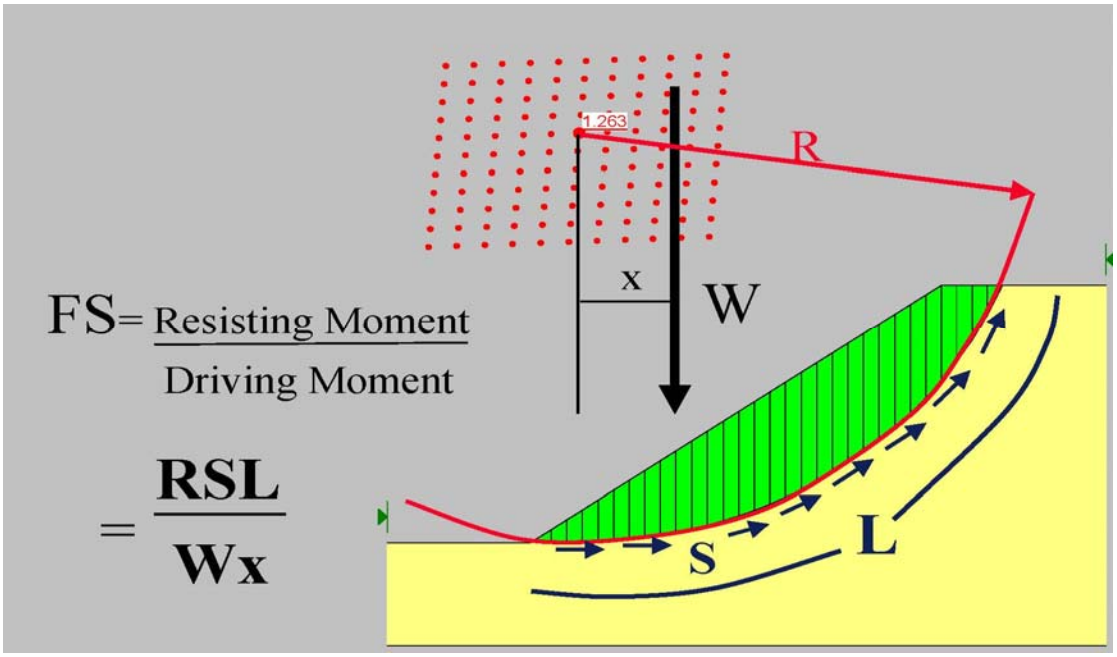
RECLAMATION

Managing Water in the West

Results of Geotechnical and Structural Analysis, Leadville Mine Drainage Tunnel

Leadville, Colorado
Great Plains Region



U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

DRAFT FINAL June 2008

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Geotechnical Engineering

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1. Introduction

This report was prepared to document studies performed to evaluate stability of the Leadville Mine Drainage Tunnel (LMDT). The LMDT is an underground excavation constructed by the Bureau of Mines during World War II and the Korean War to drain groundwater from metal mines located at Leadville in Lake County, Colorado (Figure 1). Collapse of the tunnel roof was a common occurrence during and after construction. Eventually a decision was made to place a porous bulkhead against a major zone of collapse near the portal and continue drainage operations at the LMDT using both flow through the bulkhead and by pumping water from extraction wells located upstream of the bulkhead. Expected continued roof collapse in upstream areas of the tunnel, have led to the establishment of impounded water referred to as the "mine pool" due to its connection with flooded and interconnected old mine workings.

The elevation of the mine pool water behind collapsed areas in the LMDT has been rising over the past few years. Area residents, local and state officials, and the EPA have expressed safety concerns relating to the possibility of a sudden release of water behind blockages or a massive slope failure of the hillside above the portal area. A small residential community, The Village at East Fork, is located adjacent to the LMDT portal. Also, the Bureau of Reclamation (Reclamation) operates a Water Treatment Plant located adjacent to the portal of the LMDT. In response to these concerns, Reclamation commissioned a qualitative risk assessment of the LMDT. The assessment of risks has been performed in three major steps which include:

- Determination of the existing condition of the LMDT including its history, details of construction, modifications, and current operations.
- Identification of potential failure modes and effects analysis (PFMEA) including identifying opportunities for data gathering, risk reduction, and monitoring enhancement activities which can enhance project safety.
- Structural analysis of specific LMDT features associated with potential failure modes to better understand the mechanism of failure and the likelihood of occurrence.

Finally, a review and adjustment of the PFMEA was made in light of the analysis results. Separate reports have been prepared for each of the above major steps of investigation. The report titles are:

Existing Condition of the Leadville Mine Drainage Tunnel

Potential Failure Modes and Effects Analysis, Leadville Mine Drainage Tunnel

Results of Geotechnical and Structural Analysis, Leadville Mine Drainage Tunnel

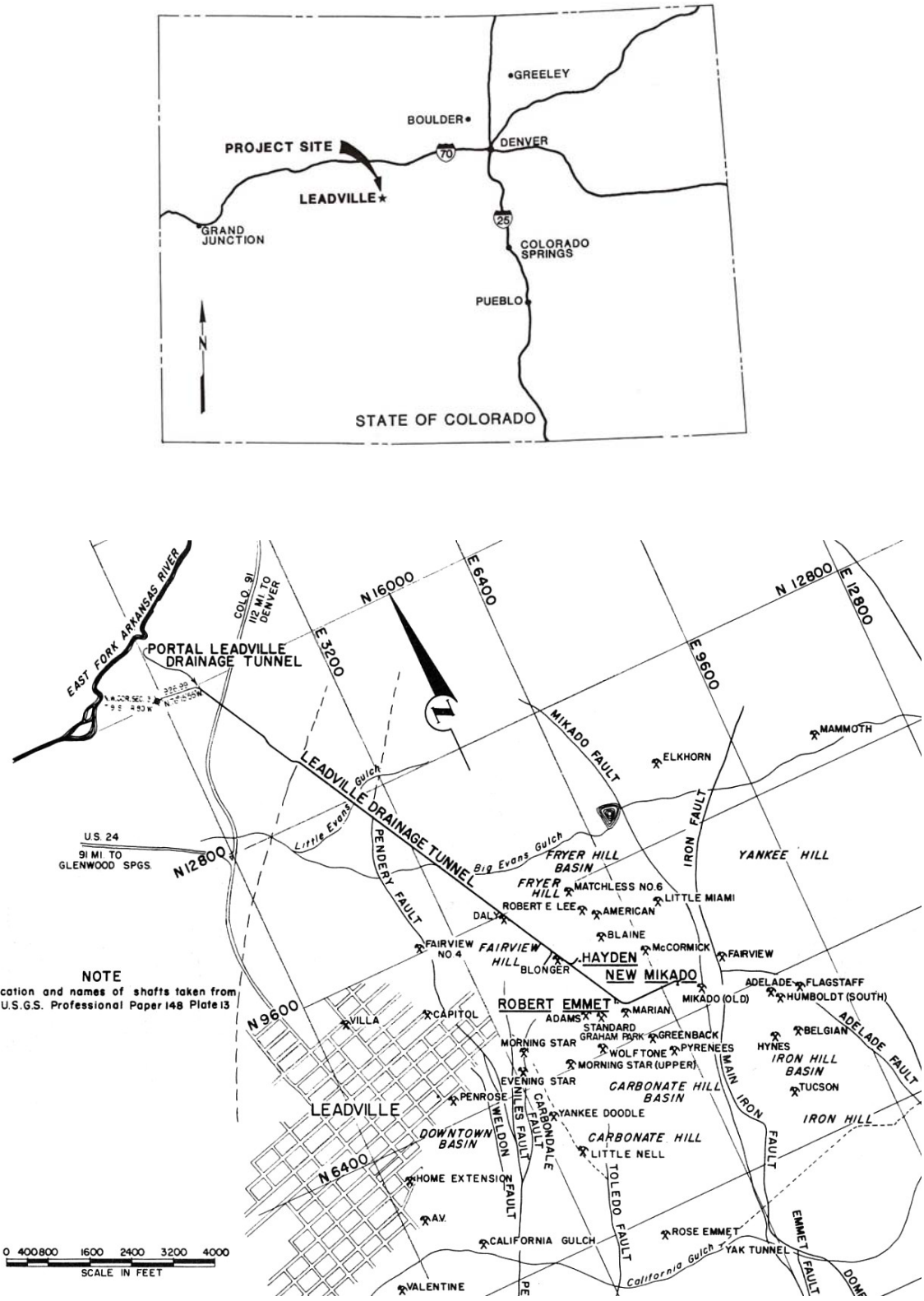

Figure 1. Location of the LMDT at Leadville, Colorado

1.1 Establishment of the Mine Pool

The LMDT has a history of roof collapse both during and after construction. Additional details about the history, operation, and features of the LMDT can be found in a report titled "Existing Condition of the Leadville Mine Drainage Tunnel" (Gobla and Vandeberg, 2008). After construction, the deterioration of the strength of some rock units, timber, and steel supports resulted in additional roof collapses throughout the LMDT which were repaired on an as needed basis until 1968 when a major collapse about 500 feet into the tunnel threatened the overlying Highway 91. The collapse dammed off much of the flow previously draining through the tunnel. Some flow continued to pass through the existing tunnel ventilation and compressed air pipes which were in the zone of collapse. The collapsed area was modified by injection of sand and gravel into the collapsed portions of the tunnel, injection of grout into voids in the overburden above the tunnel, and installation of a porous bulkhead at Station 4+66 to reinforce the downstream side of the collapsed zone, thus forming a porous hydraulic plug to support the overlying highway and hillside and protect against rapid release of the water backed up by the plug. Because this porous plug significantly reduces the transmission of water which was previously free draining from the tunnel, water extraction wells fitted with pumps were installed upstream of the plug to continue drainage operations. In the 1970s there were continued problems with minor sinkholes developing in the lower portion of the LMDT. In 1990-92 a new concrete portal and a reinforced concrete tunnel liner were constructed along the lower reaches of the tunnel to prevent further occurrences of sinkholes. Five feet of additional gravel and cobble fill was placed between the porous bulkhead at Station 4+66 and a second porous "timber lattice" bulkhead was constructed at Station 4+61 and tied to the new concrete tunnel liner.

In response to suspected additional tunnel collapses, a pool of water has been building up within the abandoned mine workings connected to the LMDT. As the water flow from the mine workings is further impeded down the tunnel, the water level rises in hydraulically interconnected mine workings, thus forming a "mine pool." Extraction wells located at Station 10+25 yield no more than about 750 gpm of water. Monitoring of water elevations along the LMDT alignment shows that there is a differential head of about 119 feet between the groundwater level in observation wells located at Station 36+77 and Station 46+66. This segment of the tunnel between these wells crosses the Pendery Fault. Based upon the geology and history of the LMDT, it is believed that a significant zone of collapse exists just downstream of the fault, just under ¾ mile in from the portal, and that this collapse is impeding flow and causing the rise in the mine pool water elevation. It is likely that the zone of collapse is not a complete barrier to flow. Ventilation and compressed air pipelines from the initial construction left inside the tunnel are believed to have been engulfed in the collapse debris in a similar fashion as to what was observed at a collapse further downstream in the tunnel. These conduits likely penetrate through the blockage and transmit limited amounts of water. In summary, there are believed to be at least two major blockages to flow in the tunnel, a lower blockage and an upper blockage. The lower blockage is the porous plug consisting of two gravel filled bulkheads supporting the overlying and adjoining collapse material below the highway which was stabilized by injection of fill. The upper blockage is believed to be a zone of collapsed rock downstream of the Pendery Fault.

1.2 Purpose

The purpose of this report is to present the results of geotechnical and structural analyses performed to evaluate the structural stability of the LMDT and portal hillside area. A total of seven potential failure modes were identified for the LMDT and documented in a separate report titled "Potential Failure Modes and Effects Analysis, Leadville Mine Drainage Tunnel" (Reclamation, 2008). The seven potential failure modes are summarized below:

Potential Failure Mode No. 1 – Breach in Upstream Tunnel Blockage results in "Blowout" of Downstream Bulkheads

Description
Breach of a tunnel blockage near the Pendery Fault results in increased head and flow in the downstream tunnel which results in breaches of the downstream tunnel blockages and bulkheads. The mine pool is released out the tunnel portal at a high flow rate.

Potential Failure Mode No. 2 – Breach in Upstream Tunnel Blockage results in Rapid Erosion Breach of Downstream Slope Materials

Description
This potential failure mode begins in a similar manner to Potential Failure Mode No. 1, except that as the increased water pressures reach the downstream blockages and bulkheads, they hold. The groundwater levels and flow rates could then rise along the outside of the tunnel. If erosion of the material at the downstream slope face begins, progressive erosion and slumping of material or "piping" could progress upstream through the hillside until a connection was made to the tunnel upstream of State Highway 91, resulting in a rapid release of water. A potential additional complication could involve collapse of the concrete tunnel lining downstream of the bulkheads (from the portal, Station 0+54, to Station 4+61), resulting in sinkholes that shorten the seepage path to the tunnel upstream of the highway.

Potential Failure Mode No. 3 – Breach in Upstream Tunnel Blockage results in High Downstream Groundwater Levels and Slope Instability

Description
Breach of an upstream tunnel blockage near the Pendery Fault could result in increased water pressure in the downstream portion of the tunnel and a rise in the adjacent groundwater level. Given that the downstream tunnel blockage under State Highway 91 and the bulkheads hold, the groundwater level outside of the tunnel could then rise to unprecedented levels. The increase in pore pressures within the gravel soils near the portal could result in slope instability, and movement of earth materials and water into and adjacent to the tunnel portal area.

Potential Failure Mode No. 4 – Breach in Upstream Tunnel Blockage results in Leakage of Contaminated Water into Downstream Areas

Description
Breach of a tunnel blockage near the Pendery Fault results in higher water pressures in the downstream tunnel and higher groundwater levels above the downstream portion of the tunnel. The blockage under State Highway 91 and bulkheads hold, but water contaminated with heavy metals seeps through the pervious gravels into low lying areas, possibly exiting at Evans Gulch, Little Evans Gulch, or more likely the tunnel portal. It is possible that water could also flow toward California Gulch if the groundwater levels over the downstream portion of the tunnel rose to high enough levels.

Potential Failure Mode No. 5 – Earthquake Triggers Slope Instability near Tunnel Portal

Description
A major earthquake causes instability of a large portion of the slope adjacent to the downstream tunnel portal resulting in impacts to this area. This could be triggered under normal groundwater conditions, or as a result of elevated groundwater conditions due to breach of a blockage upstream near the Pendery Fault.

Potential Failure Mode No. 6 – Seepage Erosion into Tunnel Causes Sinkholes and Loss of the Highway

Description
Under this scenario, high groundwater outside the tunnel would result in a gradient that could carry soil material into the tunnel. The loss of material overlying the tunnel would then result in voids that could stope to the surface, creating sinkholes that would affect State Highway 91. For this to occur, the water pressure outside the tunnel would need to be higher than inside.

Potential Failure Mode No. 7 – Flow at Tunnel Portal Plugs Off, Raising Groundwater and Causing Slope Instability

Description
For this potential failure mode to initiate, impervious fines would need to be carried into the tunnel, filling the voids in the downstream tunnel to the point where drainage through the tunnel is impeded, raising the groundwater level outside the tunnel and leading to slope instability. The initial water level outside the tunnel would need to be higher than inside the tunnel, and the tunnel would need to be acting as a drain for the slopes near the portal.

A review of the failure modes indicates that elevated groundwater conditions in and adjacent to the downstream portion of the tunnel is a common theme with respect to the loading conditions which might lead to failure. Considering the loading conditions which might reasonably occur, the following are evaluated in this report:

- Stability of a flow blockage in the vicinity of the Pendery Fault
- Stability of the porous plug above the Timber-Lattice Bulkhead
- Stability of the Timber-Lattice Bulkhead
- Stability of the Concrete Tunnel Lining
- Stability of the Hillside above the Portal

1.3 Summary of Results

Based upon the detailed analysis documented in this report, it is concluded that the blockage near the Pendery Fault is likely due to a zone of roof collapse located downstream from the fault. The blockage is stable and currently resists the force exerted by 119 feet of differential head. However, the likelihood of the blockage remaining stable decreases with increased head differential. For that reason, all analyses and potential failure mode assessment conservatively assumes rapid failure of the blockage.

The forces acting on the plug of porous material and lattice bulkhead due to the pressure wave resulting from failure of a blockage near the Pendery Fault would not be great enough to overcome the existing shear strength of the material and move it.

Wells at Station 10+25 and at Station 6+35 would be likely to experience artesian flow conditions and relieve some of the pressure. The amount and height of flow would be limited by head losses in the LMDT and by those caused by the well casing and pumping rates.

It would take a significant period of time for the increased water pressures to seep through the 130-foot-long seepage pathway from the upstream end of the plug near Station 5+92 to the soils around the LMDT near the Timber-Lattice Bulkhead and Tunnel Liner at Station 4+61. The elevated groundwater levels would likely drain off into the surrounding terrace gravels near Station 6+00. If groundwater levels near the bulkheads were to rise unexpectedly, there would be a warning because this condition would be detected by the groundwater observation well at Station 4+70.

In the very remote event that groundwater levels near the Timber-Lattice Bulkhead and Tunnel Liner at Station 4+61 were to rise to levels which could collapse the tunnel lining, and despite preventative actions a collapse occurred, a blowout is not expected to follow. It is noted that failure of the concrete liner and/or bulkhead would leave a considerable length, (130-feet) of

terrace gravels between the point of collapse and the water released from the mine pool, in turn raising the water head in the tunnel downstream from the Pendery Fault.

Failure of the hillside due to slope instability is highly unlikely. Analysis shows that soil strengths would need to be lower than currently estimated for a failure to occur. In conclusion, engineering analysis indicates that neither blowout nor slope failure are likely to occur.

2. Geotechnical and Structural Analysis

The following chapters of this report present the details of the geotechnical and structural analysis of key features associated with the LMDT. First, the stability of a tunnel collapse flow blockage in the vicinity of the Pendery Fault is evaluated. Next, it is assumed that the blockage near the Pendery Fault has failed and the effects upon the manmade and natural porous plug located between Stations 4+61 and 6+30 are analyzed. Then, the structural stability of the Timber-Lattice Bulkhead and the Concrete Tunnel Lining are evaluated. Finally, the slope stability of the hillside around the Portal is analyzed assuming an increase in the groundwater levels near the portal.

2.1 Stability of Flow Blockage in the Vicinity of the Pendery Fault

An increase in the groundwater levels in the upper reaches of the LMDT and the hydraulically connected old mine workings, has been measured and is commonly referred to as the "mine pool." The elevated groundwater levels are transmitted down the tunnel at least as far as the observation well at Station 46+66. Down station from this point, the groundwater level decreases as indicated by water level measurements from the observation well located at Station 36+77. There is approximately 119-feet of differential head due to the change in groundwater levels between the two observation well locations. A collapse in the tunnel resulting in formation of a flow blockage is believed to have occurred somewhere between the observation wells at Stations 36+77 and 46+66. The Pendery Fault forms a water barrier which apparently prevents water from circumventing the blockage and passing through defects in the rock adjacent to the LMDT. Concerns manifested that further increases in the mine pool level could increase the differential head and cause a rupture of the blockage leading to adverse consequences near the tunnel portal such as a slope failure or a "blowout" (sudden catastrophic release) of the mine pool.

2.1.1 Location of Flow Blockage

Prior to this study, it was thought that the blockage was due to tunnel roof collapse at the Pendry Fault. Detailed examination of construction and operation records indicate that the area of blockage is unlikely located within the limits of the Pendery Fault, but rather is thought to be in the adjacent area of the tunnel below the fault. This reasoning is threefold.

First, Bureau of Mine's reports from construction and post construction state that the Pendery Fault portion of the LMDT was reinforced and concreted. Later observations, through downhole camera work, found the tunnel open and well supported by the concrete encased shoring (Gobla and Vandeberg, 2008). The fault area likely remains open due to the level of support provided during tunnel construction.

Second, just downstream of the Pendry Fault, the tunnel had a history of collapse during construction through zones of quartzite which filled the tunnel for long distances with the running ground. During later operations there were several reports of heavy ground, decaying timbers, and minor collapse of supports. The most likely blockage location is just downstream of the fault between Stations 38+50 and 40+70 within the Parting Quartzite where a section of 46 consecutive timber sets showed signs of dry rot during inspections in 1955. In January, 1955 collapse of two sets near Stations 40+35 to 40+40 formed a dam which backed up water 2.5 feet deep in the tunnel and prompted the recommendation to replace all 46 timber sets. Only six light steel sets were placed in the vicinity of the collapse, the recommended replacement of all 46 decayed timber sets was never completed. Thus, a zone of roof collapse likely extends for a significant distance along the tunnel below the fault.

Third, it is unlikely that the Pendery Fault or any of the rock units in this section are pervious enough to rapidly drain the water from the tunnel, thus a collapse is necessary to explain the differential head. The Pendery Fault, encountered from Stations 40+75 to 40+95, is a normal fault. It is steeply dipping to the northwest. Geologic data defines the Pendry Fault as a hydraulic barrier that prohibits the horizontal movement of groundwater across its boundary, but which can transmit flow along (parallel to) the fault. There has been speculation that leakage of mine pool water into California Gulch is presumably occurring along the Pendery Fault; however, it is not likely that there is a significant inflow from the LMDT into this fault. When the fault was encountered during LMDT construction, it was said to have made "some water" which was a surprise to the tunnel builders who were expecting a large inrush of water. Furthermore, across the fault zone, the tunnel was reinforced with concrete which would be a barrier to flow from the LMDT to the fault. Other underground mine workings contact and penetrate the Pendery Fault, if there is significant flow of mine water along the fault it likely is due to connections with the old mine workings.

2.1.2 Nature of Flow Blockage

Based on the length of tunnel reported to contain dry rot timber supports, a considerable length of tunnel (up to about 200 feet, from Stations 38+50 to 40+75) could be collapsed. This would include both dolomite and quartzite types of rock. The rock downstream of the Pendery Fault is typically more fractured. The dolomite on the downstream, 'Hanging Wall' side of the Pendery Fault was reported to be blocky, and fractured, but was only lightly supported. As the supports deteriorate, loads from the blocky rock may become too much, and a collapse would result. Several zones of fractured dolomite, and a long zone of fractured Parting Quartzite was penetrated by the tunnel excavation downstream of the Pendery Fault.

When the tunnel would collapse during construction, large lengths of the tunnel (reports of 40 to 100 feet) would fill with flowing debris that would eventually stabilize and allow work to resume. Excavation of the inflow debris would often result in resumption of running ground until another blockage to the flow would establish. The tunnel builders eventually learned to bulkhead and inject grout into the inflow debris prior to excavation. Twenty vertical feet of roof collapse is another common dimension referenced in reports. This is the distance that stoping occurred up and beyond the original tunnel crown and typically the entire tunnel and much if not all of the stope was filled with debris.

Debris from the Parting Quartzite is likely non-plastic. A collapse zone in the Parting Quartzite would contain a mixture of blocks, gravel, and sand-sized particles which would likely form a "filter" as the finer particles catch against the coarser particles, making such a zone less susceptible to seepage erosion or piping. Even if the mixture was internally unstable and the fines were washed out, the remaining assemblage of coarse interlocked particles would limit flow through the blockage, and would retain high shear strength, see Figure 2.

The debris was likely deposited through standing water. The material should be assumed to be loose. Side pressure could increase the normal stress and shear strength of the blockage; however, the effects of side pressure from the tunnel walls were not observed in this reach of the tunnel. Therefore side pressures would only be generated by the submerged weight of the collapse debris itself pushing against the tunnel walls. It is assumed that there would not be any significant amount of overburden pressure due to eventual formation of an arch in the fractured rock over the debris pile.

The maximum head on the upstream side of a tunnel blockage likely is limited by the elevation of the top of rock where the overlying pervious terrace gravels would quickly drain away any excess head that would rise above the bedrock interface with the gravels. The exact elevation and location of this hydraulic control is unknown, as it likely occurs at a low bedrock contour elevation off the tunnel alignment. To date, the highest water elevation observed is approximately 10,150 feet which is 163 feet above the tunnel invert at the monitoring well located at Station 96+44.

Finally, based upon observations of actual flow blockage due to roof collapse further downstream in the LMDT, the 20-inch ventilation pipe is likely to be partially collapsed by the force of the collapse and the thicker 8-inch diameter compressed air pipe may still be intact and carrying significant flow (250 gpm). This would explain how observed turbidity changes and injected dye can be rapidly transmitted to the Water Treatment Plant from the upstream areas of the LMDT, and yet a blockage still exist. Although some flows are allowed to pass through the embedded pipes, the collapse debris could still be impeding most of the flow thus creating the observed differential head.

Figure 2. Excavating to remove a boulder at the top of a collapse zone of shattered quartzite in the LMDT, note the mix of material sizes, taken from (Salsbury, 1956).

2.1.3 Forces Acting on the Flow Blockage

As stated, the blockage is comprised of rock blocks as well as gravel, and sand-sized particles. Fine-grained particles are not expected to be present in appreciable quantity because of the nature of the geology at this location. The actual height and length of the collapse debris mass forming the flow blockage is unknown but likely extends above the tunnel crown and encompasses many of the "dry rot" timber sets. In order to calculate a Factor of Safety (FS), the ratio of the summation of forces resisting divided by the summation of forces driving, would require certain knowledge of the actual length, vertical extent, contact area, weight and frictional resistance of the collapse material. As these are unknown, the forces are also not calculable with certainty. However, determining the minimum shear strength required to maintain a plug in place, while resisting the differential head can begin to express the stability of the blockage with more certainty as fewer inputs are required.

The Shape of the original tunnel excavation immediately downstream of the Pendry Fault in the area where it is assumed that the collapse and subsequent blockage has resulted is a modified horseshoe 11 feet wide and 12.5 feet high. This shape is calculated to have a contact perimeter of 42 feet. The face loading area was calculated to be 125 ft^2. Based upon water level elevations measured in observation wells at Station 37+77 and 46+66, a differential head of 119 feet exists. Calculations using differential heads of 100 feet and 150 were used to compute the driving force and shearing resistance necessary to maintain a FS of 1.0 against shearing at the perimeter. Values lower than 1.0 indicate potential instability, while those increasingly higher than 1.0 indicate increasing stability.

The driving force on the face of the blockage was computed by multiplying the differential head for each case times the face loading area. Those values are:

- For 100 feet of differential head: 100 ft. x 62.4 lbs/ft^3 x 125 ft^2 = 780,000 lbs
- For 150 feet of differential head: 150 ft. x 62.4 lbs/ft^3 x 125 ft^2 = 1,170,000 lbs

Next the driving force is divided by the contact area in square inches (contact perimeter times blockage length) to obtain the shear strengths required per square inch of contact area for stability at various assumed blockage lengths. The results are presented in Table 1.

Table 1. Required shear strength for assumed blockage lengths.

Differential Head 100 feet	
Blockage Length feet	Required Shear Strength lb/in^2
10	13
25	5
50	3
100	1
Differential Head 150 feet	
Blockage Length feet	Required Shear Strength lb/in^2
10	19
25	8
50	4
100	2

The results of the calculation are shown graphically in Figure 3 where the data are plotted as required shear resistance vs. length on the y and x axis respectively. Two plots, the upper representing the differential head of 150 feet, and a lower representing the differential head of 100 feet are shown. Both are asymptotic. The curves are down trending with a decrease in required shearing resistance as the length of the blockage (and total frictional contact area) increases. As expected, more shearing resistance is required to hold the larger differential head of 150 feet. Further, the shearing resistance required is seen to decrease as the length of the blockage increases.

For a length of blockage of less than thirty feet, the amount of shear strength required to maintain the blockage in place rapidly increases. At blockages greater than about 50 feet in length, the required shear strength is small and does not change significantly with increasing blockage length. The calculation is a simplification of the actual situation, but is useful in showing the approximate range of frictional shear strength needed for various lengths of blockage to resist the driving force caused by the differential head.

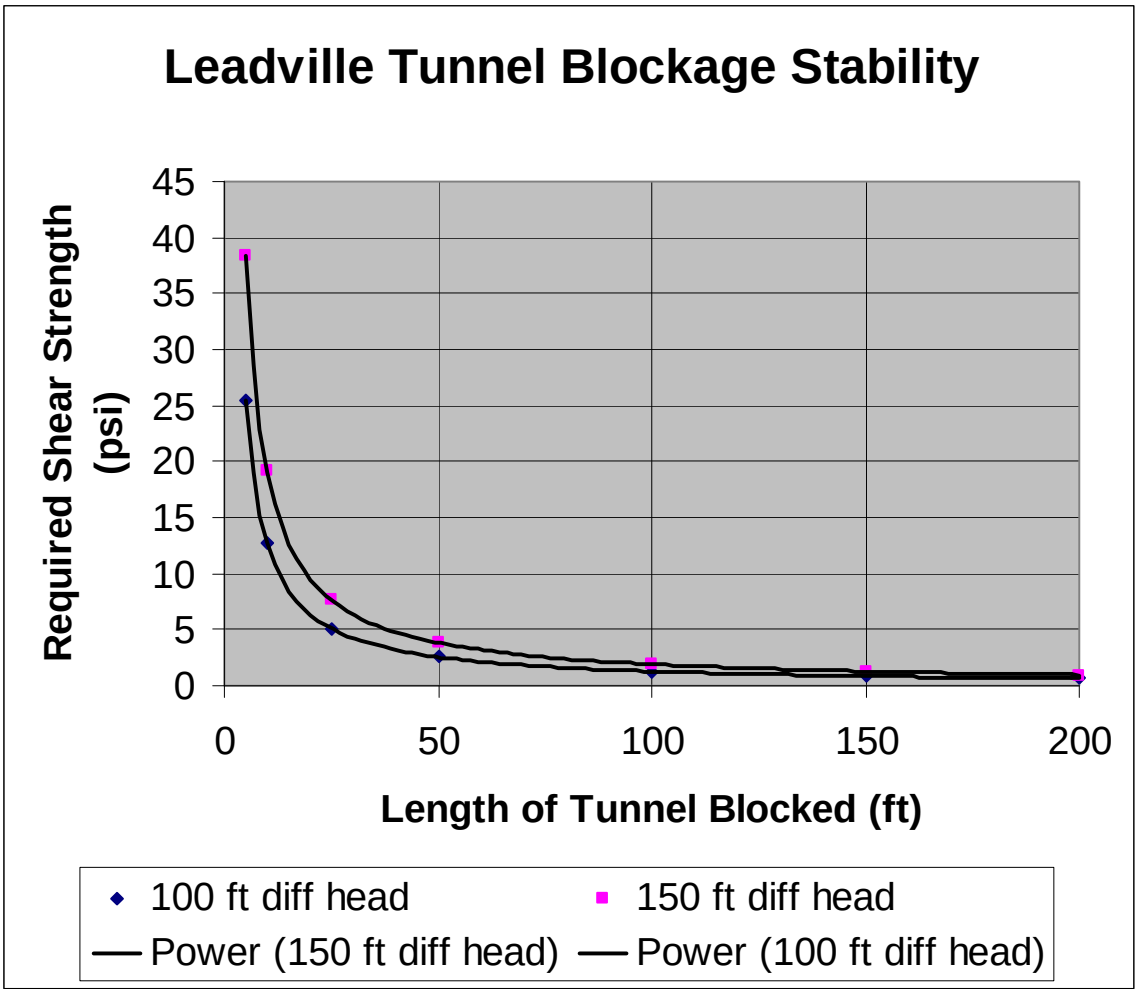

Figure 3. Required shear strength versus length of tunnel blocked from collapse in LMDT.

In reality, the roof of the tunnel above the debris is not likely to provide much frictional resistance before movement starts, if any, due to a lack of a normal force. The sides will provide less resistance than the floor prior to shearing due to differing normal forces. On the other hand, the surfaces of the tunnel in this analysis were characterized as smooth surfaces of uniform shape for the entire length. The actual conditions would be very rough and irregular, thus significantly adding to the shear strength. Also, the debris contains numerous angular rock fragments which would tend to rotate during shearing causing the fill to dilate upon initiation of movement. This would tend to increase the normal forces along the tunnel walls thus adding to the frictional strength. The material above the roof would be sheared at the roof line of the downstream tunnel opening upon movement. This would mobilize additional strength.

The next logical question is, can the required shear strength values indicated actually be achieved by a reasonable length of collapse debris in the tunnel? This question is addressed in the following section of this report.

2.1.4 Likely Strength of the Upper Flow Blockage

The shear strength derived from surface friction prior to movement is equal to the normal force times the tangent of the interface friction angle. Assuming an arched roof forms, there is no overburden pressure acting on the debris in this section of the tunnel other than the debris pile itself. Therefore, the normal force acting on the floor of the tunnel is equal to the submerged weight of the overlying debris. The normal force acting along the sides of the tunnel is equal to the submerged weight of the overlying debris times some factor for the side earth pressure. Also, the normal force along the sides varies from a maximum at the base of the debris pile to no force at the top of the pile. The 11-ft wide by 12.5-ft tall portions of the tunnel which remain intact within and below the collapsed segments will act as a shear key preventing movement of the upper portions of the debris. This would mobilize the strength of the material if movement initiates.

Using this rational, the likely shear strength due to friction is calculated. The following material properties are assumed for the quartzite debris:

Table 2. Material properties for quartzite debris.

Property	Value
Submerged Unit Weight	77.6 lb/ft^3
Friction Angle	35 degrees
Cohesion	0
Coefficient of at rest earth pressure (Ko)	0.43 (1 – sin Ø)

For a unit length of 1 foot along the tunnel, and assuming a debris pile height of 20 feet, the frictional strength is estimated as:

For the 11-foot wide tunnel floor (assuming a 20-foot high debris pile):

F = height x submerged weight x Tangent of the friction angle x width

F = 20 ft. x 77.6 lb/ft^3 x tan (35) x 11 ft. = 11,950 lbs/ft of tunnel length.

For the tunnel walls (below the roof):

P_0 = ½ K_0 x submerged weight x height^2 (for one wall)
P_0 = ½ x 0.43 x 77.6 lb/ft^3 x (12.5ft)^2 = 2,546 lbs/ft of tunnel length.
Double the value for two walls = 5,092 lbs/ft of tunnel length.

Total frictional resistance = 11,950 lbs/ft + 5,092 lbs/ft = 17,043 lb/ft of tunnel length or about 3.3 lb/in^2.

Using the estimated frictional strength, the length of collapse required downstream of a blocked section of tunnel to resist movement of the blockage would be the driving force divided by the frictional resistance per foot of tunnel length. For 100 feet of differential head, the driving force

= 100 ft. x 62.4 lbs/ft^3 x 125 ft^2 = 780,000 lbs. The required length of debris in the tunnel to resist this force without movement = 780,000 lbs / 17,043 lbs/ft = 46 feet.

For 120 feet of differential head the driving force = 120 ft. x 62.4 lbs/ft^3 x 125 ft^2 = 936,000 lbs. The required length of debris in the tunnel to resist this force without movement = 936,000 lbs / 17,043 lbs/ft = 55 feet.

For 150 feet of differential head the driving force = 150 ft. x 62.4 lbs/ft^3 x 125 ft^2 = 1,170,000 lbs. The required length of debris in the tunnel to resist this force without movement = 1,170,000 lbs / 17,043 lbs/ft = 69 feet.

The actual length of collapsed material forming a flow blockage is not known, as the differential head increases, the length of blockage required to resist movement increases. However, given that about 40 timber sets exhibiting dry rot were not replaced, the length of collapsed tunnel could easily approach 80 or more feet. Further, if the material began to shear, additional resistance would be generated at the roof line, and at the wall due to the roughness and dilation of the material.

2.2 Stability of The Porous Plug Above The Timber-Lattice Bulkhead

The following portion of this report assesses the stability of the porous plug located between Stations 4+61 and 6+32. The evaluation includes a description of the composition of the plug, its geologic environment, current groundwater conditions, and the effects of a rapid rise in groundwater pressure should the upstream tunnel blockage near the Pendery Fault fail. The results of this assessment play a key role in understanding the likelihood of Potential Failure Modes 1, 2, 3, and 4. Most importantly, this assessment shows that even if the upper blockage fails, the rapid transmission of a pulse of groundwater pressure to the porous plug likely will not result in failure of the bulkhead.

2.2.1 Description of the Porous Plug

The porous plug, which has also been referred to as the “lower blockage” is comprised of a heterogeneous matrix of naturally occurring materials and man-made construction components including two bulkheads with cobble and gravel fill, and over 100-feet of collapsed overburden plus injected sand and gravel fill. This section of the tunnel was excavated through glacial deposits and terrace gravels.

At the downstream end, from Station 4+60 to 4+61 is the Timber Lattice Bulkhead as shown in Figure 4. The timbers are held in place by an L-shaped bracket made of stainless steel which is anchored to the Concrete Tunnel Lining. From Station 4+61 to Station 4+66 there is 5-feet of cobble and gravel fill held in by the Timber Lattice Bulkheads. The fill immediately against the

bulkhead on the downstream side is comprised of cobble-size rock as seen in Figure 5.

Figure 4. - Photograph taken in 2001 of the existing Timber Lattice Bulkhead installed in 1991 at Station 4+60 to 4+61. Note clarity of water outflow and 2.5-foot high flow with has been typical since post construction.

Figure 5. Photograph taken on October 25, 1991 of the Timber-Lattice Bulkhead as construction was nearing completion showing the layer of cobbles at Station 4+62 of the LMDT.

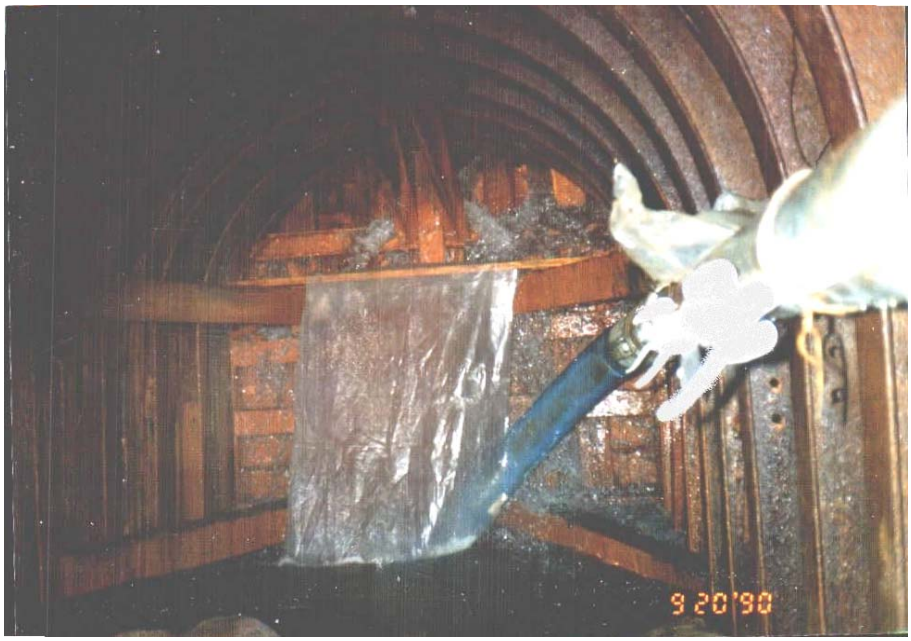

Figure 6. Photograph taken on August 20, 1990 showing the porous bulkhead located at Station 4+66.

At Station 4+66 a porous timber bulkhead was previously constructed in 1980 when the LMDT was excavated back to approximately Station 5+00 (Figure 6), new steel supports were placed, and gravel backfill was installed to bear against the sand-filled collapse zone. The tunnel along this porous plug section was originally excavated to approximately 11 feet wide and 12 feet high (Gobla and Vandeberg, 2008). Various tunnel supports and spacing were used. Both wood and steel sets were used with the only common denominator being the use of wood for blocking and lagging and the use of spiling through bad ground. The exact configuration and location of all the various support members are not documented. Spacing varied and various support members or configurations were changed over the years as maintenance and stability issues occurred.

The tunnel at Station 4+62 has a bedrock bottom and 4-foot-high bedrock side walls, above which the rest of the wall and crown are composed of unconsolidated terrace gravel materials. The bedrock surface slopes up station and by Station 6+35 the entire tunnel is completely in bedrock (Reclamation, 1989). The change from a tunnel roof in terrace gravels to a rock roof is probably the reason why the 1968 surface subsidence and the area of tunnel collapse did not extend further southeast. The thickness of the overlying overburden above the blockage area averages about 100 feet. Most of the blockage in the tunnel is from the 1968 ground subsidence that occurred some where between Station 5+00 and drill hole #2, at about Station 5+75, and extended all the way to the surface (Figure 7).

The interpretation of the length of the collapsed zone was based upon the data from five drill holes and downstream observations in the tunnel. Figure 7 shows the collapse going all the way to Station 6+32. However, no other documentation supports that this was the actual case; no back filling that was done through drill holes other than drill holes 2 and 3, at about Station 5+92, see Table 3. It is possible that this 40 foot area from Station 5+92 to Station 6+32 may have collapsed since then, but it is not known for certain. There is no direct evidence that material is plugging the tunnel any further up station than 5+92.

In 1973, Reclamation awarded a contract to clean out first 200 feet of tunnel, install new steel horseshoe shaped supports from Station 1+00 to Station 2+00, and completely backfill all remaining sinkholes, voids, and un-collapsed portions of the tunnel between approximate Stations 1+25 and 5+00 (Bennett, 1977). A bulkhead of treated timbers was also installed at Station 2+00.

In 1975 Reclamation installed a 1 cubic foot per second (cfs) capacity pump at Station 6+34 in an attempt to maintain the groundwater table at a low level in ground adjacent to the lower portion of the tunnel. This was considered to be a temporary fix. It is interesting to note that this was followed in 1976 by numerous sinkholes at the ground surface above the LMDT from Station 2+00 to approximately 6+50. It is assumed that this portion of the tunnel was almost completely filled with sloughed material.

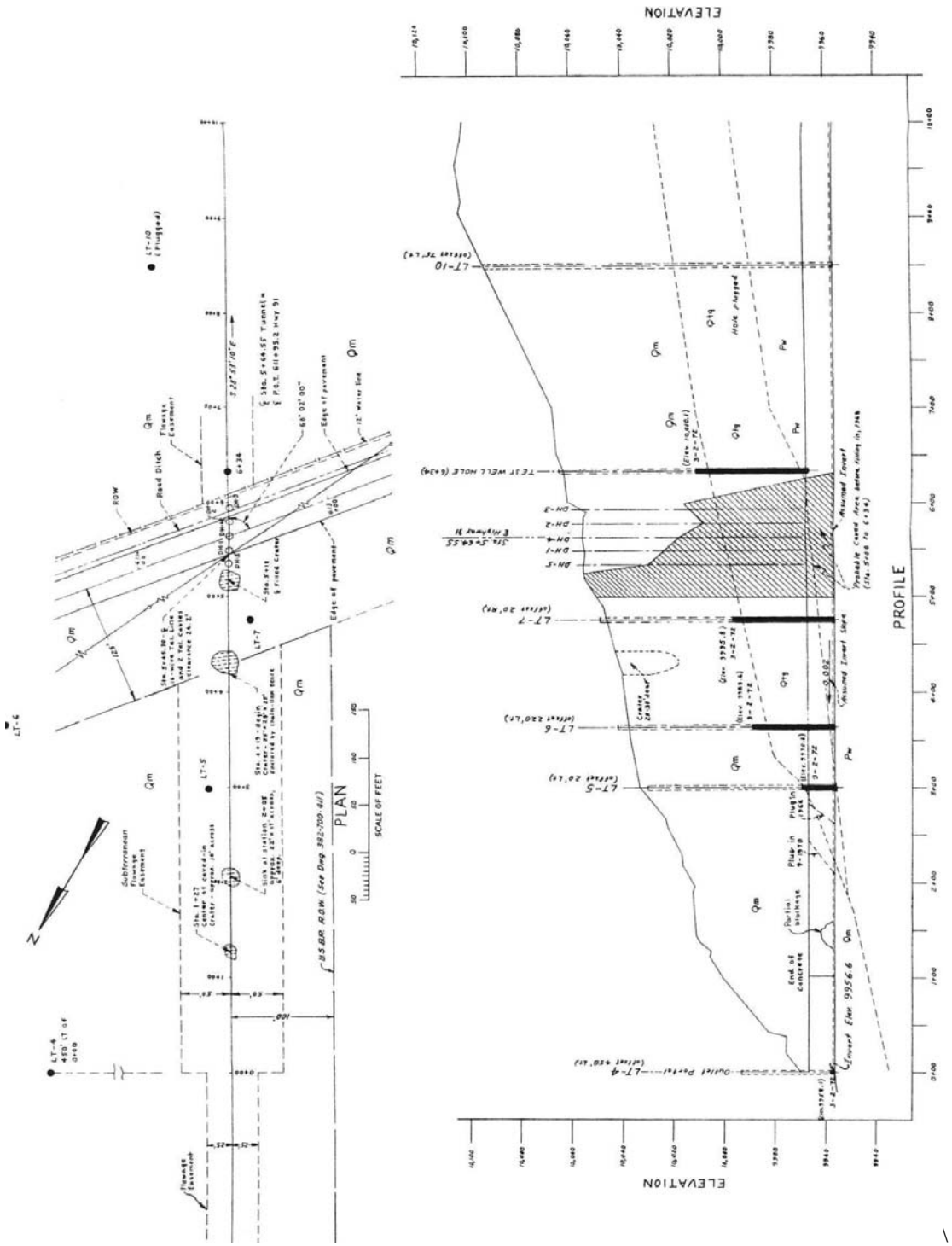

Figure 7. Plan and section showing condition of the LMDT in 1972 including the location of sinkholes, 1968 injection drill holes, and monitoring wells installed in 1968, taken from (Reclamation, 1976).

From 1978 until 1980 the bulkhead at Station 2+00 was removed and the tunnel was excavated back to perform consolidation grouting of the overburden and tunnel support rehabilitation was completed for the first five hundred feet of the tunnel. Records regarding the extent of consolidation grouting performed, if any, have not been found. The project encountered difficulty in drilling a well above the collapse area to reduce water levels in the tunnel. Most of the excavation work to open the tunnel back to Station 5+00 was performed with a constant flow of water and there were some instances of running ground filling the excavation. Eventually the tunnel was opened and the steel sets were improved to Station 5+00. Upstream from Station 5+00 the collapsed fill from 1968 remained. Gravel fill was placed against the collapsed material and continued down to Station 4+66 where a timber bulkhead was installed (Figure 6). The steel sets provide some roughness to the tunnel perimeter and the bulkhead provides some restraining force to downstream movement of the collapsed area infill and the man-made fill placed between Stations 5+00 and 4+67.

Table 3. Results of five injection drill holes into the LMDT in 1968.

Drill Hole Number	Voids Encountered	Gravel Placed yd^3	Grout Placed bags of cement	Condition of LMDT at drill hole bottom
1	5-foot cavity between 61.9 and 66.9 feet above LMDT	7 at upper void	172	Tunnel filled to crown with caved material
2	4-foot cavity between 47.7 and 51.7 feet above LMDT	12 at LMDT, 0.5 at upper void	93	Tunnel filled to within 4 feet of crown with caved material
3	10-foot cavity between 49.7 to 59.7 feet above LMDT	48 at LMDT, 23 at upper void	185	Tunnel open
4	3-foot cavity between 58.4 and 61.4 feet above LMDT	4 at upper void	155	Tunnel filled to crown with caved material
5	1-foot cavity between 74.6 and 75.6 feet above LMDT	0.25 at upper void	5	Tunnel filled to crown with caved material
Totals		94.75	610	

Transecting the 1968 collapsed portion of the tunnel is an existing 20-inch ventilation pipe and an 8-inch cast iron pipe along bottom of the east side of the tunnel. The smaller diameter pipe was estimated in 1980 to be passing 250-400 gpm while the larger diameter ventilation pipe was only estimated at 250 gpm. It is probable that the larger diameter, less crush resistant, ventilation pipe has some obstructions. It was noted after the 1968 collapse that flows continued through the 20-inch diameter steel ventilation pipeline at about 3.7 cfs (1660 gpm). Later on, an 8-inch-diameter test well was drilled 60 feet east of the highway and the tunnel was found to be open. The casing was pulled to the top of the LMDT and water levels were measured to be 23 feet above the top of the tunnel. This water level indicated that the LMDT water discharge through the ventilation pipeline required some head to force the flow through the pipe. Therefore, the flow was being partially retarded by the collapse. More importantly is that the combined flow from these two conduits equals most of the flow down at the portal so that the amount of flow through the actual material filling the collapsed area must therefore be minimal.

Based upon what could be observed, the collapse material is a heterogeneous mixture of overburden materials with steel and wooden tunnel segments spread through out the collapsed mass. These large support segments could add to the resistance of the tunnel blockage from being displaced down the tunnel. However, it is known that both the wooden and steel support members disintegrate in the mine environment and therefore could not be depended upon for the long term. Furthermore, these segments may result in voids along the boundary of the segments and the collapsed material if there was inadequate natural compaction against these segments or if these segments weather away.

It should be noted that with the double bulkheads at the downstream end of the collapse area, and the manner in which the gravels and cobbles were placed in between the bulkheads and the collapsed material, that the risk of piping or erosion of fines would be reduced, if not eliminated. In fact, the water seen exiting from the bulk head at Station 4+60 (Figure 4) has remained clean for 18 years and the drainage ditches on either side of the walkway are free of sediments. This is evidence that filters have formed and they are retaining material. The tunnel shows a pressure differential from a maximum at the upstream end of the collapse (around 50 feet of head) to a minimum at the Timber-Lattice Bulkhead, which is exposed to the atmospheric pressure and showing 2.5 feet of head.

It should be noted that the tunnel from Stations 0+54 to 4+61 was supported with steel sets, lined with reinforced concrete and the inside diameter reduced in size to an 8 feet wide by 8.5 feet high, semi-circular arch. Furthermore, the portal area has a massive concrete structure, (Figure 8) and the space behind the concrete tunnel lining was pressure grouted. Therefore, the entire structure together with the bulkhead and reduced cross sectional area would provide additional restraint to the material plugging the larger diameter tunnel from being ejected.

The geology of the LMDT along the porous plug includes three units; Quaternary glacial moraine (Qm), terrace gravels (Qtg), and Permian Minturn Formation (Pm). The glacial moraine consists of gravel, cobbles, and boulders in a silt and sand matrix and it overlies the terrace gravels. The terrace gravels make up the crown and part of the tunnel side walls until at about station 6+35 where the bedrock surface meets at the tunnel crown. Downstream of about station

3+50 the tunnel is completely in alluvial material and gets thicker below the tunnel invert until it is over 100-feet thick at the portal area. The Minturn Formation is interbedded sandstones and shales and makes up the entire tunnel floor along the tunnel plug. At Station 4+62, the lower 4-foot portion of the tunnel wall is bedrock. The height of the bedrock in the walls rises slowly in the upstream direction and, at Station 6+35 the rock reaches the crown of the tunnel.

Figure 8. Photograph taken May 21, 2008 showing the reinforced concrete LMDT Portal with 1-ft. thick wing walls.

2.2.2 Plug Material Properties

Although the material filling the tunnel is of a heterogeneous nature, simplifying assumptions were made in order to conduct an analysis. Other than the pipes that became buried along the length of the blockage, the material in the entire length of the blockage was considered to be uniform, including the bulkhead areas and any fill placed into the upstream collapse area through drill holes.

The tunnel dimensions and area were taken as uniform for the entire length of the blockage. Also, the wall of the tunnel was assumed to be smooth, which represents a more conservative case than actually exists. Approximately 130 feet of plug is known to exist, the drawing of the

1968 collapse and fill suggests that a total of 170 feet of plug could exist.

A plug length of 92 feet was used in stability calculations as a conservative estimate. Since the 1968 collapse extended to the surface and the overburden is unconsolidated, it was assumed that the full overburden load was being seen at the tunnel collapse from Station 5+00 on upstream to the end of the plug. At and below Station 5+00, the tunnel was rehabilitated with closely spaced steel sets and lagging therefore the lower 38 feet of tunnel plug has a roof and will likely not experience full overburden pressure on the backfill. The lower 38 feet of the tunnel fill was not considered to provide strength, a conservative assumption.

The two bulkheads downstream of the blockage were only considered as being important contributors to forming a filter whereby the fines and gravel size fragments could not be easily removed by erosion or piping. Although the bulkheads, and the intervening steel sets would also add shear resistance to the material in the plug, the resistance was not considered and was therefore viewed as another conservative factor in the model.

Physical and strength properties identified for use in the evaluation were based upon a 2005 valve controlled bulkhead study (Smith and others, 2005) which was based upon available project data, interviews, and site visits; however, no strength tests were undertaken. The unit weight, strengths, and other data are assumed values; however, they appear to be reasonably conservative for the types of materials involved.

The unconsolidated overburden was assumed to be all the same and a conservative value of unit weight of 110 lb/ft^3 was used. A higher value of void ratio was used, which corresponds to the glacial moraine and is in agreement with a 1988 study. The friction angle was varied from 30 to just over 46 degrees, with the higher value matching that used in a 1988 analysis of the plug stability (Reclamation 1988). The higher friction angles are possible because the tunnel cross sectional area drops from 119 ft^2 at the blockage to 66.8 ft^2 at the concreted tunnel section. In addition the tunnel side surface would not be totally smooth. An at rest lateral earth pressure coefficient of $1- \sin\varphi$ was assumed.

2.2.3 Plug Hydrogeology

The hydrogeology of the entire area of the LMDT and connected mines is quite complex. It is summarized in a report by Gobla and Vandeberg (2008) and detailed in a report by the EPA (2002). The hydrogeology of the tunnel collapses is much simpler as they block or restrict flow down the tunnel alignment and may or may not be completely filling the tunnel. Water flows through the unconsolidated glacial moraine and terrace gravels are much greater than through the bedrock, except maybe in the areas or directions of intense fracturing or solution channels. The unconsolidated gravelly aquifer is porous and tends to readily transmit groundwater flow. The geometry of the bedrock is a controlling factor in groundwater flow which is towards the river and the portal along the bedrock surface. Rock fractures and solution channels are of little importance in the bedrock formations present (shales and sandstones) along the plug portion of the LMDT.

Observation wells have been placed along and adjacent to the tunnel alignment to measure the head in the tunnel and in the overburden, and provide access for down hole camera observations in the tunnel at a few of the well locations. Also, at Station 10+25 there are two additional wells which are used to pump water from the tunnel to control the head in the tunnel upstream from the tunnel blockage between Station 4+61 and 6+32.

2.2.3 Plug Failure Analysis

The stability issue of the lower tunnel plug was first addressed in a memorandum dated August 22, 1988 from the Regional Engineer, Billings, Montana to the Project Manager in Loveland, Colorado (Reclamation, 2002). The memo concluded that the plug was not likely to blow out and had a considerable factor of safety with respect to existing conditions. After studying the 1988 memorandum, it was decided that the data and conclusions were not adequate. Some of the reasons were:

- All steps of calculations were not shown.
- It did not reference all the sources of data
- No attention was given to piping or internal erosion
- The 1988 memorandum covered a plug length of 170 feet even though the evidence suggests it may only be 130 feet long.

The only values that could be verified in the memorandum were the volume of a 170-foot-long plug and the hydraulic driving force. However, the mass of the plug at 995,000 pounds seemed like it was low, but might be the submerged buoyant weight. The mass of the 170-foot long plug should be at least 2.4 million pounds, and have a submerged weight of 1 million pounds. Lastly, the angle of internal friction used was not stated and the amount of shear resistance reported seems high. Since the previous study was poorly documented, and due to renewed concerns about the stability of the tunnel plug, a new stability analysis was carried out.

Tunnel blockage, whether from a natural collapse or a man made plug that then develops a hydraulic head behind it, can fail for one of several reasons. However, the primary reason is usually leaking around the blockage which usually leads to an erosional failure around or next to the blockage and not a blow out of the obstruction (Abel, 1998), (Harteis and Dolinar, 2006), (Fuenkajorn and Daemen, 1996). No discussion or evaluation of this critical issue was made in 1988.

The upstream end of the plug is not directly visible. On the downstream end there are man made structures from which some data can be obtained. Water flow into the concrete tunnel and monitoring wells located both upstream and downstream of the bulkhead are measured. However, no test data are available for any of the materials in the collapse zone. The upstream limit of the collapse and the extent of the sand and gravel fill (Table 3) into the collapsed material are all mostly derived by anecdotal evidence.

Since the lattice bulkhead was installed (1991), the tunnel flow has been rather consistent and,

more importantly, the volume of flow has been less than the capacity of the cast iron and vent pipe conduits that pass through the plug. Therefore, it appears that very little flow is coming directly through the surrounding ground surface or directly through the tunnel blockage. The difference in head seen in boreholes located upstream and downstream of the blockage is usually no more than 10 feet and most of the time the upstream and downstream levels are at about the same elevation. In fact, since the elevation of flow through the Station 4+61 bulkhead is 15-20 feet lower than the two nearby observation wells, groundwater flow is through soils around the tunnel and towards the bulkhead at Station 4+61.

The explanation for this behavior is that the collapsed fill portion of the plug, although somewhat pervious, is believed to be a barrier to water flow. There were fines in the terrace gravels which collapsed into the tunnel and a mix of sand and gravel was injected through boreholes into the fill. The up-station part of the plug is known to impound water. The lower portion of the plug, from Station 5+00 down to Station 4+62, is filled with clean gravels and cobbles. The absence of fines creates a much more porous and very pervious flow medium in the downstream portion of the plug. For the purposes of modeling, it was assumed that the plug is nearly impermeable over the entire cross sectional area for the upstream 92 feet, except for flow in the surrounding soils around the plug or through the two previously mentioned pipes.

It was also assumed that the plug is in contact with the entire perimeter which was conservatively modeled as smooth and uniform, and with constant normal loading. With a heterogeneous collection of material in the plug and no test data, a range of values was examined to analyze any sensitivity in the frictional strength parameter. An angle of internal friction of 46 degrees was selected as a maximum value and a conservative value of 30 degrees was selected as a minimum value. A conservative value of zero was used for the cohesion.

The head of 34 feet that has been experienced over the past several years was selected as a minimum. Maximum head was selected based upon the back calculated value of hydraulic head that would be needed to obtain a factor of safety (FS) equal to 1.0.

The shear resistance provided by the blockage was calculated using:

Shear Stress = (Normal Stress – pore pressure) * tan (phi) Shear stress was assumed to occur along the entire theoretical tunnel perimeter.

Where: Stress normal to the shear surface is broken down into vertical and horizontal surfaces
Pore pressure is the hydraulic head * 62.4 lb/ft^3
phi is the angle of internal friction between the blockage material and the tunnel perimeter

The vertical stress from the overburden on the blockage was assumed to not be arching over the blockage since it had failed to the surface and was calculated by:

Sigma-v = Unit weight *Depth of overburden /144 in^2

Where: Sigma-V is the vertical stress
Unit weight of the overburden material, lbs/ft^3
Average depth of overburden in feet

The secondary principal or horizontal stress on the sides of the blockage was calculated by

Sigma-H = Sigma-V * (1-sinφ)

Where: Sigma-H is horizontal stress

The force driving the blockage out was calculated using:

Driving force = Vertical face area of the plug times the hydraulic pressure

Where: Vertical area = nominal area of the semi-circular tunnel shape at the upstream plug end
Hydraulic pressure = the height of the water table above the invert of the tunnel, rather than at the centroid of the face, is another conservative assumption.

The Excel spread sheets in Appendixes B & C summarize the type of calculations that were made. The results are summarized in figures 9 and 10. Figure 9 shows the calculation for the plug considering an instantaneous loading condition. In other words, it assumes that the upper blockage near the Pendery Fault has failed and a pulse of groundwater pressure is rapidly transmitted down the tunnel to the upstream end of the plug.

As would be expected, the lower the friction angle, the lower the safety factors for a given hydraulic head. As the hydraulic head increases, the safety factor decreases for each value of friction. However, for each pair of curves, with a different friction angle, the two curves when plotted against safety factor are non linear and with increasing hydraulic head converge. At a factor of safety of 1, indicating the driving and resisting forces are equal, the hydraulic head is nearly 220 feet. This is more than 100 feet higher than any groundwater levels ever measured in

this part of the LMDT. The calculations show that this pressure pulse can be withstood and the plug will not "blowout."

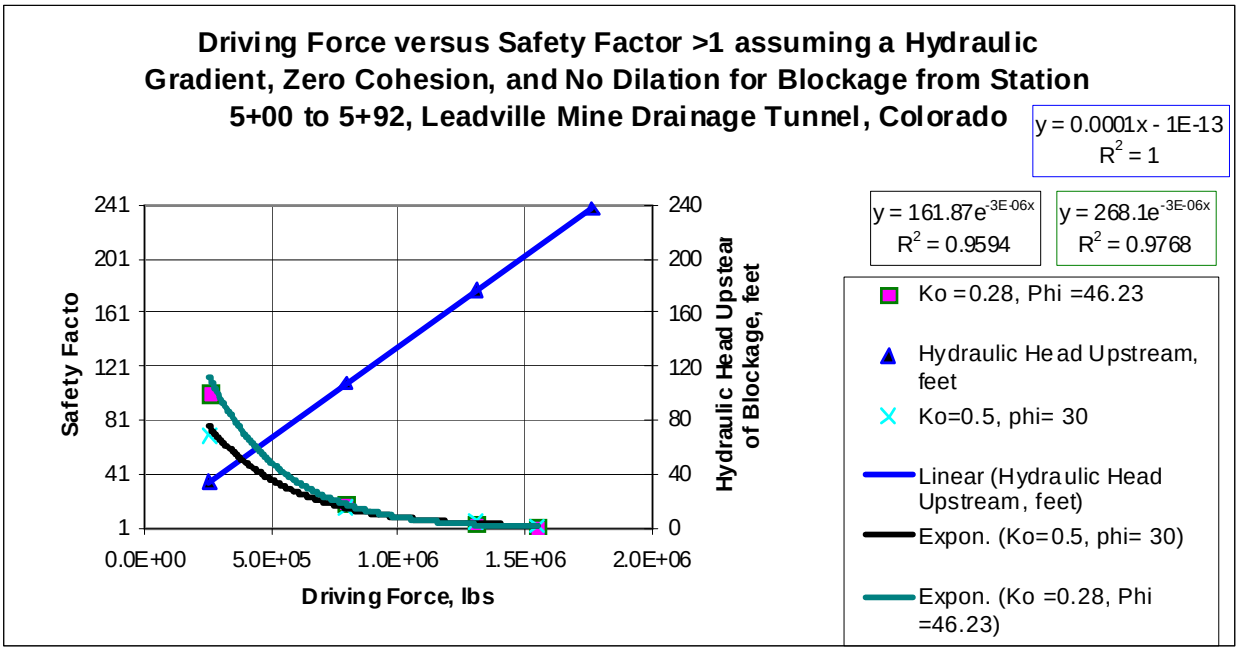

Figure 9. Driving force versus differential head and safety factor for an instantaneous increase of head at the upstream end (Station 5+92) of the plug in the LMDT. The additional resistance from the manmade bulkheads and porous cobble and gravel fills located from Station 4+61 to Station 5+00 are not included in the calculation.

A second calculation was made to consider what would happen over a period of time. In time, the initial rapid increase in water pressure would begin to inject water through the pore space in the plug thus raising the groundwater levels in the surrounding soils. Eventually elevated groundwater conditions could be experienced at a distance away from the upstream end of the plug. An assumption was made that the full upstream hydraulic head would travel to the lower end of the nearly impervious part of the plug at Station 5+00. The calculation is repeated considering a uniform elevated groundwater condition all along the plug from Station 5+92 down to Station 5+00.

The concrete tunnel lining and concrete portal with its supporting wing walls are massive concrete structures. The concrete lining has an internal area smaller than the area containing the plug. Therefore, any mass movement of the plug down into the tunnel alignment would cause dilation of the angular fill and it would be faced with a cross sectional restriction thus generating high frictional resistance. The massive, anchored concrete portal structure and concrete tunnel lining which is attached to preexisting steel tunnel supports, and any of the consolidation grouting behind the concrete lining form a structure which can resist the thrust produced by the frictional loading.

The results shown in Figure 10 for the full hydraulic head across the length of the plug indicate that the conservative factor of safety remains above 1.0 for differential heads up to about 125

feet.

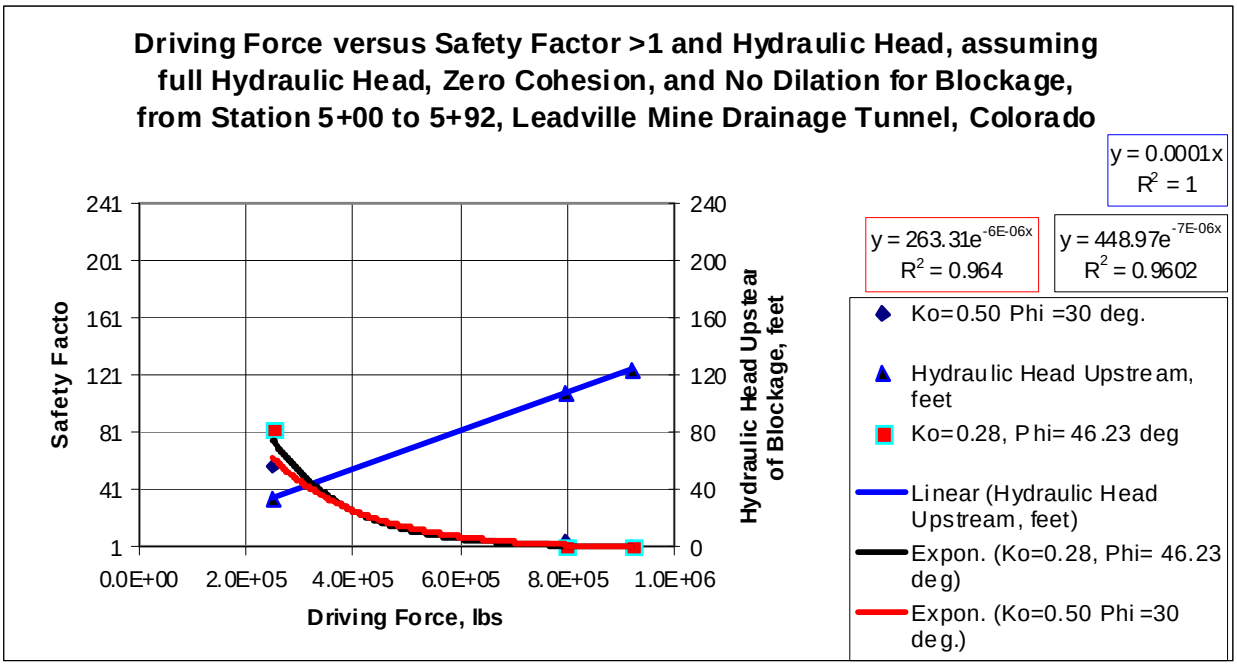

Figure 10. Driving force versus safety factor for a diffusion of head along the plug from the upstream end at Station 5+92 to just above the porous fill at Station 5+00. Again the additional resistance provided by the manmade bulkheads and porous cobble and gravel fills located from Station 4+61 to Station 5+00 are not included in the calculation.

Considering the nature of the surrounding soils it is impossible for such a condition to exist. First of all, the soil overburden is only about 100 feet thick in this area which would limit the groundwater levels above the plug to that height. Pressures beyond that level could not be transmitted downstream along the plug. Instead, the wells near and upstream of the plug would become artesian. Secondly, as the water with elevated pressure at the upstream end of the plug spreads out into the soil, it is likely to drain off laterally as well as towards the bulkhead at Station 4+62; therefore, any elevated water table forming along the plug is going to be attenuated by drainage into the adjacent soil and be substantially less than the upstream driving head. Considering the situation, it is concluded that a “blowout” of the plug along the tunnel and out the portal is virtually impossible.

2.2.4 Other Potential Failure Modes If the Plug Holds

Other modes of failure modes might be possible if most of the maximum mine pool head were seen at the lower plug and the plug holds. Failure Modes 2, 3, and 4 must be considered.

Failure Mode 2, a seepage erosion breach of the downstream portal slope is also unlikely. First a portion of the elevated groundwater levels will drain to the pervious part of the plug. Since the plug will remain intact, the gravel and cobble fill will continue to act as a filter and allow the water to pass. Even if a hydraulic fracture forms along the side of the tunnel allowing water seepage and higher pressures to reach to the lower end of the plug, the flow will still need to pass through the 38 feet of gravel and cobble fill where filtration is expected to be maintained.

For water migrating from the upstream end of the plug towards the toe of the hillside, there is a reasonably low gradient. There is at least 400 feet of soil through which the flow must travel. Regarding the potential for possible piping or backward erosion at the downstream slope, seepage should be looked for and if found evaluated and monitored. Likely seepage exit points would be on the outside of the Portal wing walls, or along the toe of the hillside on either side of the Portal. Therefore, this problem becomes an issue similar to leakage at embankment dams. The appearance of seepage, and monitoring of changes in seepage rates and turbidity with time can be used to determine the stability of seepage areas. Any changes in volume of water or observations of solids moving in the seepage areas can be used to give an early warning and the appropriate course of action taken: evacuation and/or drawdown of the mine pool, and capturing and treating seepage flows. At present there are no known seeps at the downstream toe of the hillside.

Failure Mode 3 deals with elevated groundwater levels causing slope instability of the hillside. This is addressed in a later section of this report. Failure Mode 4, Leakage of contaminated water into downstream areas is possible. If the elevated groundwater spreads out and drains into the surrounding soils as well as to the porous end of the plug, the water in the surrounding soils will likely migrate to the river and not be captured by the LMDT bulkhead flows. If the water elevations of a pressure pulse are high enough, the wells at 10+25 and 6+36 may experience artesian flow to the surface. Therefore under present conditions, some amount of loss of contaminated water into downstream areas could occur if the Pendery Fault area blockage fails.

One final observation was made for the water elevation data for drill holes at Stations 6+34 and 10+25. There appears to be a difference in head between these two wells, see Figure 11. This would suggest that a small tunnel collapse or blockage may have formed some place between the two wells. Reviewing the history of the tunnel, in 1979 a well was drilled into the tunnel at Station 6+65, but while waiting for delivery of well screen, a sinkhole appeared next to the drill rig and the hole was lost. If there is a blockage in this area, then the risk to the tunnel blockage downstream of Station 6+34 may be reduced even further since the pressure gradient down the tunnel would experience another damping effect from this blockage. This also points out an important fact that given the history of the LMDT, new collapses may form and old ones may grow. As a result, there will be even more barriers to not only impede flow down the LMDT but

to create additional hydraulic barriers, which will create additional stepped pressure drops along the drainage path.

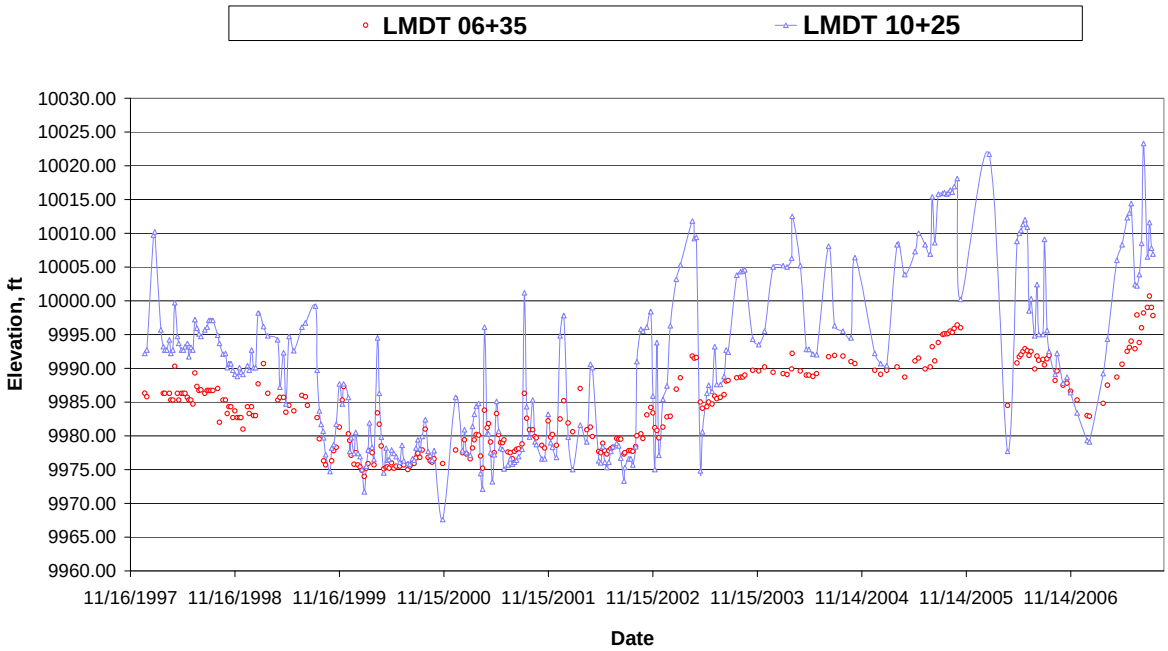

Figure 11. Water levels measured in wells at Stations 6+35 and 10+25.

2.3 Stability of Timber-Lattice Bulkhead

The timber bulkhead at Sta. 4+60 of the Leadville Mine Drainage Tunnel was inspected March 25, 2008. The dimensions and spacing of the timber members of the bulkhead were confirmed to be the same as shown on drawing no. 1335-D-123 (see Appendix A). The timbers were probed with an awl and appeared in good condition.

The size and number of stainless steel anchor bolts were also confirmed the same as shown on drawing no. 1335-D-123. The bolts and stainless steel angle brackets affixed to the concrete also appeared in good condition.

The analysis of the components of the timber bulkhead was based on the following:

- Timber beam (see Figure 12) was assumed simply supported and uniformly loaded.

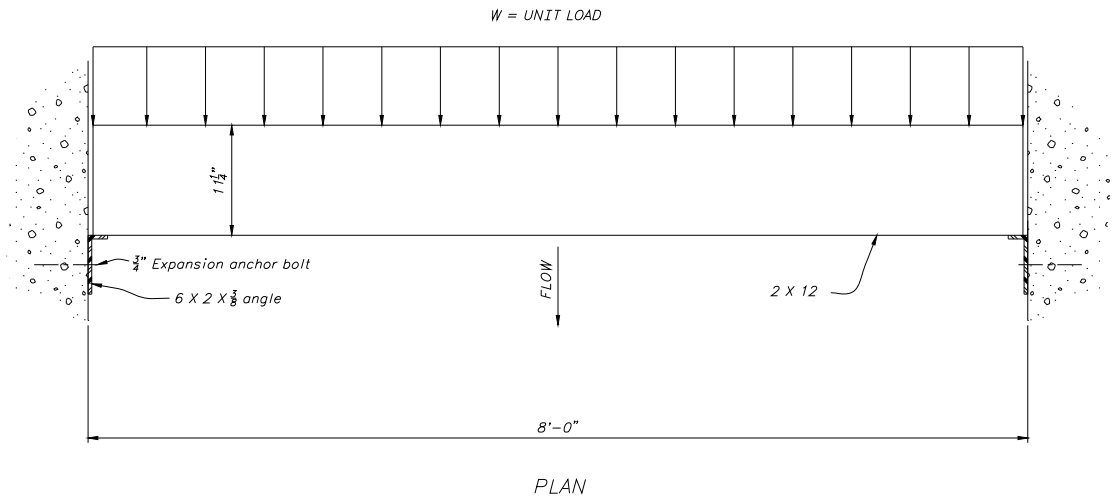

Figure 12. Plan view of timber beam.

- Length of beam for moment and bearing = 7.9 ft.
- Effective length (length – 2 x depth of beam) of beam for shear = 6.0 ft.
- The timber was assumed to be Douglas Fir Larch, select structural grade as shown on drawing no. 1335-D-123. Properties for the timber are from Timber Construction Manual, American Institute of Timber Construction, 2nd ed., 1974. Timber properties follow:
    - Allowable bending stress (repetitive member use) = 2050 lb/in^2
    - Allowable horizontal shear stress = 95 lb/in^2
    - Allowable compression perpendicular to grain = 385 lb/in^2
    - Duration factor (assume 50 years) = 0.96
    - Adjustment factor for bending when moisture content exceeds 19% = 0.86
    - Adjustment factor for shear when moisture content exceeds 19% = 0.97
    - Adjustment factor for compression perpendicular to grain when moisture content exceeds 19% = 0.67

- No reduction in allowable stresses assumed for preservative treatment

- The assumed average factor of safety is 2.5. Ninety-nine out of 100 pieces will have a safety factor greater than 1.25. See Design of Wood Structures—ASD, Breyer, Donald E., Kenneth J. Fridley, David G. Pollock and Kelly E, Cobeen., McGraw-Hill, 5th ed.

The analysis of the ¾ -inch diameter expansion anchor bolt was based on the following properties:

- Allowable shear per bolt = 5.65 kips

- Assumed safety factor (ratio of ultimate load to allowable load) = 3.8

The analysis of the 6 x 2 x 3/8 angle (see Figure 2) was based on the following properties

- Yield strength (F_y) assumed to be 39,000 lb/in^2 (type 304L)

- Assumed allowable shear stress = 0.4 x F_y

- Assumed allowable bending stress = 0.66 x F_y

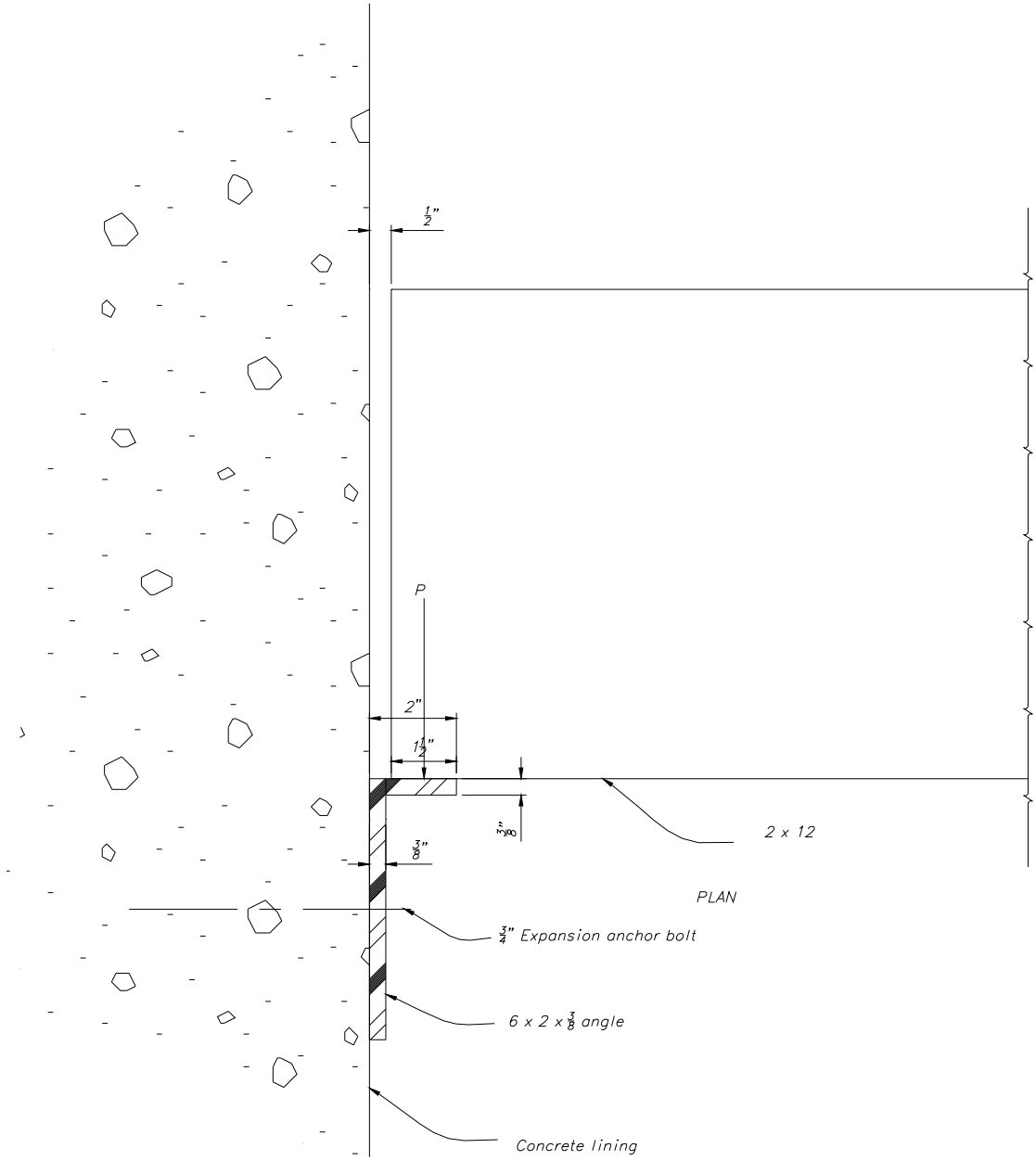

Figure 13. View of angle bracket and anchor bolt.

The soil loading against the bulkhead assumed the following:

- Assumed angle of internal friction = 36 degrees
- Assumed unit weight of moist soil = 115 lb/ft^3
- Assumed unit weight of saturated soil = 130 lb/in^2
- Assumed Jaky's at-rest coefficient = 0.412
- Assumed effective height of soil against bulkhead = 5.8 feet

The hydrostatic pressure exerted against the timber bulkhead assumed the following:

- Assumed percent of clear area between horizontal beams that is open to flow = 11%
- Assumed no hydrostatic pressure on downstream side of bulkhead

The results of the analysis of the bulkhead components are summarized in the following table:

Table 4. Results of bulkhead components analysis.

Bulkhead Component	Design Head (ft. above invert)	Computed Head at Failure (ft. above invert)	Notes
Timber beams	7.0	21.0 (52.0)	1,2,3,4
Support angles	34.0	52.0	5
Anchor bolts	30.0	116.0	6

Notes:

1) Timber beam is assumed to have an average factor of safety of 2.5 for the design head.
2) Timber beam is assumed to fail at an average factor of safety just less than 1.0.
3) The value of computed head at failure shown (not in parenthesis) represents the value that the beam would fail at in compression perpendicular to grain. This failure mode would occur at the bearing of the timber beam on the angle.
4) After the beam has initially failed in compression perpendicular to grain, it was assumed that the wood fibers would densify until the mode of failure finally becomes shear (the value in parenthesis). This should be verified by laboratory test if this value is deemed critical.
5) Support angle is assumed to fail by flexure when yield strength is reached.
6) Expansion bolt is assumed to have a safety factor of 3.8 in shear.
7) Hydrostatic pressure is assumed to develop at the face of the timber lattice; drainage at the interface is ignored.

Discussion –

The "design head" represents the height of water (relative to the tunnel invert) that would be considered a safe value for design purposes. The "assumed head at failure" represents the height of water (relative to the invert) where failure could occur for the assumptions outlined above. Reports indicate that historically the water level at the timber lattice never rises more than about 2 or 3 feet above the tunnel invert, regardless of the fact that the water level at Station 10+25 has exceeded 70 feet above the tunnel invert. There is only about 80 feet of unconsolidated materials (terrace gravel and glacial moraine) above the bulkhead. It should be pointed out that this is a conservative analysis and that is unlikely that the "assumed head at failure" values of 52 or 116 could be achieved in these materials. The porous nature of these deposits would most likely be able to dissipate water before reaching these levels. However, adding a support that would resist horizontal loads at the midspan of the beams would be a low cost but effective method of increasing "design head" and "assumed head at failure" of all bulkhead components.

2.4 Stability of Tunnel Lining

The reinforced concrete lining from Sta. 0+54 to Sta. 4+61 of the Leadville Mine Drainage Tunnel was inspected March 25, 2008. The observable concrete (above the shallow water surface in the gutters) was in good condition.

The analysis of the concrete lining was first based on the following:

- Although, drawing no. 1335-D-123 indicated that the spacing of the existing steel ribs and struts could vary from 2'-0" to 6'-0", the spacing of the steel ribs was assumed to be at 2'-0" and 4'-0" based on the following:

    - On page 56 of the Design Summary, Treatment Plant and Tunnel Lining, Leadville Mine and Drainage Project, Bureau of Reclamation, 1991, it states that "the existing tunnel consists of steel sets placed at 2-ft and 4-ft centers"

    - A check of photographs shows a 4-ft or closer spacing of the steel sets.

    - Jeff Farrar (a Reclamation employee and an inspector of the tunnel construction who was underground when the supports were being placed) said that the spacing of the steel sets sets were mostly on 2 ft. spacings where the ground was heavy and spiling had to be used, but there may have been some areas with better ground where a 4 ft. spacing was used. Jeff did not believe there were any areas with 6-ft spacings.

- The reinforcement sizes and spacings shown on drawing no.1335-D-123 are assumed as-built. The dimensions of the lining and the location of reinforcement and steel sets shown on the drawing are assumed as-built.

Assumptions made for analyses included:

- The weep holes plug and become inoperable or are overwhelmed by the volume of inflow such that external head exists above the level of weep holes. The former scenario is possible only if routine cleaning of the weep holes is not performed and the latter scenario is viewed as an extreme worst case scenario.

- The earth loading is carried entirely by the initial support (steel sets, liner plate and steel struts). The full capacity of the reinforcement in the concrete lining less any moments and shears resulting from grouting is available to resist external hydrostatic pressures.

- There is no interaction between the external soil mass and the concrete lining; i.e., the stiffness of the soil in keeping the lining from deforming was not included.

- Two sets of assumptions of the residual capacity of the steel sets were assumed:

    - The steel ribs have no residual moment carrying capacity or shear capacity remaining after supporting soil loadings. The steel struts have no moment carrying capacity but full shear capacity after supporting soil loadings.

    - The steel ribs and steel struts have a residual 1/3 of their original moment or 3/5 of their original shear carrying capacity to resist external hydrostatic loading.

- The floor of the concrete lining of the tunnel carries only the dead load of the concrete lining and the grout load. Vertical earth loadings are carried by the initial support.

- The steel strut of the floor carries all the compressive load from the lateral soil pressures

- The concrete has bonded to the steel ribs and steel struts

- The allowable concrete to steel bond stress is 160 lb/in^2

- 28-day compressive strength of concrete is 4,000 lb/in^2

- Reinforcement bars are Grade 60

- Yield strength (f_y) of steel tunnel supports and steel invert strut is 36,000 lb/in^2

During typical designs, load factors above 1.0 (overestimating loads) are used for external loads. Strengths are assumed with factors slightly less than 1.0 (underestimating strengths). The analyses were adjusted for these factors as noted below:

- A load factor of 1.0 combined with the strength reduction factors shown below was used to determine the computed head at failure

- A strength reduction factor for shear of 0.85

- A strength reduction factor for moment of 0.90

The following table summarizes the results of the analyses.

Table 5. Results of Tunnel Lining Analysis.

Spacing of Steel Sets	Original Design Head (ft. above invert) [1]	Computed Head at Failure (ft. above invert) [2][3]	Computed Head at Failure (ft. above invert) [4]
4'-0"	0	20	46
2'-0"	0	25	68

Notes:

[1] The tunnel was not designed to resist external hydrostatic pressure. See the Design Summary, Treatment Plant and Tunnel Lining, Leadville Mine and Drainage Project, Colorado, January 1991 (for specifications No. DC-7804). The author of the design summary writes that the transverse reinforcement was sized using the difference between the design loads (vertical and lateral earth pressures or grouting pressure) and the capacity provided by the steel sets. Additionally, as was done in the specification (DC-7804) it is common practice to not design for external hydrostatic pressure where feasible. This is done by specifying weep holes to keep external hydrostatic pressures from developing against tunnel linings.

[2] Assumes steel ribs and steel struts have no residual strength remaining after supporting soil loadings except for steel struts in shear.

[3] It cannot be determined with reasonable certainty what stresses the backfill grouting actually imparted to the steel supports or steel reinforcement in the floor. Assuming the floor initially acts as a transverse fixed-fixed beam with a uniform foundation pressure distributed across the bottom of the floor equal to the combined loading from the dead load of the concrete lining and a 25 lb/in^2 backfill grouting pressure over the top 90 degrees of the crown, analysis indicates the floor should have already failed in flexure. This failure would probably manifest itself as local crushing (plastic hinge) at the junction of the floor and the wall but should not constitute a catastrophic failure as the floor would begin to act as a simple beam. It is further assumed following the crushing of the concrete that while the concrete floor would have minimal capacity to resist shear at the junction of the wall and floor, the full shear capacity of the steel strut is available to resist shear. As this mode of failure is assumed not catastrophic, the value shown for assumed head at failure reflects the capacity of the lining elsewhere.

[4] Assumes steel ribs and steel struts have a residual strength of 1/3 their original flexural capacity and 3/5 of their original shear capacity after supporting soil loadings. This reflects

the excess capacity of the initial support without regard to any additional flexural capacity provided by the reinforcement embedded in the concrete lining or the shear capacity provided by the concrete itself.

Discussion –

The reinforced concrete lining was placed approximately 11 years after the tunnel supports were erected. This fact is the basis for assuming all earth loadings were carried solely by the initial support. It cannot be ascertained with any certainty how much of a steel support's flexural or shear capacity is used to resist the earth loads. Hence, the set of assumptions noted above was made regarding the residual capacity in bending and shear of the steel ribs and struts.

The assumption that the steel ribs and struts, after resisting the ground loads, have 1/3 of their original capacity to resist moments and 3/5 of their original capacity to resist shears assumes the original designers used 0.66 f_y (f_y is the yield strength of the steel supports) for allowable bending stresses and 0.4 f_y for allowable shear stresses..

If the values for original design head or computed head at failure are deemed critical to this study then the location of the steel sets should be verified by non-destructive testing and the original design head or computed head at failure should be compared to a modeled groundwater surface which would be established in the vicinity of the tunnel following a breach of the blockage near the Pendry Fault.

The fact that the water level at the timber lattice bulkhead is never seen to rise more than 2 or 3 feet above the tunnel invert, regardless of the upstream water levels is again an important observation for the tunnel lining stability. This, coupled with the weepholes in the lining, and other potentially conservative analysis assumptions, suggest that the likelihood of reaching a critical failure head is small.

2.5 Stability of Hillside in Vicinity of Portal

Even though there is a lack of specific engineering data regarding the near surface soil and bedrock units in the immediate vicinity of the tunnel portal, this area is likely to remain stable due to the free draining nature of the near surface material, the dip of the contact surface between the near surface material and the bedrock, and the physical size and scale of the portal structure. The pervious terrace gravels in the portal area are underlain by the rock surface of the Minturn formation which slopes away toward the river. The groundwater level in the gravels follows the rock surface, quickly dropping toward the river downstream of about Station 6+35.

Seismic loading contributes little to the risk of slope failure. The simultaneous occurrence of a large earthquake and high groundwater levels in the portal area, both of which are needed to approach unstable slope conditions, is a remote possibility. The earthquake hazard in the Leadville area is not high, and it is unlikely an earthquake would trigger other potential failure modes.

2.5.1 Assumptions and Data

Physical and Mechanical properties were assumed for the three geologic units that would most influence fluid transport and earth loads. The Geologic Units are Glacial Moraine (Qm), Terrace Gravels (Qtg) and bedrock comprised of shales and sandstones of the Minturn Formation (Pm).

The assessment of these properties was accomplished from review of available project data, discussions with project personnel, site visits and published data. Recent excavation for a pipeline installation indicated that the Glacial Moraine is able to stand at steep angles in excavations and in dumped fill embankments, see Figure 14. This indicates that the material has cohesion and the actual shear strength of this material is likely closer to the maximum value for the range of strengths assumed in the analysis.

Figure 14. Excavation in May, 2008 into the Glacial Moraine at Leadville for installation of a pipeline under Highway 91 along the alignment of the LMDT. Note the steep slopes of the excavation and spoil banks indicating high shear strength of the soil.

The Glacial Moraine (Qm) is a silty to clayey gravel (GM-GC) with cobbles and boulders. The Terrace Gravel (Qtg) is also silty to clayey gravels (GM-GC) without the presence of cobbles and boulders. The Minturn Formation (Pm) is a coarse-grained medium-hard sandstone with interbeded shale.

Physical Properties

The physical properties of each geologic unit are presented below in Tables 6 through 8, Physical Properties.

Table 6. Physical Properties, Glacial Moraine (Qm).

Property		Minimum	Maximum	Average
Unit Weight	lbs/ft^3	115	130	125

Table 7. Physical Properties, Terrace Gravels (Qtg)

Property		Minimum	Maximum	Average
Unit Weight	lbs/ft^3	110	120	115

Table 8. Physical Properties, Minturn Formation (Pm).

Property		Minimum	Maximum	Average
Unit Weight	lbs/ft^3	142	150	146

Mechanical Properties

The mechanical properties of each geologic unit are presented below in Tables 9 through 11, Mechanical Properties.

Table 9. Mechanical Properties, Glacial Moraine (Qm)

Property		Minimum	Maximum	Average
Cohesion	lbs/in^2	2	10	5
Friction Angle	Degrees	32	45	40

Table 10. Mechanical Properties, Terrace Gravels (Qtg).

Property		Minimum	Maximum	Average
Cohesion	lbs/in^2	5	15	10
Friction Angle	Degrees	35	41	38

Table 11. Mechanical Properties, Minturn Formation (Pm).

Property		Minimum	Maximum	Average
Cohesion	lbs/in^2	10	40	25
Friction Angle	Degrees	50	60	55

2.5.2 Seismicity

Estimated seismic loadings used in these studies as pseudo static loadings are presented in the Table 12, Seismic Loading.

Table 12. Seismic Loading for the LMDT Slope Stability Analysis.

Estimated Return Period	**Probable Horizontal Acceleration**
Years	g
500	0.05
2,500	0.15
10,000	0.35

2.5.3 Slope Stability Cases

Twelve slope stability cases were considered by varying the physical and mechanical properties. The physical and mechanical properties utilized in the analyses were the minimum, maximum, and average properties. All properties (unit weight, cohesion, and friction angle) were varied at the same time. Additionally, the piezometric water surface was conservatively modeled at elevations approximately 40 feet greater than historically observed. Modeled water elevations in wells along the tunnel alignment are shown below in Table 13.

Table 13. Piezometric Levels for Slope Stability Analysis LMDT.

Piezometric Head @ Station	Elevation in Feet
3+00	10023.9
4+70	10040.8
6+35	10056.1
10+25	10,060.0

Effects of earthquake loading were modeled by imposing pseudo static earthquake loadings for three return periods, using the seismic coefficients equal to the peak horizontal ground acceleration 0.05g, 0.15g and 0.35g for 500, 2500 and 10,000 year return periods respectively.

Two additional slope stability cases were also computed. One case is the static stability using the minimum properties and assuming no soil cohesion. The other case is an analysis to determine the yield acceleration under seismic loading. The yield acceleration is that level of seismic loading at which the Factor of Safety is 1.0.

The slope stability analysis computed both static and pseudo static Factors of Safety (FS) for the slope between the portal and LMDT station 10+25 using the computer software SLOPE W (GEO-SLOPE International Ltd). The critical failure surface is determined by an automatic search that initially involves a circular plane, but according to the geologic configuration the surface can be modified to straight line segments.

The analysis results are presented in Table 14.

Factor of Safety values are most influenced by physical and mechanical properties and seismic loading conditions as is indicated by the results of Slope Stability Analysis.

Consideration was also given to the observation that the site shows no indications of previous gross instability, other than the reported localized areas of sinkholes due to collapse in the tunnel. Output files from the SlopeW software are included in Appendix D.

Table 14. Results of Slope Stability Analysis

Seismic Loading Case (Pseudo Static Coefficient)	Factor of Safety using Minimum Material Properties	Factor of Safety using Average Material Properties	Factor of Safety using Maximum Material Properties
Static with cohesion	1.96	3.12	4.44
Static with no cohesion	1.54	2.03	-
0.05	1.68	2.68	3.67
0.15	1.30	2.02	2.59
0.35	0.87	1.22	1.59

Please note that pseudo static stability conditions would exist during an earthquake for just an instant, when the accelerations in the failure mass are as high as the coefficient shown (in the units of acceleration due to gravity). During an earthquake, factors of safely fluctuate. Since liquefaction of the gravelly soils is not considered to be a reasonable possibility at this site, it is only while the factor of safety drops below 1.0 that permanent deformation of the slide mass would occur. Since all pseudo static factors of safety are above 1.0, except for the highest earthquake loading and minimum strength parameters analyzed, very little to no deformations are expected to be caused by an earthquake even under extremely conservative assumptions.

The minimum factor of safety of 0.87 was obtained by combining the extreme conditions of a maximum seismic loading with minimum soil properties and elevated groundwater in the downstream hillside. Although a result with a Factor of Safety of less than 1.0 was calculated for this case, it does not automatically follow that the hillside will fail even if these conditions were to manifest. For seismic loading one must consider the amount of ground deformation that will occur. Empirical correlations between yield acceleration and calculated "Newmark" type rigid body movements (Jibson, 2007) were used to estimate the likely maximum amount of movement that would occur under the worst-case scenario using the following equation:

$$\log D_N = 0.215 + \log\left[\left(1 - \frac{a_c}{a_{\max}}\right)^{2.341}\left(\frac{a_c}{a_{\max}}\right)^{-1.438}\right] + 0.510$$

where, D_N is the estimated displacement (in centimeters), a_c is the yield or critical acceleration, a_{max} is the peak earthquake acceleration, and 0.510 is a factor to account for the mean plus one standard deviation of the data. A yield acceleration for the soil mass was calculated to be about 0.197g (the pseudo-static coefficient resulting in a F.S. = 1.0 using lower shear strength and high ground water estimates). Using the yield acceleration and peak earthquake acceleration, the empirical relationship indicates maximum displacements would be on the order of 0.7 inches. It is generally accepted that it takes predicted displacements at least on the order of 6 to 12 inches before stability is considered to be threatened.

Therefore the analysis indicates that the gross stability of the portal area, defined for this study as extending to LMDT Station 10+25 is adequate for the ground conditions, water loading and seismic conditions as currently assumed. Although it is believed that groundwater levels near the portal cannot rise to dangerous levels, it is recommended that the groundwater wells at Stations 3+00, 4+70, and 6+35 be instrumented with pressure transducers and data be tied to the EWS.

3. Conclusions

Based upon the detailed analysis documented in this report, it is concluded that the upper blockage near the Pendery Fault is likely due to a zone of roof collapse located downstream from the fault. This blockage is likely to contain tunnel utility conduits which remained intact in the collapse which still convey some flow directly through this part of the tunnel. The upper blockage is estimated to be stable under the current conditions with 100 to 119 feet of differential head. However, the likelihood of the blockage remaining stable declines with increased head differential. For that reason, all analysis and potential failure modes conservatively assumed rapid failure of the blockage.

If the blockage near the Pendery Fault were to fail, the likelihood of uncontrolled seepage would increase and some property damage could occur, but loss of life would not be expected. A pressure wave would travel down the LMDT and might damage the extraction wells at Station 10+25. Further down the LMDT, it is very unlikely that the failure could cause a "blowout" of the porous plug and Timber-Lattice Bulkhead. The forces acting on the plug of porous material due to the pressure wave would not be great enough to overcome the existing shear strength of the material and move it. Flow down the tunnel from the failure of the upper blockage near the Pendery Fault would be limited by head losses through the system and by the diameter of the casings in the wells at Station 10+25 which would be expected to experience artesian flow conditions. Because the LMDT is full of water below the upper blockage, and the porous plug would hold, a small flow would accompany the pressure pulse, not a massive "blowout" type of flood wave. The well at Station 6+35 is likely to also experience artesian flow. The artesian flow conditions at one or possibly two wells could last for a significant period of time (days to weeks) until the head in the mine pool was lowered.

It is very unlikely that failure of the upper blockage near the Pendery Fault could result in failure of the Timber-Lattice Bulkhead or in failure of the Tunnel Liner. A large mass of soil would need to experience elevated ground water conditions. It would take a significant period of time (days) for the increased water pressures to seep through the 130-foot-long seepage pathway from the upstream end of the plug near Station 5+92 to the soils around the LMDT near the Timber-Lattice Bulkhead and Tunnel Liner at Station 4+62. Although specific seepage modeling has not been performed, it is expected that the mounding groundwater levels would drain off and thus be attenuated by the surrounding Terrace Gravels. If groundwater levels were to rise, this changing condition would be detected by the monitoring well at Station 4+70 and by increased flow at the Timber-Lattice Bulkhead.

In the very unlikely event that groundwater levels near the Timber-Lattice Bulkhead and Tunnel Liner at Station 4+60 were to rise to levels which could collapse the bulkhead or tunnel liner, a "blowout" would not be expected to follow. Rather, some of the surrounding soils would be pushed into the tunnel, but eventually the shear strength of the soil would act against the floor and walls of the lower portion of the LMDT and prevent a "blowout." It is noted that failure of the liner and/or bulkhead would leave a considerable length, (several hundred feet) of concrete lined tunnel and the massive concrete portal intact. The remaining mass would be able to resist

the thrust generated by the force of fill being pushed into the collapsed tunnel opening and eventually a stable plug would form.

Analysis shows that movement of the hillside could only occur in model runs by combining the extreme conditions of elevated groundwater, a maximum seismic loading, and minimum soil properties. None of these conditions are considered to be likely.

The likelihood of the upper blockage near the Pendery Fault remaining stable decreases as the level of the mine pool, and subsequent head differential, increases. If the upper blockage were to fail, the likelihood of uncontrolled seepage would increase and some property damage could occur, but loss of life would not be expected. A more thorough and complete assessment of the likelihood of these combination of events, other failure modes, and the consequences of failure events is presented in the "Potential Failure Modes and Effects Analysis" report (Reclamation, 2008).

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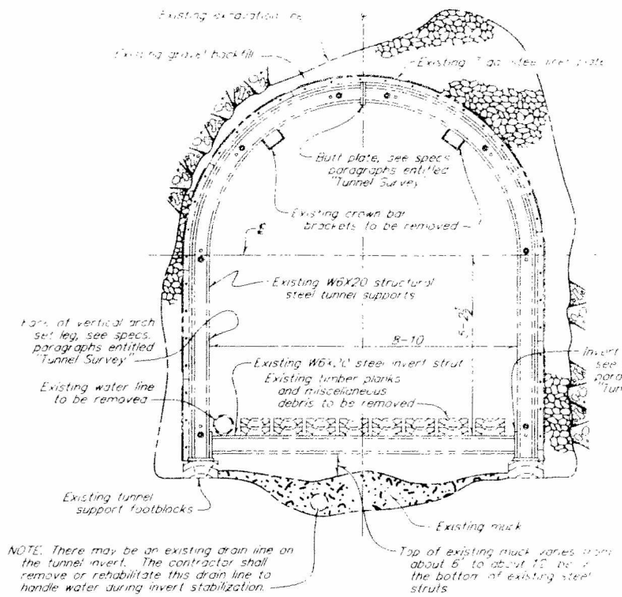
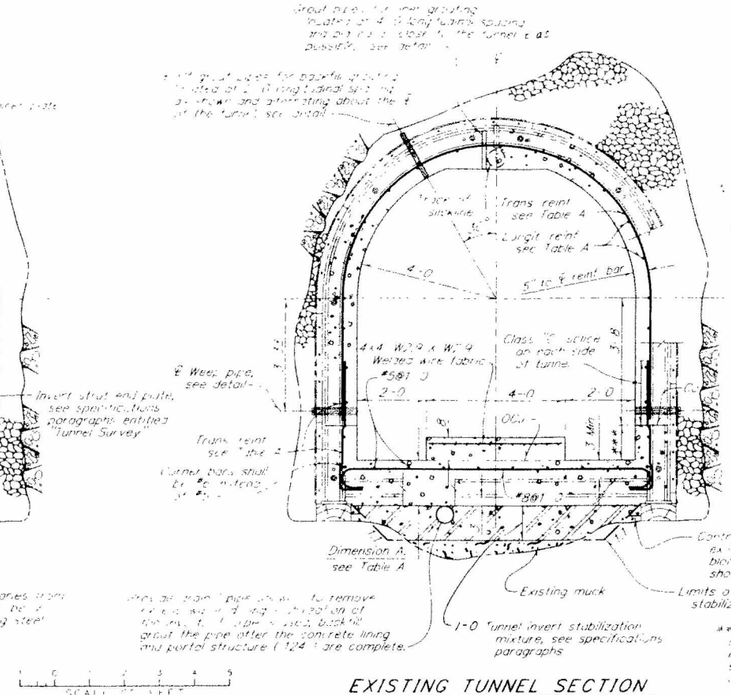
Appendix A

Drawing no. 1335-D-123 showing the Timber Bulkhead.



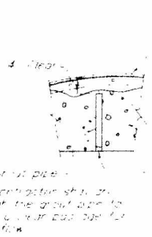
EXISTING TUNNEL SECTION



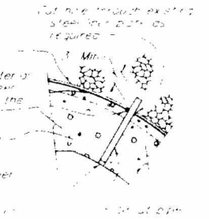
EXISTING TUNNEL SECTION WITH NEW CONCRETE LINING

TABLE A

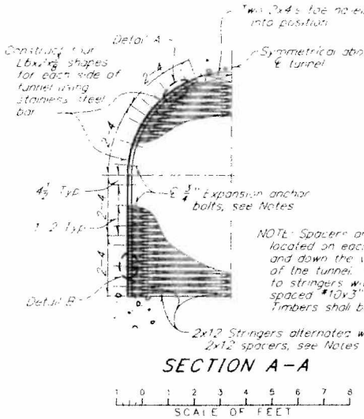
STATIONS	DIMENSION A	TRANS. REINF.	LONGIT. REINF.
STA. 0+54 to STA. 0+94	1'-4" Min.	#7@1'-0	#7@1'-0
STA. 0+94 to STA. 4+26	1'-0" Min.	#5@1'-0	#5@1'-0
STA. 4+26 to End of concrete lining (± 4+61±)	1'-0" Min.	#5@1'-0	#8@1'-0



GROUT PIPE DETAIL FOR LINER GROUTING

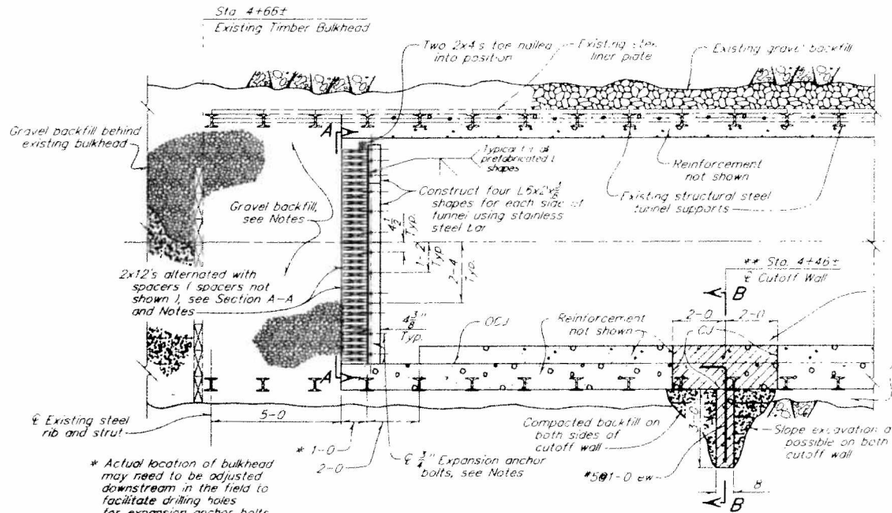


GROUT PIPE DETAIL FOR BACKFILL GROUTING



SECTION A-A

SCALE OF FEET



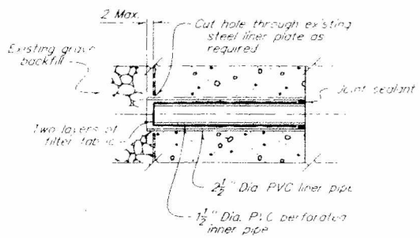
* Actual location of bulkhead may need to be adjusted downstream in the field to facilitate drilling holes for expansion anchor bolts where existing ribs and or concrete reinforcement may be in the way.

NOTE: Existing steel ribs and struts may be from 2-0 to 6-0 spacing, 2-0 spacing shown.

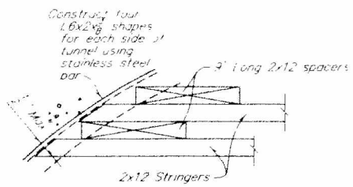
PROFILE OF EXISTING TUNNEL WITH NEW CONCRETE LINING NEAR EXISTING BULKHEAD

** Actual location of cutoff wall may need to be adjusted in the field if existing steel strut is in the way. The contractor may remove one existing steel strut if struts are on 2'-0" spacing.

NOTE: The contractor shall not substitute backfill concrete for compacted backfill around cutoff wall.



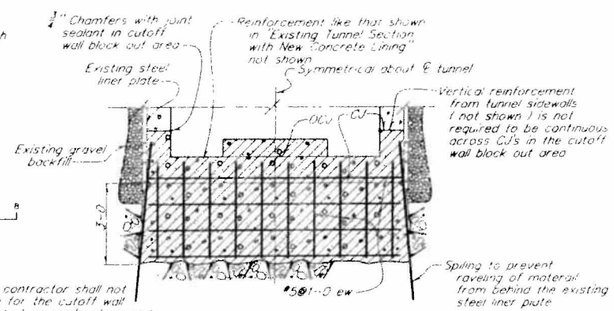
WEEP PIPE DETAIL



DETAIL A
DETAIL B SIMILAR

Reinforcement in cutoff wall blockout same as "Existing Tunnel Section with New Concrete Lining". Longitudinal reinforcement is not required to be continuous across CJ.

For details of invert stabilization, see "Existing Tunnel Section with New Concrete Lining" and specifications paragraphs.



NOTE: The contractor shall not excavate for the cutoff wall until the last concrete placement of the tunnel lining within 25'-0" of either side of cutoff wall has attained a compressive strength (f'c) of 4,000.

SECTION B-B

NOTES

For details of water handling during construction, see (160).
For alignment and profile, see (122).
For ventilation details, see (156).
For lighting details, see (158).
For general concrete outline notes, see 40-D-651.
Chamfers are required on all exposed concrete edges, see 40-D-651.
For general notes and minimum requirements for detailing reinforcement, Class 60, see 40-D-626.
Tunnel lining design is based on a specified compressive concrete strength (f'c) of 4,000 lbs. per square inch at 28 days, and reinforcement with a specified minimum yield strength (fy) of 60,000 lbs. per square inch.
Splices for all longitudinal reinforcement bars shall be staggered.
Backfill grouting pressure shall be 25 pounds per square inch. Liner grouting pressure shall be 20 pounds per square inch. Grouting shall not begin until the floor blockout and the cutoff wall have been placed. Grouting shall be in accordance with the specifications paragraphs.
2x12 Timber stringers and spacers used in timber bulkhead shall be Douglas Fir-Larch, stud grade.
¾" Expansion anchor bolts shall be stainless steel and embedded 9" min. into concrete.
Holes drilled in L6x2½x⅜'s shall be ⅞" dia. Holes may be slotted in vertical direction to aid in installation.
Gravel backfill shall be 1½" to 2½" nominal size. Gradation shall meet ASTM C 33, size No. 2.

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

LEADVILLE MINE DRAINAGE TUNNEL PROJECT - COLORADO

TUNNEL LINING

TYPICAL TUNNEL SECTION, CUTOFF WALL, AND TIMBER BULKHEAD

DESIGNED: TECHNICAL APPROVAL:
DRAWN: SUBMITTED:
CHECKED: APPROVED:
CHIEF, DESIGN & CONSTRUCTION BRANCH

DENVER, COLORADO — FEBRUARY 16, 1989 — 1335-D-123

D (22 X 34)

Appendix B

Spreadsheet for calculation of the plug stability against blowout for the instantaneous pressure condition.

5+00 to 5+92 Hydraulic Gradient Through the Tunnel Plug

1988 Calculations		Square section of Tunnel			Curved section		Total
		Height, ft	Width, ft	Area, ft^2	radius, ft	Area, ft^2	Area, ft^2
Cross sectional area of	Inside A-Line	5.21	8.83	46.01	4.42	30.64	76.65
	Area	6.00	8.83	53.00	4.42	30.64	83.64
	Outside B-line	5.21	9.83	51.22	4.92	37.97	89.18707
	Area	6.00	9.83	59.00	4.92	37.97	96.97179
Surface Area						Length	
		10.42	8.83		4.42	13.88	33.13
		12.00	8.83		4.42	13.88	34.71

Calculations

Density of material used 2005 and in this report	110	lb_f/ft^3	This number used by this study			
Density of material used 1988	100	lb_f/ft^3				
Collapsed Length of tunnel	92	feet				
Assumed Internal Friction and Poisson's ratio	30	degrees		0.45	Rock Tunnel Wall Height	
Pages 43 and 61 Gobla's report	11	feet wide	R=	5.5	Sta	height, feet
excavation dimensions	12	feet tall	H=	6.5	4+62	4
Max Area of Plug, estimated in this study	119.02	$feet^2$		46	5+00	5.79
More accurate Volume of Plug this study	10,950	Cu. Feet			5+75	9.32
Hydraulic Driving Force					5+92	10.12
This Study max area & correct head	252,506	pounds			6+34	12
This Study max area & artesian flow @ well 6+34	796,878	pounds				
This Study maximum area & max head	922,685	pounds				
Shear Calculations						
Average Depth of Collapse	100	feet				
Average vertical stress this study	76.39	psi				
Average horizontal stress this study	38.19	psi				
Invert elevation 0+00	9957	feet	Jaky Ko=	0.50		
Surface Elevation well 6+34	10,064.30	feet	Sigma H = Sigma V*(1-sin(phi))			
Head and Pressure at well 6+34 for artesian flow	107.30	feet	Sigma H = Sigma V*(1-sin(phi))			
Maximum head elevation Pendry Blockage	10148.81	feet				
Maximum Potential water head at 6+34	191.81	feet				
Maximum hydraulic pressure-Non Leaky Tunnel	83.12	psi	46.50	psi		
Hydraulic head and pressure for uplift/fracturing 2008	176.28	head	Mine pool head 192 feet, 9/13/2007. Doubtful hydraulic fracturing			
Water head and pressure at 6+34 for FS=1	124.24	feet	or uplift may occur where overburden less than about half the head			
shear calculations			Jaky (1948)[1] for normally consolidated soils			
Perimeter of horseshoe tunnel	34.71	feet	Jaky (1948)[1] for normally consolidated soils			

Bottom of horseshoe	8.83	feet	76.39	psi
Sides of horseshoe tunnel x2	12	feet	53.84	psi
Top of horseshoe tunnel	13.88	feet		
Shear Equation used in 1988 memorandum				
Shear strength 1988 back calculation for Tan A	Reported 22.9 x10^6 lbs shear		S=(p-u) tan Ø	
Fiction Angle between Rock and fill	30	degrees		Input phi value here:
Corrected Head and pressure	34	feet	14.73	psi
Shear strength, pounds this study w/1988 area	14,818,187	pounds	Used true head.	Not used
Shear strength, this study w/ max area	14,681,664	pounds		
Shear strength, this study uplift head non leaky	-7,011,342	pounds	Without a leaky tunnel pressure could liftup overburden	
Shear strength, this study max head non leaky	923,240	pounds	Without a leaky tunnel pressure could blowout plug or overburden	
Shear strength, this study max head, leaky	3,505,994	pounds	With a leaky tunnel pressure could not blowout plug or overburden	

Location	Description	Date Completed [MM/DD/YY]	Total Depth [feet]	TOC elevation [ft. ASL]
LDT03+00	Alluvial well 20' off set from LDT, Station 3+00	1976	78	10035.1
LDT04+70	Alluvial well 20' off set from LDT, Station 4+70	1976	92	10048.8
LDT06+34	LMDT Well Previous DWW, Station 06+34	1976	108	10099.7

10,064.30 feet TOC for LDT 06+34 Error in table

Parameters for this study

Water Elev. feet	Water Pressure psi	Head at 6+34 feet	Shear Resistance, lbs	Driving force, lbs	S.F.
9991.00	14.73	34.00	14,681,664	252,506	58.14
10064.30	46.50	107.30	3,505,994	796,878	4.40
10081.24	53.84	124.24	923,240	922,685	1.00

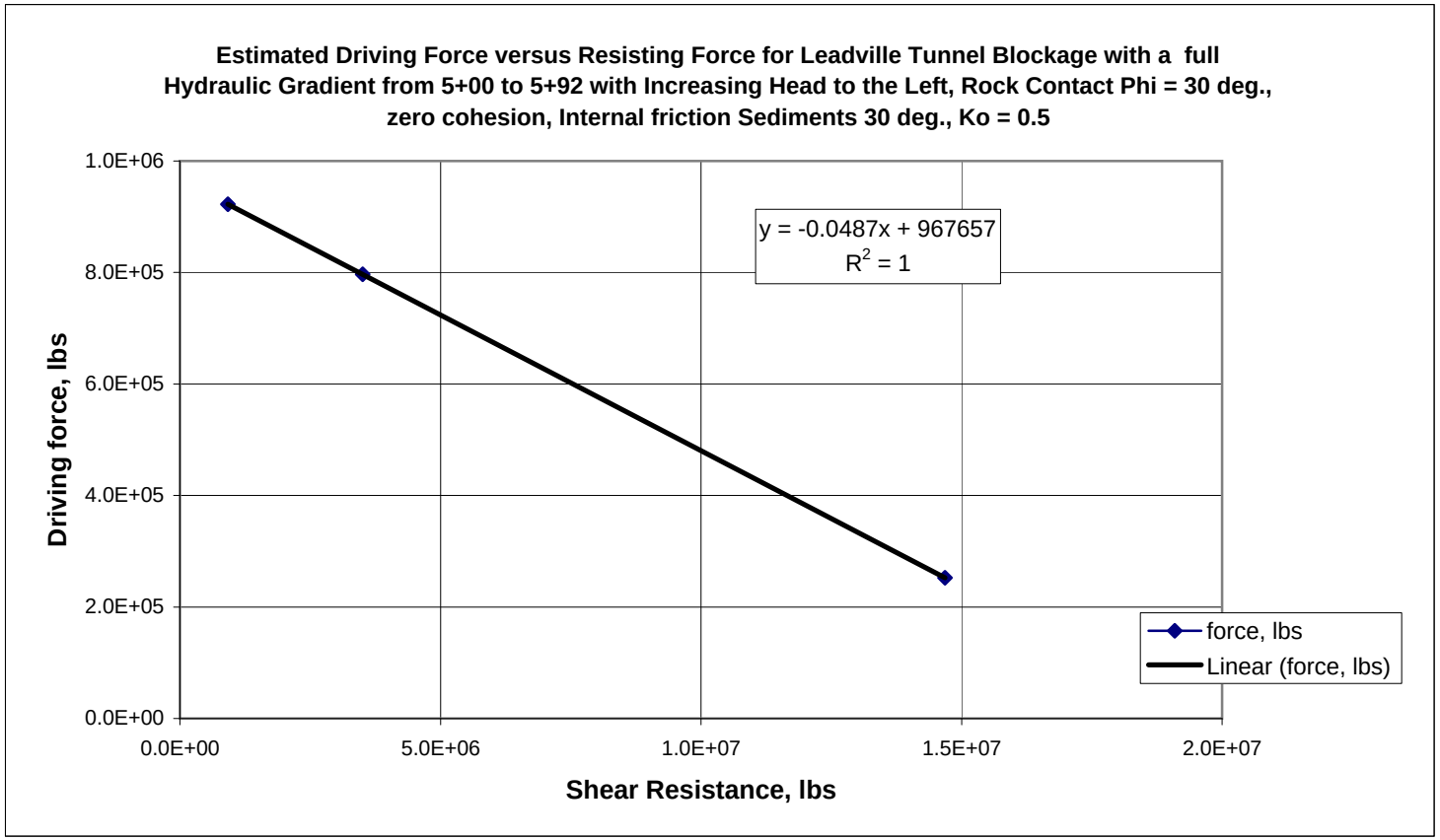
Equation Parameters

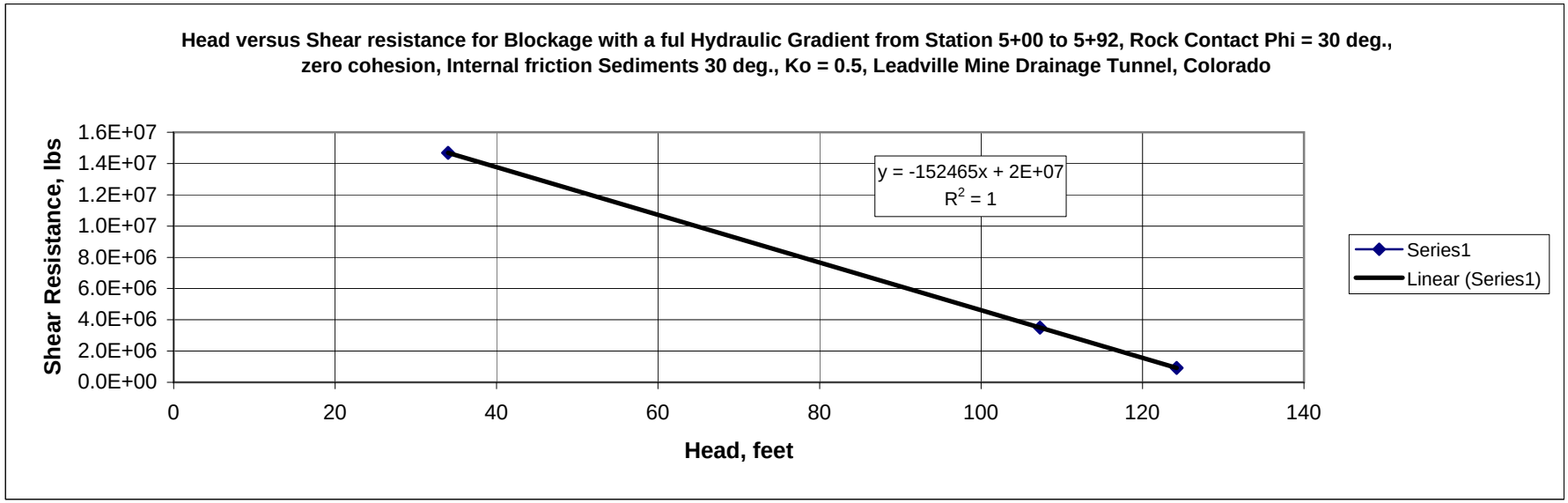
	B	A			
	-0.048710487	967,657		Shear resistance SF=1	922,711
	-152464.8116	19865468.06	922,711	Head for FS=1	124.24

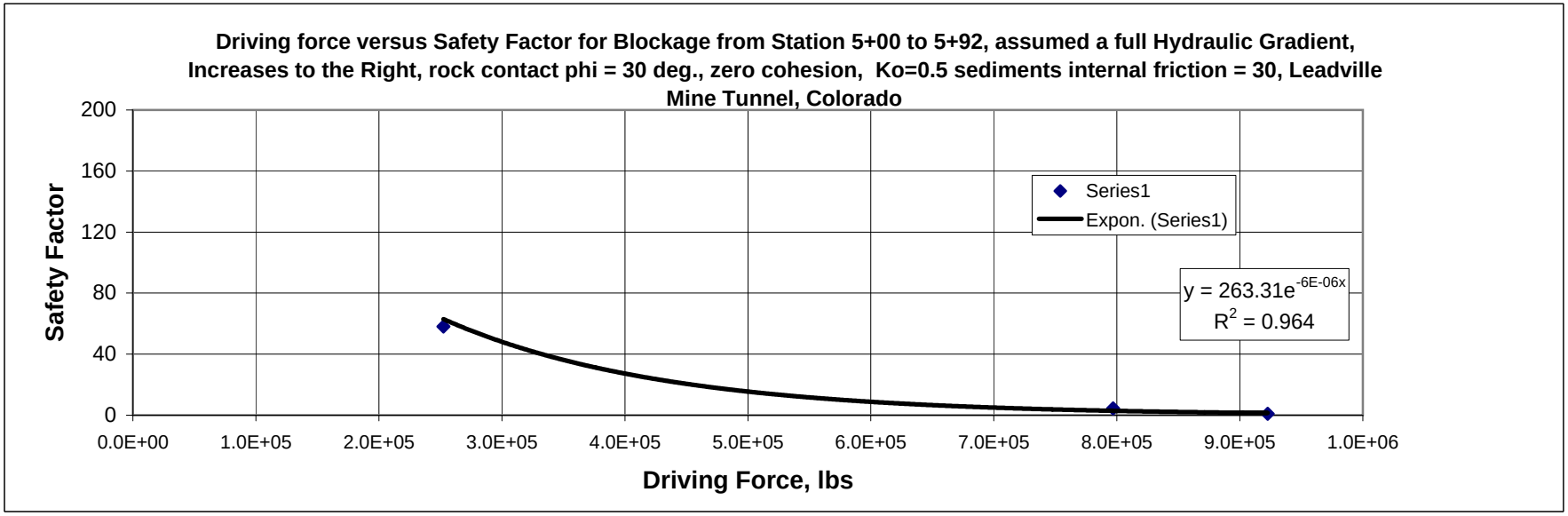
Check

lbs 922,711 lbs

feet


Estimated Driving Force versus Resisting Force for Leadville Tunnel Blockage with a full Hydraulic Gradient from 5+00 to 5+92 with Increasing Head to the Left, Rock Contact Phi = 30 deg., zero cohesion, Internal friction Sediments 30 deg., Ko = 0.5
y = -0.0487x + 967657
R2 = 1
Driving force, lbs
1.0E+06
8.0E+05
6.0E+05
4.0E+05
2.0E+05
0.0E+00
0.0E+00
5.0E+06
1.0E+07
1.5E+07
2.0E+07
Shear Resistance, lbs
force, lbs
Linear (force, lbs)


Head versus Shear resistance for Blockage with a ful Hydraulic Gradient from Station 5+00 to 5+92, Rock Contact Phi = 30 deg., zero cohesion, Internal friction Sediments 30 deg., Ko = 0.5, Leadville Mine Drainage Tunnel, Colorado
Shear Resistance, lbs
1.6E+07
1.4E+07
1.2E+07
1.0E+07
8.0E+06
6.0E+06
4.0E+06
2.0E+06
0.0E+00
0
20
40
60
80
100
120
140
Head, feet
y = -152465x + 2E+07
R2 = 1
Series1
Linear (Series1)


Driving force versus Safety Factor for Blockage from Station 5+00 to 5+92, assumed a full Hydraulic Gradient, Increases to the Right, rock contact phi = 30 deg., zero cohesion, Ko=0.5 sediments internal friction = 30, Leadville Mine Tunnel, Colorado
Safety Factor
200
160
120
80
40
0
0.0E+00
1.0E+05
2.0E+05
3.0E+05
4.0E+05
5.0E+05
6.0E+05
7.0E+05
8.0E+05
9.0E+05
1.0E+06
Driving Force, lbs
Series1
Expon. (Series1)
y = 263.31e-6E-06x
R2 = 0.964

5+00 to 5+92

1988 Calculations		Square section of Tunnel			Curved section		Total
		Height, ft	Width, ft	Area, ft^2	radius, ft	Area, ft^2	Area, ft^2
Cross sectional area of	Inside A-Line	5.21	8.83	46.01	4.42	30.64	76.65
	Area	6.00	8.83	53.00	4.42	30.64	83.64
	Outside B-line	5.21	9.83	51.22	4.92	37.97	89.18707
	Area	6.00	9.83	59.00	4.92	37.97	96.97179
Surface Area						Length	
		10.42	8.83		4.42	13.88	33.13
		12.00	8.83		4.42	13.88	34.71

Calculations

Density of material used 1988	100	lb_f/ft^3				
Density of material used 2005 and in this report	110	lb_f/ft^3	This number used by this study			
Collapsed Length of tunnel	92	feet				
Assumed Internal Friction and Poisson's ratio	46.23	degrees		0.45	Rock Tunnel Wall Height	
Pages 43 and 61 Gobla's report	11	feet wide	1988 Conservative value inside A-line		Sta	height, feet
excavation dimensions	12	feet tall	R=	5.5	4+62	4
Max Area of Plug, estimated in this study	119.02	$feet^2$	H=	6.5	5+00	5.79
More accurate Volume of Plug this study	10,950	Cu. Feet		46	5+75	9.32
Hydraulic Driving Force					5+92	10.12
This Study max area & correct head	252,506	pounds				
This Study max area & artesian flow @ well 6+34	796,878	pounds				
This Study maximum area & max head	794,501	pounds				
Shear Calculations						
Average Depth of Collapse	100	feet				
Average vertical stress this study	76.39	psi				
Average horizontal stress this study	21.23	psi				
Invert elevation at station 0+00	9957	feet	Jaky Ko =	0.28		
Surface Elevation well 6+34	10,064.30	feet	Sigma H = Sigma V*(1-sin(phi))		Jaky Coefficient used	
Head and Pressure at well 6+34 for artesian flow	107.30	feet	Sigma H = Sigma V*(1-sin(phi))		Jaky Coefficient used	
Maximum head elevation Pendry Blockage	10148.81	feet				
Maximum Potential water head at 6+35	191.81	feet				
Maximum hydraulic pressure-Non Leaky Tunnel	83.12	psi	Head may be incorrect. Appears depth to the water table used as			
Hydraulic head and pressure for uplift/fracturing 2008	176.28	feet	the head. 77 feet was only used to check original calculations.			
Water head and pressure at 6+35 for FS=1	106.98	feet	46.50	psi		
shear calculations			Mine pool head 192 feet, 9/13/2007. Doubtful hydraulic fracturing			
Perimeter of horseshoe tunnel	34.71	feet	or uplift may occur where overburden less than about half the head			
Bottom of horseshoe	8.83	feet				
Sides of horseshoe tunnel x2	12	feet	76.39	psi		
Top of horseshoe tunnel	13.88	feet	46.36	psi		
Shear Equation used in 1988 memorandum			S=(p-u) tan Ø			
Friction Angle between fill and rock surfaces	46	degrees				
Corrected Head and pressure in 1988	34	feet				
Shear strength, pounds this study w/1988 area	23,976,341	pounds	14.73	psi		
Shear strength, this study w/ max area	20,913,668	pounds	Used true head.	This was not used		
Shear strength, this study max head non leaky	-18,308,588	pounds	Without a leaky tunnel pressure could blowout lift up overburden.			
Shear strength, this study max head non leaky	795,598	pounds	Without a leaky tunnel pressure could blowout plug or overburden.			
Shear strength, this study max head, leaky	707,385	pounds	With a leaky tunnel pressure could not blowout plug or overburden.			

Location	Description	Date Completed [MM/DD/YY]	Total Depth [feet]	TOC elevation [ft. ASL]
LDT03+00	Alluvial well 20' off set from LDT, Station 3+00	1976	78	10035.1
LDT04+70	Alluvial well 20' off set from LDT, Station 4+70	1976	92	10048.8
LDT06+34	LMDT Well Previous DWW, Station 06+34	1976	108	10099.7

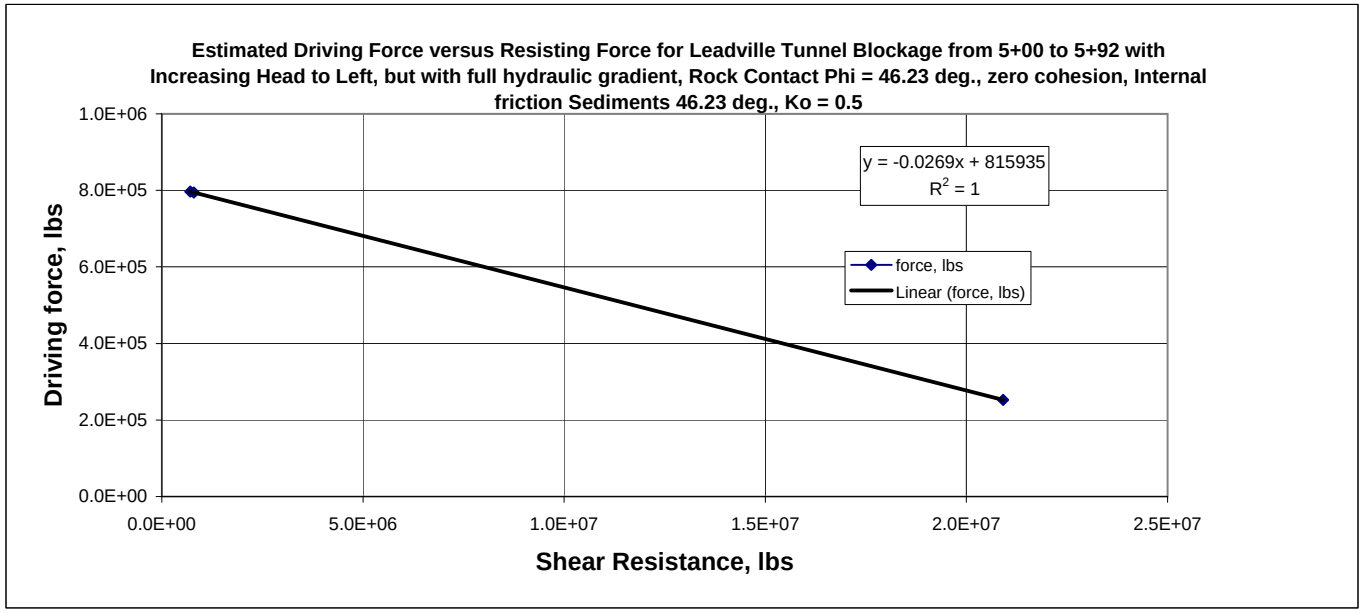
10,064.30 feet TOC for LDT 06+34 Error in table

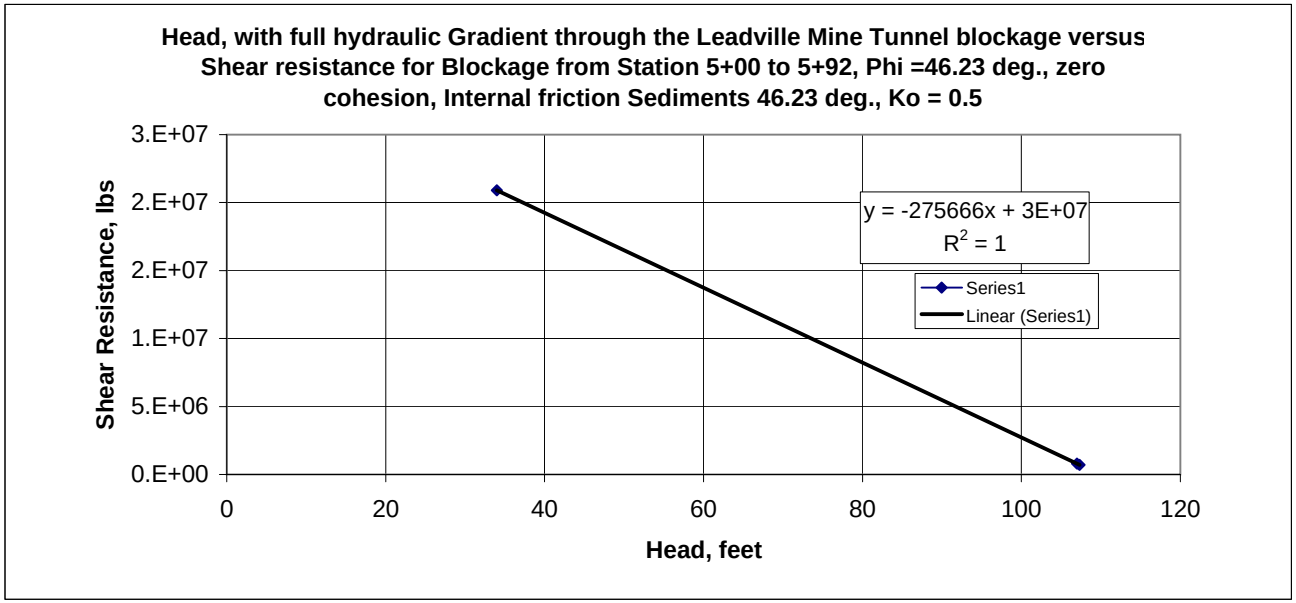
Parameters for this study

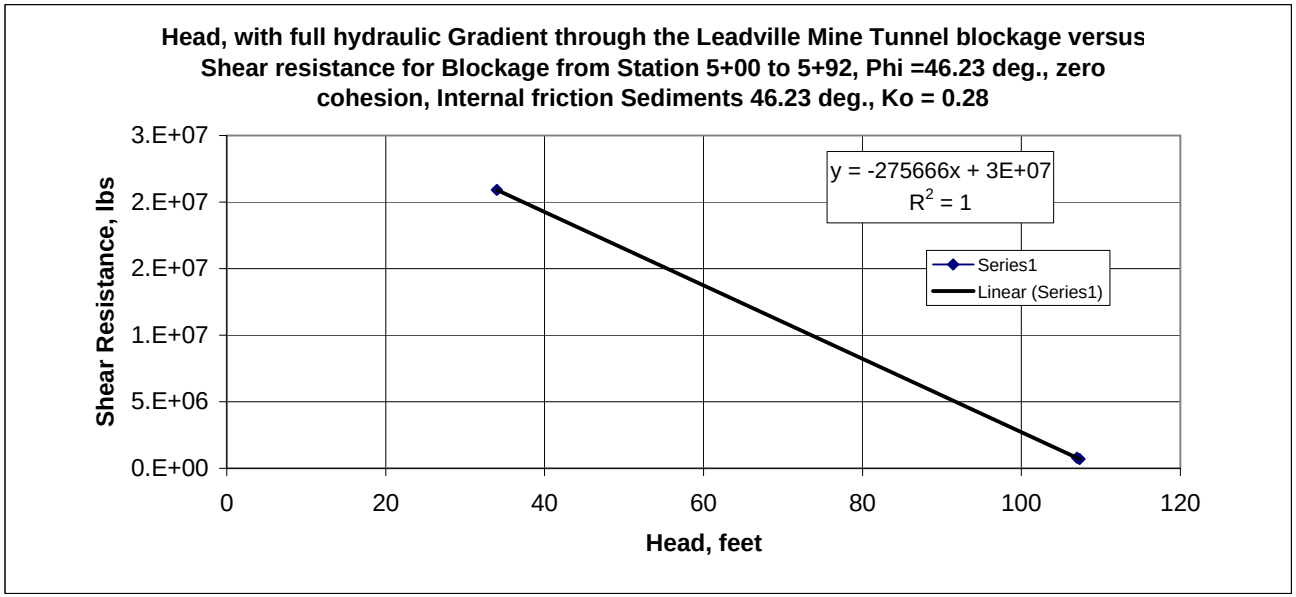
Water Elev. feet	Water Pressure psi	Head at 6+34 feet	Shear Resistance, lbs	Driving force, lbs	S.F.
9991.00	14.73	34.00	20,913,668	252,506	82.8
10064.30	46.50	107.30	707,385	796,878	0.89
10063.98	46.36	106.98	795,598	794,501	1.0

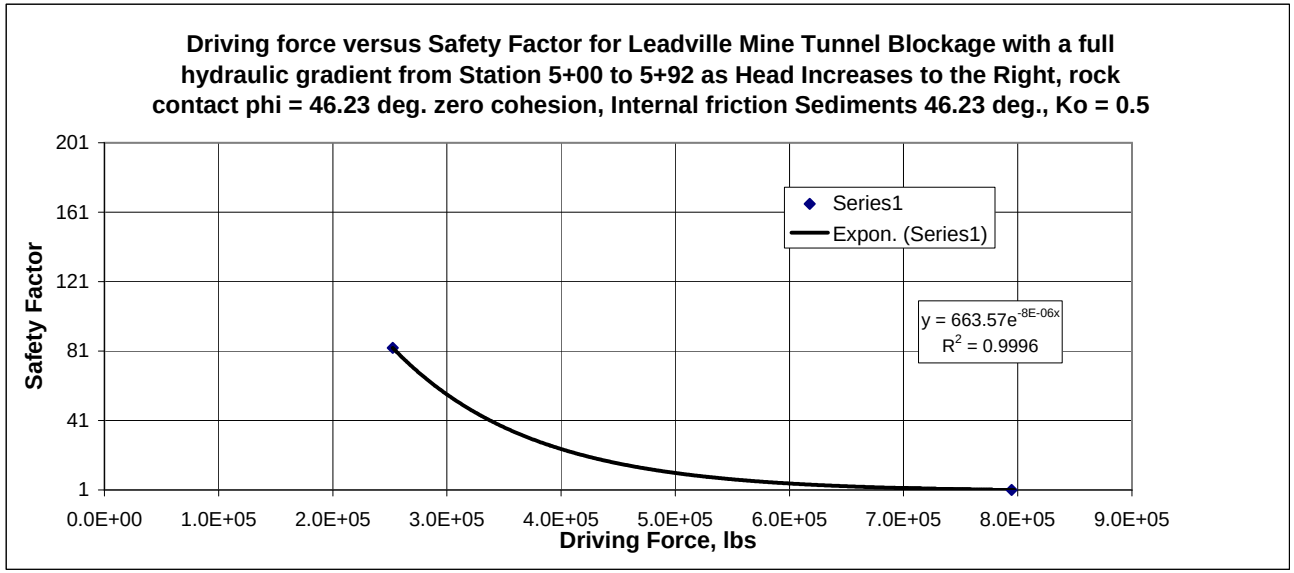
Equation Parameters

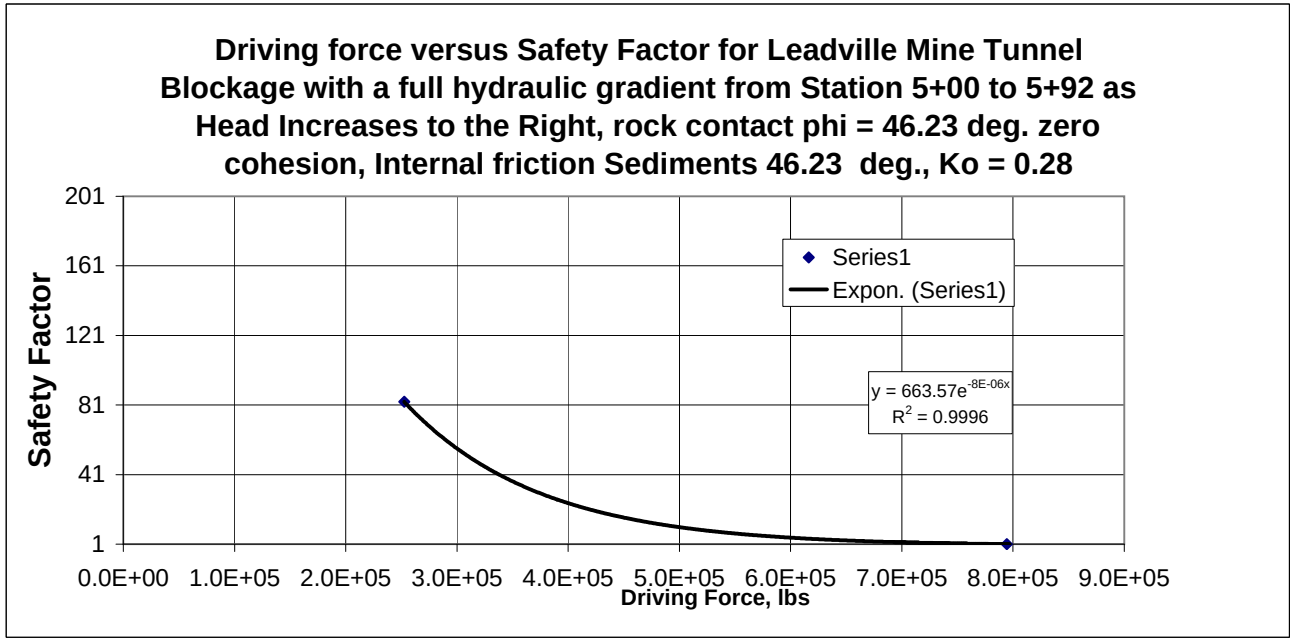
B	A					Check
-0.026940747	815,935		Shear resistance SF=1	794,530 lbs		794,530
-275665.5256	30286295.95	794,530	Head for FS=1	106.98 feet		lbs

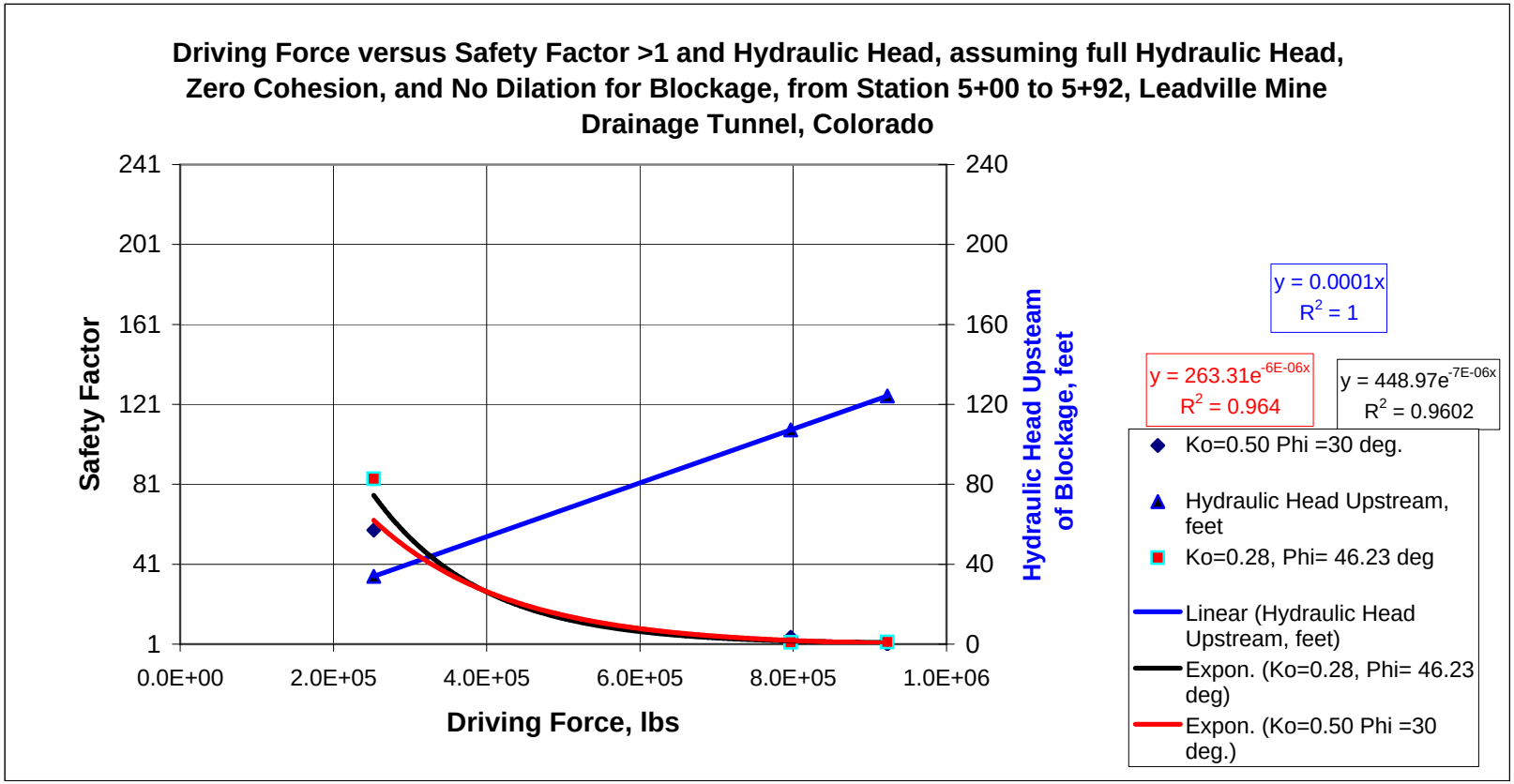

Estimated Driving Force versus Resisting Force for Leadville Tunnel Blockage from 5+00 to 5+92 with Increasing Head to Left, but with full hydraulic gradient, Rock Contact Phi = 46.23 deg., zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.5
y = -0.0269x + 815935
R2 = 1
force, lbs
Linear (force, lbs)
Driving force, lbs
1.0E+06
8.0E+05
6.0E+05
4.0E+05
2.0E+05
0.0E+00
0.0E+00
5.0E+06
1.0E+07
1.5E+07
2.0E+07
2.5E+07
Shear Resistance, lbs


Head, with full hydraulic Gradient through the Leadville Mine Tunnel blockage versus Shear resistance for Blockage from Station 5+00 to 5+92, Phi =46.23 deg., zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.5
Shear Resistance, lbs
3.E+07
2.E+07
2.E+07
1.E+07
5.E+06
0.E+00
0
20
40
60
80
100
120
Head, feet
y = -275666x + 3E+07
R2 = 1
Series1
Linear (Series1)


Head, with full hydraulic Gradient through the Leadville Mine Tunnel blockage versus Shear resistance for Blockage from Station 5+00 to 5+92, Phi =46.23 deg., zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.28
Shear Resistance, lbs
3.E+07
2.E+07
2.E+07
1.E+07
5.E+06
0.E+00
0
20
40
60
80
100
120
Head, feet
y = -275666x + 3E+07
R2 = 1
Series1
Linear (Series1)


Driving force versus Safety Factor for Leadville Mine Tunnel Blockage with a full hydraulic gradient from Station 5+00 to 5+92 as Head Increases to the Right, rock contact phi = 46.23 deg. zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.5
Safety Factor
201
161
121
81
41
1
0.0E+00
1.0E+05
2.0E+05
3.0E+05
4.0E+05
5.0E+05
6.0E+05
7.0E+05
8.0E+05
9.0E+05
Driving Force, lbs
Series1
Expon. (Series1)
y = 663.57e-8E-06x
R2 = 0.9996


Driving force versus Safety Factor for Leadville Mine Tunnel Blockage with a full hydraulic gradient from Station 5+00 to 5+92 as Head Increases to the Right, rock contact phi = 46.23 deg. zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.28
Safety Factor
201
161
121
81
41
1
0.0E+00
1.0E+05
2.0E+05
3.0E+05
4.0E+05
5.0E+05
6.0E+05
7.0E+05
8.0E+05
9.0E+05
Driving Force, lbs
Series1
Expon. (Series1)
y = 663.57e-8E-06x
R2 = 0.9996


Driving Force versus Safety Factor >1 and Hydraulic Head, assuming full Hydraulic Head, Zero Cohesion, and No Dilation for Blockage, from Station 5+00 to 5+92, Leadville Mine Drainage Tunnel, Colorado
Safety Factor
241
201
161
121
81
41
1
240
200
160
120
80
40
0
Hydraulic Head Upsteam of Blockage, feet
0.0E+00
2.0E+05
4.0E+05
6.0E+05
8.0E+05
1.0E+06
Driving Force, lbs
y = 0.0001x
R2 = 1
y = 263.31e-6E-06x
R2 = 0.964
y = 448.97e-7E-06x
R2 = 0.9602
Ko=0.50 Phi =30 deg.
Hydraulic Head Upstream, feet
Ko=0.28, Phi= 46.23 deg
Linear (Hydraulic Head Upstream, feet)
Expon. (Ko=0.28, Phi= 46.23 deg)
Expon. (Ko=0.50 Phi =30 deg.)

Appendix C

Spreadsheet for calculation of the plug stability against blowout for the diffused pressure condition.

5+00 to 5+92 Hydraulic Gradient Through the Tunnel Plug

Calculations		Square section of Tunnel			Curved section		Total
		Height, ft	Width, ft	Area, ft^2	radius, ft	Area, ft^2	Area, ft^2
Cross sectional area of	Inside A-Line	5.21	8.83	46.01	4.42	30.64	76.65
	Area	6.00	8.83	53.00	4.42	30.64	83.64
	Outside B-line	5.21	9.83	51.22	4.92	37.97	89.18707
	Area	6.00	9.83	59.00	4.92	37.97	96.97179
Surface Area						Length	
		10.42	8.83		4.42	13.88	33.13
		12.00	8.83		4.42	13.88	34.71

Calculations

Density of material used 1988	100	lb_f/ft^3				
Density of material used 2005 and in this report	110	lb_f/ft^3	This number used by this study			
Collapsed Length of tunnel	92	feet				
Assumed Internal Friction and Poisson's ratio	30	degrees		0.45	Rock Tunnel Wall Height	
Pages 43 and 61 Gobla's report	11	feet wide	1988 Conservative value inside A-line		Sta	height, feet
excavation dimensions	12	feet tall	R=	5.5	4+62	4
Max Area of Plug, estimated in this study	119.02	$feet^2$	H=	6.5	5+00	5.79
More accurate Volume of Plug this study	10,950	Cu. Feet		46	5+75	9.32
Correct Mass of Collapsed Tunnel plug 1988	1,094,953	lbs			5+92	10.12
Hydraulic Driving Force					6+34	12
This Study max area & correct head	252,506	pounds				
This Study max area & artesian flow @ well 6+34	796,878	pounds				
This Study maximum area & max head	1,763,455	pounds				
Shear Calculations			Not used			
Average Depth of Collapse	100	feet	Not used			
Average vertical stress this study	76.39	psi				
Average horizontal stress this study	38.19	psi	Jaky Ko=			
Invert elevation 0+00	9957	feet	Sigma H = Sigma V*(1-sin(phi))			
Surface Elevation well 6+34	10,064.30	feet	Sigma H = Sigma V*(1-sin(phi))			
Water head used in 1988 , 6+34 (11/24/1976)	77	feet				
1988 hydraulic pressure	33.37	psi		0.50		
Head and Pressure at well 6+34 for artesian flow	107.30	feet	Head may be incorrect. Appears depth to the water table used as			
Maximum head elevation Pendry Blockage	10148.81	feet	the head. 77 feet was only used to check original calculations.			
Maximum Potential water head at 6+34	191.81	feet	46.50			
Maximum hydraulic pressure-Non Leaky Tunnel	83.12	psi	Mine pool head 192 feet, 9/13/2007. Doubtful hydraulic fracturing			
Hydraulic head and pressure for uplift/fracturing 2008	176.28	head	76.39			

Water head and pressure at 6+34 for FS=1	237.45	feet	102.90	
shear calculations				psi
Perimeter of horseshoe tunnel	34.71	feet		psi
Bottom of horseshoe	8.83	feet		psi
Sides of horseshoe tunnel x2	12	feet	S=(p-u) tan Ø	
Top of horseshoe tunnel	13.88	feet		
Shear Equation used in 1988 memorandum			Friction of	Jaky (1948)[1] for normally consolidated soils
Fiction Angle between Rock and fill	30	degrees		Jaky (1948)[1] for normally consolidated soils
Corrected Head and pressure in 1988	34	feet	14.73	
Shear strength, pounds this study w/1988 area	14,818,187	pounds	Used true h	Input phi value here:
Shear strength, this study w/ max area	17,273,566	pounds		46.23
Shear strength, this study uplift head non leaky	6,427,063	pounds	Without a leaky tunnel pr	degrees found by back calculation
Shear strength, this study max head non leaky	1,764,083	pounds	Without a le	psi
Shear strength, this study max head, leaky	11,685,731	pounds	With a leaky tunnel pressure could not blowout plug or overburden	

Location	Description	Date Completed [MM/DD/YY]	Total Depth [feet]	TOC elevation [ft. ASL]
LDT03+00	Alluvial well 20' off set from LDT, Station 3+00	1976	78	10035.1
LDT04+70	Alluvial well 20' off set from LDT, Station 4+70	1976	92	10048.8
LDT06+34	LMDT Well Previous DWW, Station 06+34	1976	108	10099.7

10,064.30 feet TOC for LDT 06+34 Error in table

Parameters for this study

Water Elev. feet	Water Pressure psi	Head at 6+34 feet	Shear Resistance, lbs	Driving force, lbs	
9991.00	14.73	34.00	17,273,566	252,506	
10064.30	46.50	107.30	11,685,731	796,878	
10133.28	76.39	176.28	6,427,063	1,309,182	S.F.
10194.45	102.90	237.45	1,764,083	1,763,455	

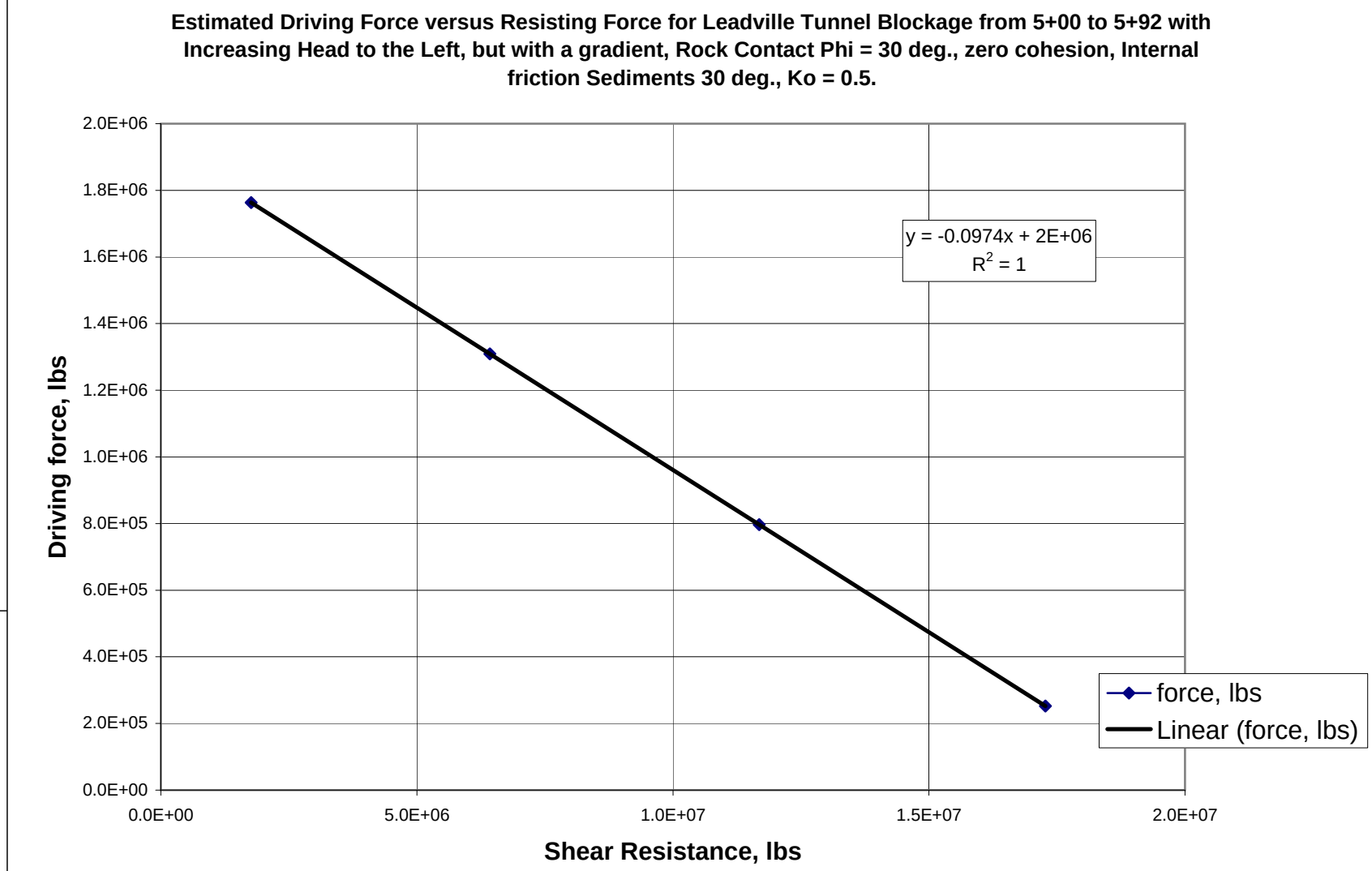
Equation Parameters

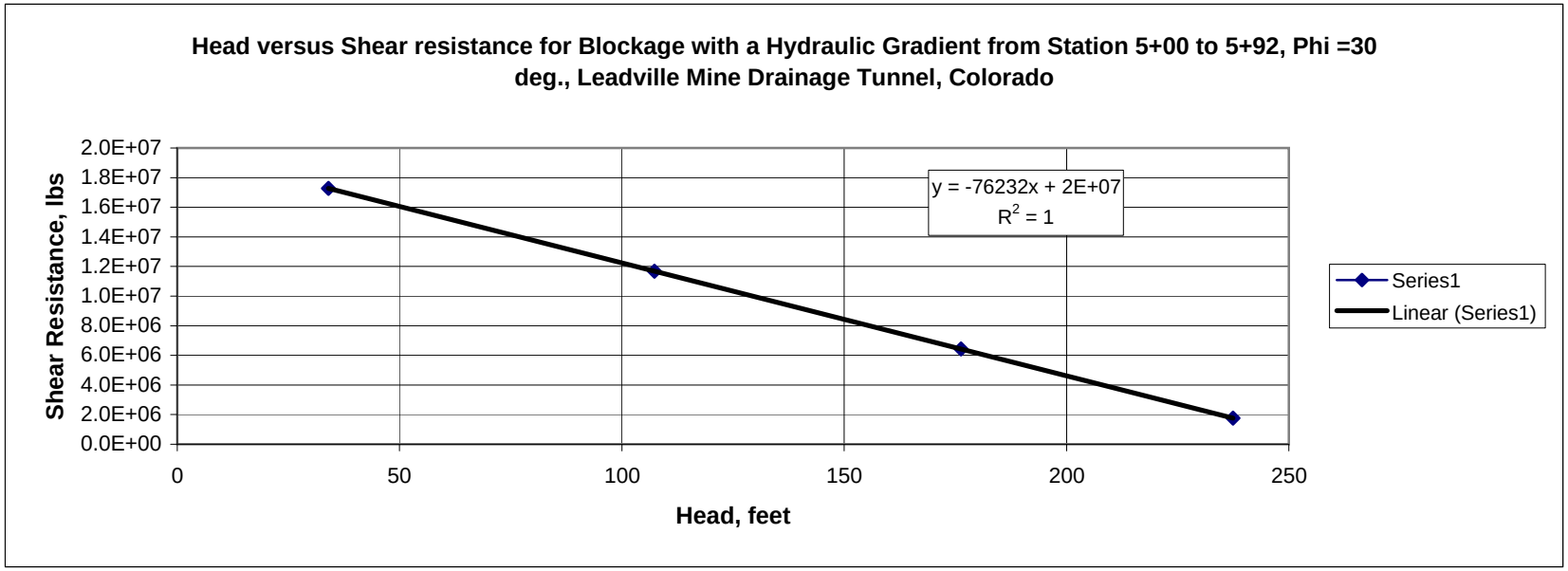
				68.41
B	A			14.66
-0.097420973	1,935,313		Shear resistan	4.91
-76232.40578	19865468.06	1,763,510	Head for FS=	1.00

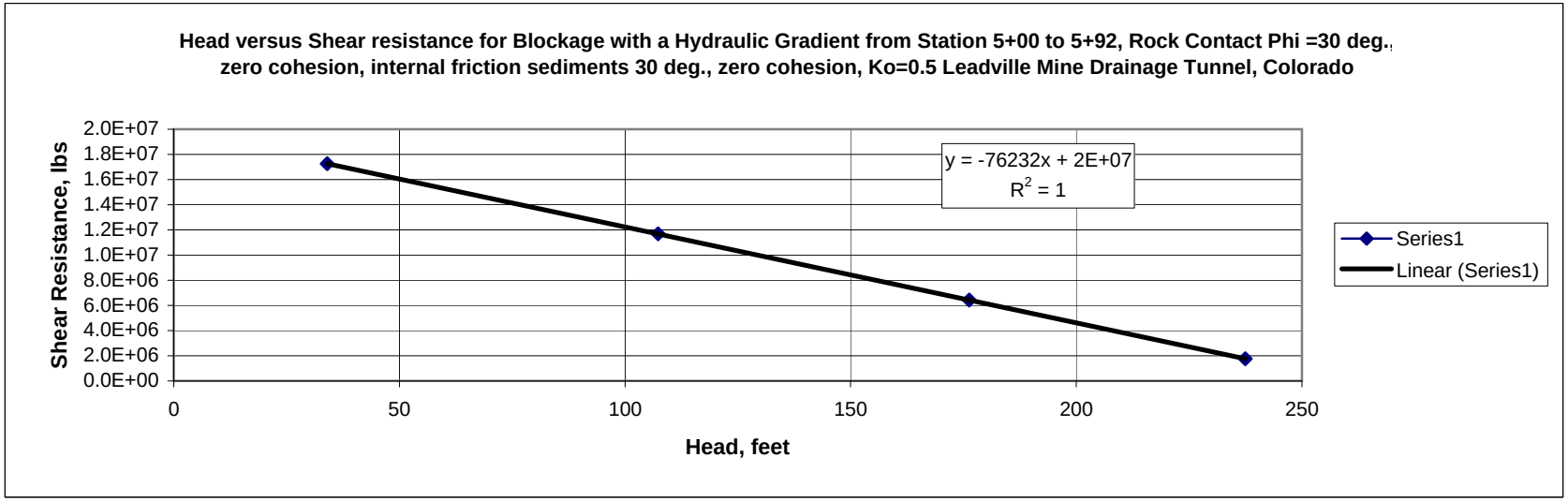
1,763,510 Check

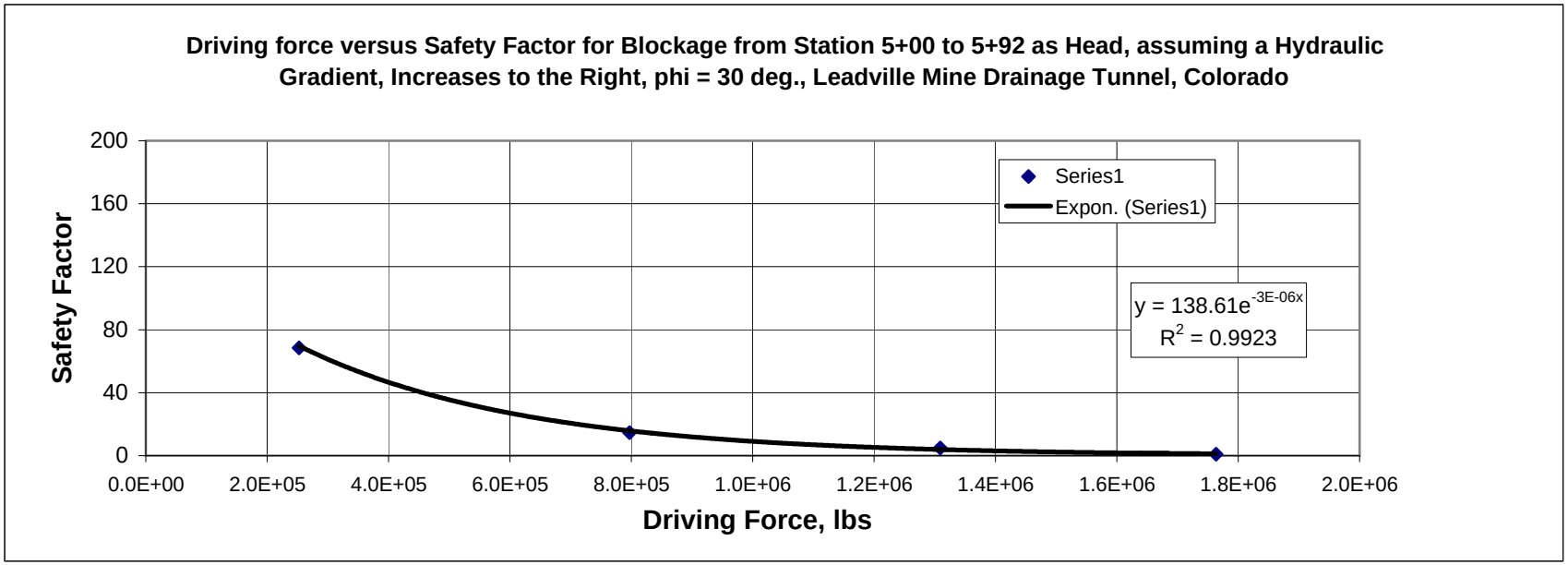
237.46 lbs 1,763,510 lbs

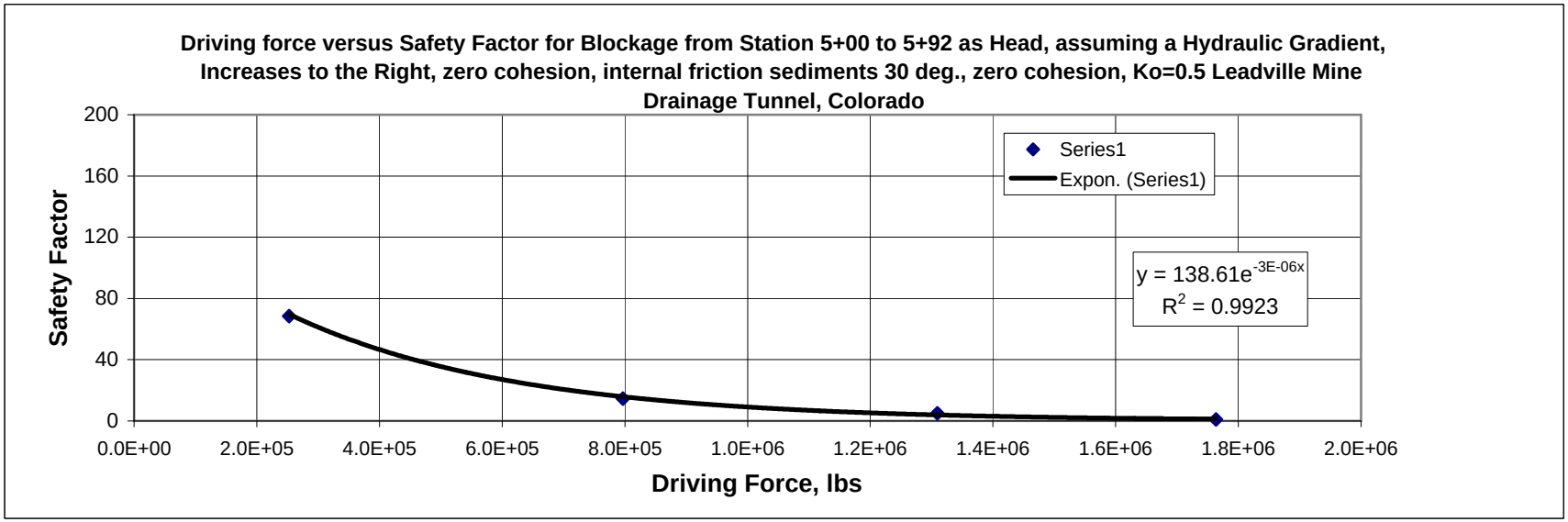
feet


Estimated Driving Force versus Resisting Force for Leadville Tunnel Blockage from 5+00 to 5+92 with Increasing Head to the Left, but with a gradient, Rock Contact Phi = 30 deg., zero cohesion, Internal friction Sediments 30 deg., Ko = 0.5.
Driving force, lbs
2.0E+06
1.8E+06
1.6E+06
1.4E+06
1.2E+06
1.0E+06
8.0E+05
6.0E+05
4.0E+05
2.0E+05
0.0E+00
0.0E+00
5.0E+06
1.0E+07
1.5E+07
2.0E+07
Shear Resistance, lbs
y = -0.0974x + 2E+06
R2 = 1
force, lbs
Linear (force, lbs)


Head versus Shear resistance for Blockage with a Hydraulic Gradient from Station 5+00 to 5+92, Phi =30 deg., Leadville Mine Drainage Tunnel, Colorado
Shear Resistance, lbs
2.0E+07
1.8E+07
1.6E+07
1.4E+07
1.2E+07
1.0E+07
8.0E+06
6.0E+06
4.0E+06
2.0E+06
0.0E+00
0
50
100
150
200
250
Head, feet
y = -76232x + 2E+07
R2 = 1
Series1
Linear (Series1)


Head versus Shear resistance for Blockage with a Hydraulic Gradient from Station 5+00 to 5+92, Rock Contact Phi =30 deg., zero cohesion, internal friction sediments 30 deg., zero cohesion, Ko=0.5 Leadville Mine Drainage Tunnel, Colorado
Shear Resistance, lbs
2.0E+07
1.8E+07
1.6E+07
1.4E+07
1.2E+07
1.0E+07
8.0E+06
6.0E+06
4.0E+06
2.0E+06
0.0E+00
0
50
100
150
200
250
Head, feet
y = -76232x + 2E+07
R2 = 1
Series1
Linear (Series1)


Driving force versus Safety Factor for Blockage from Station 5+00 to 5+92 as Head, assuming a Hydraulic Gradient, Increases to the Right, phi = 30 deg., Leadville Mine Drainage Tunnel, Colorado
Safety Factor
200
160
120
80
40
0
0.0E+00
2.0E+05
4.0E+05
6.0E+05
8.0E+05
1.0E+06
1.2E+06
1.4E+06
1.6E+06
1.8E+06
2.0E+06
Driving Force, lbs
Series1
Expon. (Series1)
y = 138.61e^-3E-06x
R^2 = 0.9923


Driving force versus Safety Factor for Blockage from Station 5+00 to 5+92 as Head, assuming a Hydraulic Gradient, Increases to the Right, zero cohesion, internal friction sediments 30 deg., zero cohesion, Ko=0.5 Leadville Mine Drainage Tunnel, Colorado
Safety Factor
200
160
120
80
40
0
0.0E+00
2.0E+05
4.0E+05
6.0E+05
8.0E+05
1.0E+06
1.2E+06
1.4E+06
1.6E+06
1.8E+06
2.0E+06
Driving Force, lbs
Series1
Expon. (Series1)
y = 138.61e^-3E-06x
R^2 = 0.9923

5+00 to 5+92

Calculations		Square section of Tunnel			Curved section		Total
		Height, ft	Width, ft	Area, ft^2	radius, ft	Area, ft^2	Area, ft^2
Cross sectional area of	Inside A-Line	5.21	8.83	46.01	4.42	30.64	76.65
	Area	6.00	8.83	53.00	4.42	30.64	83.64
	Outside B-line	5.21	9.83	51.22	4.92	37.97	89.187066
	Area	6.00	9.83	59.00	4.92	37.97	96.971789
Surface Area						Length	
		10.42	8.83		4.42	13.88	33.13
		12.00	8.83		4.42	13.88	34.71

\Calculations

Density of material used 1988	100	lb_f/ft^3				
Density of material used 2005 and in this report	110	lb_f/ft^3	This number used by this study			
Collapsed Length of tunnel	92	feet				
Assumed Internal Friction and Poisson's ratio	46.23	degrees		0.45	Rock Tunnel Wall Height	
Area of Plug, estimated in 1988	83.6	$feet^2$	1988 Conservative value inside A-line		Sta	height, feet
Pages 43 and 61 Gobla's report	11	feet wide	R=	5.5	4+62	4
excavation dimensions	12	feet tall	H=	6.5	5+00	5.79
Max Area of Plug, estimated in this study	119.02	$feet^2$		46	5+75	9.32
More accurate Volume of Plug this study	10,950	Cu. Feet			5+92	10.12
Hydraulic Driving Force					6+34	12
This Study max area & correct head	252,506	pounds				
This Study max area & artesian flow @ well 6+34	796,878	pounds				
This Study maximum area & max head	1,548,528	pounds				
Shear Calculations						
Average Depth of Collapse	100	feet				
Average vertical stress this study	76.39	psi				
Average horizontal stress this study	21.23	psi	Jaky Ko =	0.28		
Invert elevation at station 0+00	9957	feet	Sigma H = Sigma V*(1-sin(phi))			
Surface Elevation well 6+34	10,064.30	feet				
Head and Pressure at well 6+34 for artesian flow	107.30	feet	the head. 77 feet was only used to check original calculations.			
Maximum head elevation Pendry Blockage	10148.81	feet	46.50	psi		
Maximum Potential water head at 6+35	191.81	feet	Mine pool head 192 feet, 9/13/2007. Doubtful hydraulic fracturing			
Maximum hydraulic pressure-Non Leaky Tunnel	83.12	psi	or uplift may occur where overburden less than about half the head			
Hydraulic head and pressure for uplift/fracturing 2008	176.28	feet				
Water head and pressure at 6+35 for FS=1	208.51	feet	76.39	psi		

shear calculations 90.35 psi

Perimeter of horseshoe tunnel	34.71	feet		
Bottom of horseshoe	8.83	feet		
Sides of horseshoe tunnel x2	12	feet		
Top of horseshoe tunnel	13.88	feet		
Shear Equation used in 1988 memorandum				
Friction Angle between fill and rock surfaces	46	degrees	S=(p-u) tan Ø	
Corrected Head and pressure in 1988	34	feet	14.73 psi	
Shear strength, pounds this study w/1988 area	23,976,341	pounds	Used true head.	This was not used
Shear strength, this study w/ max area	25,599,982	pounds		
Shear strength, this study max head non leaky	5,988,854	pounds	Without a leaky tunnel pressure could blowout lift up overburden.	
Shear strength, this study max head non leaky	1,546,787	pounds	Without a leaky tunnel pressure could blowout plug or overburden.	
Shear strength, this study max head, leaky	15,496,841	pounds	With a leaky tunnel pressure could not blowout plug or overburden.	

Location	Description	Date Completed [MM/DD/YY]	Total Depth [feet]	TOC elevation [ft. ASL]
LDT03+00	Alluvial well 20' off set from LDT, Station 3+00	1976	78	10035.1
LDT04+70	Alluvial well 20' off set from LDT, Station 4+70	1976	92	10048.8
LDT06+34	LMDT Well Previous DWW, Station 06+34	1976	108	10099.7

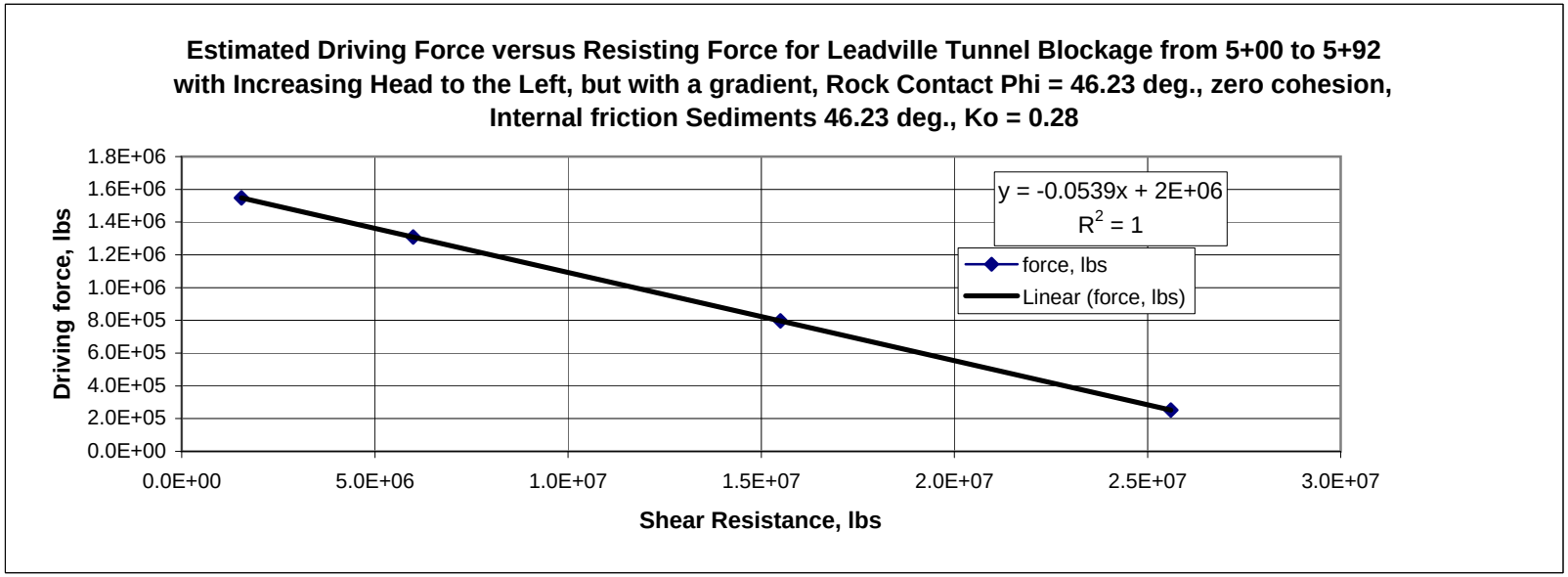
10,064.30 feet TOC for LDT 06+34 Error in table

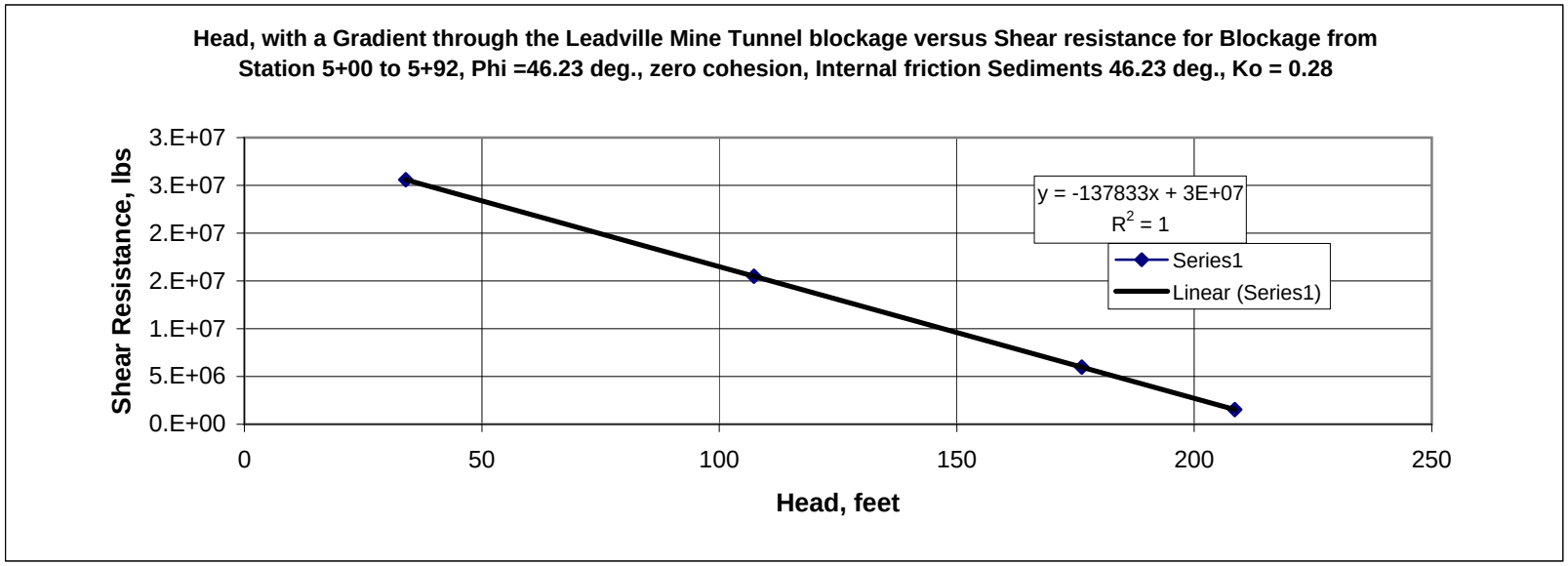
Parameters for this study

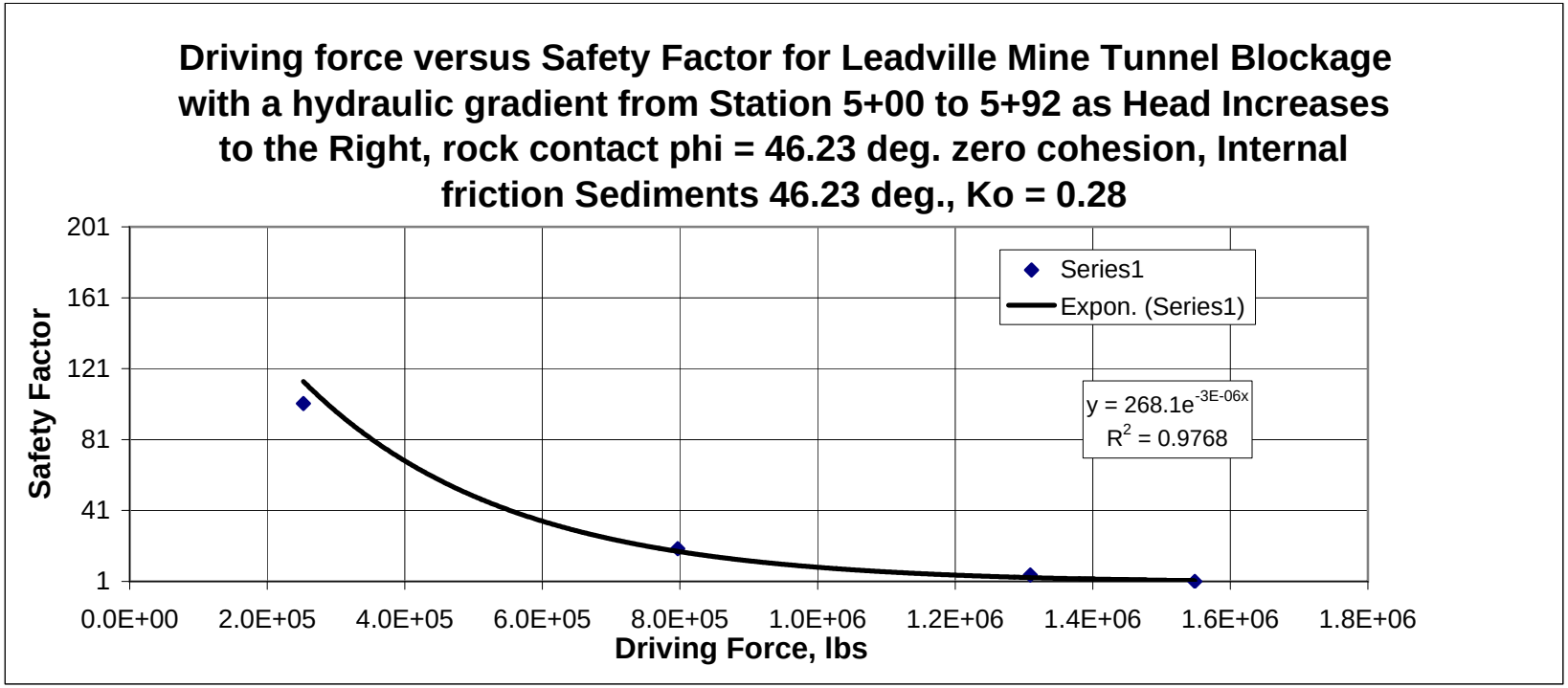
Water Elev. feet	Water Pressure psi	Head at 6+34 feet	Shear Resistance, lbs	Driving force, lbs	S.F.
9991.00	14.73	34.00	25,599,982	252,506	101.4
10064.30	46.50	107.30	15,496,841	796,878	19.4
10133.28	76.39	176.28	5,988,854	1,309,182	4.6
10165.51	90.35	208.51	1,546,787	1,548,528	1.0

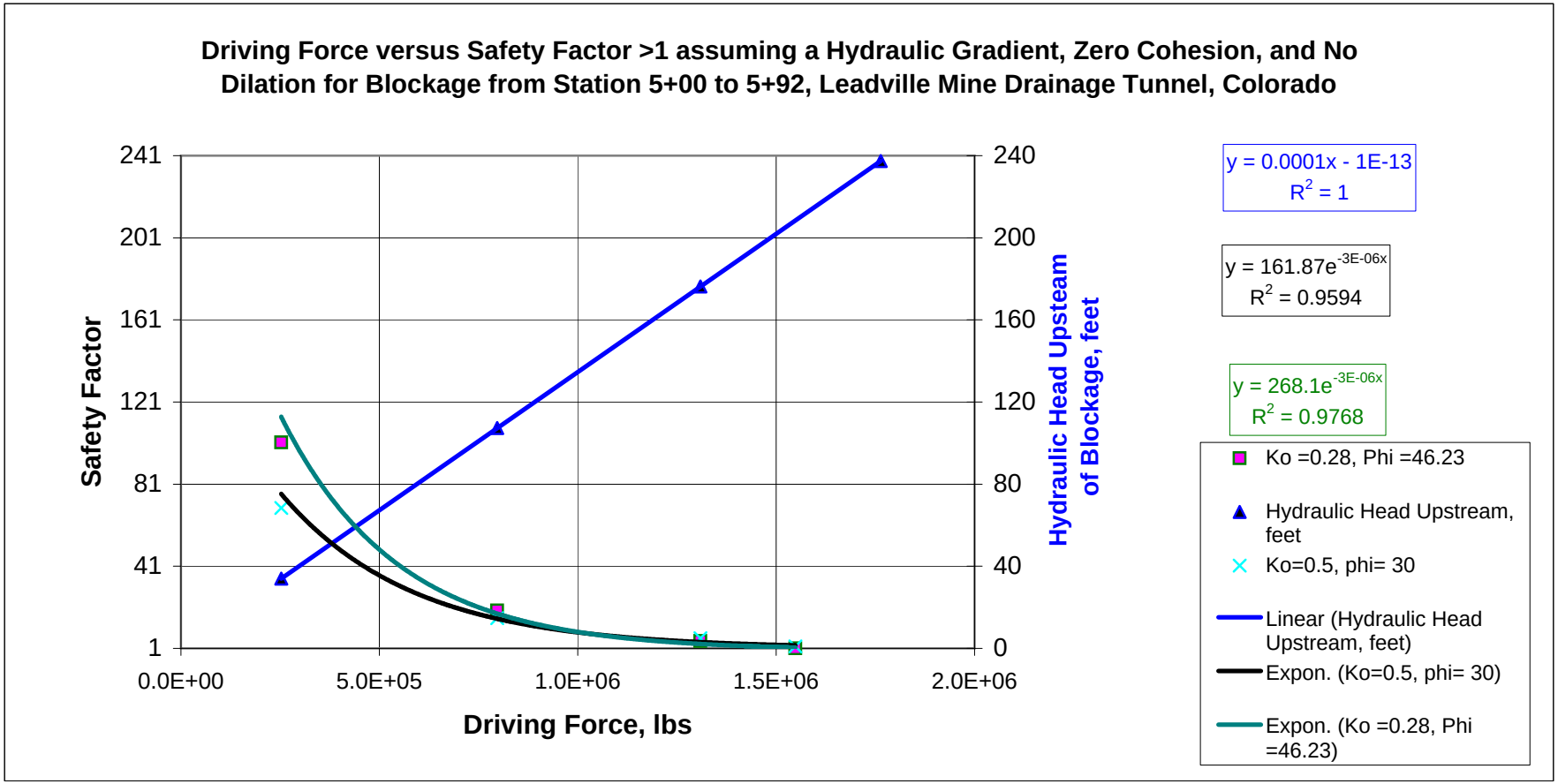
Equation Parameters

B	A				
-0.053881494	1,631,871		Shear resistance SF=1	1,548,439	
-137832.7628	30286295.95	1,548,439	Head for FS=1	208.50	Check
				lbs	1,548,439 lbs
				feet	


Estimated Driving Force versus Resisting Force for Leadville Tunnel Blockage from 5+00 to 5+92 with Increasing Head to the Left, but with a gradient, Rock Contact Phi = 46.23 deg., zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.28
Driving force, lbs
1.8E+06
1.6E+06
1.4E+06
1.2E+06
1.0E+06
8.0E+05
6.0E+05
4.0E+05
2.0E+05
0.0E+00
0.0E+00
5.0E+06
1.0E+07
1.5E+07
2.0E+07
2.5E+07
3.0E+07
Shear Resistance, lbs
y = -0.0539x + 2E+06
R2 = 1
force, lbs
Linear (force, lbs)


Head, with a Gradient through the Leadville Mine Tunnel blockage versus Shear resistance for Blockage from Station 5+00 to 5+92, Phi =46.23 deg., zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.28
Shear Resistance, lbs
3.E+07
3.E+07
2.E+07
2.E+07
1.E+07
5.E+06
0.E+00
0
50
100
150
200
250
Head, feet
y = -137833x + 3E+07
R2 = 1
Series1
Linear (Series1)


Driving force versus Safety Factor for Leadville Mine Tunnel Blockage with a hydraulic gradient from Station 5+00 to 5+92 as Head Increases to the Right, rock contact phi = 46.23 deg. zero cohesion, Internal friction Sediments 46.23 deg., Ko = 0.28
Safety Factor
201
161
121
81
41
1
0.0E+00
2.0E+05
4.0E+05
6.0E+05
8.0E+05
1.0E+06
1.2E+06
1.4E+06
1.6E+06
1.8E+06
Driving Force, lbs
Series1
Expon. (Series1)
y = 268.1e^-3E-06x
R^2 = 0.9768


Driving Force versus Safety Factor >1 assuming a Hydraulic Gradient, Zero Cohesion, and No Dilation for Blockage from Station 5+00 to 5+92, Leadville Mine Drainage Tunnel, Colorado
Safety Factor
241
201
161
121
81
41
1
240
200
160
120
80
40
0
Hydraulic Head Upsteam of Blockage, feet
0.0E+00
5.0E+05
1.0E+06
1.5E+06
2.0E+06
Driving Force, lbs
y = 0.0001x - 1E-13
R2 = 1
y = 161.87e-3E-06x
R2 = 0.9594
y = 268.1e-3E-06x
R2 = 0.9768
Ko =0.28, Phi =46.23
Hydraulic Head Upstream, feet
Ko=0.5, phi= 30
Linear (Hydraulic Head Upstream, feet)
Expon. (Ko=0.5, phi= 30)
Expon. (Ko =0.28, Phi =46.23)

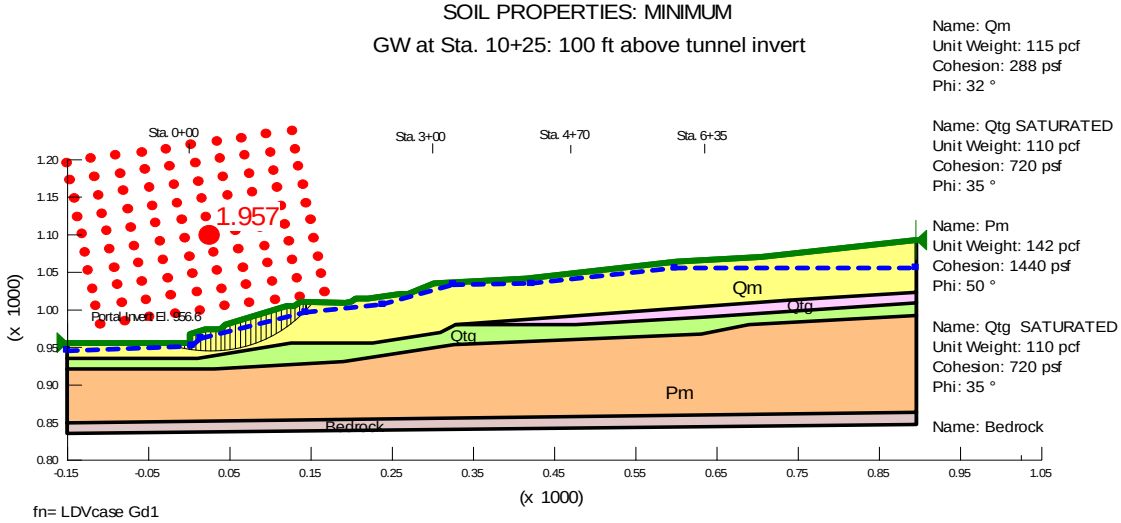
Appendix D

Slope Stability Calculation Results

APPENDIX D - FIGURE 1
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.000g
SOIL PROPERTIES: MINIMUM

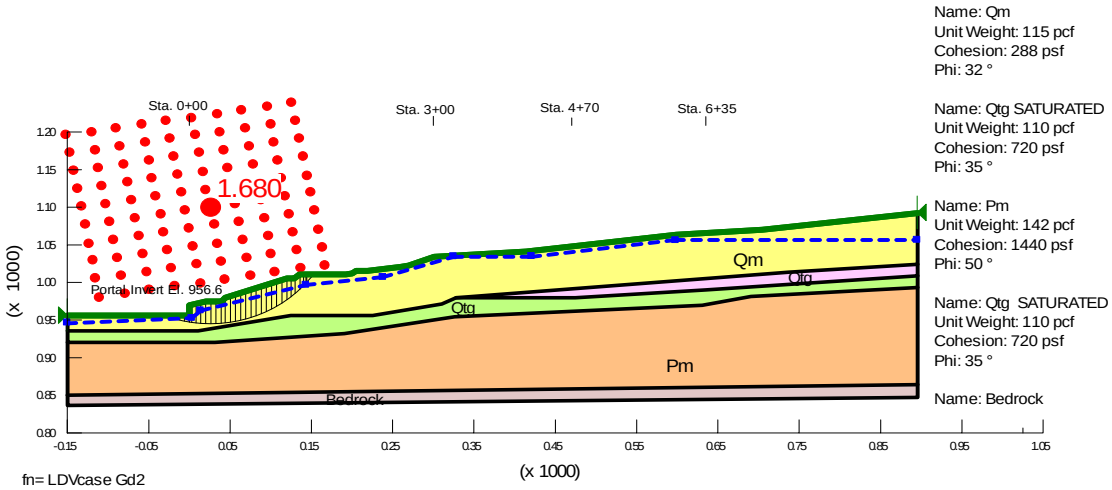
GW at Sta. 10+25: 100 ft above tunnel invert



APPENDIX D - FIGURE 2
LEADVILLE DRAINAGE TUNNEL

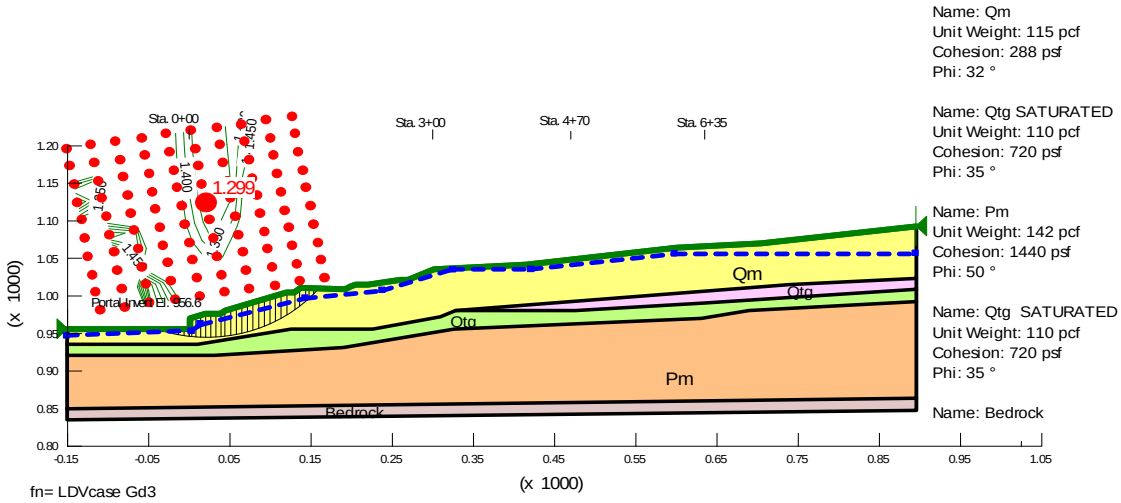
STUDY 2008 - SEISMIC COEFFICIENT: 0.05g
SOIL PROPERTIES: MINIMUM

GW at Sta. 10+25: 100 ft above tunnel invert



APPENDIX D - FIGURE 3
LEADVILLE DRAINAGE TUNNEL

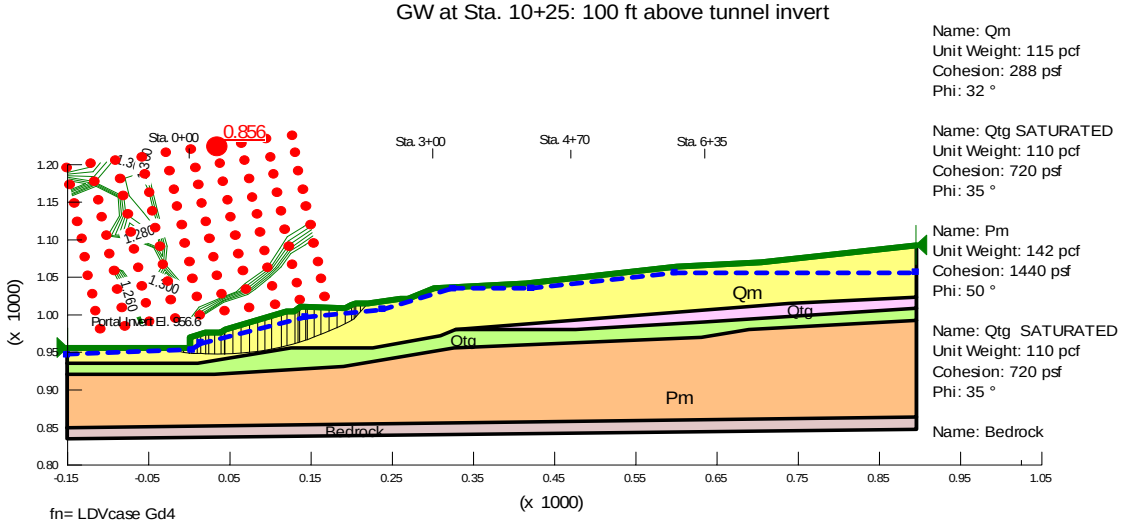
STUDY 2008 - SEISMIC COEFFICIENT: 0.15g
SOIL PROPERTIES: MINIMUM
GW at Sta. 10+25: 100 ft above tunnel invert



fn= LDVcase Gd3

APPENDIX D - FIGURE 4
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.35g
SOIL PROPERTIES: MINIMUM
GW at Sta. 10+25: 100 ft above tunnel invert

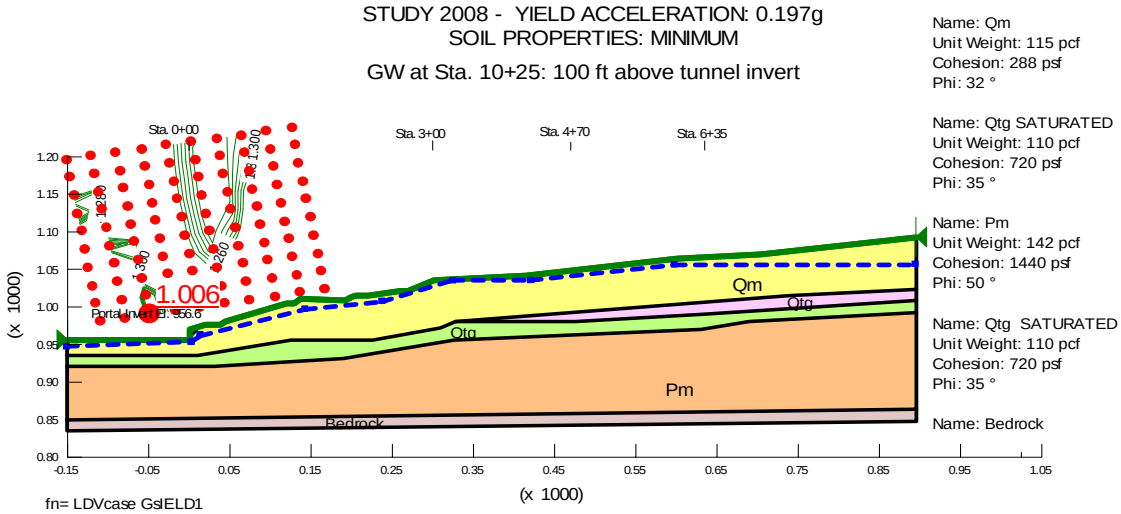


fn= LDVcase Gd4

APPENDIX D - FIGURE 5
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - YIELD ACCELERATION: 0.197g
SOIL PROPERTIES: MINIMUM

GW at Sta. 10+25: 100 ft above tunnel invert

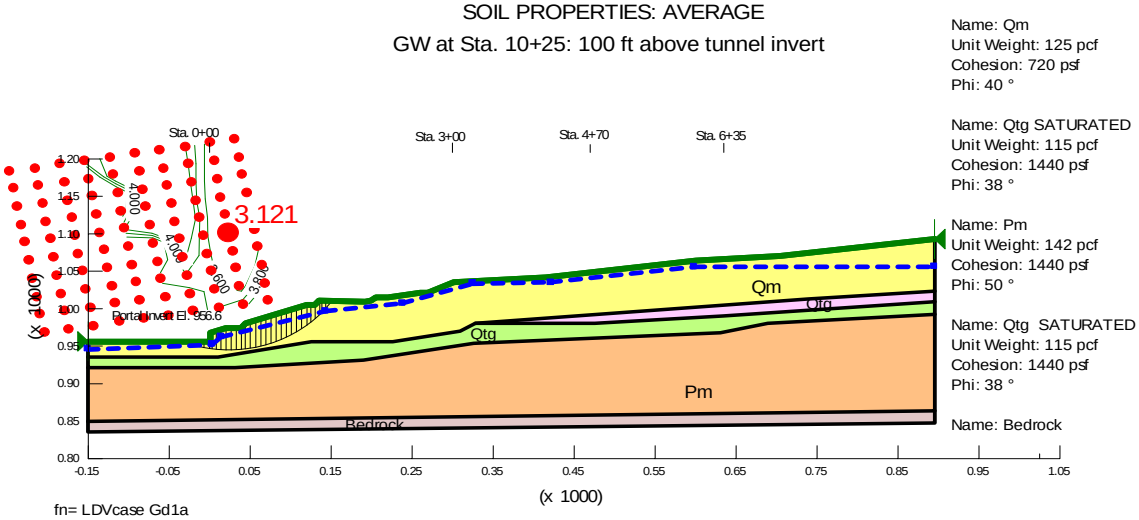


fn= LDVcase GslELD1

APPENDIX D - FIGURE 6
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.000g
SOIL PROPERTIES: AVERAGE

GW at Sta. 10+25: 100 ft above tunnel invert

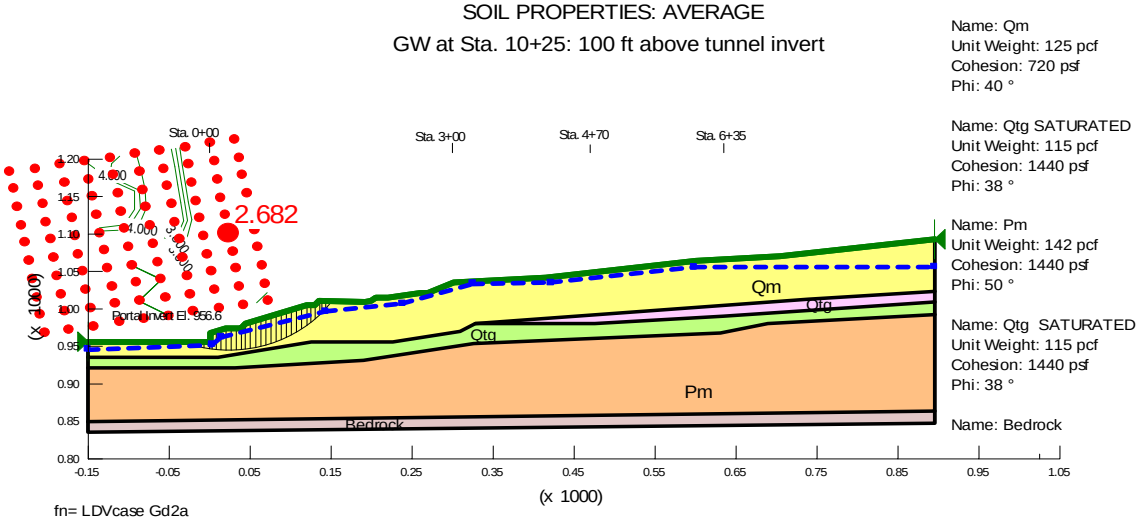


fn= LDVcase Gd1a

APPENDIX D - FIGURE 7
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.050g
SOIL PROPERTIES: AVERAGE

GW at Sta. 10+25: 100 ft above tunnel invert

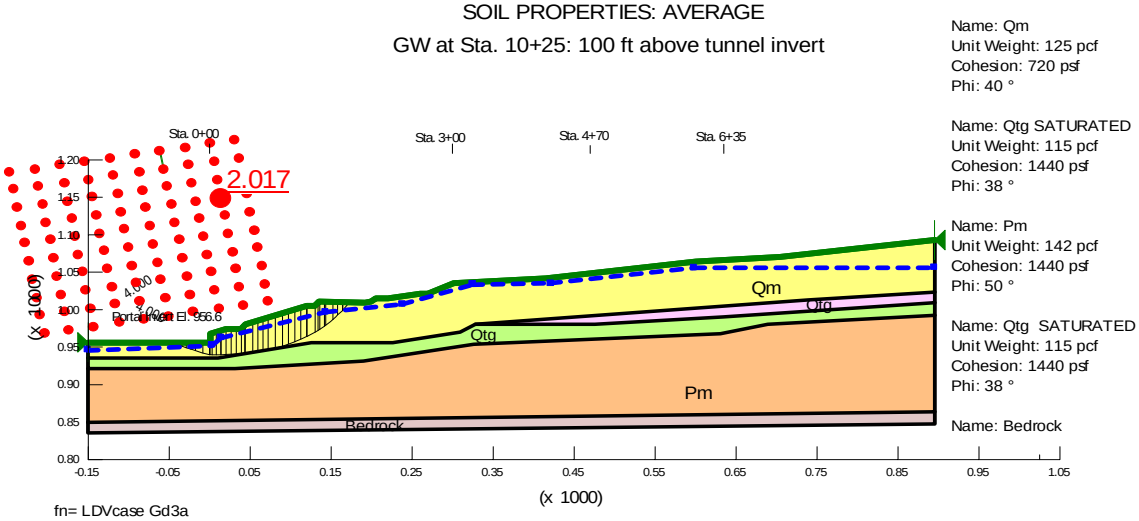


fn= LDVcase Gd2a

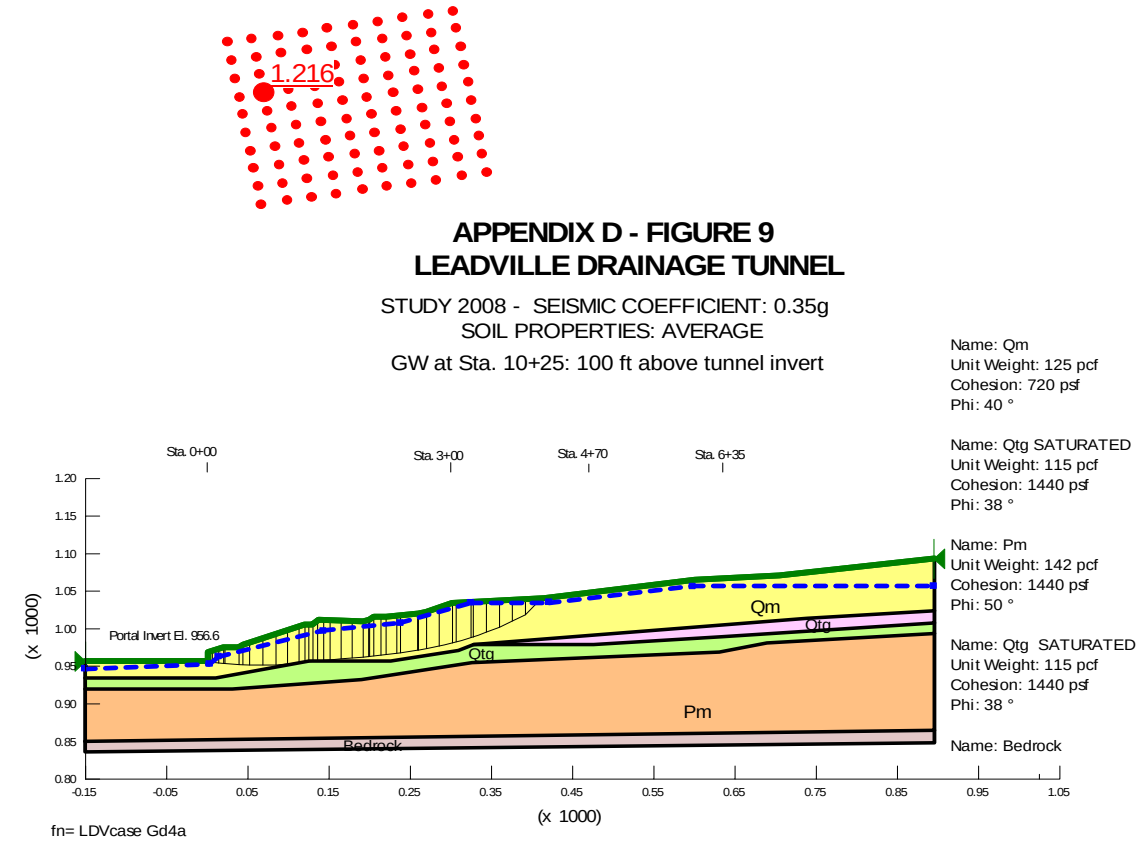
APPENDIX D - FIGURE 8
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.15g
SOIL PROPERTIES: AVERAGE

GW at Sta. 10+25: 100 ft above tunnel invert



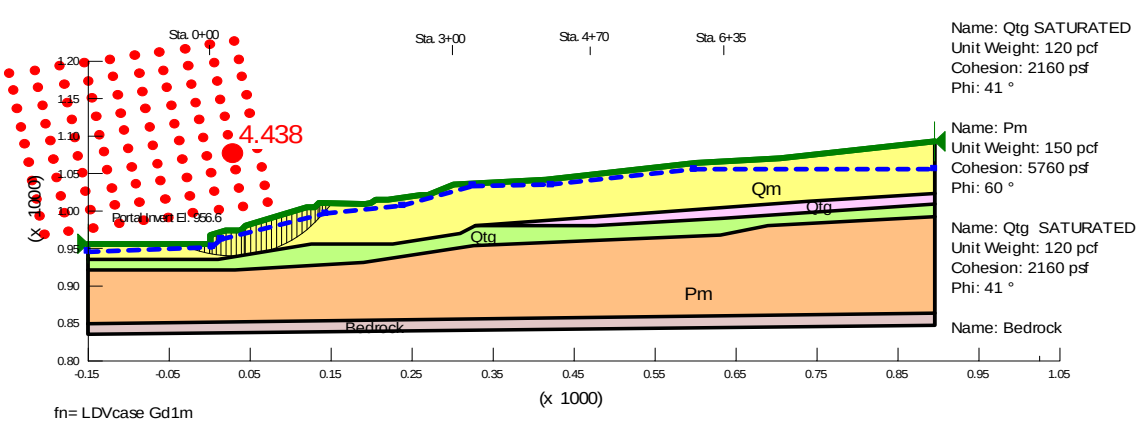
fn= LDVcase Gd3a



APPENDIX D - FIGURE 9
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.35g
SOIL PROPERTIES: AVERAGE
GW at Sta. 10+25: 100 ft above tunnel invert

fn= LDVcase Gd4a



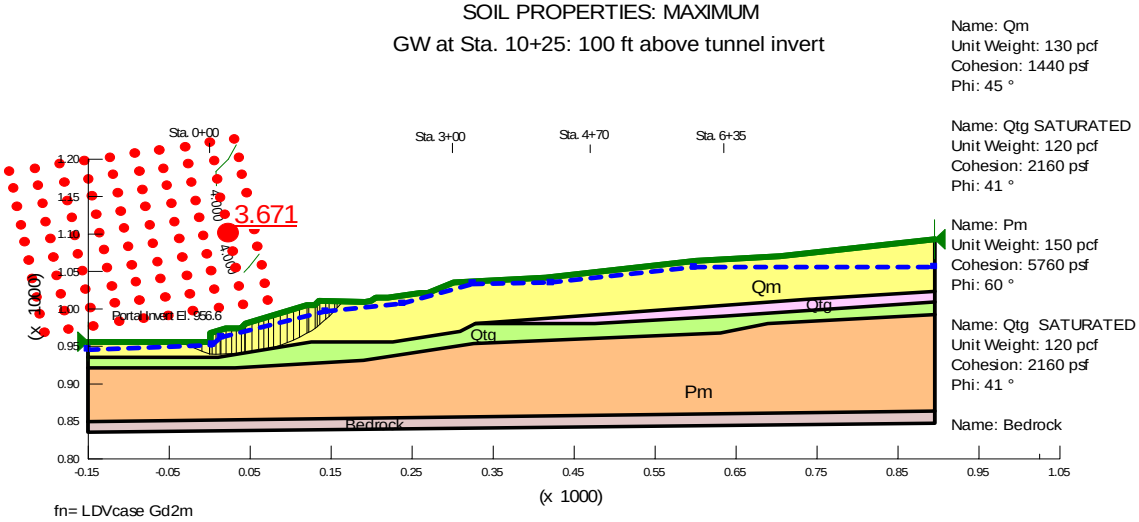
APPENDIX D - FIGURE 10
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.000g
SOIL PROPERTIES: MAXIMUM
GW at Sta. 10+25: 100 ft above tunnel invert

fn= LDVcase Gd1m

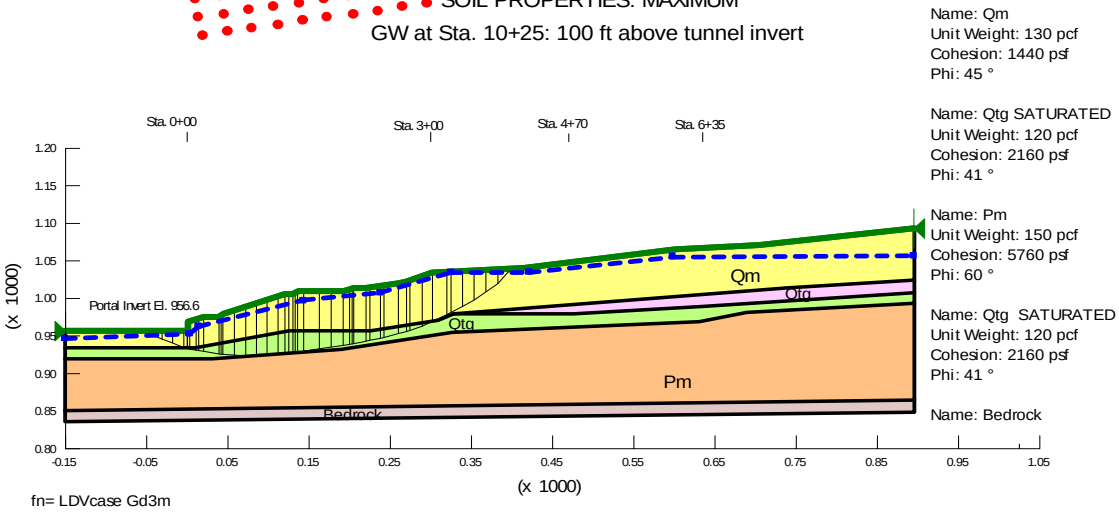
APPENDIX D - FIGURE 11
LEADVILLE DRAINAGE TUNNEL

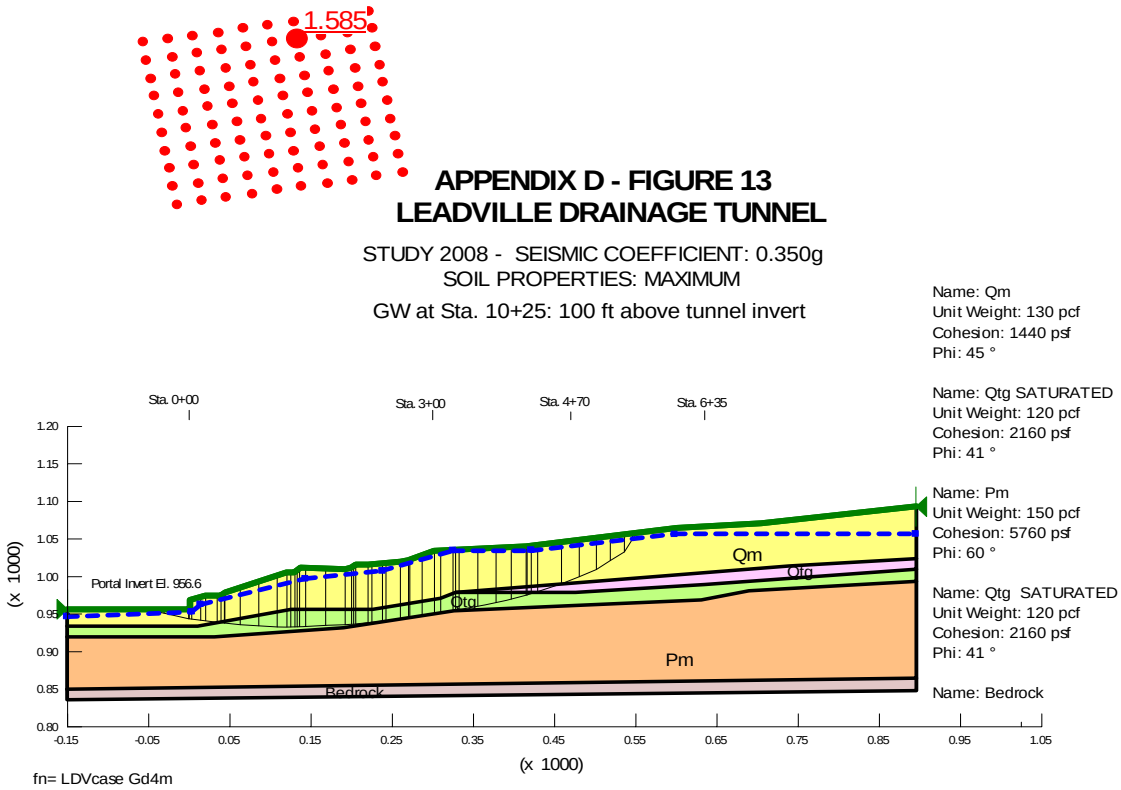
STUDY 2008 - SEISMIC COEFFICIENT: 0.050g
SOIL PROPERTIES: MAXIMUM
GW at Sta. 10+25: 100 ft above tunnel invert



APPENDIX D - FIGURE 12
LEADVILLE DRAINAGE TUNNEL

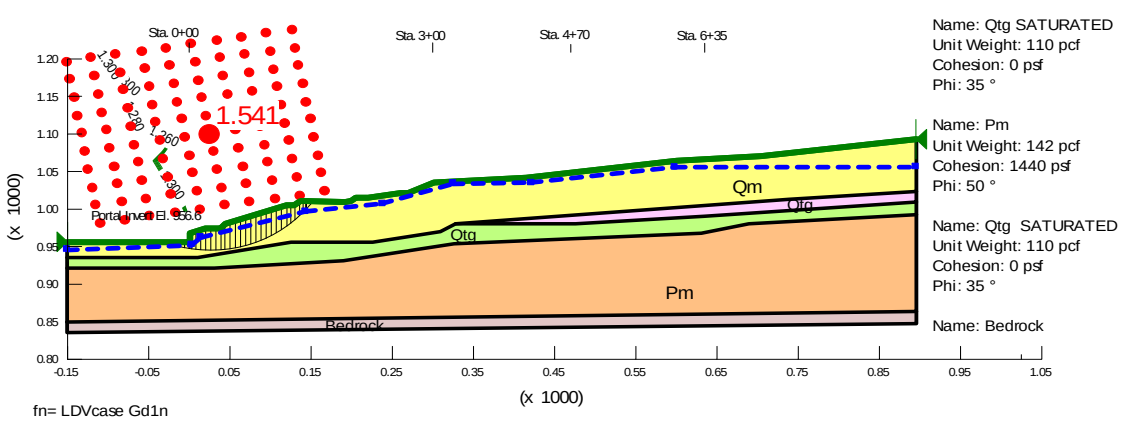
STUDY 2008 - SEISMIC COEFFICIENT: 0.150g
SOIL PROPERTIES: MAXIMUM
GW at Sta. 10+25: 100 ft above tunnel invert





APPENDIX D - FIGURE 13
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.350g
SOIL PROPERTIES: MAXIMUM
GW at Sta. 10+25: 100 ft above tunnel invert

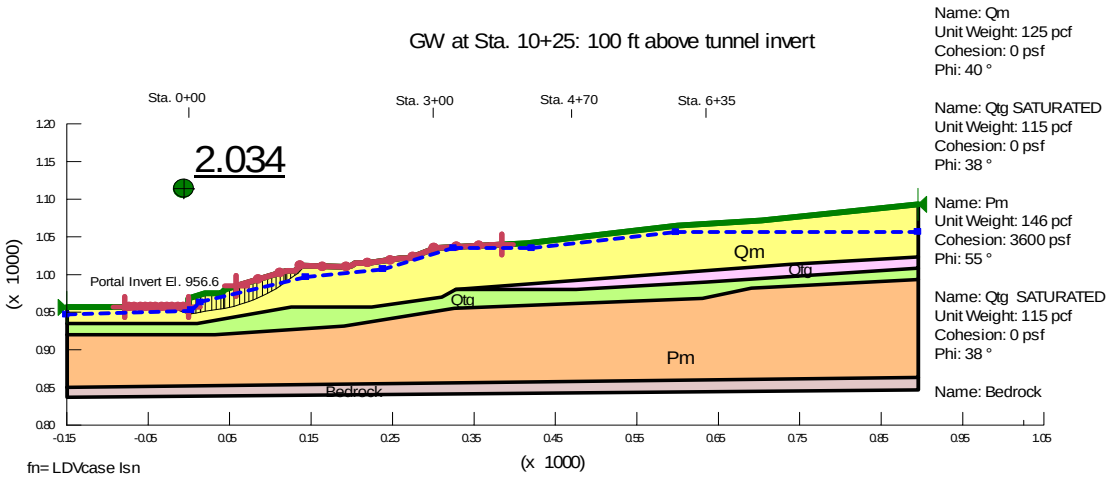


APPENDIX D - FIGURE 14
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.000g
SOIL PROPERTIES: MINIMUM & NO COHESION
GW at Sta. 10+25: 100 ft above tunnel invert

APPENDIX D - FIGURE 15
LEADVILLE DRAINAGE TUNNEL

STUDY 2008 - SEISMIC COEFFICIENT: 0.00g
SOIL PROPERTIES: AVERAGE & NO COHESION



fn= LDVcase lsn