhoe | FOREMAN | CSK |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. TP-8

Page 1 of 1

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE<br>**Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

Approx. Surface Elev.: 611.7 ft

| DESCRIPTION | Depth of layer, ft | Elevation, ft | DEPTH, ft. | USCS SYMBOL | SAMPLES NUMBER | SAMPLES TYPE | SAMPLES RECOVERY, in. | SAMPLES SPT-N BLOWS / ft. | TESTS WATER CONTENT, % | TESTS DRY UNIT WT pcf | TESTS UNCONFINED COMPRESSION, psf | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **WELL GRADED SAND** with silt, brown | 4 | 607.5 | | SW<br>SM | 1 | BS | | | 11 | | | S-1<br>-#200=8% |
| **SILTY SAND** dark gray, with hydrocarbon odor | 5 | 606.5 | 5 | SM | 2 | BS | | | 34 | | | S-2<br>-#200=28% |
| **SAND** brown ▽ | 9 | 602.5 | | SP | | | | | | | | |
| BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 7 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | Trackhoe | FOREMAN | CSK |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. TP-9

**Page 1 of 1**

CLIENT
**Tulsa County**

SITE **Zink Dam**
**Tulsa, Oklahoma**

PROJECT
**Proposed Zink Dam Modifications**

| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 615.4 ft | DEPTH, ft. | USCS SYMBOL | SAMPLES: NUMBER | SAMPLES: TYPE | SAMPLES: RECOVERY, in. | SAMPLES: SPT-N BLOWS / ft. | TESTS: WATER CONTENT, % | TESTS: DRY UNIT WT pcf | TESTS: UNCONFINED COMPRESSION, psf | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | **POORLY GRADED SAND**<br>brown<br>▽ | | SP | 1 | BS | | | 4 | | | S-1<br>-#200=0% |
| | 5  610.5 | 5 | | | | | | | | | |
| | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 3 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | |
|---|---|
| BORING STARTED | 5-29-12 |
| BORING COMPLETED | 5-29-12 |
| RIG Trackhoe | FOREMAN CSK |
| APPROVED CSK | JOB # 04125129 |

# LOG OF BORING NO. TP-10

Page 1 of 1

| | |
|---|---|
| CLIENT<br>**Tulsa County** | |
| SITE **Zink Dam**<br>**Tulsa, Oklahoma** | PROJECT<br>**Proposed Zink Dam Modifications** |

| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 614.4 ft | DEPTH, ft. | USCS SYMBOL | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| | **POORLY GRADED SAND**<br>brown ▽ | | SP | 7 | BS | | | 6 | | | S-1<br>-#200=0% |
| | | 5 | | | | | | | | | |
| | 7 607.5 | | | | | | | | | | |
| | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 2 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

Terracon

| | | | |
|---|---|---|---|
| BORING STARTED | | | 5-29-12 |
| BORING COMPLETED | | | 5-29-12 |
| RIG | Trackhoe | FOREMAN | CSK |
| APPROVED | CSK | JOB # | 04125129 |

# LOG OF BORING NO. TP-11

Page 1 of 1

CLIENT
**Tulsa County**

SITE **Zink Dam**
**Tulsa, Oklahoma**

PROJECT
**Proposed Zink Dam Modifications**

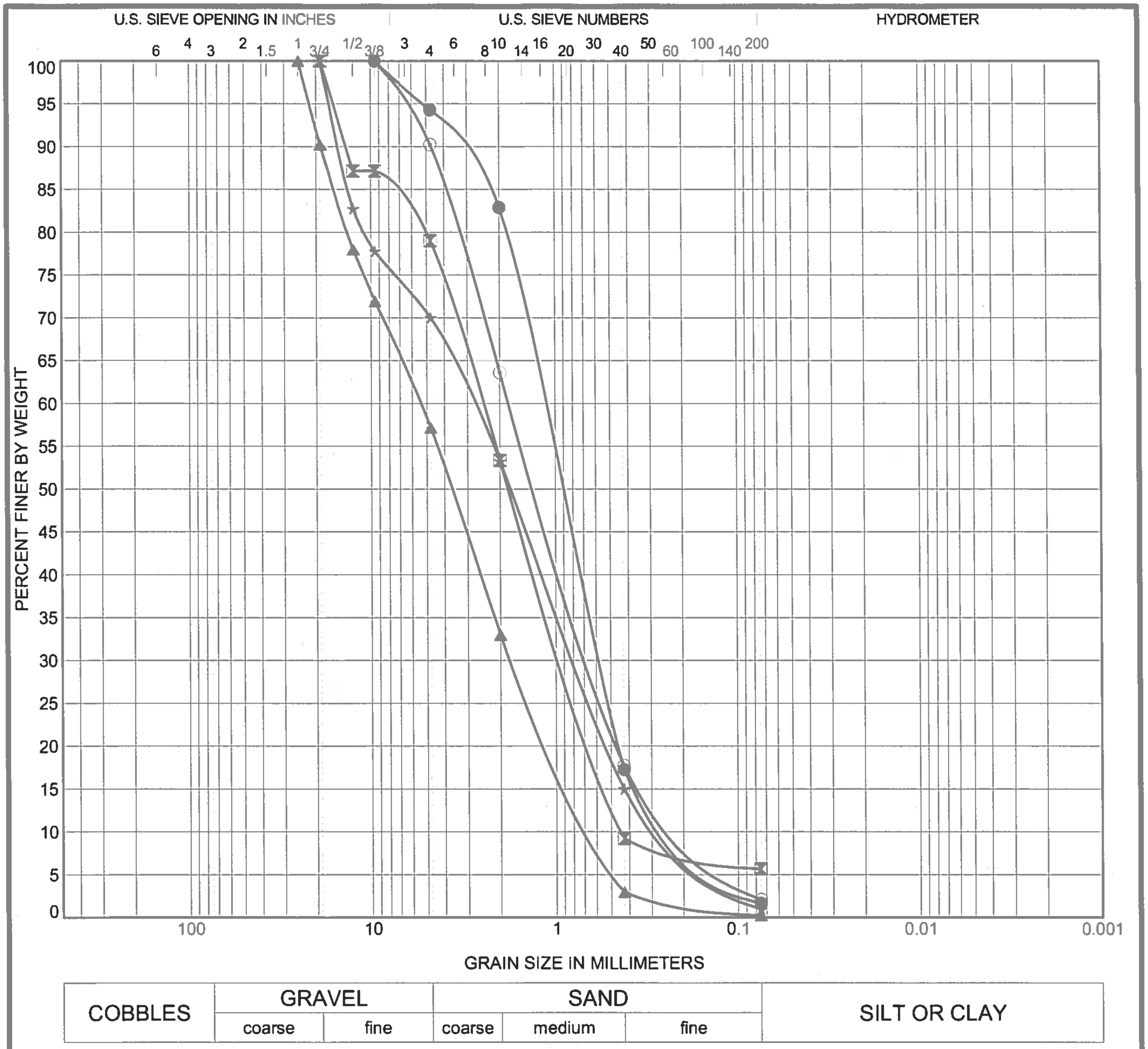
| | | | | SAMPLES | | | | TESTS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| GRAPHIC LOG | DESCRIPTION<br>Approx. Surface Elev.: 615.2 ft | DEPTH, ft. | USCS SYMBOL | NUMBER | TYPE | RECOVERY, in. | SPT-N BLOWS / ft. | WATER CONTENT, % | DRY UNIT WT pcf | UNCONFINED COMPRESSION, psf | |
| | **POORLY GRADED SAND**<br>brown<br>▽<br>5 610 | 5 | SP | 1 | BS | | | 4 | | | S-1<br>-#200=0% |
| | BOTTOM OF TEST PIT - TRACKHOE REFUSAL | | | | | | | | | | |

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

| WATER LEVEL OBSERVATIONS, ft | | |
|---|---|---|
| WL | ▽ 3 | ▼ |
| WL | ▼ | ▼ |
| WL | | |

**Terracon**

| | |
|---|---|
| BORING STARTED | 5-29-12 |
| BORING COMPLETED | 5-29-12 |
| RIG Trackhoe | FOREMAN CSK |
| APPROVED CSK | JOB # 04125129 |



| | Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|
| ● | **B-1** | **WELL-GRADED SAND(SW)** | | | | **1.49** | **6.15** |
| ⊠ | **B-3** | **WELL-GRADED SAND (SW)** | | | | **0.71** | **5.73** |
| ▲ | **TP-1** | **POORLY GRADED SAND with GRAVEL(SP)** | | | | **0.89** | **8.88** |
| ★ | **TP-2** | **POORLY GRADED SAND with GRAVEL(SP)** | | | | **0.94** | **12.35** |
| ⊙ | **TP-3** | **WELL-GRADED SAND(SW)** | | | | **1.30** | **9.82** |

| | Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|
| ● | **B-1** | **9.5** | **1.165** | **0.574** | **0.19** | **6** | **93** | **2** | |
| ⊠ | **B-3** | **19** | **2.502** | **0.881** | **0.437** | **21** | **73** | **6** | |
| ▲ | **TP-1** | **25** | **5.431** | **1.716** | **0.611** | **43** | **57** | **0** | |
| ★ | **TP-2** | **19** | **2.831** | **0.78** | **0.229** | **30** | **69** | **1** | |
| ⊙ | **TP-3** | **9.5** | **1.771** | **0.643** | **0.18** | **10** | **88** | **2** | |

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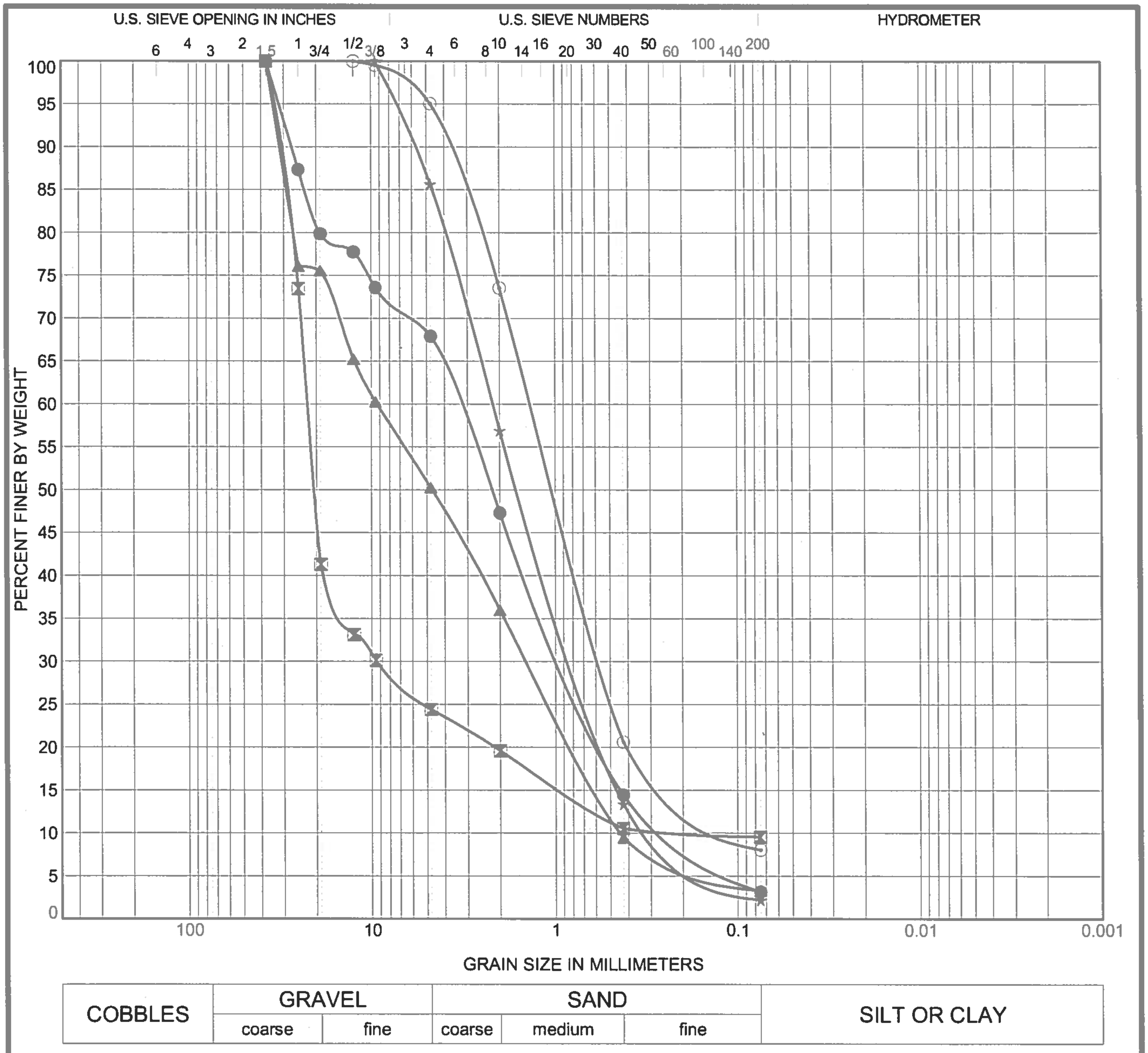
## GRAIN SIZE DISTRIBUTION

Project: Proposed Zink Dam Modifications
Site: Zink Dam Tulsa, Oklahoma
Job #: 04125129
Date: 6-14-12



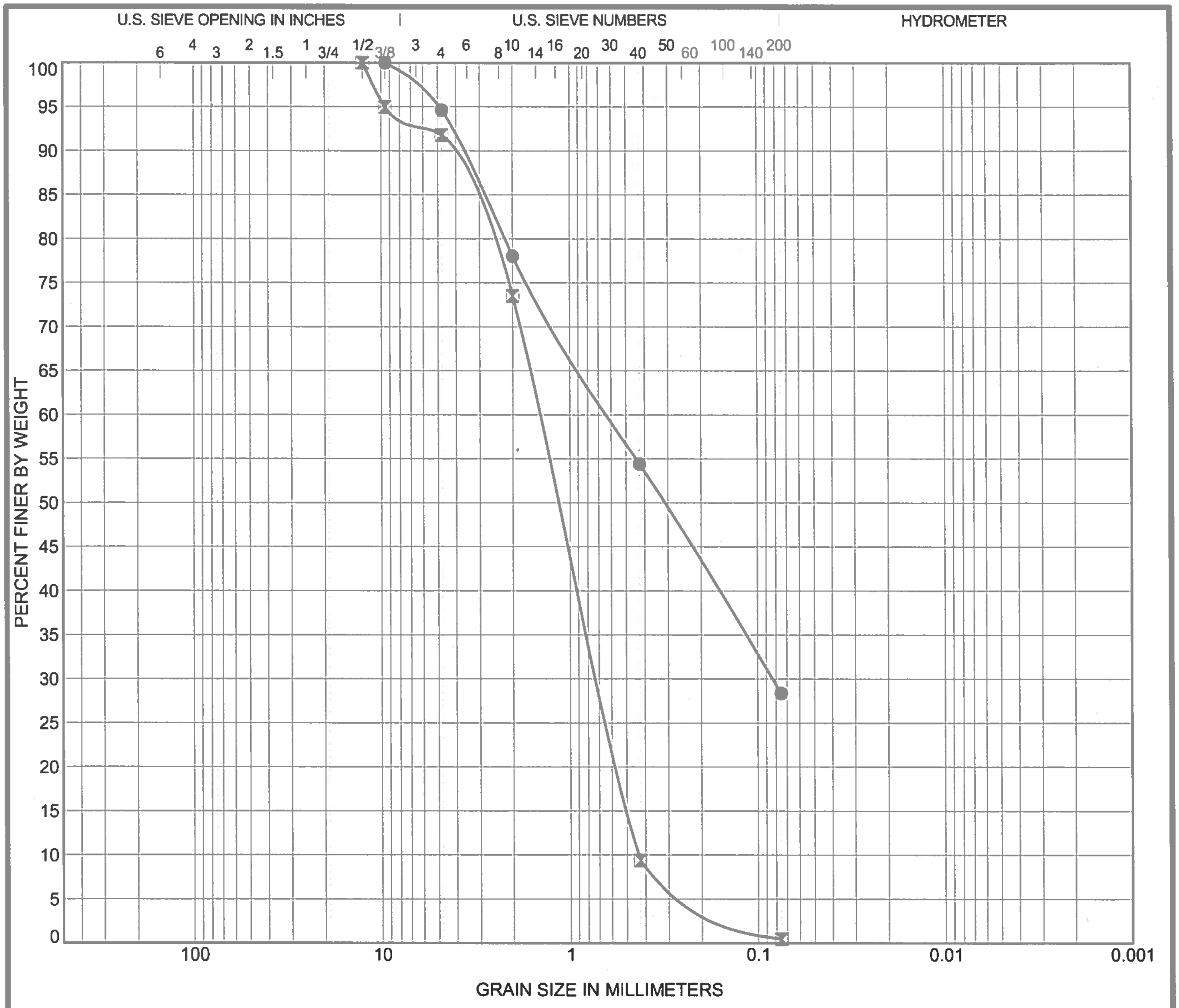
| Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|
| ● TP-4 | **WELL-GRADED SAND with GRAVEL(SW)** | | | | 1.07 | 15.84 |
| ⧗ TP-5 | **POORLY GRADED GRAVEL W/SILT AND SAND (GP-GM)** | | | | 23.97 | 135.66 |
| ▲ TP-6 | **POORLY GRADED GRAVEL with SAND(GP)** | | | | 0.49 | 21.24 |
| ★ TP-7 | **WELL-GRADED SAND(SW)** | | | | 1.07 | 8.71 |
| ⊙ TP-8 | **WELL GRADED SAND W/SILT (SW-SM)** | | | | 2.36 | 13.69 |

| Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|
| ● TP-4 | 37.5 | 3.407 | 0.885 | 0.215 | 32 | 65 | 3 | |
| ⧗ TP-5 | 37.5 | 22.287 | 9.368 | 0.164 | 76 | 15 | 10 | |
| ▲ TP-6 | 37.5 | 9.321 | 1.411 | 0.439 | 50 | 47 | 3 | |
| ★ TP-7 | 9.5 | 2.195 | 0.768 | 0.252 | 14 | 83 | 2 | |
| ⊙ TP-8 | 12.5 | 1.346 | 0.56 | 0.098 | 5 | 87 | 8 | |

**Terracon**

# GRAIN SIZE DISTRIBUTION

Project: Proposed Zink Dam Modifications
Site: Zink Dam Tulsa, Oklahoma
Job #: 04125129
Date: 6-14-12



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

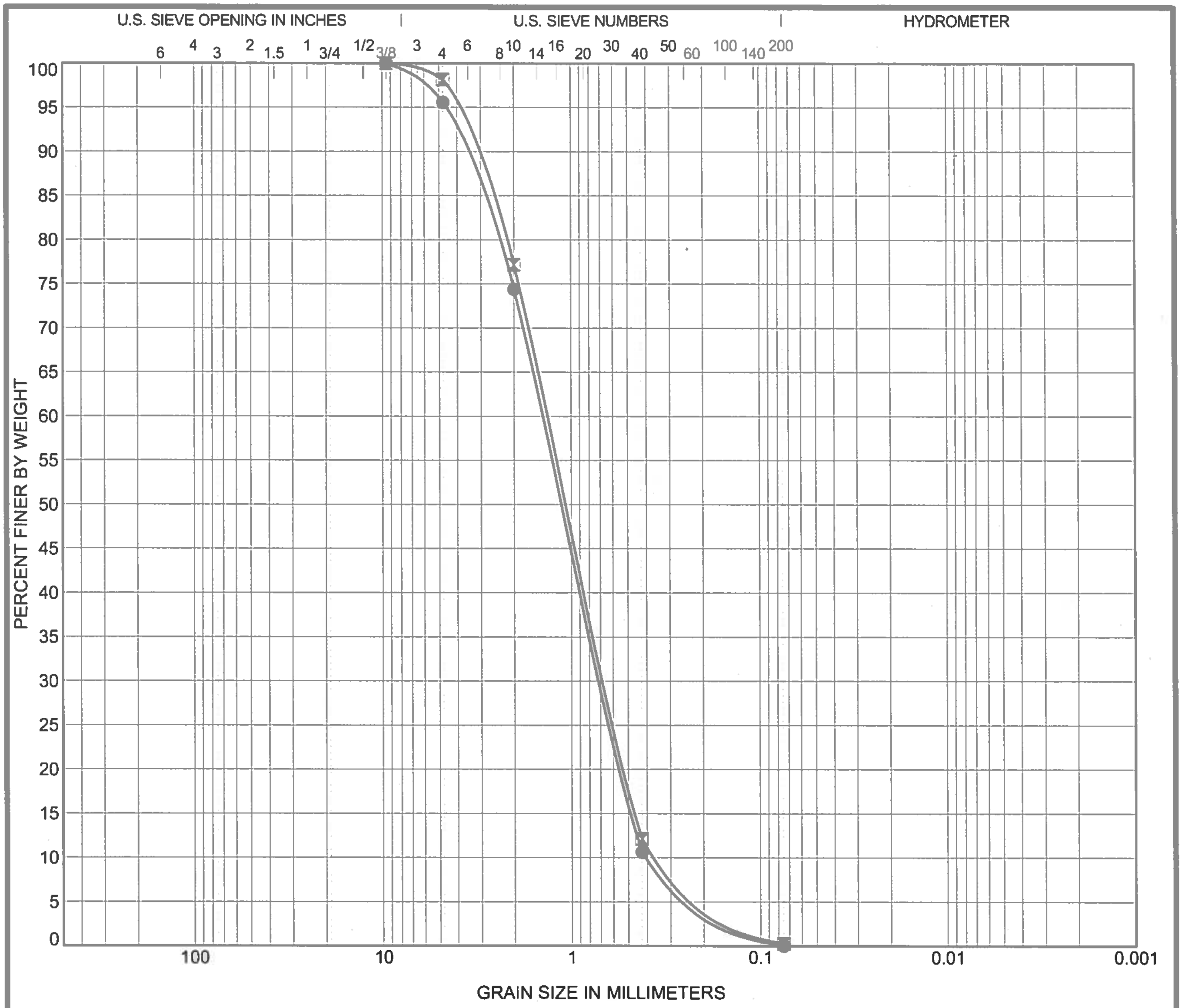
| | Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|
| ● | TP-8 | **SILTY SAND (SM)** | | | | | |
| ☒ | TP-9 | **POORLY GRADED SAND(SP)** | | | | **0.79** | **3.35** |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

| | Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|
| ● | TP-8 | **9.5** | **0.613** | **0.084** | | **5** | **66** | **28** | |
| ☒ | TP-9 | **12.5** | **1.443** | **0.699** | **0.431** | **8** | **91** | **0** | |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

## GRAIN SIZE DISTRIBUTION

Terracon

Project: Proposed Zink Dam Modifications
Site: Zink Dam Tulsa, Oklahoma
Job #: 04125129
Date: 6-14-12



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| | Specimen Identification | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|
| ● | TP-10 | **POORLY GRADED SAND(SP)** | | | | **0.86** | **3.68** |
| ⧗ | TP-11 | **POORLY GRADED SAND(SP)** | | | | **1.02** | **4.26** |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

| | Specimen Identification | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|
| ● | TP-10 | **9.5** | **1.41** | **0.68** | **0.383** | **4** | **96** | **0** | |
| ⧗ | TP-11 | **9.5** | **1.329** | **0.65** | **0.312** | **2** | **98** | **0** | |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

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## GRAIN SIZE DISTRIBUTION

Project: Proposed Zink Dam Modifications
Site: Zink Dam Tulsa, Oklahoma
Job #: 04125129
Date: 6-14-12

# PHOTOGRAPHIC ROCK CORE LOGS
## Zink Dam Modifications
Tulsa, Oklahoma
Terracon Project No. 04125129

B-1 5'-15'

B-1 15'-25'

# PHOTOGRAPHIC ROCK CORE LOGS
## Zink Dam Modifications
## Tulsa, Oklahoma
## Terracon Project No. 04125129

B-1 25’-35’

B-2 3’-13’

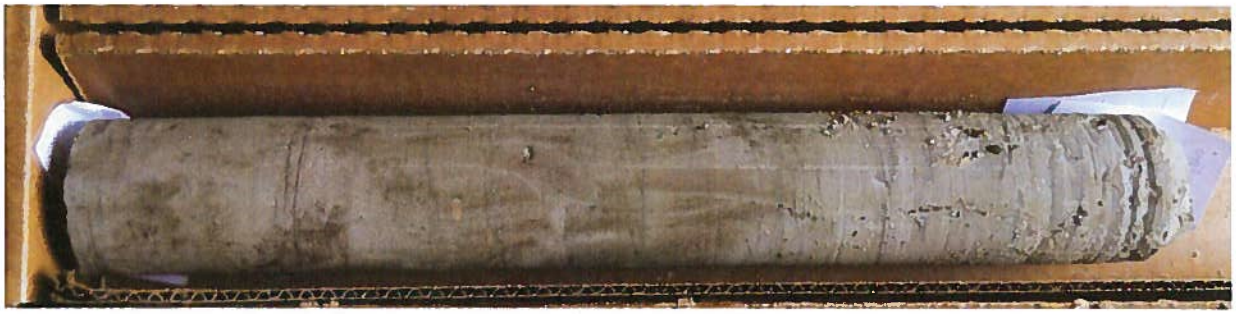
# PHOTOGRAPHIC ROCK CORE LOGS
## Zink Dam Modifications
## Tulsa, Oklahoma
## Terracon Project No. 04125129

B-2 13'-23'

B-2 23'-33'

# PHOTOGRAPHIC ROCK CORE LOGS
## Zink Dam Modifications
## Tulsa, Oklahoma
## Terracon Project No. 04125129

B-2 33'-38'



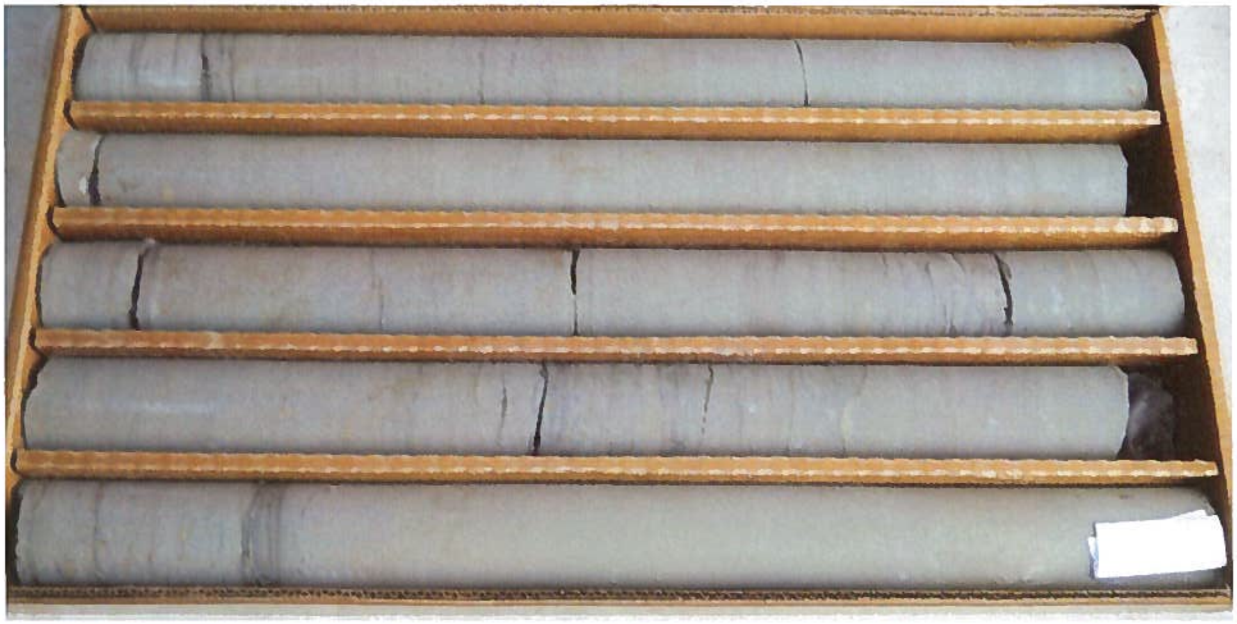
B-3 4'-5'

# PHOTOGRAPHIC ROCK CORE LOGS
## Zink Dam Modifications
## Tulsa, Oklahoma
## Terracon Project No. 04125129

B-3 5'-15'

B-3 15'-25'

# PHOTOGRAPHIC ROCK CORE LOGS
## Zink Dam Modifications
## Tulsa, Oklahoma
## Terracon Project No. 04125129



B-3 25'-35'

B-3 35'-40'

Test Pit: TP-1

Test Pit: TP-1

**Zink Dam Modifications**
**Tulsa, Oklahoma**
**Terracon Project No. 04125129**

Test Pit: TP-2

Test Pit: TP-3

**Zink Dam Modifications**
**Tulsa, Oklahoma**
**Terracon Project No. 04125129**

Test Pit: TP-3

Test Pit: TP-4

**Zink Dam Modifications**
**Tulsa, Oklahoma**
**Terracon Project No. 04125129**

Test Pit: TP-4

Test Pit: TP-5

**Zink Dam Modifications**
**Tulsa, Oklahoma**
**Terracon Project No. 04125129**

Test Pit: TP-5

Test Pit: TP-6

Test Pit: TP-6

Test Pit: TP-7

**Zink Dam Modifications**
**Tulsa, Oklahoma**
**Terracon Project No. 04125129**

Test Pit: TP-7

Test Pit: TP-8

Test Pit: TP-8

Test Pit: TP-8

# Zink Dam Modifications
Tulsa, Oklahoma
Terracon Project No. 04125129

Test Pit: TP-8

Test Pit: TP-10

**Zink Dam Modifications**
**Tulsa, Oklahoma**
**Terracon Project No. 04125129**

Test Pit: TP-10

Test Pit: TP-10

**Zink Dam Modifications**
**Tulsa, Oklahoma**
**Terracon Project No. 04125129**

Test Pit: TP-10

Test Pit: TP-11

# Zink Dam Modifications
Tulsa, Oklahoma
Terracon Project No. 04125129

Test Pit: TP-11

# GENERAL NOTES

**DRILLING & SAMPLING SYMBOLS:**

| | | | |
|---|---|---|---|
| SS: | Split Spoon - 1-$^{3}/_{8}$" I.D., 2" O.D., unless otherwise noted | HS: | Hollow Stem Auger |
| ST: | Thin-Walled Tube – 2" O.D., 3" O.D., unless otherwise noted | PA: | Power Auger (Solid Stem) |
| RS: | Ring Sampler - 2.42" I.D., 3" O.D., unless otherwise noted | HA: | Hand Auger |
| DB: | Diamond Bit Coring - 4", N, B | RB: | Rock Bit |
| BS: | Bulk Sample or Auger Sample | WB | Wash Boring or Mud Rotary |

The number of blows required to advance a standard 2-inch O.D. split-spoon sampler (SS) the last 12 inches of the total 18-inch penetration with a 140-pound hammer falling 30 inches is considered the "Standard Penetration" or "N-value".

**WATER LEVEL MEASUREMENT SYMBOLS:**

| | | | | | |
|---|---|---|---|---|---|
| WL: | Water Level | WS: | While Sampling | BCR: | Before Casing Removal |
| WCI: | Wet Cave in | WD: | While Drilling | ACR: | After Casing Removal |
| DCI: | Dry Cave in | AB: | After Boring | N/E: | Not Encountered |

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. Groundwater levels at other times and other locations across the site could vary. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels may not be possible with only short-term observations.

**DESCRIPTIVE SOIL CLASSIFICATION:** Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

**CONSISTENCY OF FINE-GRAINED SOILS**

| Unconfined Compressive Strength, Qu, psf | Standard Penetration or N-value (SS) Blows/Ft. | Consistency |
|---|---|---|
| < 500 | 0 - 1 | Very Soft |
| 500 – 1,000 | 2 - 4 | Soft |
| 1,000 – 2,000 | 4 - 8 | Medium Stiff |
| 2,000 – 4,000 | 8 - 15 | Stiff |
| 4,000 – 8,000 | 15 - 30 | Very Stiff |
| 8,000+ | > 30 | Hard |

**RELATIVE DENSITY OF COARSE-GRAINED SOILS**

| Standard Penetration or N-value (SS) Blows/Ft. | Relative Density |
|---|---|
| 0 – 3 | Very Loose |
| 4 – 9 | Loose |
| 10 – 29 | Medium Dense |
| 30 – 50 | Dense |
| > 50 | Very Dense |

**RELATIVE PROPORTIONS OF SAND AND GRAVEL**

| Descriptive Term(s) of other constituents | Percent of Dry Weight |
|---|---|
| Trace | < 15 |
| With | 15 – 29 |
| Modifier | ≥ 30 |

**GRAIN SIZE TERMINOLOGY**

| Major Component of Sample | Particle Size |
|---|---|
| Boulders | Over 12 in. (300mm) |
| Cobbles | 12 in. to 3 in. (300mm to 75mm) |
| Gravel | 3 in. to #4 sieve (75mm to 4.75mm) |
| Sand | #4 to #200 sieve (4.75 to 0.075mm) |
| Silt or Clay | Passing #200 Sieve (0.075mm) |

**RELATIVE PROPORTIONS OF FINES**

| Descriptive Term(s) of other constituents | Percent of Dry Weight |
|---|---|
| Trace | < 5 |
| With | 5 – 12 |
| Modifier | > 12 |

**PLASTICITY DESCRIPTION**

| Term | Plasticity Index |
|---|---|
| Non-plastic | 0 |
| Low | 1-10 |
| Medium | 11-30 |
| High | > 30 |

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# GENERAL NOTES

## Sedimentary Rock Classification

### DESCRIPTIVE ROCK CLASSIFICATION:

Sedimentary rocks are composed of cemented clay, silt and sand sized particles. The most common minerals are clay, quartz and calcite. Rock composed primarily of calcite is called limestone; rock of sand size grains is called sandstone, and rock of clay and silt size grains is called mudstone or claystone, siltstone, or shale. Modifiers such as shaly, sandy, dolomitic, calcareous, carbonaceous, etc. are used to describe various constituents. Examples: sandy shale; calcareous sandstone.

| | |
|---|---|
| LIMESTONE | Light to dark colored, crystalline to fine-grained texture, composed of $CaCo_3$, reacts readily with HCl. |
| DOLOMITE | Light to dark colored, crystalline to fine-grained texture, composed of $CaMg(CO_3)_2$, harder than limestone, reacts with HCl when powdered. |
| CHERT | Light to dark colored, very fine-grained texture, composed of micro-crystalline quartz ($Si0_2$), brittle, breaks into angular fragments, will scratch glass. |
| SHALE | Very fine-grained texture, composed of consolidated silt or clay, bedded in thin layers. The unlaminated equivalent is frequently referred to as siltstone, claystone or mudstone. |
| SANDSTONE | Usually light colored, coarse to fine texture, composed of cemented sand size grains of quartz, feldspar, etc. Cement usually is silica but may be such minerals as calcite, iron-oxide, or some other carbonate. |
| CONGLOMERATE | Rounded rock fragments of variable mineralogy varying in size from near sand to boulder size but usually pebble to cobble size (½ inch to 6 inches). Cemented together with various cementing agents. Breccia is similar but composed of angular, fractured rock particles cemented together. |

### PHYSICAL PROPERTIES:

#### DEGREE OF WEATHERING

| | |
|---|---|
| Slight | Slight decomposition of parent material on joints. May be color change. |
| Moderate | Some decomposition and color change throughout. |
| High | Rock highly decomposed, may be extremely broken. |

#### HARDNESS AND DEGREE OF CEMENTATION

**Limestone and Dolomite:**

| | |
|---|---|
| Hard | Difficult to scratch with knife. |
| Moderately Hard | Can be scratched easily with knife, cannot be scratched with fingernail. |
| Soft | Can be scratched with fingernail. |

**Shale, Siltstone and Claystone**

| | |
|---|---|
| Hard | Can be scratched easily with knife, cannot be scratched with fingernail. |
| Moderately Hard | Can be scratched with fingernail. |
| Soft | Can be easily dented but not molded with fingers. |

**Sandstone and Conglomerate**

| | |
|---|---|
| Well Cemented | Capable of scratching a knife blade. |
| Cemented | Can be scratched with knife. |
| Poorly Cemented | Can be broken apart easily with fingers. |

#### BEDDING AND JOINT CHARACTERISTICS

| Bed Thickness | Joint Spacing | Dimensions |
|---|---|---|
| Very Thick | Very Wide | >10′ |
| Thick | Wide | 3′ - 10′ |
| Medium | Moderately Close | 1′ - 3′ |
| Thin | Close | 2″ - 1′ |
| Very Thin | Very Close | .4″ - 2″ |
| Laminated | — | .1″ - .4″ |

| | |
|---|---|
| Bedding Plane | A plane dividing sedimentary rocks of the same or different lithology. |
| Joint | Fracture in rock, generally more or less vertical or transverse to bedding, along which no appreciable movement has occurred. |
| Seam | Generally applies to bedding plane with an unspecified degree of weathering. |

#### SOLUTION AND VOID CONDITIONS

| | |
|---|---|
| Solid | Contains no voids. |
| Vuggy (Pitted) | Rock having small solution pits or cavities up to ½ inch diameter, frequently with a mineral lining. |
| Porous | Containing numerous voids, pores, or other openings, which may or may not interconnect. |
| Cavernous | Containing cavities or caverns, sometimes quite large. |

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