Shell Canal Tunnel

Level II Feasibility Study
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DEERE & AULT
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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Topic</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1</td>
<td>Purpose</td>
<td>1</td>
</tr>
<tr>
<td>1.2</td>
<td>Scope</td>
<td>1</td>
</tr>
<tr>
<td>1.2.1</td>
<td>Task 1 - Scoping and Project Meetings</td>
<td>1</td>
</tr>
<tr>
<td>1.2.2</td>
<td>Task 2 - Review of Background Information</td>
<td>1</td>
</tr>
<tr>
<td>1.2.3</td>
<td>Task 3 - Access</td>
<td>1</td>
</tr>
<tr>
<td>1.2.4</td>
<td>Task 4 - Inspection and Condition of Structure</td>
<td>1</td>
</tr>
<tr>
<td>1.2.5</td>
<td>Task 5 - Geotechnical Investigation</td>
<td>2</td>
</tr>
<tr>
<td>1.2.6</td>
<td>Task 6 - Alternatives Analysis</td>
<td>3</td>
</tr>
<tr>
<td>1.2.7</td>
<td>Task 7 - Conceptual Design and Cost Estimates</td>
<td>3</td>
</tr>
<tr>
<td>1.2.8</td>
<td>Task 8 - Permits</td>
<td>3</td>
</tr>
<tr>
<td>1.2.9</td>
<td>Task 9 - Economic Analysis and Project Financing</td>
<td>3</td>
</tr>
<tr>
<td>1.2.10</td>
<td>Task 10 - Recommendations</td>
<td>3</td>
</tr>
<tr>
<td>1.2.11</td>
<td>Task 11 - Discretionary Task</td>
<td>3</td>
</tr>
<tr>
<td>1.2.12</td>
<td>Task 12 - Reports</td>
<td>4</td>
</tr>
<tr>
<td>2.0</td>
<td>BACKGROUND</td>
<td>5</td>
</tr>
<tr>
<td>3.0</td>
<td>TUNNEL ACCESS</td>
<td>6</td>
</tr>
<tr>
<td>4.0</td>
<td>TUNNEL INSPECTION and ASSESSMENT OF CONDITION</td>
<td>8</td>
</tr>
<tr>
<td>5.0</td>
<td>GEOTECHNICAL INVESTIGATION</td>
<td>10</td>
</tr>
<tr>
<td>5.1</td>
<td>Geologic Reconnaissance</td>
<td>10</td>
</tr>
<tr>
<td>5.2</td>
<td>Tunnel Liner Investigations</td>
<td>12</td>
</tr>
<tr>
<td>5.2.1</td>
<td>Tunnel Liner Drilling</td>
<td>12</td>
</tr>
<tr>
<td>5.3</td>
<td>Bedrock Ridge Investigations</td>
<td>13</td>
</tr>
<tr>
<td>5.3.1</td>
<td>Geotechnical Drilling</td>
<td>13</td>
</tr>
<tr>
<td>5.3.2</td>
<td>Laboratory Testing</td>
<td>14</td>
</tr>
<tr>
<td>6.0</td>
<td>ALTERNATIVES ANALYSIS</td>
<td>16</td>
</tr>
<tr>
<td>6.1</td>
<td>Alternatives</td>
<td>17</td>
</tr>
<tr>
<td>6.1.1</td>
<td>Tunnel Sleeve</td>
<td>17</td>
</tr>
<tr>
<td>6.1.2</td>
<td>Open Cut Channel</td>
<td>19</td>
</tr>
<tr>
<td>6.1.3</td>
<td>Conventional Rehabilitation of the Tunnel</td>
<td>19</td>
</tr>
<tr>
<td>6.1.4</td>
<td>Summary of Analysis</td>
<td>20</td>
</tr>
</tbody>
</table>
List of Appendices

Appendix A  Tunnel Inspection
Appendix B  Tunnel Liner Drilling
Appendix C  Laboratory Test Results
Appendix D  Tunnel Sleeve Conceptual Drawings
Appendix E  Open Cut Conceptual Drawings
1.0 INTRODUCTION

1.1 Purpose

This report describes the Level II Feasibility Study of the Shell Canal Tunnel completed for the Shell Valley Watershed Improvement District (Project Sponsor) near Shell, Wyoming. The Shell Canal annually carries water through approximately 56 miles of canals and laterals to irrigate about 3,500 acres of crop land. The purpose of the study is to evaluate alternatives for the rehabilitation of the tunnel and to refine conceptual engineering designs and cost estimates for the rehabilitation work.

1.2 Scope

Work completed for the Level II Feasibility Study of the Shell Canal Tunnel included the following tasks:

1.2.1 Task 1 - Scoping and Project Meetings

We conducted four meetings within the watershed coordinated with the Wyoming Water Development Commission (WWDC) Project Manager. The main purpose of the meetings was to keep the Project Sponsor, the Shell Valley Watershed Improvement District, and the community informed of the project progress.

1.2.2 Task 2 - Review of Background Information

We performed a review of available information on the Shell Canal Tunnel. Our review included any construction records and earlier reports and evaluations. In addition, we checked with local sources, including the staff at the Greybull Museum, where we tried to identify local historical societies and interviewed local individuals who might have knowledge regarding the tunnel construction. We also met with the staff at the Worland BLM office, which provided historical information on the canal, the tunnel, and the BLM right-of-way.

1.2.3 Task 3 - Access

We obtained access as required for the project tasks and acquired all necessary permits and clearances.

1.2.4 Task 4 - Inspection and Condition of Structure

With the Canal Company’s permission, D&A personnel completed a detailed examination of the Shell Canal Tunnel on March 18, 2010, the day following the pre-proposal meeting in Shell. The work was completed in compliance with all OSHA requirements, including air monitoring. Photographs and video were taken as the tunnel was inspected, measurements of tunnel dimensions and significant cracks and other features were made, and detailed notes were kept.
A memorandum was prepared documenting the examination methods and results, including our opinion of the causes of the deteriorated condition of the structure. The memorandum is included in Appendix A of this report.

1.2.5 Task 5 - Geotechnical Investigation

We developed a geotechnical investigation program to provide information for conceptual level designs and cost estimates of the various rehabilitation alternatives.

- **Tunnel Liner Investigations**

  1. We chose seven locations along the length of the tunnel to core and/or hammer drill the liner concrete in the walls and crown; 3-inch diameter cores of the concrete were obtained from four locations and numerous hammer drill holes were drilled in the walls and crown at each location.

  2. Each liner drill hole was examined to document the thickness of the liner and to observe the material and explore for voids behind the liner.

  3. The concrete core samples were tested in the laboratory for unconfined compressive strength.

  4. Samples of the bedrock material behind the liner were obtained through the core holes; one sample was tested in the laboratory for index properties.

  5. Each core hole, hammer drill hole, and anchor hole was patched with high strength grout after completion of the work.

  6. Data obtained from the liner investigations was summarized and analyzed for this report.

- **Investigations of the Bedrock Ridge**

  1. We drilled an exploratory boring through the Cloverly Formation bedrock that forms the ridge penetrated by the tunnel using a truck-mounted drilling rig. The boring was approximately 60 feet deep to penetrate all the strata from the top of the ridge to below the invert of the tunnel. The boring was located 20 feet from the centerline of the tunnel so that it would not encounter the tunnel.

  2. The boring was logged by a D&A engineering geologist as the work progressed to document the geologic conditions penetrated.

  3. Core samples of the bedrock retrieved from the bore hole were tested in the laboratory for various physical and engineering properties.

  4. Data from the core boring were summarized and analyzed for this report.
1.2.6 Task 6 - Alternatives Analysis

We considered several alternatives for rehabilitating or replacing the Shell Canal Tunnel, including those proposed in the Level I study. We then focused our analysis on three alternatives: 1) sleeving the tunnel with large diameter pipe, similar to the option proposed in the Level I study; 2) constructing an open cut canal through the bedrock ridge; and 3) conventional tunnel rehabilitation.

1.2.7 Task 7 - Conceptual Designs and Cost Estimates

We prepared conceptual designs for two of the three alternatives. The conceptual designs include all appurtenances and structures required. Conceptual design drawings were developed for the two alternatives, and cost estimates were prepared and broken down into WWDC eligible and non-eligible costs. We also prepared life cycle cost analysis for each alternative. The costs include items for design, permitting, land acquisition, construction engineering, quality control, construction, construction contingencies, and O&M estimates. The cost estimates are based on 2011 costs. We worked with the WWDC Project Manager to develop an appropriate inflation factor based on the timeline developed for the project.

1.2.8 Task 8 - Permits

We identified permits, easements, and clearances that may be required for construction of the selected alternatives. We consulted with the BLM regarding required supporting documentation.

1.2.9 Task 9 - Economic Analysis and Project Financing

We developed recommendations for the annual financial commitments the project beneficiaries can make to retire the construction debt. We understand project funding will be based on a 67 percent grant from the WWDC and 33 percent loan.

1.2.10 Task 10 - Recommendations

Based on consultation with the WWDC and the Project Sponsor, we selected a preferred alternative to rehabilitate the Shell Canal Tunnel to be considered for Level III construction funding.

1.2.11 Task 11 - Discretionary Task

Fifteen thousand dollars ($15,000) of the project budget were placed in a discretionary task. This was to allow for changes in scope that may be required as the work proceeds. We requested that in order to pursue detailed conceptual design and cost estimates for the open cut option that we be allowed to use discretionary task funds to pay for an additional survey. This would allow us to create a base map for the conceptual designs for both alternatives, provide accurate quantities for cost estimates, and to allow for a hydraulic review of the rehabilitation impacts on the canal capacity. We proposed to split the cost of this survey with our budget because we had some money for survey of the inlet and outlet structures in our original budget. The survey was approved and completed during our field investigation in November of 2010.
1.2.12 Task 12 - Reports

D&A has prepared this report describing the results of all work completed in this study. One project notebook containing the working files used in this project will also be provided. The project notebook files will include descriptions of the assumptions and methodologies used in the project analysis. The notebooks shall be organized in such a way as to allow replication of the steps, calculations, and procedures used by Deere & Ault Consultants to reach the conclusions described in the final report.
2.0 BACKGROUND

The Shell Canal Tunnel carries the flows of the Shell Canal through a narrow bedrock ridge in the Northwest Quarter of the Southeast Quarter of Section 3, Township 52 North, Range 92 West in eastern Bighorn County, Wyoming (Figure 1). Upstream and downstream of the tunnel, the canal is in 25-foot (+/-) deep cut sections. The concrete lined tunnel is approximately 562 feet long. The tunnel is a horseshoe section approximately 7.5 feet wide at the invert and 6 feet in height from invert to crown.

The tunnel was reportedly constructed in the early 1900s. Water for the irrigation of approximately 3,500 acres passes through the tunnel annually. Based on information provided at the pre-proposal site visit, the tunnel typically carries flows in the range of 47 to 50 cfs, but could be required to carry peak flows of up to 100 cfs. We understand the canal typically starts running the first week in April and runs until about November 1st of each year.

The Shell Valley Watershed Plan Level I Study completed by Engineering Associates in March of 2010, included a preliminary structural assessment of the Shell Canal Tunnel and seepage losses caused by the tunnel. The assessment found the “structure has far outlived its useful design life.”

The purpose of this new work is to perform a Level II study for the Shell Valley Watershed Improvement District (Project Sponsor) to evaluate alternatives and to refine conceptual engineering designs and cost estimates for the rehabilitation of the Shell Canal Tunnel.
3.0 TUNNEL ACCESS

The tunnel is located on Bureau of Land Management (BLM) property (Figure 2). The property boundaries shown on Figure 2 are based on mapping by the Bighorn County Assessor. The tunnel can be accessed by two routes (colored yellow on Figure 2) from U.S. Highway 14. The first access route from Highway 14 is the road between parcels 3 and 4. The physical address of parcel 4 is 798 Highway 14. This road goes south approximately 0.90 miles to the intersection with a small road heading southeast. The smaller road heads southeast for approximately 0.37 miles to a fork. The left fork then heads northeast for approximately 0.48 miles to the tunnel site. In addition to BLM land and parcels 3 and 4, the route also crosses parcels 20 and 21 (Figure 2). Therefore the parties owning land along this route are as follows:

1. Parcel 3: Cole J. Clawson (2955 North Custer Road, Monroe, MI 48161)
2. Parcel 4: Larry E. Regnier (P.O. Box 384, Greybull, WY 82426)
3. Parcel 1: Bureau of Land Management, Worland Office
4. Parcel 21: Red Canyon Ranch Partnership (2745 Beaver Creek Road, Shell, WY 82441)
5. Parcel 20: Michael Comstock (847 Hansmore Place Knoxville, TN 37919)

The second access route from Highway 14 is the road between parcels 11 and 12. The physical address of parcel 12 is 954 Highway 14. This road goes south approximately 0.36 miles to the Shell Canal and then follows the canal for another 0.63 miles to the tunnel site. The parties owning land along this route are:

1. Parcel 11: Charles Q. Noyes (P.O. Box 769, Greybull, WY 82426)
2. Parcel 12: Michael R. Greene (954 Highway 14, Greybull, WY 82426)
3. Parcel 1: Bureau of Land Management, Worland Office

The BLM office that administers the parcel of land containing the Shell Canal Tunnel is located in Worland, Wyoming. Contact with this BLM office was made prior to performing the geotechnical investigations at the site. No special permits were required. Discussions with BLM personnel in Worland indicate that the Shell Canal has a 50-foot easement as shown on Figure 10. During the visit to the Worland BLM office documents describing the location of the Shell Canal and the canal’s easement were obtained. The easement was granted under Title 43, Section 946 of the U.S. Code. The act was passed on March 3, 1891 and is entitled “Right of way to canal ditch companies and irrigation or drainage districts for irrigation or drainage purposes and operation and maintenance of reservoirs, canals, and laterals.” The specific language of the act describing the easement is as follows:

“The right of way through the public lands and reservations of the United States is granted to any canal ditch company, irrigation or drainage district formed for the purpose of irrigation or drainage, and duly organized under the laws of any State or Territory, and which shall have filed, or may hereafter file, with the Secretary of the Interior a copy of its articles of incorporation or, if not a private corporation, a copy of the law under which the same is formed and due proof of its organization under the same, to the extent of the ground occupied by the water of any reservoir and of any canals and laterals and fifty feet on each side of the marginal limits thereof, and, upon presentation of satisfactory showing by the applicant, such additional rights of
way as the Secretary of the Interior may deem necessary for the proper operation and maintenance of said reservoirs, canals, and laterals; also the right to take from the public lands adjacent to the line of the canal or ditch, material, earth, and stone necessary for the construction of such canal or ditch: Provided, That no such right of way shall be so located as to interfere with the proper occupation by the Government of any such reservation, and all maps of location shall be subject to the approval of the department of the Government having jurisdiction of such reservation; and the privilege herein granted shall not be construed to interfere with the control of water for irrigation and other purposes under authority of the respective States or Territories.” (U.S. Code, 2011)

The section was “... repealed by Pub. L. 94-579, title VII, Sec. 706(a), Oct. 21, 1976, 90 Stat. 2793, effective on and after Oct. 21, 1976, insofar as applicable to the issuance of rights-of-way over, upon, under, and through the public lands and lands in the National Forest System” (U.S. Code, 2011). Because the act was repealed, the right-of-way cannot be amended. Any additional rights-of-way must be obtained through a new application for rights-of-way.
4.0 TUNNEL INSPECTION and ASSESSMENT OF CONDITION

A visual evaluation was conducted on the Shell Canal Tunnel on March 18, 2010. The tunnel was reportedly constructed by blasting through the bedrock with black powder. The tunnel was lined with unreinforced concrete as it was excavated. The tunnel has a horseshoe shaped arch cross-section that is 6 feet tall from the invert to the crown of the arch, it has a 7.5-foot bottom width with a 2.5-foot high vertical straight leg portion. The tunnel concrete was formed with wooden forms. Concrete was placed between the forms and the tunnel excavation. Cold joints are located every 12 feet along the tunnel length, which indicates the forms were 12 feet long. The length of the tunnel is approximately 562 feet and was constructed with concrete transition headwall structures at the inlet and outlet.

The inspection began at the inlet structure (Station 0+00) and continued downstream through the length of the tunnel to the outlet structure (Station 5+62). The description of the tunnel condition references the left and right side of the tunnel facing downstream. For each reach of the tunnel the section was rated as good, fair, poor, and very poor. This rating reflects the following general description of the tunnel.

- **Good** - Tunnel has full functionality and only exhibits minor concrete damage or deterioration
- **Fair** - Tunnel exhibits moderate damage to the concrete and some deterioration
- **Poor** - Tunnel exhibits severe damage to the concrete liner
- **Very Poor** - Tunnel exhibits severe damage to the concrete liner and the liner appears in danger of failing

The summary of the tunnel section lengths, general description of conditions, and the condition rating for each section is provided in the following table:

<table>
<thead>
<tr>
<th>Section</th>
<th>General Description of Conditions</th>
<th>Condition Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet Structure, Station 0+00</td>
<td>Very cracked and broken, pulling away from tunnel</td>
<td>Very Poor</td>
</tr>
<tr>
<td>Station 0+00 to 1+00</td>
<td>Invert entirely broken up to Station 0+90, regular arch shape, small offsets on cracks</td>
<td>Poor</td>
</tr>
<tr>
<td>Station 1+00 to 2+00</td>
<td>Invert appears sound, regular arch, small offsets on cracks</td>
<td>Good</td>
</tr>
<tr>
<td>Station 2+00 to 3+00</td>
<td>Invert appears sound, regular arch shape, small offsets on cracks, some effervescence</td>
<td>Fair</td>
</tr>
<tr>
<td>Station 3+00 to 4+00</td>
<td>Invert appears sound, arch locally cracked with offsets up to 2&quot;, exposed aggregate, generally more broken</td>
<td>Poor</td>
</tr>
<tr>
<td>Station 4+00 to 5+00</td>
<td>Invert appears to be sound, longitudinal cracks both sides with large offsets up to 4&quot;, arch quite broken</td>
<td>Very Poor</td>
</tr>
<tr>
<td>Station 5+00 to 5+62</td>
<td>Large longitudinal cracks both sides of arch with large offsets, severe cracking on crown and invert, severe transverse crack</td>
<td>Very Poor</td>
</tr>
<tr>
<td>Outlet Structure, Station 5+62</td>
<td>Structure is failing, void behind liner and large area of missing concrete on right side, transition on left cracked and broken</td>
<td>Very Poor</td>
</tr>
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</table>
Photographs were taken of the concrete tunnel liner to document its condition as the inspection of the tunnel proceeded. Details of the tunnel inspection and the photographs documenting the work are provided in Appendix A. The tunnel inspection was used to develop a crack map of the tunnel provided on Figure 3.

Based on the investigations of the existing liner, the damage and cracking in the tunnel arch and walls appears to be a result of swelling of the bedrock. The damage to the tunnel invert in the upstream 90 or so feet is probably the result of a combination of frost heave and swelling bedrock.
The geotechnical investigation program was developed to answer several fundamental questions for analyzing, designing, and costing the proposed tunnel rehabilitation alternatives. These questions are:

1. How thick is the concrete tunnel liner?
2. How strong is the liner concrete?
3. What is behind the concrete liner (i.e., are there voids, timbers, bedrock, sloughed material)?
4. What are the properties of the bedrock in the ridge penetrated by the tunnel? Can these materials be excavated by scraper or excavator, or will blasting be required?

The site geotechnical investigations at the tunnel included a general geologic reconnaissance at the site, as well as investigations of the tunnel liner and the bedrock in the ridge penetrated by the tunnel. The field work was completed between November 16 and 19, 2010. During the geotechnical field investigations a detailed survey was also performed by Mr. Paul Reid, PLS. The survey encompasses an area of approximately 16 acres at the site, ranging in width from 300 to 500 feet in the vicinity of the tunnel. The profile and several cross sections of the Shell Canal were also surveyed from the north portal of the tunnel approximately 1,300 feet upstream to the siphon and from the south portal approximately 1,000 feet downstream. The survey resulted in a detailed topographic map of the tunnel site.

5.1 Geologic Reconnaissance

The site is located in the Bighorn Basin in the Central Rocky Mountains Physiographic Province of Wyoming. The regional geology consists of Cretaceous age sedimentary rocks of the Cloverly Formation and Jurassic age sedimentary rocks of the Morrison Formation. These rocks are folded into a series of anticlines and synclines. The tunnel site is situated on the northeastern limb of the Devil’s Kitchen Anticline (Reppe, 1986). The northeastern limb of this anticline dips towards the northeast at approximately five to eight degrees. More recent geologic processes have resulted in the deposition of alluvial, colluvial and residual deposits on the bedrock surface.

**Figure 4** is a geologic map of the site. The map shows the tunnel alignment and the location of the geotechnical boring drilled at the site to investigate the bedrock penetrated by the tunnel. **Figure 5** is a geologic profile along the tunnel alignment showing the tunnel alignment, the tunnel liner investigation sites and the location of the geotechnical boring.

The Cloverly Formation bedrock at the site generally consists of two distinct rock types: shale and sandstone. The bulk of the Cloverly formation is characterized by massive grey to purple to red shale deposits. The shale unit is the principal unit that impacts the tunnel project. The bedrock structure at the site dips to the north-northeast at approximately 6°.

The primary surficial deposit at the tunnel site is residual soil derived from weathered bedrock. These soils appear to be highly plastic and prone to erosion. A relatively large landslide was observed at the site west of the tunnel (**Figure 4**). The landslide is a rotational block slide of Cloverly Formation shale beds that was dislodged from the steep slopes of the hill to the west. The
landslide can be clearly seen as the mound just behind and to the left of the drill rig in the photograph below:

Photograph showing the landslide at the site. Notice the red beds on the hill to the right and the corresponding bed outlined near the center of the landslide mass.

The landslide indicates the relatively weak nature of the bedrock.

The erodible nature of the Cloverly Formation, characterized by badlands-type topography, is also clearly visible at the tunnel site. Piping type erosion is also common in the Cloverly Formation shale. This form of erosion typically occurs along large scale fractures in the rock where precipitation is able to enter the fracture and erode the softer material along the fracture beneath the ground surface. The eroded material is then deposited in topographically low areas that intersect the fracture. This type of erosion can be noticed by the presence of “chimneys” oriented in a line. The chimneys are basically large holes in the ground where material has collapsed into a larger natural pipe underground. Examples of these features measuring about 1-foot in diameter are shown in the photograph below:

Photograph showing three chimneys connecting the ground surface to an underground natural pipe.
5.2  **Tunnel Liner Investigations**

The tunnel liner investigations consisted of coring and hammer drilling through the tunnel’s concrete liner to investigate the thickness of the concrete and explore for voids behind the liner.

5.2.1  **Tunnel Liner Drilling**

Seven sites were selected along the tunnel to investigate the concrete liner. These investigation sites are numbered A through G and are shown on Figures 3 and 5. The drilling methods used to investigate the tunnel liner included coring and hammer drilling methods.

The concrete coring was completed using a portable core machine to obtain 3-inch diameter cores of the concrete. Concrete cores were collected from Sites A, B, C and D.

The hammer drilling method used a 3/8-inch diameter, 18-inch long hammer bit to penetrate the concrete. A total of 21 hammer holes were drilled in the concrete liner, 19 in the arch (3 at Sites A-F; 1 at Site G) and two into the sides (Sites E and G). The three arch holes at Sites A-F were drilled on the centerline of the crown and to the left and to the right of the crown.

Once the holes were drilled in the liner, the concrete thickness was measured. This was done using a tape measure for the core holes and the hammer bit for the hammer holes. Then a 1/4-inch aluminum probe was used to measure the length of void behind each hole as the difference between the length of the probe in the hole and the measured thickness of the concrete. The probe was also used to help obtain small amounts of bedrock from behind the liner. Photographs were taken of the concrete cores and of the holes made by the coring machine. After the investigations of the concrete liner were completed, all the core borings, hammer drill holes, and anchor holes were patched with high strength grout.

Cross-sections at each site are presented on Figure 6 showing the location of the drill holes, the thickness of the concrete liner and the dimensions of the void between the liner and the bedrock. Details of the tunnel liner drilling and findings with photographs of the work are provided in Appendix B. The description of the tunnel investigation sites references the left and right side of the tunnel facing downstream.

The drilling of the tunnel liner found the concrete thickness to range from about 7 inches to more than 18 inches. No timbers or other temporary support was found behind the concrete liner. The tunnel was apparently constructed by excavating a 12-foot long section, installing the wooden forms, placing the unreinforced liner concrete, and then repeating the excavation and support sequence. The bedrock apparently was sufficiently competent to support itself until the concrete liner was in-place.

The liner concrete includes up to 2-inch diameter rounded, alluvial gravels. The large aggregate made the concrete difficult to drill, but probably enhances the concrete strength and durability. Although much of the concrete was dense and appeared to be relatively well consolidated, zones of honeycombing and voids were common in the cores that were obtained and in the hammer drill holes.
Several voids were encountered between the concrete and the bedrock outside the liner. The voids were up to 19 inches between the concrete and the bedrock at the drill locations. At most of the locations, however, the bedrock was found to be in contact with the outside of the liner. Figure 6 shows the voids and general conditions encountered at the boring locations.

Four concrete cores were tested in the laboratory for the unconfined compressive strength. The test results are summarized on Table 1 and the detailed data sheets are presented in Appendix C. The average unconfined compressive strength of the concrete cores is approximately 4,810 psi (pounds per square inch). The measured strength of the core from Site A (Station 0+80) was 3,350 psi; the core from Site B (Station 1+50) was 4,890 psi; the core from Site C (Station 2+30) was 5,000 psi; and the core from Site D (Station 3+10) was 6,010 psi. These strengths are consistent with or higher than modern normal strength concrete.

5.3 Bedrock Ridge Investigations

The investigations of the bedrock ridge penetrated by the tunnel consisted of drilling of a geotechnical test boring and laboratory analysis of selected soil and bedrock samples collected during the investigations. In addition to the samples collected during the drilling, one sample of the bedrock collected from Site B of the tunnel liner investigations and one from the downstream invert were also tested in the laboratory.

5.3.1 Geotechnical Drilling

The exploratory boring was drilled on the site to a total depth of 59.6 feet, using hollow stem auger methods to 20.8 feet deep and HQ coring methods for the remaining 38.8 feet. The location of the boring, along with other pertinent site features is shown on Figure 4 and on the geologic profile on Figure 5. The boring was logged as the drilling progressed, and samples were obtained of the various soils and bedrock encountered. The summary log of the exploratory boring is provided on Figure 7. Photographs of the individual core runs as they were extruded from the sampler are shown on Figure 8. After drilling, the hole was backfilled with a cement-bentonite grout mix.

The soils encountered during drilling consist of approximately 6.5 feet of residual soils overlying Cloverly Formation bedrock. The residual soils on-site are derived from weathering of the underlying bedrock. The soils consist of variegated fat clay that is moist, stiff, and has high to very high plasticity depending on the ratio of clay to sand or silt.

Bedrock was encountered in the boring at 6.5 feet below the ground surface. The Cloverly Formation bedrock consists of shale, sandy shale and shale with limestone concretions (Figure 7). The shale has a claystone texture and is hard, very low strength to low strength, slightly moist to moist, slightly weathered to fresh, massive, highly plastic and moderately fractured. The shale is variegated, ranging from grey to blue-grey to various shades of purple (Figure 8). It is locally sandy or grades to clayey sandstone. It also contains zones of thin (~1/8-inch thick) and soft bentonite clay beds. The shale is generally non-calcareous, but contains local calcareous zones. The shale in the bottom 17 feet of the boring contains relatively large limestone concretions. Standard penetration test blow counts in the shale are shown on the left side of the summary log on Figure 7. The blow counts range from 66 blows per foot to 87 blows per 11 inches. The last count (76 blows per 10 inches) was judged to be auger refusal because the final 25 blows bounced on the
rock. Rock Quality Designation (RQD) values for core obtained from the Cloverly Formation bedrock ranged from 19 to 100 percent, averaging 71 percent. If the RQD data is weighted by run length, the weighted average is 77 percent. This indicates that the rock quality can be described as fair to good (Deere & Deere, 1989). Groundwater was not encountered at the time of drilling, however, groundwater levels may fluctuate seasonally.

5.3.2 Laboratory Testing

Samples of the soils and bedrock were tested in a laboratory for various physical and engineering properties, including: moisture content, density, gradation (including hydrometer and percent passing the #200 sieve), Atterberg Limits, swell consolidation, unconfined compressive strength, soluble sulfate content and resistivity. Laboratory tests were performed on soil and rock samples obtained from the geotechnical boring, from the core hole at Site B of the tunnel liner investigation and from the outcrop at the downstream invert of the tunnel. The laboratory test results are summarized on Table 1. Laboratory data and gradation plots are included in Appendix C.

Laboratory testing indicates the residual soils are very fine grained fat clays with liquid limits of up to 91 percent and plasticity indices of up to 65 percent. One swell test indicates the soils exhibit up to 3 percent swell. The soils have natural moisture contents up to 22 percent and natural dry densities up to 95.7 pcf. The soils have soluble sulfate contents which are indicative of a high degree of attack for normal strength concrete and electrical resistivity, indicating a very severe degree of corrosivity for buried metal structures.

Natural moisture contents of the shale bedrock ranged from 9 to 17 percent, averaging 13 percent and the natural dry density ranged from 101.5 to 131.1 pcf, averaging 112.5 pcf. The percent passing the No. 200 sieve ranged from 71 to 99 percent, averaging 94 percent. The hydrometer tests indicate that the shale has clay contents ranging from 38 to 50 percent and silt contents ranging from 37 to 61 percent, averaging 44 and 51 percent, respectively. The Atterberg Limits tests performed on the shale indicate liquid limits ranging from 54 to 179 percent and the plasticity indices ranging from 32 to 155 percent, averaging 80 and 57 percent respectively. The Atterberg Limits data is presented in graphical form on Figure 9. Swell/consolidation tests indicate that the shale will swell between 1 and 12 percent, an average of 7 percent. Unconfined compressive strengths of the shale range from 190 to 3,050 psi, averaging approximately 1,200 psi. The shale has a water soluble sulfate content of 0.17 percent, which indicates a moderate to severe attack for normal strength concrete. The electrical resistivity was 236 ohm-centimeters indicating the shale has a very severe degree of corrosivity for buried metal structures.

Pinhole dispersivity tests were attempted on both soil and bedrock samples to quantify the erosive potential of the material. However, due to the highly plastic nature of the shale and residual soil, the holes would close when water was added prematurely terminating the tests. However, observations of the badlands topography and the chimneys and pipes clearly demonstrate the high erosion potential and the dispersive nature of the soil and bedrock.
In summary, the laboratory testing indicates the bedrock surrounding the tunnel is corrosive, has high sulfate contents, and is very plastic and will swell and expand when wetted. The damage documented in the existing tunnel liner arch and walls appears to be the result of the swelling properties of the bedrock documented by the laboratory testing. Similarly, the lab testing indicates the bedrock has index properties that indicate it is subject to frost heave. Combined with the swelling properties this most likely explains the damage to the upstream 90 feet of the tunnel invert.
6.0 **ALTERNATIVES ANALYSIS**

The goal of our investigations is to develop practical conceptual alternatives for rehabilitating the Shell Canal Tunnel. The criteria we used for this alternatives analysis includes the following factors:

- **Costs** - The alternatives must be cost effective.
- **Longevity** - The alternatives should ideally provide a service life similar to the original tunnel.
- **Maintenance** - The alternatives should not be maintenance intensive.
- **Constructability** - The alternatives should be readily constructible considering the site location and bedrock conditions. The construction should be possible within the annual ditch “off” season between approximately November 1 and April 1 (approximately five months).

In addition, the alternatives must provide the required capacity, typically 47 to 50 cfs with possible peak flow requirements of 100 cfs.

The Level I study (Engineering Associates, 2010) suggested four possible alternatives:

1. Slewing the tunnel with 63-inch diameter “Snap-Tite” HDPE pipe and grouting the annulus (estimated 2010 cost = $838,000)
2. Slewing the tunnel with 54-inch diameter “Snap-Tite” HDPE pipe and grouting the annulus (estimated 2010 cost = $721,000)
3. Concrete lining the tunnel (estimated 2010 cost = $1,198,000)
4. Excavating the overburden and tunnel liner and placing the canal in a 6-foot by 6-foot concrete box culvert (estimated 2010 cost without backfill = $3,729,000, with backfill = $4,562,000)

Based on the Level II investigations we have discarded Alternatives 3 and 4. In our opinion, concrete lining the tunnel has costs and constructability issues because of the existing tunnel dimensions. The Alternative 4 costs are prohibitive and in our opinion there are other more cost effective alternatives for the rehabilitation. Alternatives 1 and 2 are essentially the same alternatives, but with different hydraulic capacities.

Our inspections and investigations indicate that:

1. The tunnel is in a very poor condition, especially near the downstream end. Failure and collapse of the concrete liner and subsequent loss of use is a real possibility.
2. The dimensions of the tunnel are large enough for conventional tunnel rehabilitation with rockbolts, steel ribs or channel, and shotcrete; or rehabilitation by sleeving with a 63-inch, thick wall HDPE pipe.

3. The bedrock penetrated by the tunnel is mainly low to very low strength shales and sandstones of the Cloverly Formation. Since no temporary support, such as timber, was encountered behind the concrete liner, the rock apparently was firm to slow raveling ground, providing sufficient standup time for forming and placing the concrete before excavating the next 12-foot long section. The bedrock appears to be excavatable with heavy duty equipment using rippers. Blasting may be required locally in the confined excavations.

4. The soils and bedrock at the tunnel site are mainly very erodible, frost susceptible, plastic clays and shales with significant swell potential. The soils and bedrock have high soluble sulfate contents detrimental to normal strength concrete, and have severe corrosion potential for buried metal structures. Alternatives designed for rehabilitating the tunnel will need to account for the soil and bedrock properties.

6.1 Alternatives

There are several feasible alternatives that appear possible for rehabilitating the Shell Canal Tunnel. Based on the investigations and the factors listed above, we have focused on three alternatives. These are:

1. **Tunnel Sleeve**: The tunnel would be sleeved with a large diameter heavy duty HDPE pipe. Our analysis indicates a 63-inch diameter thick walled HDPE pipe with a DR 32.5 pressure rating will provide the strength and flow capacity required. The pipe would be blocked up on cradles or installed with rails and encased in cellular concrete. The pipe encased in cellular concrete will be a suitable design to resist the swell pressures and frost heave potential of the surrounding bedrock.

2. **Open Cut Channel**: Replace the tunnel with an open channel cut through the bedrock ridge. This alternative involves complete demolition and removal of the tunnel. The Cloverly Formation shales and sandstones would be excavated to match the exiting Shell Canal flow lines and dimensions.

3. **Conventional Rehabilitation of the Tunnel**: This work would be done using conventional tunneling methods; channel steel rockbolted in-place encased in shotcrete. The concrete tunnel invert would be demolished and replaced with a reinforced concrete invert.

The three alternatives are discussed in more detail in the following sections.

6.1.1 **Tunnel Sleeve**

This option probably has the least unknowns and construction risks, and is relatively simple. The tunnel invert would be removed, as appropriate, where it has been bowed upward or needs to be removed in order to maintain the required pipe grade. Cradle blocks would be installed along the length of the tunnel to block the pipe up at the required elevation. The thick walled HDPE pipe
would be pushed or pulled into the tunnel by cables from the downstream or upstream end of the tunnel or both. Each 40-foot segment of pipe would be butt welded as the pipe is pulled in. After the pipe is installed along the entire length of the tunnel, the ends would be temporarily bulkheaded and the annulus space between the pipe and the concrete tunnel liner (as well as possible voids between the existing concrete liner and the bedrock, if required) would be filled with cellular concrete backfill. The grouting would be completed in stages to keep from floating the pipe, or the pipe would be filled with water prior to grouting.

Canal hydraulics will be impacted with the installation of a pipe to sleeve the tunnel. We are proposing installing a 63-inch thick walled HDPE pipe with a DR 32.5 pressure rating. In addition new inlet and outlet structures will be constructed to replace the existing deteriorated structures. HEC-RAS computer models of existing and proposed conditions were used to evaluate the hydraulic impact of the proposed improvements for flows up to 100 cfs. Survey data provided us with a profile of the canal from the tunnel inlet structure upstream 1,450 feet to an existing inverted siphon. Additional surveying was completed for the inverted siphon and canal upstream of the inverted siphon inlet structure. The results of the model show the capacity of the tunnel lining exceeds 100 cfs, but that there will be a rise in the water surface upstream of the tunnel inlet. This rise is a result of the decreased cross sectional area of the pipe compared to the existing tunnel cross-section. At 100 cfs the proposed improvements will result in an increased water depth of 1.92 feet at the tunnel inlet structure. The water depth will translate upstream beyond the inverted siphon because there is very little slope on the canal profile upstream of the inlet to the existing inverted siphon. At flow rates of 50 cfs and higher, an increase in the water surface elevation at the downstream end of the inverted siphon will translate to an increase at the upstream end of the inverted siphon. The impacts of the tunnel sleeve option on the inlet to the inverted siphon are shown in the following table:

<table>
<thead>
<tr>
<th>Flow rate (cfs)</th>
<th>Water Surface Elevations (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inverted Siphon Inlet</td>
</tr>
<tr>
<td></td>
<td>Existing</td>
</tr>
<tr>
<td>25</td>
<td>4165.17</td>
</tr>
<tr>
<td>50</td>
<td>4165.50</td>
</tr>
<tr>
<td>75</td>
<td>4169.69</td>
</tr>
<tr>
<td>100</td>
<td>4175.33</td>
</tr>
</tbody>
</table>

For flow up to 45 cfs, the water surface at the inverted siphon and the canal upstream of the inverted siphon will show no rise from the tunnel sleeve. At flow rates above 50 cfs, capacity will have to be monitored to make sure the ditch bank is not overtopped. Our model shows that the effect of the tunnel sleeve alternative on the existing conditions of the siphon will not handle flows of 70 cfs and above. Downstream of the tunnel there will be increased velocity as water discharges from the pipe to the canal due to the decreased roughness of the pipe compared to the existing concrete tunnel lining. We have proposed riprap protection to control the erosion generated by the higher velocity.

The existing concrete inlet and outlet structures would be demolished and replaced with new reinforced concrete structures. The steel reinforcement will be designed to resist damage from differential swelling or frost heave of the foundation bedrock.

**Sheets 1 through 7 in Appendix D** show conceptual plans for this option.
6.1.2 Open Cut Channel

This option is mainly an earth and rock excavation project, but includes demolition and removal of the concrete tunnel liner and headwall structures. The soils and Cloverly Formation bedrock overlying the tunnel would be excavated and the tunnel itself removed. This project would involve a benched cut excavation approximately 60 feet deep. The cut slopes would be made at approximately 1.5:1 (horizontal to vertical), with the maximum vertical height between benches equal to 20 feet. A maintenance bench would be provided above the canal for equipment access. The canal cross-section at the base of the cut would be designed to match the existing upstream and downstream cross-sections. The earth and rock from the excavation would need to be disposed on adjacent properties.

Sheets 1 through 6 in Appendix E show conceptual plans for this option.

The open cut option will excavate through the entire bedrock ridge penetrated by the tunnel. The cut will have a maximum width of approximately 250 feet. Excavatability of the bedrock was considered a major concern to be addressed for this alternative. The bedrock penetrated by the exploratory borings found very low to low strength Cloverly Formation bedrock. In general, it appears the upper rock should be excavatable with heavy duty equipment equipped with rippers. However, blasting will probably be required, especially in the deeper bedrock and in more confined excavations.

A second major concern for this alternative is the erodibility of the soils and bedrock, and the potential for slope instability in the high cut slopes. The soils and bedrock are very erodible and maintenance of the slopes and benches should be anticipated. Sediment from the erosion will probably need to be removed from the ditch on a regular basis and may be especially high in the first year or two following excavation.

The proposed side slope angle of 1.5:1 (horizontal to vertical) was selected based on matching the existing cut slopes and natural slopes surrounding the project area. A minor shallow landslide is present in the channel at approximately this same slope angle downstream of the tunnel (see Photograph P41 in Appendix A). This slope was probably undercut with the water flows in the canal. Similar slope failures may occur along the new channel slopes in response to canal flows and there is always some risk for larger scale landslides.

The excavated soils and bedrock, as well as concrete from demolition of the tunnel liner, will need to be permanently disposed on nearby adjacent lands. The materials will need to be placed in a relatively stable fill configuration with graded side slopes. Given the badlands nature of the surrounding areas, vegetating the stockpiled materials is probably not feasible.

6.1.3 Conventional Rehabilitation of the Tunnel

This option would utilize conventional methods for tunnel rehabilitation. The concept would involve using two piece channel steel prefabricated in the shape of the tunnel. The steel channels would be bolted in-place on approximately 4-foot centers along the entire tunnel using 5-foot long epoxy resin encapsulated and anchored rock bolts drilled through the concrete liner into the rock above and outside the liner. The channel sets would then be encased in a 4-inch thick layer of high
strength shotcrete applied to the inside of the liner. As well as providing additional strength to the liner, the shotcrete encasement will also provide corrosion protection for the steel sets and rockbolts.

This alternative will also require the demolition and replacement of the tunnel invert with reinforced concrete, and the construction of reinforced concrete inlet and outlet structures to replace the existing deteriorated structures.

The tunnel presents challenges for conventional tunnel rehabilitation, mainly because of the relatively small tunnel dimensions. The cross-section will be tight for both the drilling of the rockbolts and the shotcrete application. The work will be mainly manual, labor intensive work with the drilling completed with mining type “jack legs” and shotcrete would be applied by a nozzleman rather than a remote controlled machine. Because of the small dimensions, considerable rebound and waste would be expected during shotcrete application. Unit costs for this option are expected to be very high.

6.1.4 Summary of Analysis

In order to provide a basis for selecting an alternative to carry forward to construction, we prepared a matrix of technical advantages and disadvantages for the three alternatives. This is shown on the following table. Based on this analysis, sleeving the tunnel with heavy duty HDPE appears to be the best alternative with the least risk for both short-term construction issues and long-term operational and maintenance issues. The open channel option appears to provide a secondary practical alternative that merits costing and conceptual design as well. Based on the high level of uncertainty and anticipated high costs associated with the labor intensive work, conventional tunnel rehabilitation was dropped from further consideration and was not costed.
## Alternatives Analysis

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Sleeve the Tunnel with Heavy Duty HDPE Pipe</td>
<td>▪ Fewest unknowns for design and construction</td>
<td>▪ Impacts on canal hydraulic capacity (reduced conveyance cross-section)</td>
</tr>
<tr>
<td></td>
<td>▪ Relatively simple construction sequence</td>
<td>▪ Requires specialty contractor and equipment</td>
</tr>
<tr>
<td></td>
<td>▪ Short construction schedule</td>
<td>▪ Requires construction of new inlet/outlet structures</td>
</tr>
<tr>
<td></td>
<td>▪ All construction within existing BLM right-of-way</td>
<td></td>
</tr>
<tr>
<td></td>
<td>▪ Long expected service life</td>
<td></td>
</tr>
<tr>
<td></td>
<td>▪ Low maintenance (same as present)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>▪ Long expected service life</td>
<td></td>
</tr>
<tr>
<td>2. Open Cut Channel</td>
<td>▪ Removes a structure from the canal</td>
<td>▪ Requires additional right-of-way</td>
</tr>
<tr>
<td></td>
<td>▪ Maintains or improves existing canal capacity</td>
<td>▪ Results in high, erodible cut slopes; risks of slope failure for cut slopes and ditch banks</td>
</tr>
<tr>
<td></td>
<td>▪ Local earthwork contractor can probably complete work</td>
<td>▪ Risks for encountering hard bedrock requiring blasting for excavation</td>
</tr>
<tr>
<td></td>
<td>▪ No corrosion risks</td>
<td>▪ Removes access to the other side of the canal at this location</td>
</tr>
<tr>
<td>3. Conventional Tunnel Rehabilitation with Steel Sets, Rockbolts and Shotcrete</td>
<td>▪ All construction within existing BLM right-of-way</td>
<td>▪ Largest construction unknowns for required scope, schedule, and costs</td>
</tr>
<tr>
<td></td>
<td>▪ Low maintenance (same as present)</td>
<td>▪ Highest labor requirements</td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Requires specialty contractors</td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Some impacts on canal hydraulic capacity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Corrosion risks for steel sets and bolts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Requires demolition and replacement of tunnel invert</td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Requires construction of new inlet/outlet structures</td>
</tr>
</tbody>
</table>
7.0 CONCEPTUAL DESIGNS and COST ESTIMATES

The two alternatives carried into conceptual design and costing are the tunnel sleeve and the open cut options. The conceptual design plan sets for each alternative are presented in Appendices D and E, respectively. This section will outline the conceptual design and costs of the pertinent construction items, contingencies, permitting, engineering, observation and quality control associated with each alternative. A life cycle cost analysis including costs associated with operation, maintenance, replacement, and annual permit fees was also completed for each alternative.

7.1 Tunnel Sleeve

The results of the alternatives analysis indicate that sleeving the tunnel is the best rehabilitation option for the Shell Canal Tunnel in terms of service life, constructability, risk of failure and long term maintenance. Construction of the tunnel sleeve will occur after the irrigation season when the canal is out of service. A summary of the engineer’s opinion of costs is presented in Table 2. The conceptual design plans are included in Appendix D.

The first construction items considered for the tunnel sleeve alternative include mobilization, bonding, insurance; site access; demolition; and earthwork. Mobilization, bonding and insurance costs are estimated to be 10 percent of the construction cost. Earthwork will be required to improve the site access roads, and to provide access to both the tunnel inlet and outlet. Access roads to these structures will be cut into the existing canal banks on the eastern side of the canal. Once access to the tunnel is open, the inlet and outlet structures, as well as the first 90 feet of the invert, will be demolished and disposed offsite. Additionally, the remainder of the existing tunnel invert will be thoroughly cleaned of any sediment/debris deposits. The total cost for these items is estimated to be approximately $106,500.

After demolition, pipe skids (casing spacers), pipe cradles, or rails will be used to aid in the pipe installation and to help establish the design invert for the pipe. The 63-inch thick wall HDPE pipe will be installed in the existing tunnel by jacking the pipe into place on top of the railing. The entire length of the existing tunnel (562 feet) will be sleeved and approximately 500 cubic yards of cellular concrete backfill will be used for the annulus backfill and to fill the larger holes/cracks in the existing tunnel liner. The total cost of these items including construction of the temporary bulkheads for placement of the cellular concrete comes to approximately $605,250.

When the cellular concrete backfill is set and the sleeve is in place, construction can begin on the inlet and outlet structures. These structures will be built using approximately 84 cubic yards of reinforced concrete. Approximately 150 cubic yards of grouted riprap will be placed upstream of the inlet and downstream of the outlet to define and stabilize the channel cross-section. Additional erosion control will be employed in the vicinity of the two structures to minimize erosion from the surrounding area around the structures. The total estimated cost for construction of the inlet and outlet structures is approximately $85,500, bringing the total estimated construction cost for the tunnel sleeve option to approximately $797,250.
In addition to the construction items, we have estimated costs for 20 percent contingency, engineering/contracts/bidding, permitting, construction observation and quality control testing, totaling approximately $322,500. The total estimated cost of the Shell Canal Tunnel sleeve alternative, rounded to the nearest $1,000, is approximately $1,120,000.

Life cycle costs were also analyzed for the tunnel sleeve alternative. Annual operation and maintenance (O&M) costs associated with this alternative were considered to be similar to those accrued for the existing structure. The primary O&M activity associated with the existing tunnel is maintaining the canal channel upstream and downstream of the structure. We feel that proposed improvements for the tunnel sleeving option will not increase existing O&M in this reach of the canal.

Based on the current condition of the existing inlet and outlet structures and the high potential for sulfate attack on the concrete, the replacement costs of the concrete comprising the proposed inlet and outlet structures was also analyzed. Replacement of the tunnel sleeve itself is not considered necessary and therefore was not analyzed as a replacement cost. Our cost estimate to construct the inlet and outlet structures and to replace riprap revetment at both structures along with engineering, is estimated at $103,000. Given a 50-year life, an annual interest rate of four percent, and an inflation factor of three percent, the present value replacement costs of the inlet and outlet structures will be approximately $63,000.

7.2 Open Cut Channel

The results of the alternatives analysis identified the open cut channel alternative as a practical and effective option for rehabilitating the Shell Canal Tunnel. This alternative appears to have more inherent risk associated primarily with slope instability and long term maintenance. However, channel hydraulics will improve, and it removes a structure from the canal. Like the tunnel sleeve alternative, construction of the open cut will also have to be performed after the irrigation season when the canal is out of service. A summary of the engineer’s opinion of costs is presented in Table 3. The conceptual design plans are included in Appendix E.

The first construction items evaluated for cost are mobilization, bonding, insurance and site access. Again, mobilization, bonding and insurance costs are estimated to be approximately 10 percent of the construction cost. Site access is an important construction consideration for this alternative because the excavated material will have to be transported and disposed offsite. Therefore, the site access road will have to be improved to be able to accommodate construction traffic. The site access road will be converted into a haul road between the tunnel site and the material stockpile area offsite about one-half mile to the south (see Figure 2). This will require widening the road and surfacing it with about four inches of gravel or road base material. The total estimated cost for these items is approximately $89,900.

The grading plan for the open cut alternative will consist of 1.5:1 (horizontal to vertical) slopes with eight-foot wide intermediate benches, a 25-foot wide maintenance road on the west side of the cut and a 12-foot wide access road on the east side of the cut. The benches will be sloped toward the canal and away from the longitudinal center of each bench to promote sheet flow and minimize rill erosion. Based on the results of the geotechnical investigations, it will likely be easier to excavate the soil and rock from the existing grade down to 10 feet compared to the harder rock below 10 feet.
Due to the likelihood that local blasting may be required for the rock below 20 feet, different unit costs were estimated for the two methods of excavation. The total volume to be removed is approximately 118,000 cubic yards, of which 50,000 cubic yards is estimated to be above 10 feet deep and 68,000 cubic yards below 10 feet. It is assumed that the material removed from the tunnel site will be disposed offsite approximately one-half mile to the south. Additional haul distances will increase the unit cost of removing the material. The material will probably be used to fill in deep gulches in the area. The estimated cost of the earthwork construction items is estimated to be approximately $676,000.

Demolition of the existing tunnel liner is a separate construction item because the concrete rubble can probably be used as riprap for erosion control in the drainage(s) where the fill is placed. When the open cut is finished, road surfacing will be applied to the maintenance bench and the access bench. The road surfacing will consist of a four-inch thick bed of gravel or road base material. This will help maintain the benches and associated maintenance access by discouraging erosion. The total estimated cost for these construction items is approximately $62,000, bringing the overall total estimated cost for construction items for the open cut to approximately $827,900.

In addition to the construction items, we have estimated costs for 20 percent contingency, engineering/contracts/bidding, permitting, construction observation and quality control testing, totaling approximately $331,000. Therefore, the total estimated cost of the Shell Canal Tunnel open cut alternative, rounded to the nearest $1,000, is approximately $1,159,000.

A life cycle cost analysis was performed for the open cut alternative as well. The O&M costs associated with the open cut will be significantly higher than those currently incurred at the existing structure because there will be approximately 600 feet of new channel, as well as about 3,000 feet of benches to maintain. The primary maintenance activities will be cleaning sediment deposits out of the channel and the maintenance of access roads. We feel that additional O&M will be primarily mobilizing in heavy equipment and paying for importing road surfacing material. We estimate an additional O&M expense of $3,000 per year.

As previously discussed, due to the relatively steep cut slopes adjacent to the canal, slope instability will be a maintenance concern. Throughout the life of the open cut channel there will be a risk that a large slope failure could occur and block the canal. However, because this will not occur on an annual basis, this was not included in the future O&M costs. Alternatively, the cost of removing such a mass of soil & rock from the open cut is better represented as a replacement cost. The total cost of removing a landslide mass from the canal is difficult to estimate without knowledge of the volume of material to excavate. For costing purposes therefore, the volume is assumed to be equal to about one-third of the volume excavated to build the open cut, or approximately 37,000 cubic yards. Based on this volume and a $5/yard cut and fill unit cost (future value), the replacement cost will have a future value of approximately $185,000.
8.0 PERMITS

Possible permits, easements and clearances were identified for construction of the tunnel rehabilitation alternatives. The BLM, Wyoming Department of Environmental Quality (DEQ), and the U.S. Army Corps of Engineers (Corps) were the principal federal and state agencies contacted to understand the permit issues.

8.1 U.S. Army Corps of Engineers

Conversations with the Corps indicate that no permits are required for construction of either alternative.

8.2 Bureau of Land Management

Meetings and correspondence with the BLM suggest that as long as work to rehabilitate the tunnel falls within the existing easement that only temporary use permits will be required to complete this project. Additionally, because the canal is listed on the national register of historic places a Class III Cultural Resource Inventory will need to be conducted prior to rehabilitating the tunnel.

The canal right-of-way act of 1891 (U.S. Code, 2011) governs the right-of-way for the Shell Canal and specifies a 50-foot easement. However, there is some uncertainty in the language of the act, specifically:

“… fifty feet on each side of the marginal limits thereof…”

The uncertainty lies in what the definition of “marginal limits” is. For the purposes of this investigation, the “marginal limits” of the Shell Canal has been interpreted to be the access road, or top of bank upstream and downstream of the tunnel. This interpretation is based on the fact that the Shell Valley Watershed Improvement District is responsible for maintaining the canal and the slopes of the banks on either side of it. Therefore, using surveyed cross-sections of the canal, the marginal limits were delineated both upstream and downstream of the tunnel. The 50-foot easement was applied to these boundaries and then projected along the existing tunnel alignment. The resulting area ranges from approximately 270 to 310 feet wide in the vicinity of the tunnel and is considered to be the right-of-way easement for this project (Figure 10).

The construction limits of the two tunnel rehabilitation alternatives identified were compared to the right-of-way easement in Figure 10. The tunnel sleeve and open cut alternatives appear to be constructible within the existing easement (Appendix D and E). This area has been reviewed by the BLM and it appears as long as the permanently disturbed ground is within these limits, then no additional right-of-way will be required. Therefore, the BLM requirements for this project are as follows:

1. Road Right-of-Way Amendment: The Shell Canal Company currently has an existing road right-of-way (WYW-089779). This will require a temporary use amendment/permit to allow for improvements of the road for construction.
2. **Temporary Use Permit**: The construction limits will be determined during final design. A temporary use permit will be required for areas outside of the easement. This area will be minimized and will be kept off BLM land, if possible. Once construction is complete and the land is restored to existing conditions, then the permit will terminate.

3. **Class III Cultural Resource Inventory**: This is required as part of the National Historic Preservation Act and will need to be complete prior to the construction of the project.

The BLM charges fees to process applications for temporary use permits. The amount is determined by the amount of time required to review the application. Most fees fall within Category 2 (8 to 24 hours, $394.00). It is anticipated this review would fall in that range. Additional time would result in higher fees. A Category 4 Review is the highest at 36 to 50 hours and a fee of $1,063. We have included permitting in our cost estimate to cover fees and the submittal of the applications.

8.3 **Wyoming DEQ**

A Storm Water Pollution Prevention Plan will be required by the state for this project. This will require filling out an application and submitting a report and plan sheets showing the Best Management Practices to be used to control storm water on-site. We have included time to complete the requirements for the application and design in the cost estimate under permitting and engineering.
9.0 ECONOMIC ANALYSIS and PROJECT FINANCING

9.1 Economic Analysis - Quantification of Losses

The economic impact of this project has been assessed in terms of the effect of a catastrophic failure of the existing Shell Canal Tunnel. Such a failure would prevent the delivery of irrigation water to approximately 3,500 acres of cropland. Such a failure is expected if the tunnel is not rehabilitated or replaced. We reviewed three areas of negative economic impact on the local economy: land value reduction, net revenue reduction, and total dollar losses. The purpose of this economic analysis is twofold; first to demonstrate the negative economic impact associated with a tunnel failure, and second, to demonstrate the Sponsor’s ability to pay for its portion of the project.

9.2 Land Value Reduction

According to the Bighorn County Assessor’s Office, irrigated cropland under the Shell Canal is valued at approximately $2,250 per acre. Because this area typically receives about eight inches of precipitation annually, irrigation is a critical component of crop production. Without an irrigation supply, this land would most likely be converted to dry land pasture, which has an estimated value of $750 per acre. Therefore, if the tunnel fails and the land reverts to dry land pasture there will be a reduction of approximately two-thirds of the existing land value, or $5.25 million. This reduction would adversely impact the regions property taxes for the length of time the canal is inoperable.

9.3 Revenue Reduction

In order to demonstrate the Sponsor’s ability to pay, the net revenues from irrigated crop production were estimated. The distribution of crops produced on the subject 3,500 acres were reported by the Sponsor as follows:

<table>
<thead>
<tr>
<th>Crop</th>
<th>Percent of Acreage</th>
<th>Acreage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alfalfa</td>
<td>50</td>
<td>1,750</td>
</tr>
<tr>
<td>Corn</td>
<td>12.5</td>
<td>437.5</td>
</tr>
<tr>
<td>Beans</td>
<td>12.5</td>
<td>437.5</td>
</tr>
<tr>
<td>Wheat</td>
<td>5</td>
<td>175</td>
</tr>
<tr>
<td>Barley</td>
<td>5</td>
<td>175</td>
</tr>
<tr>
<td>Oats</td>
<td>5</td>
<td>175</td>
</tr>
<tr>
<td>Pasture</td>
<td>10</td>
<td>350</td>
</tr>
<tr>
<td>Total</td>
<td>100</td>
<td>3,500</td>
</tr>
</tbody>
</table>

The yield for each crop was estimated using National Agricultural Statistics Service (NASS) average irrigated crop yields for 2004 through 2008 in Bighorn County. This represents the most recent available five year period for which crop yield and price data were available. Irrigated alfalfa yield data was not available in 2008, so 2003 through 2007 yield values were used. The market price for each crop product was estimated using NASS market year average prices for the State of Wyoming. Farm production costs are highly variable, and were estimated based on our knowledge of similar cropping systems in other locations. These quantities are summarized below:
The revenue reduction that would result from removing the irrigation supply from the subject 3,500 acres was also estimated. The net revenue generated on irrigated cropland under the Shell Canal is estimated to be approximately $94.05 per acre. The net revenue for the subject lands without irrigation water is estimated to be $7.50 per acre, which is the NRCS Grassland Reserve Program rental rate in Wyoming. This results in approximately $86.55/acre net revenue attributed to irrigation. Therefore, the loss of irrigation would result in a net revenue reduction of $86.55 per acre for every year the canal is out of operation due to a failure of the tunnel.

### 9.4 Indirect and Direct Losses

Direct and indirect losses stem from the economic multiplier effect of decreases in income on a regional economy. For example, if a farmer loses income because a failure at the tunnel prevents the irrigation of his farmland, then that farmer will have less money to spend in the local economy. This effect results in additional losses to other sections of the Wyoming economy. Therefore, failure to maintain the tunnel and continue irrigation water deliveries will directly affect the farmers’ incomes and indirectly affect the local economy.

The Bureau of Economic Analysis of United States Department of Commerce (USDCO) has produced estimates of the indirect income multipliers for Wyoming’s agricultural sector. Their latest published estimate of this multiplier blended for crop distribution of the affected area is 1.7045. Using this rationale, for every dollar of reduction of farm income in Wyoming, total income will decrease 1.7045.

Consequently, applying this multiplier to the decrease in farm income of $86.55/acre for 3,500 acres results in a loss of $516,336 of income for every year the land is not irrigated due to the tunnel failing.

### 9.5 Project Financing and Ability to Pay

It is estimated that the project costs will be $1,120,000 for the tunnel sleeve option and $1,159,000 for the open cut option. The chosen alternative will be financed with a mix of a grant and a loan to the Sponsor. Based on WWDC programs operating criteria, the grant percentage was assumed to be 67 percent and the loan percentage was assumed to be 33 percent. A loan interest rate of 4 percent
was assumed. The loan periods examined were 20, 30, 40, and 50 years. The table below presents the different loan options:

Tunnel Sleeve Option:

<table>
<thead>
<tr>
<th>Loan Period (Years)</th>
<th>Total Project Cost</th>
<th>Grant Amount</th>
<th>Loan Amount</th>
<th>Total Annual Loan Payment</th>
<th>Average Annual Cost per Acre</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>$1,120,000</td>
<td>$750,400</td>
<td>$369,600</td>
<td>$26,876</td>
<td>$7.68</td>
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<tr>
<td>30</td>
<td>$1,120,000</td>
<td>$750,400</td>
<td>$369,600</td>
<td>$21,173</td>
<td>$6.05</td>
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<tr>
<td>40</td>
<td>$1,120,000</td>
<td>$750,400</td>
<td>$369,600</td>
<td>$18,535</td>
<td>$5.30</td>
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<tr>
<td>50</td>
<td>$1,120,000</td>
<td>$750,400</td>
<td>$369,600</td>
<td>$17,106</td>
<td>$4.89</td>
</tr>
</tbody>
</table>

Open Cut Option:

<table>
<thead>
<tr>
<th>Loan Period (Years)</th>
<th>Total Project Cost</th>
<th>Grant Amount</th>
<th>Loan Amount</th>
<th>Annual Loan Payment</th>
<th>Annual O&amp;M</th>
<th>Total Annual Expenditures</th>
<th>Average Annual Cost per Acre</th>
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<tbody>
<tr>
<td>20</td>
<td>$1,159,000</td>
<td>$776,530</td>
<td>$382,470</td>
<td>$27,811</td>
<td>$3,000</td>
<td>$30,811</td>
<td>$8.80</td>
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<tr>
<td>30</td>
<td>$1,159,000</td>
<td>$776,530</td>
<td>$382,470</td>
<td>$21,911</td>
<td>$3,000</td>
<td>$24,911</td>
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<td>40</td>
<td>$1,159,000</td>
<td>$776,530</td>
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<td>$22,296</td>
<td>$6.37</td>
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<td>50</td>
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<td>$776,530</td>
<td>$382,470</td>
<td>$17,106</td>
<td>$3,000</td>
<td>$20,701</td>
<td>$5.91</td>
</tr>
</tbody>
</table>

The average annual cost of the project to the Sponsor is estimated to be between four and seven percent of the current net revenue of the irrigated crop production on the subject 3,500 acres. This indicates that the cost of the project is within the Sponsor’s ability to pay.
10.0 CONCLUSIONS and RECOMMENDATIONS

The Shell Canal Tunnel Level II Feasibility Study was performed to identify potential tunnel rehabilitation alternatives. Deere & Ault Consultants has worked closely with the Project Sponsor and the Project Manager during the investigation. Historical information and documents were reviewed to understand the construction and operational background of the tunnel. Access to the tunnel was identified in terms of property ownership and easements.

On March 18, 2010 a thorough inspection of the tunnel was performed. The inspection identified areas of the tunnel that were in poor condition and warranted rehabilitation and/or replacement. In November, 2010 several geotechnical investigations were performed at the tunnel site. These investigations consisted of performing geologic reconnaissance of the site; drilling a geotechnical boring through the bedrock at the site; drilling several core and hammer holes in the existing tunnel liner; and testing samples of soil, rock and concrete for physical and engineering properties in a laboratory.

Using the data collected from the geotechnical investigations, an alternatives analysis was performed. The results of the analysis identified two practical and constructible tunnel rehabilitation alternatives: a tunnel sleeve and an open cut channel. These two rehabilitation alternatives were carried forward into conceptual design and cost estimates. The engineer’s opinion of costs show that construction of the tunnel sleeve will be approximately $1,120,000 while the open cut channel will be approximately $1,159,000.

Permits, easements and clearances needed to build each rehabilitation alternative were identified. Construction of the tunnel sleeve can be done within the existing easement as it is understood. However, to build the open cut option, a new right-of-way easement may need to be obtained, which will include an annual fee be paid to BLM.

Finally, an economic analysis was performed to show how each project would be financed. The results of this analysis show that the average annual cost of this project will be between four and seven percent of the current net revenue of the irrigated crop production. This indicates that the cost of the project is within the Sponsor’s ability to pay.

Based on the conclusions of the Shell Canal Tunnel Level II Feasibility Study, we recommend that the tunnel sleeve rehabilitation alternative be carried into final design for the following reasons. First, the cost estimates for each of the alternatives are similar, but the tunnel sleeve appears to cost about $39,000 less. Second, the tunnel sleeve option presents the fewest risks and uncertainties associated with design, construction, operation and maintenance. Conversely, the open cut option presents a risk of slope instability and will require additional maintenance because of erosion and sedimentation beyond what is currently needed at the tunnel. The increased maintenance will primarily include cleaning sediment from the channel and from the maintenance and access road benches. Further, the potential risk of a large landslide blocking the open cut channel will result in replacing (re-excavating) at least part of the open cut. Finally, the life of the tunnel sleeve will likely equal or surpass the 100-year service life of the existing tunnel liner.
11.0 DISCRETIONARY TASK

Additional tasks that discretionary funds have been used for include:

1. Topographic survey of the canal, access roads, inverted siphon, and waste area
2. Site visits for survey work and inverted siphon inspection
3. Inverted siphon and canal modeling
12.0 REFERENCES


TABLES
### Table 1 - Laboratory Test Results

#### Shell Canal Tunnel

<table>
<thead>
<tr>
<th>Hole</th>
<th>Depth (feet) or Tunnel Station (feet)</th>
<th>Sample Collected</th>
<th>Field Classification</th>
<th>Natural Moisture Content (%)</th>
<th>Natural Dry Density (pcf)</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
<th>Liquid Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Swell/Settlement Consolidation @ 1.0 KSF (%)</th>
<th>Unconfined Compressive Strength (psi)</th>
<th>Soluble Sulfates Content (%)</th>
<th>Resistivity (ohm cm)</th>
<th>Unified Soil Classification Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Rock Boring</strong></td>
<td></td>
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</tr>
<tr>
<td>B-1</td>
<td>5.0-6.0 Zip Residual Soil</td>
<td></td>
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<tr>
<td>B-1</td>
<td>6.5 Cal Residual Soil</td>
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<td>B-1</td>
<td>20.8 Cal Shale</td>
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<td>B-1</td>
<td>Run 2: 23.7-24.3 HQ Core Shale</td>
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<td>B-1</td>
<td>Run 4: 30.2-30.7 HQ Core Shale</td>
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<td>3,050</td>
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<td>Run 5: 37.0-38.0 HQ Core Shale</td>
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<td>Run 6: 40.5-41.4 HQ Core Shale</td>
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<td>B-1</td>
<td>Run 7: 44.1-45.0 HQ Core</td>
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<td>B-1</td>
<td>Run 9: 56.0-57.0 HQ Core Shale</td>
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</tr>
<tr>
<td><strong>Rock Samples Behind Tunnel Liner</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>B3</td>
<td>Sla. 1+50 Ziplock Shale</td>
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<tr>
<td>D/S Invert</td>
<td>Sla. 5+60 Ziplock Shale</td>
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<tr>
<td><strong>Tunnel Liner Concrete Cores</strong></td>
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</tr>
<tr>
<td>A1</td>
<td>Sla. 0+80 3-inch core Concrete</td>
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<td>3,350</td>
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<td>B3</td>
<td>Sla. 1+50 3-inch core Concrete</td>
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<td></td>
<td></td>
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<td></td>
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<td>4,890</td>
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<tr>
<td>C1</td>
<td>Sla. 2+30 3-inch core Concrete</td>
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<td></td>
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<td>Sla. 3+10 3-inch core Concrete</td>
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<td></td>
<td></td>
<td>6,010</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **Silt & Clay Materials (71 to 99%)**
- **Plasticity Index** - Indication of expansive nature of bedrock, 34 to 155% (CH) is very expansive.
- **Swell Potential** is Low = 1% to Very High = 12%
- **Unconfined Compressive Strength** - Bedrock is 190 to 3,050 psi - very low strength bedrock
- Bedrock is 3,350 to 6,010 psi - normal strength to high strength concrete

- **Swell Settling** - Consolidation @ 1.0 KSF (%) (-Value = Consolidation)
- **Soluble Sulfates Content (%)**
- **Resistivity (ohm cm)**
- **Unified Soil Classification Symbol**

*S Soluble Sulfate Content of 0.17 to 1.05% indicates moderate to severe conditions for concrete. Specify Type II concrete and a water/cement ratio (by weight) of 0.45. Specify high strength concrete 4,250 psi (typ.).

**Resistivity of 79 to 236 ohm cm indicates the soils and bedrock are extremely corrosive for buried metal structures or metals in contact with soils or bedrock.
# TABLE 2 - TUNNEL SLEEVE

**SHELL CANAL TUNNEL**  

**ENGINEER’S OPINION OF PROBABLE CONSTRUCTION COSTS**  

*September 6, 2011*

<table>
<thead>
<tr>
<th>Construction Item</th>
<th>Quantity</th>
<th>Unit</th>
<th>Cost</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Mobilization, Bonding, Insurance @ 10%</td>
<td>1</td>
<td>LS</td>
<td>$72,500</td>
<td>$72,500</td>
</tr>
<tr>
<td>2 Site Access</td>
<td>1</td>
<td>LS</td>
<td>$9,000</td>
<td>$9,000</td>
</tr>
<tr>
<td>3 Demolition</td>
<td>1</td>
<td>LS</td>
<td>$15,000</td>
<td>$15,000</td>
</tr>
<tr>
<td>4 Earthwork</td>
<td>1</td>
<td>LS</td>
<td>$10,000</td>
<td>$10,000</td>
</tr>
<tr>
<td>5 Pipe Installation (63” Dia. HDPE DR 32.5)</td>
<td>565</td>
<td>LF</td>
<td>$850</td>
<td>$480,250</td>
</tr>
<tr>
<td>6 Cellular Concrete</td>
<td>500</td>
<td>CY</td>
<td>$250</td>
<td>$125,000</td>
</tr>
<tr>
<td>7 Concrete Inlet Structure</td>
<td>1</td>
<td>LS</td>
<td>$30,000</td>
<td>$30,000</td>
</tr>
<tr>
<td>8 Grouted Ripap Inlet</td>
<td>90</td>
<td>CY</td>
<td>$150</td>
<td>$13,500</td>
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<tr>
<td>9 Concrete Outlet Structure</td>
<td>1</td>
<td>LS</td>
<td>$30,000</td>
<td>$30,000</td>
</tr>
<tr>
<td>10 Grouted Ripap Outlet</td>
<td>60</td>
<td>CY</td>
<td>$150</td>
<td>$9,000</td>
</tr>
<tr>
<td>11 Erosion Control/ Storm Water Management</td>
<td>1</td>
<td>LS</td>
<td>$3,000</td>
<td>$3,000</td>
</tr>
</tbody>
</table>

**Total Construction Items** $797,250  
Engineering/Contracts/Bidding @ 10% $80,000  
Permitting $7,500  
Construction Observation @ 8% $64,000  
Quality Control Testing $12,000  
Contingency @ 20% $159,000

**ESTIMATED TOTAL (rounded to nearest $1,000)** $1,120,000

**Notes:**
1. Permitting costs are estimated based on preliminary discussions with the BLM and Corp of Engineers.
2. Demolished concrete from headwalls and tunnel invert is assumed be stockpiled within a 1/2 mile haul of the tunnel location.
3. Construction observation is based on one full time employee for a 3 month construction schedule.
# TABLE 3 - OPEN CUT

**SHELL CANAL TUNNEL**

**ENGINEER’S OPINION OF PROBABLE CONSTRUCTION COSTS**

*September 6, 2011*

<table>
<thead>
<tr>
<th>Construction Item</th>
<th>Quantity</th>
<th>Unit</th>
<th>Cost</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mobilization, Bonding, Insurance @ 10%</td>
<td>1</td>
<td>LS</td>
<td>$74,900</td>
<td>$74,900</td>
</tr>
<tr>
<td>Site Access</td>
<td>1</td>
<td>LS</td>
<td>$15,000</td>
<td>$15,000</td>
</tr>
<tr>
<td>Earthwork - Cut and Fill (Existing Grade - 10 FT Deep)</td>
<td>50,000</td>
<td>CY</td>
<td>$4</td>
<td>$200,000</td>
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<tr>
<td>Earthwork - Cut and Fill (Below 10 FT Deep)</td>
<td>68,000</td>
<td>CY</td>
<td>$7</td>
<td>$476,000</td>
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<tr>
<td>Demolition - Concrete Tunnel</td>
<td>1</td>
<td>LS</td>
<td>$40,000</td>
<td>$40,000</td>
</tr>
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<td>Road Surfacing</td>
<td>1,200</td>
<td>CY</td>
<td>$15</td>
<td>$18,000</td>
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<tr>
<td>Erosion Control/ Storm Water Management</td>
<td>1</td>
<td>LS</td>
<td>$4,000</td>
<td>$4,000</td>
</tr>
</tbody>
</table>

**Total Construction Items** $827,900
- Engineering/Contracts/Bidding @ 10% $83,000
- Permitting $10,000
- Construction Observation @ 8% $66,000
- Quality Control Testing $6,000
- Contingency @ 20% $166,000

**ESTIMATED TOTAL (rounded to nearest $1,000)** $1,159,000

### Notes:
1. Excavated soil assumed to be hauled and placed within 1/2 mile from the tunnel location with earth moving scrapers and or off road haul trucks
2. Demolished concrete from tunnel is assumed be stockpiled within a 1/2 mile haul of the tunnel location
3. Permitting costs are estimated based on preliminary discussions with the BLM and Corp of Engineers
4. Construction observation is based on one full time employee for a 3 month construction schedule
FIGURES
Legend

Geotechnical Boring B-1

Approximate Tunnel Centerline

Strike and Dip of Bedding (Degrees)

6°

Geology, contacts approximately located (after Reppe, 1986)

- Qt4 - Upper Shell Creek Terrace Deposits.
- Ksm - Sykes Mountain Member of the Cloverly Formation. Light brown siltstone and sandstone.
- Kcl - Cloverly Formation, Undivided. Variegated mudstone and shale.
Atterberg Limits Plot

- Residual Soil
- Cloverly Formation

Plasticity Index (PI) vs Liquid Limit (LL)

- "U" line
- "A" line

Scale: 1 inch = 30 percent
Legend

- Shell Canal
- Approximate Tunnel Centerline
- Tunnel Access Road
- Topographic Contour Lines

Shell Canal
Approximate Tunnel Centerline
Tunnel Access Road
Topographic Contour Lines

Surveyed Marginal Limits
Tunnel Inlet
Tunnel Outlet

50-FOOT EASEMENT BOUNDARIES
(Offset 50 feet from Marginal Limits Projected Along Tunnel)
APPENDIX A
TUNNEL INSPECTION
INTRODUCTION

The detailed inspection of the Shell Canal Tunnel was performed on March 18, 2010. The inspection began at the tunnel inlet and progressed downstream. The following discussion is presented in eight sections describing the tunnel condition in 100-foot intervals. Selected photographs supporting the text are included at the back of this appendix. The Tunnel Liner Crack Map (Figure 3) is the primary figure summarizing the inspection and can be referred to throughout.

A.1 Inlet Structure (Station 0+00)

The inlet structure at Station 0+00 is on the north end of the tunnel. The crown of the arch is cracked through the full thickness of the liner, which appears to be approximately 10 inches (Photograph P1). The transition wall on the left side of the inlet structure is cracked and broken away from the tunnel headwall. The crack is about 5 to 6 inches wide at the bottom and extends from the tunnel floor approximately 2 feet up the straight leg where it branches off into two cracks (P2 and P3). The main crack narrows to about 3 to 4 inches wide and separates the transition wall from the headwall (P4), while the other pinches down to an aperture of about 1/4-inch. The large opening at the bottom is about 1.5 feet deep (P5) and allows water to get behind the tunnel liner. The invert leading into the tunnel is gone for the first few feet, which has resulted in undermining both the left (P2 and P3) and right (P6 and P7) head wall. The undermining of the foundation has exposed aggregate in the liner and allows cracks to propagate towards the cold joint between the transition wall and the headwall. The diagonal crack is shown on P6 and P7, begins at the top of the undermined part of the head wall, and appears to be propagating toward the transition wall. The overall condition at the inlet structure is classified as very poor due to the failing left transition wall and the absence of the invert.

A.2 Station 0+00 to 1+00

The tunnel cross-section is generally in good condition with minor longitudinal cracks on the left and right sides of the tunnel. This pervasive longitudinal cracking is present along the tunnel walls at 2.5 to 3 feet off the tunnel invert. The cracks are mainly along the junction of the vertical straight leg portion of the wall and the beginning of the arch and are relatively tight. The longitudinal cracking appears to be from compressive forces exerted on the tunnel by either swelling soils or expansion and contraction movement related to freeze and thaw of water behind the walls.

The first 20 feet of this reach has several transverse cracks that run around the cross-section of the tunnel (Figure 3). These cracks are newer than the longitudinal cracks. Photograph P8 shows an example of these transverse cracks at approximately Station 0+10. Photograph P9
shows an area of overbreak associated with the transverse crack near Station 0+10. These areas of overbreak exhibit exposed aggregate.

Transverse cracks are also located at most of the cold joints from the construction of the tunnel. The transverse cracks at the cold joints from approximately Station 0+20 to 1+00 appear to be old. The transverse cracks at cold joints typically occur from the expansion and contraction of the concrete. These cracks are common and are expected to develop in concrete.

Some aggregate is exposed on the lower portion of the tunnel walls at the intersection with the tunnel floor, mainly at the cold joints (P10). In these areas the longitudinal cracks typically intersect the areas of exposed aggregate. Damage is old. There is some evidence of patching that was done with skim coating or some type of epoxy painting over the cracks.

The concrete tunnel invert has heaved and is buckled and completely broken across the width of the tunnel bottom from the inlet structure to Station 0+90 (P11). The unreinforced concrete is broken into pieces that are roughly 2 to 3 feet wide by 2 to 3 long. The buckling is most severe along the centerline of the invert exhibiting a maximum amplitude of approximately 12 inches. Beyond Station 0+90 the invert is intact. Standing water and deposits of sand and gravel 2 to 3 inches thick occur across the invert.

Some of this tunnel reach is in very poor condition because of the condition of the floor and the transversal cracks in the first 20 feet of the tunnel. However, the tunnel cross-section from 0+20 to 1+100 appears to be in good condition. The overall rating of the tunnel liner from 0+00 to 1+00 is poor.

A.3 Station 1+00 to 2+00

Pervasive longitudinal cracking continues on tunnel walls at 2.5 to 3 feet above the tunnel invert. The crack on the left side exhibits small displacement (less than a 1/2-inch) in several locations. The concrete invert is generally structurally sound and in good condition, but exhibits some exposed aggregate and occasional contraction/expansion cracks. Sand, gravel and debris occur locally on the invert, approximately 2 to 5 inches thick. At Station 1+45 there is a noticeably larger longitudinal crack at the cold joint (Figure 3; P12 and P13). The longitudinal crack in this reach starts to meander up and down the wall on both the left and right sides from roughly 2 feet above the invert to about three-quarters of the way up the arch (4.5 feet). The longitudinal crack also splays into cracks going into different directions (spider cracking) along this reach. There is evidence that this crack has been patched in the past (P14).

The shape of the arch in this reach is regular and is in good condition. Exposed aggregate was observed at the cold joints on the right and left sides along the bottom of the wall at the intersection with the invert. Some of these areas exhibit damage in the form of holes up to about 2 inches deep, such as the one at Station 1+92 (P15). The damage is old, but it could be a maintenance issue in the future. There was approximately 3 inches of water across the invert with a layer of ice forming during the colder months. This reach is in generally good condition.
A.4 Station 2+00 to 3+00

The pervasive longitudinal cracking continues on the sides of the tunnel. The cracks are relatively tight, with a slight offset at Station 2+50 (P16). The cracks meander up and down the wall from roughly 2 feet off the invert to about three-quarters of the way up the arch. There is exposed aggregate at the cold joints on the right and left sides along the bottom of the wall at the intersection with the invert with some small 2-inch holes in the concrete at several locations along this reach. The damage is old.

Between Stations 2+70 and 3+00 the concrete in the crown of the arch is showing signs of deterioration. In this interval the concrete is covered in a white effervescent crystalline deposit (Figure 3; P17 and 18). The deposit is likely a calcium deposit that has precipitated out of groundwater leaking into the tunnel. Reddish brown iron stains occur on the walls from groundwater leaking through the longitudinal cracks and running down the walls of the tunnel (P17). Additionally, there is exposed aggregate from a thin layer of the surface breaking off in several areas.

Some patching, possibly skim coating, was observed on the walls and arch. Some patches have broken off exposing aggregate behind the patch. During the inspection about 4 to 6 inches of water with a layer of ice was in the invert. The invert appears to be solid with some sand and gravel deposits across the tunnel. This reach is rated to be in fair condition.

A.5 Station 3+00 to 4+00

The pervasive longitudinal cracking continues along the walls of the tunnel. The cracks on the right side between Stations 3+11 and 3+23 rise about 2 feet up the arch, are open and offset up to 2 inches (Figure 3; P19 and P20). Effervescent calcium deposits and iron stains are also present up to approximately Station 3+30. The longitudinal cracks meander up and down the wall, from roughly 2 feet off the invert to about three-quarters of the way up the arch. The cracks show signs of displacement, indicating that the walls are increasingly deteriorating towards the downstream end of the tunnel.

The tunnel appears to have newer damage in the arch from Station 3+25 to 4+00. The concrete surface is breaking away and exposing aggregate, especially at the cold joints. This damage looks relatively new compared to the pervasive longitudinal cracks documented all along the tunnel at the straight leg/arch interface. Starting at about Station 3+50 a large, open longitudinal crack with about 2 inches of offset rises about a foot into the arch and extends to approximately Station 3+70 (Figure 3; P21). The crack is more open and offset at its highest point along the arch. It then becomes tighter as it meanders back into the straight leg portion of the liner about 2 feet off the invert. The crack rises again to about a foot into the arch at about Station 3+80 and becomes open and exhibits about 2 inches of offset (Figure 3; P22).

There is exposed aggregate at the cold joints on the right and left sides along the bottom of the wall at the intersection with the invert. This damage appears to be old and is characterized by small 2-inch holes at several locations. There is some patching on the walls and on the arch with exposed aggregate. In this reach there was about 6 to 10 inches of water with a layer of ice on
The sand and gravel deposits are about 4 to 6 inches deep. The invert is clean and solid in between the sediment deposits. The overall rating for this reach is poor.

A.6 Station 4+00 to 5+00

The pervasive longitudinal cracking continues on the left and right side of the tunnel. The cracks are offset up to 4 inches through this reach. These cracks meander up and down the wall, from roughly 2 feet off the invert to about three-quarters of the way up the arch. The cracks show more displacement indicating that the concrete liner is experiencing more deterioration towards the downstream end of the tunnel. From Station 4+35 to 4+45 the longitudinal cracks on the right side (P23 and P24) and left side (P25 and P26) move up the side of the wall into the arch and exhibit about 2 to 3 inches of displacement away from the wall (Figure 3). This damage appears newer. The wall on both sides of the tunnel in this 10 to 15-foot section could fail and break away from the tunnel. Irrigation water can easily get behind the liner in this reach and wash out soils and rock from behind the liner.

From Station 4+75 to 4+95 there are very severe longitudinal cracks on the left and right sides with large displacements of up to 4 inches and apertures of 1 to 2 inches (Figure 3). The crack on the right side is shown in photographs P27 and P28, while those on the left side are shown in photographs P29 and P30. In our opinion, these areas are in danger of failing. At Station 4+95 there is a severe transverse crack with displacement measuring about 2 inches around the entire cross-section of the tunnel. Through this section there was 10 to 12 inches of standing water with a layer of ice on top. The sand and gravel deposits locally occur on the invert. The invert appears solid where it is not covered with sediment. The overall rating for this reach is very poor.

A.7 Station 5+00 to 5+62

The longitudinal cracks on both sides of the tunnel and on the tunnel arch within this reach of the tunnel are severe, exhibiting a spider web appearance, offsets of 2 to 3 inches and apertures generally greater than 1 inch (Figure 3). Photographs P31, P32 and P33 document the severe cracking along the right side of the tunnel from Station 5+00 to 5+62. Photographs P34, P35 and P36 document the severe cracking along the left side of the tunnel from Station 5+00 to 5+62. The crown of the tunnel has a crack running down the center of the arch (P35), so where it intercepts the cold joints there are areas of significant aggregate exposure (Figure 3). The transverse cracks located at the cold joints contribute to the spider web cracking where they intersect the longitudinal and crown cracks. Due to the spider web character of the cracking in this reach, there is an increased probability of failure of individual blocks bounded by the cracks. Evidence of such failure can be seen where the last four feet of the straight leg portion of the right side of the tunnel is gone (P33).

There was about 12 to 15 inches of water with a layer of ice on top. The sand and gravel deposits were about 6 to 12 inches deep. Generally the tunnel liner is severely broken up and in danger of failure for the last 50 feet, and its condition is classified as very poor.
A.8 Outlet Structure (Station 5+62)

The outlet structure of the tunnel appears to be failing (Figure 3). The crown of the arch is cracked through the entire 10-inch thickness of the liner (P37 and P38). The entire transition wall on the right side of the outlet structure is gone along with the last four feet of the straight leg portion of the tunnel wall on the right side (P38 and P39). A very large void exists behind the wall on the right side of the outlet structure (P38 and P39). This void is experiencing headward erosion behind the tunnel liner. The erodible nature of the soil and bedrock has allowed water flow to undermine the foundation of the outlet structure. The transition wall on the left side of the outlet structure is cracked and broken and the lower part of straight leg portion of the tunnel wall and the headwall on the left side is missing (P36 and P40). Photographs document the poor condition of the tunnel liner, the right side of the arch at the outlet structure (P38), and the straight leg portion of the left side of the tunnel liner (P36). The outlet structure is classified as being in very poor condition.

The low strength, highly erosive character of the soil and bedrock at the site contributes problems at the tunnel outlet in other ways. The steep slopes leading down to the canal just downstream of the tunnel outlet are subject to slope stability problems, such as landslides, as shown in photograph P41. This photograph shows a slide deposit in the canal about 30 feet downstream of the tunnel outlet. The slide material in the channel causes water to backup into the canal, seep through the many cracks in the tunnel liner and further erode, or undermine the outlet’s foundation.

There are two gullies that have been carved into the shale bedrock on the east and west sides of the downstream tunnel portal (P42 and P43). The gulley on the west side is much larger than the one on the east side. During heavy rainstorms or rapid snowmelt, these gullies transport water and sediment down into the canal at the outlet works. In addition to flowing irrigation water during the irrigation season, the runoff also erodes and undermines the tunnel outlet.
Inlet Structure (Station 0+00)
P1 - Photograph showing the inlet structure of the tunnel. Notice the heaved invert in the tunnel, the absence of the invert in the first 3 to 4 feet and the through-going crack at the crown of the arch.

P2 - Photograph showing a large, open crack on the left transition wall of the inlet structure. The crack is about 3 to 4 inches wide.
P3 - Flash photograph showing the same large, open crack as in P2, and the character of the concrete forming the liner.

P4 - Photograph showing a branch of the large crack shown in P2 and P3 defining the separation between the left transition wall and the headwall.
P5 - Close up photograph of the same open crack for P2 and P3 showing the character of the concrete, the depth of the crack (~1.5 feet) and the thickness of the concrete (~12 inches).

P6 - Photograph showing undermining of the right transition wall where the floor is missing and aggregate is exposed at the bottom of the wall. There is also a relatively tight crack leading up the wall from the floor.
P7 - Close up photograph of the bottom of the right transition wall at the inlet showing the exposed aggregate.
P8 - Photograph showing example of a relatively large transverse crack around the entire arch at approximately Station 0+10 exhibiting about 1 to 2 inch offset and an overbreak near the crown (upper right).

P9 - Photograph showing the overbreak area near the crown and associated exposed aggregate along the transverse crack at approximately Station 0+10.
P10 - Photograph showing an example of exposed aggregate on a straight leg portion of the tunnel near the invert. Notice the longitudinal crack intersecting this exposed aggregate on the right side of the photo.

P11 - Photograph showing the heaved invert looking downstream from approximately Station 0+05. Notice the broken slabs of the invert measuring approximately 4 to 6 inches thick.
Station 1+00 to 2+00
P12 - Photograph showing a longitudinal crack at Station 1+45 approximately three quarters of the way up the arch on the right side of the tunnel. The crack has about 1 inch of offset.

P13 - Photograph showing the longitudinal crack at Station 1+45 terminating at a cold joint.
P14 - Photograph showing the transverse crack becoming smaller at Station 1+50. Notice the evidence of patching above this crack on the arch.

P15 - Photograph showing an example of exposed aggregate at a cold joint at Station 1+92. The hole here is about 2 inches deep.
Station 2+00 to 3+00
P16 - Photograph showing longitudinal crack with minor offset at Station 2+50.

P17 - Photograph showing white effervescent calcium deposits and reddish brown stains on the arch starting at Station 2+60 and becoming abundant at Station 2+70. These features are likely due to groundwater leaking into the tunnel.
P18 - Photograph showing heavy effervescent calcium deposits and exposed aggregate on the arch of the tunnel at Station 3+00.
Station 3+00 to 4+00
P19 - Photograph showing an open and offset (~1 to 2 inches) longitudinal crack about 2 feet up the arch between Stations 3+11 and 3+23. Also, notice the reddish brown iron stains and the effervescent calcium deposits on the wall.

P20 - Photograph showing the open and offset crack between Stations 3+11 and 3+23 along with effervescent calcium deposits and iron stained wall especially below the crack. This shows that water has seeped into the tunnel through this crack.
P21 - Photograph showing a large, open crack with about 2 inches of offset on the left side of the tunnel at about Station 3+60 to 3+65.

P22 - Photograph showing the large, open crack with about 2 inches of offset on the left side of the tunnel at about Station 3+80.
Station 4+00 to 5+00
P23 - Photograph showing the severe longitudinal crack on the right side of the tunnel from Stations 4+35 to 4+45.

P24 - Photograph showing a 2-inch offset and about 1/2 to 1 inch aperture on the same longitudinal crack in photo P23 from Stations 4+35 to 4+45.
P25 - Photograph showing the severe longitudinal crack on the left side of the tunnel from Stations 4+35 to 4+45.

P26 - Photograph showing a 3 - inch offset and about 1/2-inch aperture on the same longitudinal crack in photo P25 from Stations 4+35 to 4+45.
P27 - Photograph showing severe longitudinal cracking along the right side of the tunnel between Stations 4+75 and 4+95.

P28 - Photograph showing the severe offset (4 inches) and aperture (1 to 2 inches) associated with the longitudinal crack on the right side of the tunnel between Stations 4+75 and 4+95.
P29 - Photograph showing the severe longitudinal cracking on the left side of the tunnel between Stations 4+75 and 4+95. Notice the two sub-parallel cracks just to the left of the hammer.

P30 - Photograph showing the severe offset and aperture associated with the longitudinal crack on the left side of the arch at about Station 4+80.
Station 5+00 to 5+62
P31 - Photograph showing severe longitudinal cracking along the right side of the tunnel from about Station 5+30 looking towards the outlet.

P32 - Photograph showing the last six feet of the right side of the tunnel. Notice the spider web appearance of the cracking.
P33 - Photograph showing the void where the last 4 feet of the straight leg of the right side of the tunnel is missing.

P34 - Photograph showing severe longitudinal cracking along the left side of the tunnel from about Station 5+00 looking towards the outlet.
P35 - Photograph showing the severe spider web cracking on the left side of the tunnel and on the arch from approximately Station 5+40 to 5+55.

P36 - Photograph showing ongoing failure of the last 5 feet of the bottom of the straight leg portion on the left side of the tunnel liner.
Outlet Structure (Station 5+62)
P37 - Photograph showing the outlet structure. Notice the through going crack in the crown and the failing lower headwall.

P38 - Photograph showing the failure of the right side of the outlet structure. The right side of the straight leg wall and the right headwall are completely gone. The right side of the arch also appears to be on the verge of failure.
P39 - Photograph showing the void caused by headward erosion behind the right side of the outlet structure. The erosive character of the soil and bedrock at the tunnel contributes to the failure of the outlet structure.

P40 - Photograph showing the left side of the outlet structure with a large through going crack that is connected to severe longitudinal cracks on the left side of the tunnel liner. Also notice the undermining of the lower part of the headwall.
P41 - Photograph showing a large deposit of material in the canal about 30 feet downstream of the outlet structure. Notice the small landslide scarp just above the deposit on the right side of the photograph.

P42 - Photograph from the access road showing the two gullies on either side of the outlet structure.
P43 - Photograph from the canal showing the two gullies on the west (left) and east (right) sides of the outlet structure. Notice that the western gully is larger.
APPENDIX B
TUNNEL LINER DRILLING
TUNNEL LINER DRILLING

INTRODUCTION

The tunnel liner investigation was conducted during November 16 through 19, 2010. The following detailed discussion of the results of the investigation is organized into seven sections representing the Tunnel Liner Investigation Sites A through G shown in plan view on Figure 3 and in profile on Figure 5. The results are summarized in cross-sections on Figure 6, which can be referred to throughout.

B.1 Tunnel Liner Investigation Site A (Station 0+80)

At Site A, a three-inch concrete core was obtained on the right side of the tunnel roughly four feet off the tunnel floor. The concrete was measured to be 18 inches thick at this location. The first foot of the core was sound and contained abundant aggregate. However, the last six inches of the core contained several voids. The concrete core representing the tunnel liner at Site A is shown below.

[Photograph showing the concrete core collected from Site A (Station 0+80). Notice the relative sound nature of the concrete in the first foot of the core and the large, abundant voids in the last six inches.]

Shale bedrock was observed behind the concrete liner as well as a small void between the liner and the bedrock. The shale behind the liner was hard and grey. The void between the liner and the rock measured approximately 1 to 2 inches. The photo below shows the core hole in the tunnel liner at Site A.
After the coring machine penetrated the tunnel liner at Site A, water from the coring operations was observed flowing out of the longitudinal cracks in the liner near the drill hole. This indicates that these cracks penetrate the full thickness of the concrete liner.
Three holes were hammer drilled in the concrete liner on the arch at Site A. The following observations and measurements were made regarding liner thickness, void and bedrock type.

<table>
<thead>
<tr>
<th>Site A Arch Hammer Drilling Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>Left Side of Arch (29 inches from crown)</td>
</tr>
<tr>
<td>Crown</td>
</tr>
<tr>
<td>Right Side of Arch (22 inches from crown)</td>
</tr>
<tr>
<td>Concrete Liner Thickness (inches)</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>&gt;16</td>
</tr>
<tr>
<td>12</td>
</tr>
<tr>
<td>Void (inches)</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>Not Measured</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>Bedrock Type</td>
</tr>
<tr>
<td>Gray Shale</td>
</tr>
<tr>
<td>Not Observed</td>
</tr>
<tr>
<td>Gray Shale</td>
</tr>
</tbody>
</table>

Voids were encountered in concrete while drilling the last nine inches of the hole in the crown.

B.2 Tunnel Liner Investigation Site B (Station 1+50)

A three-inch concrete core was drilled on the left side of the tunnel at Site B roughly 2 feet 5 inches off the tunnel floor. The concrete was measured to be 11 inches thick at this location. The concrete core obtained from this part of the tunnel was very sound and had abundant aggregate, some up to two inches in diameter. Gray shale bedrock was observed behind the concrete liner. There was no void between the liner and the bedrock. Some of the shale bedrock was attached to the concrete core after it was removed. The photograph below shows the core from Site B.

Photograph showing the concrete core representing the tunnel liner at Site B. Notice the large two-inch aggregate on the left and the gray shale bedrock on the right.

Three holes were hammer drilled in the concrete liner on the arch at Site B. The following observations and measurements were made regarding liner thickness, void and bedrock type.
Site B Arch Hammer Drilling Observations

<table>
<thead>
<tr>
<th></th>
<th>Left Side of Arch (32 inches from crown)</th>
<th>Crown</th>
<th>Right Side of Arch (25 inches from crown)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Liner Thickness (inches)</td>
<td>11.5</td>
<td>9</td>
<td>&gt;16</td>
</tr>
<tr>
<td>Void (inches)</td>
<td>6.5</td>
<td>5</td>
<td>Unknown</td>
</tr>
<tr>
<td>Bedrock Type</td>
<td>Fine Clayey Sandstone</td>
<td>Not Observed</td>
<td>Not Observed</td>
</tr>
</tbody>
</table>

While drilling the hammer hole on the crown, a void was encountered at approximately eight inches into the liner. The hole on the right side of the arch was drilled to the maximum depth of the drill (16 inches) and neither the drill nor the probe could break through the concrete. Voids were encountered in the hole on the right side of the arch between 12 and 16 inches. The liner is thicker than 16 inches at this location. While drilling the hole on the left side of the arch, voids were encountered between 7.5 and 11.5 inches. The probe was able to dislodge fine grained clayey sandstone pieces.

B.3 Tunnel Liner Investigation Site C (Station 2+30)

A 3-inch concrete core was sampled from the right side of the tunnel liner roughly 2 feet 4 inches off the tunnel floor at Site C. The concrete was measured to be 13 inches thick at this location. A piece of brown poorly graded, fine-grained sandstone bedrock was observed attached to the core and behind the concrete liner. Therefore, there was no void between the liner and the bedrock. The concrete core was very sound and had a small crack in it at a depth of approximately 12 inches. The photograph below shows the core recovered from the right wall at Site C.

Photograph showing the concrete core representing the tunnel liner at Site C. Notice the brown sandstone bedrock on the right attached to the concrete.

Three holes were hammer drilled in the concrete liner on the arch at Site C. The following observations and measurements were made regarding liner thickness, void and bedrock type.
Voids were felt at about 8.5 inches while drilling the crown hole, at 1 to 2 inches in the left hole and at 7 to 15 inches in the right hole. In the right hole the drill penetrated 16 inches of concrete.

**B.4 Tunnel Liner Investigation Site D (Station 3+10)**

At Site D a concrete core was obtained from the left side of the tunnel roughly 2 feet 4 inches off the tunnel floor. The concrete was measured to be 18 inches thick at this location. Shale bedrock was observed behind the concrete liner and there was no void between the liner and the bedrock. The concrete at this location is very sound, having only a few voids near the very end and a crack at about 13 inches. A photograph of the core is shown below.

![Photograph showing the concrete core representing the tunnel liner at Site D. Notice the sound character of the concrete and the small crack at about 1.1 feet.](image)

The shale bedrock observed in the core hole was grey and purple and appeared to have limestone concretions in it.
Three holes were hammer drilled in the concrete liner on the arch at Site D. The following observations and measurements were made regarding liner thickness, void and bedrock type.

<table>
<thead>
<tr>
<th>Site D Arch Hammer Drilling Observations</th>
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</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td><strong>Left Side of Arch</strong></td>
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<tr>
<td>(17 inches from crown)</td>
</tr>
<tr>
<td>Concrete Liner Thickness (inches)</td>
</tr>
<tr>
<td>16</td>
</tr>
<tr>
<td>Void (inches)</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>Bedrock Type</td>
</tr>
<tr>
<td>Not Observed</td>
</tr>
<tr>
<td><strong>Crown</strong></td>
</tr>
<tr>
<td>12</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>Not Observed</td>
</tr>
<tr>
<td><strong>Right Side of Arch</strong></td>
</tr>
<tr>
<td>(19 inches from crown)</td>
</tr>
<tr>
<td>14</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>Not Observed</td>
</tr>
</tbody>
</table>

Voids in the concrete were felt while drilling the left hole at 5 inches and in the right hole at 10 inches and again at 12 to 13 inches.

**B.5  Tunnel Liner Investigation Site E (Station 3+90)**

One hole was hammer drilled in the left wall of the tunnel liner about 2 feet 4 inches off the floor. This hole encountered 12 inches of concrete, no void and purple shale behind the liner. Three holes were hammer drilled in the concrete liner on the arch at Site E. The following observations and measurements were made regarding liner thickness, void and bedrock type.
Site E Arch Hammer Drilling Observations

<table>
<thead>
<tr>
<th></th>
<th>Left Side of Arch (13 inches from crown)</th>
<th>Crown</th>
<th>Right Side of Arch (17 inches from crown)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Liner</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness (inches)</td>
<td>11</td>
<td>9.5</td>
<td>10</td>
</tr>
<tr>
<td>Void (inches)</td>
<td>0</td>
<td>3.5</td>
<td>3</td>
</tr>
<tr>
<td>Bedrock Type</td>
<td>Shale</td>
<td>Purple Shale</td>
<td>Not Observed</td>
</tr>
</tbody>
</table>

Voids in the concrete were felt while drilling the left hole at 4 to 5 inches.

B.6  Tunnel Liner Investigation Site F (Station 4+70)

Three holes were hammer drilled in the concrete liner on the arch at Site F. The following observations and measurements were made regarding liner thickness, void and bedrock type.

Site F Arch Hammer Drilling Observations

<table>
<thead>
<tr>
<th></th>
<th>Left Side of Arch (19 inches from crown)</th>
<th>Crown</th>
<th>Right Side of Arch (19 inches from crown)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Liner</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness (inches)</td>
<td>13</td>
<td>8.5</td>
<td>11</td>
</tr>
<tr>
<td>Void (inches)</td>
<td>19</td>
<td>3.5</td>
<td>6</td>
</tr>
<tr>
<td>Bedrock Type</td>
<td>Shale</td>
<td>Not Observed</td>
<td>Not Observed</td>
</tr>
</tbody>
</table>

Voids in the concrete were felt while drilling the right hole at 7 inches and 9 inches.

B.7  Tunnel Liner Investigation Site G (Station 5+20)

One hole was hammer drilled in the left wall of the tunnel liner about 2 feet 4 inches off the floor. This hole encountered 14 inches of concrete, no void and purple shale behind the liner. Only one hole was hammer drilled in the concrete liner on the crown of the arch at Site G. The following observations and measurements were made regarding liner thickness, void and bedrock type.

Site G Arch Hammer Drilling Observations

<table>
<thead>
<tr>
<th></th>
<th>Crown</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Liner</td>
<td>7</td>
</tr>
<tr>
<td>Thickness (inches)</td>
<td></td>
</tr>
<tr>
<td>Void (inches)</td>
<td>2</td>
</tr>
<tr>
<td>Bedrock Type</td>
<td>Purple Shale</td>
</tr>
</tbody>
</table>

Voids in the concrete were felt while drilling the crown at 5 inches and 8 inches.
APPENDIX C
LABORATORY TEST RESULTS
Samples of soils and bedrock collected during the Shell Canal Tunnel geotechnical investigations were tested in the laboratory for various physical and engineering properties. The tests were generally completed in accordance with applicable ASTM Standard Procedures. The tests on the soils and bedrock were completed by J.R. Valentine, Inc., Knight Piesold Consulting and Terracon. Unconfined compression tests on the concrete liner core were performed by CTL Thompson.

The physical property testing included natural moisture content, natural dry density, gradation and hydrometer (particle size distribution), and Atterberg limits. Engineering property tests included swell/consolidation, pin hole dispersivity, and unconfined compressive strength. As discussed in the report, the pin hole dispersivity tests could not be performed on the bedrock samples because the holes would swell shut when water was added during the test.

The laboratory testing results are summarized on Table 1. Laboratory data sheets and test results for the gradations, swell/consolidation tests, bedrock unconfined compression tests and concrete unconfined compression tests are provided here in Appendix C.
GRADATION TESTS
## ASTM D422 - Particle Size Analysis of Soil

### Hydrometer Analysis

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>% Passing</th>
<th>Particle Size - Millimeters</th>
</tr>
</thead>
<tbody>
<tr>
<td>3&quot;</td>
<td>100</td>
<td>Sand</td>
</tr>
<tr>
<td>2&quot;</td>
<td>100</td>
<td>Sand</td>
</tr>
<tr>
<td>1 1/2&quot;</td>
<td>100</td>
<td>Silt</td>
</tr>
<tr>
<td>1&quot;</td>
<td>100</td>
<td>Silt</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>100</td>
<td>Clay</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>100</td>
<td>Clay</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>100</td>
<td>Clay</td>
</tr>
<tr>
<td>#4</td>
<td>100</td>
<td>Clay</td>
</tr>
<tr>
<td>#10</td>
<td>100</td>
<td>Clay</td>
</tr>
<tr>
<td>#20</td>
<td>100</td>
<td>Clay</td>
</tr>
<tr>
<td>#40</td>
<td>100</td>
<td>Silt</td>
</tr>
<tr>
<td>#80</td>
<td>100</td>
<td>Sand</td>
</tr>
<tr>
<td>#200</td>
<td>95</td>
<td>Gravel</td>
</tr>
<tr>
<td>0.074mm</td>
<td>94</td>
<td></td>
</tr>
<tr>
<td>0.005mm</td>
<td>83</td>
<td></td>
</tr>
<tr>
<td>0.001mm</td>
<td>68</td>
<td></td>
</tr>
</tbody>
</table>

**Moisture Content (as received)** - 22.0%

**Dry Density (pcf)** - 95.7

**Liquid Limit** - 91

**Plastic Limit** - 26

**Plasticity Index** - 65
Particle Size Distribution Report

GRAIN SIZE - mm.

<table>
<thead>
<tr>
<th>% +3&quot;</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Fines</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Coarse</td>
<td>Fine</td>
<td>Medium</td>
</tr>
<tr>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>3.2</td>
</tr>
</tbody>
</table>

**SIEVE SIZE** | **PERCENT FINER** | **SPEC.** | **PASS?** | **(X=NO)**
--- | --- | --- | --- | ---
#4 | 100.0 | | | |
#10 | 96.8 | | | |
#20 | 93.5 | | | |
#40 | 91.9 | | | |
#60 | 90.8 | | | |
#100 | 89.5 | | | |
#200 | 86.7 | | | |
0.0526 mm. | 84.7 | | | |
0.0378 mm. | 82.0 | | | |
0.0272 mm. | 79.3 | | | |
0.0176 mm. | 74.8 | | | |
0.0104 mm. | 70.4 | | | |
0.0054 mm. | 61.7 | | | |
0.0028 mm. | 53.9 | | | |
0.0012 mm. | 43.2 | | | |

**Soil Description**

fat clay

**Atterberg Limits**

| PL= 24 | LL= 179 | PL= 155 |

**Coefficients**

| D85= 0.0550 | D60= 0.0047 | D50= 0.0020 |
| D30= | D15= | D10= |
| C_u= | C_s= |

**Classification**

USCS= CH  AASHTO= A-7-6(151)

**Remarks**

Sample No.: Run 2  Source of Sample:  Date: 1/18/11
Location: B-1  Elev./Depth: 23.7-24.3’

Client: Deere & Ault  Project: Shell Canal Tunnel Geotechnical Study

Project No: DV108-214/03  Fig.
### Soil Description

clayey sand

### Atterberg Limits

- PL = 24
- LL = 56
- PI = 32

### Coefficients

- $D_{85} = 0.6552$
- $D_{60} = 0.3449$
- $D_{50} = 0.2544$
- $D_{15} = $
- $C_u =$
- $C_c =$

### Classification

- USCS = SC
- AASHTO = A-2-7(3)

### Remarks

- (no specification provided)

---

### Particle Size Distribution Report

<table>
<thead>
<tr>
<th>SIEVE SIZE</th>
<th>PERCENT FINER</th>
<th>SPEC.* PERCENT</th>
<th>PASS? (X=NO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#10</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#20</td>
<td>93.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#40</td>
<td>67.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#60</td>
<td>49.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#100</td>
<td>38.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#200</td>
<td>30.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0675 mm.</td>
<td>29.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0482 mm.</td>
<td>27.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0343 mm.</td>
<td>25.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0220 mm.</td>
<td>22.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0128 mm.</td>
<td>20.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0064 mm.</td>
<td>18.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0031 mm.</td>
<td>16.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.0013 mm.</td>
<td>15.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sample No.: Run 5  Source of Sample:  Date: 1/18/11
Location: B-1  Elev./Depth: 37.5-37.8'

Client: Deere & Ault
Project: Shell Canal Tunnel Geotechnical Study
Project No: DV108-214/03  Fig.
Particle Size Distribution Report

Soil Description
- fat clay

Atterberg Limits
- PL = 21
- LL = 62
- PI = 41

Coefficients
- D85
- D60
- D30
- D15
- Cc
- Co

Classification
- USCS = CH
- AASHTO = A-7-6(45)

Remarks

Sample No.: Run 6
Source of Sample: Deere & Ault
Date: 1/18/11
Location: B-1
Elev./Depth: 40.5-41.4'

Client: Deere & Ault
Project: Shell Canal Tunnel Geotechnical Study
Project No.: DV108-214/03
Fig.
### Particle Size Distribution Report

#### GRAIN SIZE - mm.

<table>
<thead>
<tr>
<th>Size</th>
<th>PERCENT FINER</th>
<th>SPEC.* PERCENT</th>
<th>PASS? (X=NO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#10</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#20</td>
<td>99.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#40</td>
<td>99.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#60</td>
<td>99.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#100</td>
<td>99.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#200</td>
<td>99.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### Soil Description

- Fat clay

#### Atterberg Limits

- \( PL = 24 \)
- \( LL = 60 \)
- \( PI = 36 \)

#### Coefficients

- \( D_{85} = \)
- \( D_{60} = \)
- \( D_{30} = \)
- \( D_{15} = \)
- \( C_u = \)
- \( C_c = \)

#### Classification

- USCS: CH
- AASHTO: A-7-6(41)

#### Remarks

- (no specification provided)

### Sample Information

- **Sample No.:** Run 7
- **Source of Sample:**
- **Location:** B-1
- **Date:** 1/18/11
- **Elev./Depth:** 44.1-45.0'
- **Client:** Deere & Ault
- **Project:** Shell Canal Tunnel Geotechnical Study
- **Project No.:** DV108-214/03
- **Fig.:**
**Particle Size Distribution Report**

**Soil Description**
- sandy fat clay

**Atterberg Limits**
- PL = 20
- LL = 54
- PI = 34

**Coefficients**
- D85 = 0.5482
- D60 = 0.0828
- D30 = 0.0045
- D15 = C_u =
- D10 = C_c =

**Classification**
- USCS = CH
- AASHTO = A-7-6(17)

**Remarks**

---

**Sample No.:** Run 9  | **Source of Sample:**
**Location:** B-1 | **Date:** 1/18/11  
**Client:** Deere & Ault  
**Project:** Shell Canal Tunnel Geotechnical Study  
**Elev./Depth:** 56-57'  
**Project No:** DV108-214/03  
**Fig.**
SWELL/CONSOLIDATION TESTS
Notes: Water Added at 1 ksf.
Consolidation/Swell Test

Specimen Identification | Classification | \( \gamma_s \), pcf | WC, %
--- | --- | --- | ---
#1 | SHALE | 102 | 17

Notes: Water Added at 1 ksf.
Moisture Content = 13.6 percent
Dry Unit Weight = 123.4 pcf
Sample of: Claystone
From: B-1 @ 29.4'-34.6'

Expansion Upon Wetting

APPLIED PRESSURE — ksf

SWELL-CONSOLIDATION TEST RESULTS
UNCONFINED COMPRESSIVE STRENGTH TESTS CLOVERLY FORMATION
POINT LOAD TEST
ASTM D 5731
**DIAMETRAL POINT LOAD TEST**

**ASTM D 5731**

<table>
<thead>
<tr>
<th>Specimen ID Boring, Depth(ft.)</th>
<th>Length (in.)</th>
<th>Diameter (in.)</th>
<th>De^2</th>
<th>Gauge Failure Load (psig)</th>
<th>P (lb)</th>
<th>Is</th>
<th>F</th>
<th>Is(50)</th>
<th>C</th>
<th>Compressive Strength (psi)</th>
<th>Loading with respect to Fracture/ Bedding</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1, 37.5-37.8</td>
<td>2.661</td>
<td>2.477</td>
<td>6.136</td>
<td>79</td>
<td>163.5</td>
<td>26.7</td>
<td>1.1</td>
<td>29.3</td>
<td>25.3</td>
<td>740</td>
<td>Perpendicular</td>
<td>C</td>
</tr>
</tbody>
</table>

**Notes:**

- **L:** Sample Length
- **D:** Sample Diameter
- **De^2:** Equivalent Diameter = D^2
- **Piston Area (in^2):** 2.07
- **P:** Gauge Failure Load * Piston area (in^2)
- **Is:** Point Load Index Strength = P/De^2
- **F:** Size Correction Factor to 2.0 in = (De/2.0)^0.45
- **Is(50):** Size Corrected Index Strength = F * Is
- **C:** Factor to Estimate Compressive Strength related to Core Diameter

Compressive Strength in psi = C * Is(50)

**Data Entered By:**

<table>
<thead>
<tr>
<th>Data Entered By</th>
<th>Date:</th>
<th>01/04/2011</th>
</tr>
</thead>
</table>

**Data Checked By:**

<table>
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<tr>
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<th>Date:</th>
<th>04/04/2011</th>
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**Filename:**

<table>
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<tr>
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<th>KPPD131A</th>
<th>01/04/2011</th>
</tr>
</thead>
</table>
UNCONFINED COMPRESSIVE STRENGTH
ASTM D 7012 Method C
## UNCONFINED COMPRESSIVE STRENGTH

**ASTM D 7012; Method C (Previously ASTM D 2938)**

**CLIENT:** Knight Piésold  
**LOCATION:** Shell Canal Tunnel  
**PROJECT:** DV108-214.03 Task 400  
**DATE TESTED:** 1/4/11 HN/BL  

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Diameter (in.)</th>
<th>Length (in.)</th>
<th>Mass (gms)</th>
<th>Wet Density (pcf)</th>
<th>Failure Load (lb)</th>
<th>Failure Types</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1, 23.7-24.3</td>
<td>2.504</td>
<td>5.770</td>
<td>979.57</td>
<td>131.3</td>
<td>935</td>
<td>F</td>
<td>190</td>
</tr>
<tr>
<td>B-1, 40.5-41.4</td>
<td>2.487</td>
<td>5.759</td>
<td>1030.30</td>
<td>140.3</td>
<td>3,110</td>
<td>F/S</td>
<td>640</td>
</tr>
</tbody>
</table>

**Notes and Comments:**  
* Failure types:  
  - S: Shear Failure  
  - M: Matrix Failure  
  - F: Failure due to Fracture/Bedding  
  - V: Void Failure  
  - C: Combination
UNCONFINED COMPRESSIVE STRENGTH
ASTM D 7012 Method D
**UNCONFINED COMpressive STRENTh**
With Stress / Strain Measurements
ASTM D 7012; Method D (Previously ASTM D 3148)

**CLIENT:** Knight Piesold And Co.
**Project:** Shell Canal Tunnel

**Location:**
- Specimen 10: Boring, Sample I
  - Depth (ft.): Ranges from 44.1-45.0 to 56.0-59.0
  - Rock type: Shale

**Unconfined Compressive Strength With Stress/Strain Measurements**

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Diameter (in.)</th>
<th>Length (in.)</th>
<th>Mass (gms)</th>
<th>Wet Density (pcf)</th>
<th>Failure Load (lb)</th>
<th>Failure Type *</th>
<th>Compressive Strength (psi)</th>
<th>Young's Modulus (X10^6 psi)</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1, Run 6</td>
<td>2.488</td>
<td>5.652</td>
<td>1003.39</td>
<td>139.1</td>
<td>8,200</td>
<td>F</td>
<td>1,690</td>
<td>0.40</td>
<td>0.097</td>
</tr>
<tr>
<td>B-1, Run 9</td>
<td>2.480</td>
<td>5.653</td>
<td>1027.01</td>
<td>143.3</td>
<td>4,100</td>
<td>F/S</td>
<td>850</td>
<td>0.27</td>
<td>0.094</td>
</tr>
</tbody>
</table>

**Notes and Comments:**
* Failure Type:
  - S: Shear Failure, M: Matrix Failure, F/V Fracture, Bedding/Void Collapse, C: Combination

**Data Entered By:**
**Data Checked By:**
**Filename:** KFUCS129
**Job No.:** 2061-129
**Date Tested:** 12/21/10 HN
<table>
<thead>
<tr>
<th>Boring, Sample</th>
<th>Depth (ft)</th>
<th>Rock type</th>
<th>Diameter (in)</th>
<th>Length (in)</th>
<th>Mass (g)</th>
<th>Defl (Radial) (in)</th>
<th>Defl (Axial) (in)</th>
<th>Gage Length (Radial) (in)</th>
<th>Scale (Axial) (in)</th>
<th>Scale (Radial) (in)</th>
<th>Area (in²)</th>
<th>Measurement Load (lbs)</th>
<th>Failure Load (lbs)</th>
<th>Failure Type</th>
<th>Poisson's Ratio</th>
<th>Young's Modulus (psi)*10⁶</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1, Run 6</td>
<td>44.1-45.0</td>
<td>Shale</td>
<td>2.48</td>
<td>5.65</td>
<td>1003.39</td>
<td>0.25</td>
<td>1.35</td>
<td>3.25</td>
<td>0.0125</td>
<td>0.0025</td>
<td>4.862</td>
<td>10000</td>
<td>8200</td>
<td>F</td>
<td>0.097</td>
<td>0.40</td>
<td>1600</td>
</tr>
<tr>
<td>B-1, Run 9</td>
<td>56.0-59.0</td>
<td>Shale</td>
<td>2.48</td>
<td>5.65</td>
<td>1027.01</td>
<td>0.18</td>
<td>3.25</td>
<td>1.24</td>
<td>0.0125</td>
<td>0.0025</td>
<td>4.831</td>
<td>5000</td>
<td>4100</td>
<td>F/S</td>
<td>0.094</td>
<td>0.27</td>
<td>850</td>
</tr>
</tbody>
</table>
Knight Pizzold, Inc.
#2061-129
8-1, Rm 2, 56-0-57-0, Shale
Failure Type: F/S
UNCONFINED COMPRESSION TEST

Job No.: 10014  
Hole No.: B-1  
Depth No. 29.4' 34'  
Blow Count:  
Date: 12/4/10  
Load Cell No.: 1  
Strain in/min:  
Diameter, in: 2.490

<table>
<thead>
<tr>
<th>Load (lbs)</th>
<th>Load on Specimen (psi)</th>
<th>Strain</th>
<th>Unit Strain</th>
<th>Area</th>
<th>Total Strain (%)</th>
<th>Stress (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14,850</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3049.6 PSI</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>431137 PSF</td>
</tr>
</tbody>
</table>

Density

- Length, in.: 5.080
- Volume, cu.ft.: 919.5
- Wet wt., g.: 124.5
- Dry wt., g.:  
- Dry density, pcf:  

Moisture

- Dish no.: 98
- Dish/soil wt., g.: 154.1
- Dish/dry soil, g.: 135.6
- Dish wt., g.:  
- Wt. of water, g.:  
- Wt. of dry soil, g.:  
- Moisture, %: 13.6

Save for:  

Sketch

**Unit strain = \( \frac{\Delta L}{L} \)**  
**Corrected area = \( \frac{A}{1 \text{ Unit str} \text{ain}} \)**

Tested by: JRV  
Calculated by: JRV  
Reviewed by: JRV
UNCONFINED COMPRESSION STRENGTH TESTS
TUNNEL LINER CONCRETE
December 9, 2010

Deer & Ault Consultants, Inc.
600 S. Airport Road
Building A, Suite 205
Longmont, Colorado 80503

Attention: Mr. Victor deWolfe III

Subject: Concrete Core Test Results
Shell Canal Tunnel Geotechnical Study
Project No. CT15033.003-455

Dear Mr. deWolfe III:

This report presents compressive strength results for 4 concrete cores extracted from the Shell Canal Tunnel. The date of placement for the concrete cored was around the 1920’s.

The cores were tested in general conformance with ASTM C 42, “Obtaining and Testing Drilled Cores of Concrete”. The average compressive strengths and densities are included.

If you have any questions regarding the information in this report, please feel free to contact us.

Respectfully submitted,

CTL | THOMPSON MATERIALS ENGINEERS, INC.

Daniel L. Barrett
Materials Lab Manager

Reviewed by:

Damon B. Thomas, P.E.
Division Manager / Associate

DLB:DBT/dbb
Enclosures

(1 copy sent)
Client: Deer & Ault Consultants, Inc.
Project: Shell Canal Tunnel Geotechnical Study
Project No. CT15033.003

Report Date: December 9, 2010
Specified Strength (f’c): Unknown psi

Placement Date: 1920's
Date cored: 11/2010
Cure Date: December 2, 2010
Date Tested: December 7, 2010

Direction of Load with Respect to Placement: Perpendicular
Nominal Maximum Aggregate Size: 1 inch
Method of Determining Density: Dimensional Volume

Location of Cores:
Cores received 12/2/10.

<table>
<thead>
<tr>
<th>Core ID</th>
<th>Age</th>
<th>Density (pcf)</th>
<th>Height (Capped)</th>
<th>Diameter (in)</th>
<th>Area (in2)</th>
<th>Height/Dia Ratio</th>
<th>Correction Factor</th>
<th>Load (lbs)</th>
<th>Corrected Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>5</td>
<td>142.5</td>
<td>5.51</td>
<td>2.70</td>
<td>5.73</td>
<td>2.04</td>
<td>1.000</td>
<td>19,200</td>
<td>3,350</td>
</tr>
<tr>
<td>B-3</td>
<td>5</td>
<td>147.8</td>
<td>5.51</td>
<td>2.70</td>
<td>5.73</td>
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<td></td>
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<td></td>
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<td></td>
<td><strong>Average 4,810</strong></td>
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Interpretation of Results:
Average Compressive Strength (Set 1) 4,810 psi
Minimum Individual Compressive Strength (Set 1) 3,350 psi
APPENDIX D
TUNNEL SLEEVE CONCEPTUAL DRAWINGS
SURVEY CONTROL NOTES (TUNNEL & CANAL)


2. Vertical Datum: MHO 95 completed using (GSD90).

3. Field Work for Tunnel & Canal Survey Conducted in Nov. 2019 by Paul Reid, PLS.

NOTES:

1. There are two contour map surfaces used in this project. The primary survey of the area was conducted in November 2019 using MHO 95. Survey outside of the limits of the primary survey is shown for reference only and is from less maps using MHO 95.

2. We recommend obtaining further survey data to define the construction limits and complete the tunnel & proposed grades with existing grades.

3. All grading quantities based on Tunnel & Canal Survey.

4. Further survey required to complete grading plan of waste area for designated materials.

DRAFT
NOTES:

1. CONTRACTOR TO FIELD VERIFY MEASUREMENTS PRIOR TO ORDERING HOLE PIPES.
2. CLEARING DEPT OF SAND AND GRAVEL DEPOSITS TO PROVIDE A CLEAR SURFACE PRIOR TO INSTALLING LINER PIPE.
3. INSTALL LINER PIPE TO HAVE A CONSTANT GRADE (PER PLAN) WITHOUT HIGH AND LOW POINTS.
4. WASHING & RINSING SHALL BE USED TO PREVENT MOVEMENT OF THE PIPE DURING PLACEMENT OF THE CELLULAR CONCRETE.
5. DAMAGED LINER OF TUNNEL LINER INCLUDING FROM STA 488 TO 979 TO BE REMOVED AND DEPOSED OFF SITE.
6. BACKHOES WILL BE REQUIRED AT EACH END OF HOPE PIPES TO REMOVE CELLULAR CONCRETE AND ELL PLACEMENT SHOULD GENERALLY PROCEED FROM DOWNSTREAM TO UPSTREAM.
APPENDIX E
OPEN CUT CONCEPTUAL DRAWINGS
SHELL CANAL TUNNEL

Survey Control Points Table

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<th>No.</th>
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<td>3</td>
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NOTES:
1. ALL SURVEY DATA IN THIS PLANSHOT ARE BASED ON THE ORIGINAL SURVEY DATA PROVIDED.
2. THE SURVEY DATA IS SHOWN AS A NULL SCALE. THE SCALE IS SHOWN FOR REFERENCE ONLY.
3. ALL SURVEY DATA IS SHOWN AS A NULL SCALE. THE SCALE IS SHOWN FOR REFERENCE ONLY.

DRAFT
NOTES:
1. ALL CROSS SECTIONS ARE CUT LOOKING UPSTREAM
2. TYPICAL SECTION DIMENSIONS SEE SHEET 6