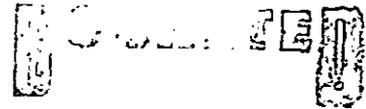




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STABILITY OF THE *A-00116257*
EAST BRADY TUNNEL *6-00981552*
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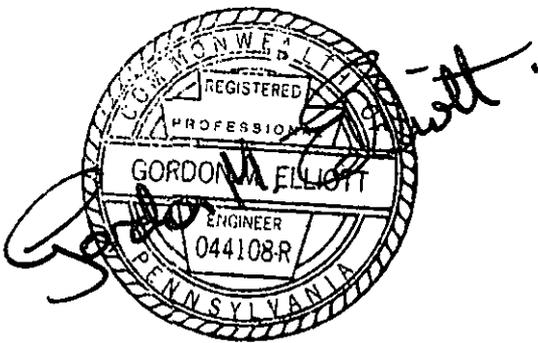
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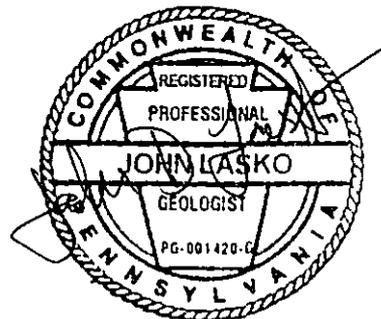
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INTRODUCTION

The East Brady Tunnel is located approximately 1.7 miles east of the town of East Brady in Brady Township, Clarion County, Pennsylvania (Figures 1 and 2). Its construction was likely completed in 1915, according to the date on the keystone at each portal, and was last operated by Consolidated Rail Corporation. It has been reported¹ that the line ceased to be operated prior to 1984, and since then, the track has been removed. The right-of-way was deeded over to the Allegheny Valley Land Trust in a quit claim deed dated January 7 1992². The opinions expressed herein are within a reasonable degree of professional certainty.

This study has been carried out to address a request by Judge Michael A. Nemecek of the Pennsylvania Public Utility Commission for substantial probative evidence that the continued existence of the East Brady Tunnel does not constitute a threat to the public highways that cross over it. Accordingly this report summarizes an engineering analysis, supported by investigated facts, that establishes that the tunnel was hewn from solid rock and is sound and stable. Its collapse in the foreseeable future is improbable, and in the unlikely event that a collapse occurs, it would not pose a threat to the highways that cross over it. The scope of work that was carried out in this study includes the following:

- An intrusive inspection of the tunnel lining. A series of probe holes were drilled through the tunnel lining at sections beneath each of the roads that cross over the tunnel, and a qualitative assessment of the degree of drilling difficulty was noted. A fiber-optic borescope was used to visually inspect the condition of each hole over its full length. With this information it was possible to characterize the nature of the materials through which the holes were drilled, and construct a cross-section at each location in the tunnel to represent the actual thickness of the lining and the significant geologic features that exist immediately behind the lining.
- A geologic study was carried out to determine the stratigraphic sequence at the site and to characterize the geology in the vicinity of the tunnel. A closer field inspection of the geology of the hillside was carried out to establish more precisely the relative location of the tunnel within the regional geologic sequence. In this way it was possible to establish the geology through which the tunnel was excavated, and into which the probe holes would have penetrated.
- Using the information obtained by the methods identified above, it was possible to construct a conceptual model of the tunnel in cross-section, and identify mechanisms controlling ground behavior around the tunnel.
- Then using the conceptual model of the tunnel environment, engineering calculations were carried out to estimate, within a range of probable parameter values, the "factor of safety" of the ground and the lining. That is, engineering calculations that simulated the geology, the tunnel geometry, and the height of rock between the tunnel and the roadways, were carried out to estimate of the ratio of material strength to the in situ state of stress. These calculations were carried out for a probable range of material properties to explore the issues and consequences of weathering of the rock and deterioration of the tunnel lining. The results were interpreted in the light of observations during the tunnel inspection.

¹ Recommended Decision on Docket Nos. C-00981956 and E-00116297, issued May 24, 2000

² Recommended Decision on Docket Nos. C-00981956 and E-00116297, issued May 24, 2000, Finding of Fact #5

The remainder of this report presents the information and evidence that was collected during this study, presents a summary of the engineering calculations that were carried out, and presents the reasoning that supports our conclusions on the stability of the tunnel.

TUNNEL GEOMETRY

The tunnel length has been previously reported as 2,464 ft from portal to portal³. It was measured approximately as 2,480-ft during this field study. It is laid out on an approximately 1,650-ft radius curve. The invert is located at approximately 847-ft MSL at the south portal. This elevation was assessed on the basis of benchmarks at nearby Philipston and Red Bank Station. Based on water levels in the ditches at least, the tunnel appears to grade north to south.

The cross-section of the tunnel at each portal was surveyed as part of this investigation. The results are presented in Figure 3. The tunnel shape is referred to as an inverted-U because the arch is cylindrical, the walls are vertical, and the invert is approximately flat. The results indicate that the tunnel is approximately 29.8 ft wide, and approximately 23.3-ft high from center of crown to invert. The elevations of each portal shown in Figure 3 are approximate, and are estimated to be more accurately represented at the south portal than the north portal.

The roadways of interest to this study are State Route 68 and State Route 2023. SR68 crosses over the tunnel approximately 820-ft from the north portal⁴ and at an elevation of approximately 1,350-ft MSL⁵. SR2023 crosses over the tunnel approximately 814-ft from the south portal and at an elevation of approximately 1,210-ft MSL⁶. Consequently, the height of overburden above the tunnel invert is approximately 503 ft and 363 ft for SR68 and SR2023, respectively.

GEOLOGIC SETTING

The geologic setting for the tunnel was established through a literature review and a two-day field reconnaissance. The results of these activities have been summarized in Figures 4 and 5.

According to the available literature, the bedrock at the site originates from the Pennsylvanian and Mississippian Systems. The Mississippian Period represents a transition from the prograding deltas of the Middle and Late Devonian Period to the cyclic alluvial and paludal environments of the Pennsylvanian Period. The rocks of the Pennsylvania System are predominantly clastic and contain subordinate amounts of coal and limestone. A section of the stratigraphic column in the vicinity of the site is presented in Figure 4.

Bedrock of the Conemaugh Group of the Pennsylvanian System occupies the highest elevations of the hillside above the tunnel. The Upper Freeport coal, which marks the contact between the Conemaugh Group and the Allegheny Formation is at approximate elevation 1,340-ft MSL to 1,360-ft MSL. The Upper Freeport coal has been extensively strip-mined in the area.

Numerous mineable coal units of the Allegheny Formation outcrop in the valley walls below the Upper Freeport coal. These include, in descending order, the Lower Freeport, Upper Kittanning, Middle Kittanning, Lower Kittanning and Clarion coals. The remains of what might have been a coal chute can be seen above the south portal. However, this could also have been constructed to control surface water. The Vanport Limestone, whose upper surface has been extensively

³ Recommended Decision on Docket Nos. C-00981956 and E-00116297, issued May 24, 2000, Finding of Fact #11

⁴ Recommended Decision on Docket Nos. C-00981956 and E-00116297, issued May 24, 2000, Finding of Fact #11

⁵ Recommended Decision on Docket Nos. C-00981956 and E-00116297, issued May 24, 2000, Finding of Fact #9

⁶ Recommended Decision on Docket Nos. C-00981956 and E-00116297, issued May 24, 2000, Finding of Fact #10

mapped, and serves as a prominent marker bed in the region, occurs above the tunnel at approximate elevation 1,060 MSL.

The District Mining Office of the Pennsylvania Department of Environmental Protection in McMurray was visited to obtain information on subsurface mining in the vicinity of the East Brady Tunnel. Based on a review of the information on record, there has been no subsurface mining over the East Brady Tunnel. Strip mines and/or deep mines are recorded a significant distance to the east of the tunnel in the, Lower Clarion coal, Upper Freeport, Middle Kittanning coal and Lower Kittanning coal seam. The thickness of the coal seams in the vicinity of the East Brady tunnel are considered to be too thin to be of commercial value to mining companies. The field reconnaissance of the hillside in the vicinity of the tunnel did reveal evidence of very small, unrecorded, entries into the coal seams, but these are likely to have been made by private individuals seeking coal for domestic use rather than for commercial gain.

The Brookville coal marks the contact between the Allegheny and Pottsville Groups. The Brookville coal outcrops on the hillside at approximate elevation 1000-ft MSL at the south portal and approximately elevation 1020-ft to 1040-ft MSL at the north portal. This difference in outcrop elevation between portals is probably the result of the Brady's Bend Syncline, a linear geologic depression in an otherwise flat lying geology. The axis of the syncline is aligned approximately N64E and crosses the tunnel in its central region at approximately right-angles to its axis, so that the bedding dips into the hillside at both portals.

The Pottsville Group is generally described as consisting of light to dark gray, fine-grained to coarsely conglomeratic sandstone with subordinate amounts of gray shale, siltstone, limestone, coal and under-clay. The bedding of the group is generally well developed, and the sandstone and siltstone beds are commonly cross-bedded. The bedding thickness ranges from less than an inch in shale to several feet in sandstone. Joints are moderately well formed with a moderate to high distribution. The spacing of the joints is generally wide (3-ft to 10-ft) to moderate (2-ft to 3-ft) in the sandstone beds and closely spaced (2-in to 2-ft) in shale. The patterns are generally regular open and vertical. All these geologic characteristics were observed during the field work.

The base of the Pottsville Group is marked by an erosional unconformity between the Lower Connoquenessing sandstone and the Mauch Chunk Formation of the Mississippian System. According to the literature, the base of the Pottsville is at approximate elevation 880 to 900 MSL at the north portal and at approximate elevation 820-ft MSL to 840-ft MSL at the south portal.

The Mauch Chunk Formation is dominantly shale and claystone with some sandstone and siltstone beds. The sandstone and siltstone beds are typically fine to medium grained and are cross-bedded. The shale and claystone beds are red and the sandstone and siltstone beds are usually gray, green, and greenish gray. The bedding of the formation is generally moderately well bedded, thin and flaggy. Joints are abundant and moderately well formed. The joints are regularly spaced at close (2-in to 2-ft) to moderate (2-ft to 3-ft) distances and are open and vertical.

Our own observations and geologic delineations suggest that the Lower Connoquenessing is the sandstone unit located immediately adjacent to the south portal, and the large overhang immediately above the portal is the base of the Upper Connoquenessing. The weathered shale and claystone bed immediately below the overhang is probably associated with the Quakertown Coal. These features are illustrated in the photographs included with the stratigraphic column in Figure 4. At the north portal, the rock types adjacent to the portal are largely claystone and shale, that we delineate to be part of the Mauch chunk formation. The overhangs high up on this

exposed rock face (see Figure 4) are considered to be at the base of the Lower Connoquenessing that was surveyed at approximately 915-ft MSL to 920-ft MSL. The bedding of the sandstone dips into the hillside at both portals suggesting that the stratigraphy takes the form of a shallow trough in section along the tunnel.

Our interpretation of these findings as they apply to a section through the hillside is shown in Figure 5. The assumed dip of the Upper and Lower Connoquenessing sandstones also takes into account the observations made in the probe hole drilling program described in the next section. It would appear that the tunnel was excavated under largely favorable conditions in blocky sandstone over much of its length, and in shale and claystone in the northern third. Bedding is almost horizontal, and joints are largely vertical, which is a favorable combination for stress-redistribution, and makes the existence of large, kinematically unstable, wedge shape blocks highly unlikely. Temporary support during construction would likely have had to address local instabilities of thin slabs of rock separating on bedding planes.

PROBE HOLE DRILLING

Close inspection of the tunnel lining was carried out with the aid of a program of probe hole drilling. The purpose of the probe holes was to make numerous penetrations of the liner and surrounding material that could be inspected visually. This provided information to characterize the nature of the materials around the tunnel, and infer the possible and likely loading conditions.

A probe hole is a 1.625-in diameter hole drilled using a hand-held pneumatic percussion drill, that is known in the industry as a "jack-leg". At cross-sections beneath each highway, eight holes were drilled at approximately 4-ft centers around the perimeter of the arch. Holes were generally drilled 10-ft long in order to penetrate the tunnel liner and the immediate zone of influence of the tunnel excavation. In addition, one hole was drilled through the side-wall at approximately 4-ft above the invert to verify the thickness of the lining in the lower elevations. Drilling was generally slow and "hard", indicating the presence of competent lining materials and competent rock beyond the limits of the observed tunnel lining.

After drilling was completed, each of the holes was visually inspected using a fiber-optic borescope. Using this tool it is possible to differentiate bedding contacts, changes in texture of side-wall materials, smoothness and regularity of hole diameter, concrete aggregate, open joints or voids, and solid-versus-broken material. Using these observations and observations made during hole drilling, reasonable inferences can be made regarding hardness or softness of materials, relative weathering, and the presence of groundwater. Pictorial logs of all the probe holes were then plotted collectively in cross-section so that an interpretation of significant and pervasive geologic features could be made. This plot is presented in Figure 6

In each of the holes we initially encountered a surprisingly significant thickness of sound concrete immediately behind the brick layer that is observed in the arch. The thickness of the concrete is generally 2.75-ft or more in the crown, 4-ft or more in the lower regions of the arch, and at least 6-ft thick at a point approximately 4-ft above the invert. We found that the brickwork in the arch is only a veneer, being only one course thick.

Outside of the concrete lining, we encountered what appeared to be unweathered sandstone beds and variably weathered shale or claystone beds. At Station 16+40 (measured from the south portal) beneath SR68, we found a considerable thickness of concrete (between 5-ft and 9.5-ft) which limited our observation beyond the liner. Generally no void was observed between the concrete and the excavation, and the rock was found to be either a hard, grainy material

interpreted to be unweathered sandstone, or a softer weathered shale or claystone. At Station 8+14, beneath SR 2023, we observed what appeared to be bedding separation, and debris immediately outside the lining, particularly on the right side. At both locations, the variability of materials encountered, and the relatively small thickness of beds of different rock types suggests that the material in the arch of the tunnel is the Quakertown bed between the Upper and Lower Connequenessing Sandstone beds. A study of the photographs of the surface exposures of these materials in the rock face at the south portal (Figure 5.) supports this interpretation.

These observations of the materials on the outside of the concrete lining would confirm that the tunnel was excavated through solid rock, and that the liner was constructed inside of the excavation perimeter. The lining was likely constructed by creating a form to the shape of the finished tunnel shape, placing a veneer of brick around the arch, and then pumping concrete into the space above the form. The evidence of broken and weathered rock immediately behind the lining suggests that the concrete did not completely fill this void behind the form, and some isolated air pockets remained at the end of construction. As time passed, the freshly exposed rock would have slowly weathered and pieces of weathered rock would have fallen into the void creating debris on the lining. Once filled, this void would not have provided space for further loosening of the rock mass, and further deterioration of the rock mass would have become confined, thereby diminishing the possibility of large-scale failure.

In general holes were dry to damp, and free water was only observed in three holes. At station 8+14, beneath SR2023, strong flows discharged from the lowest holes on either side of the tunnel immediately the holes broke through the concrete. This flow dissipated to a small run over a period of 5 to 10 minutes, and to a steady drip after 24-hours. No other free water was observed in the remaining holes at this station, despite penetrating what appeared to be bedding separations and other voids. At station 16+40, beneath SR68, water was only encountered in one hole, and here it entered the hole before the hole broke through the concrete, and disappeared back into the concrete before it was able to discharge into the tunnel. Nevertheless, it appears that the hydrostatic loading on the lining is generally low in magnitude, and localized.

The results of an unconfined compressive strength test (see Figure 7) on a 3-in diameter core recovered from the wall at Station 8+14, yielded an uncorrected failure stress of 7,582 psi, which is approximately twice the strength of concrete used in construction applications today. This would suggest that the quality of the concrete in the arch is excellent.

A visual inspection of the internal perimeter of the concrete lining over its entire length generally verified the observations made by others⁷. The brick damage depicted on perimeter mapping sheets were often found upon closer inspection to be largely surficial in nature and often simply staining. Some areas of surficial damage areas were not reported. However, with the exception of the deterioration of the portal structures and the lining immediately inside the north portal, the overall condition of the lining is considered to be "good-to-excellent", with a need for only minor maintenance work. No signs of liner overstress or cracking were observed that might signal a potentially unstable situation.

The observed deterioration of the arch at the north portal is not attributable to overstress, but rather is the result of prevailing water and weather conditions that are not experienced farther into the tunnel. Being on the north side of the hill, the portal does not benefit from the warming

⁷ Grau, R.H., Pappas, D.M. and Fisher, J.T. (1998). Overview Stability Assessment of the Brady's Bend Tunnel, Clarion County, PA. Report prepared by National Institute for Occupational Safety and Health, Pittsburgh, Research Laboratory for Allegheny Valley Land Trust.

of the sun, and as a result experiences greater ice formation and accumulation. It is firmly believed that a complete collapse of the liner at this location would not precipitate a collapse of the rock mass that is of such proportions as to threaten the highways above.

STABILITY ANALYSIS

The forgoing discussion has described site investigations that were carried out to gather information about the nature of the materials through which the tunnel passes, and to establish, at least through investigative evidence, the apparent state of stability of the existing tunnel configuration. To support these observations and interpretations, a series of engineering analyses have been carried out using finite element methods. The purpose of these calculations is to estimate the ratio of strength-to-stress for the components of the ground and liner system, and thereby, establish the factor of safety against failure under a range of possible conditions.

Two types of analysis were used for this study, one assumed the concrete lining to be an integral component of the ground system, the second assumed the arch to be free-standing. In the first type of analysis, the concrete lining is assumed to be constructed the instant the opening is excavated, so that the interaction between the in situ and constructed materials is optimized. However, based on our knowledge of construction methods in the early part of the 20th century, there would almost certainly have been a delay between excavating the opening and constructing the lining. Consequently, for much of its early existence, the lining may only have experienced self-weight and debris loads. For this reason, a free-standing arch analysis, in which the arch is treated independently of the rock mass, might be expected to provide a better representation of the arch behavior. However, this approach is not ideal because the conclusions are dependent on the relevance of the assumed boundary conditions. Consequently, the most realistic interpretation of the actual arch behavior probably lies somewhere between the two types of analysis.

The material properties that were used in the analyses are presented in Table 1.

INTERPRETATION OF RESULTS

Integral Model

The suite of analyses that were run for the integral model is summarized in Table 2. The first set of analyses was carried out for the excavation only, without the presence of the concrete lining. These analyses were carried out to assess the stability of the excavation under undisturbed and disturbed conditions. The results allow observations to be made concerning the need for the lining, and the potential for large scale collapse of the rock mass if the lining should fail. A comparison of the strength factor diagrams for high and low strength parameters, and disturbed and undisturbed materials in the arch and invert, is provided in Figure 8. The corresponding comparison of displacements of points in the model is provided in Figure 9.

As indicated earlier, the strength factor is the ratio of strength-to-stress, and is calculated for a multitude of points in the model. These data can then be contoured to highlight regions that have similar strength factors. The resulting diagram is referred to as a strength factor diagram. Using the color codes adopted for these presentations, areas colored in red and gray indicate zones in which the material strength is exceeded. For design, one would seek to minimize the extents of the areas colored in yellow.

In the displacement diagram the magnitude and direction of the movement of all the calculation points in the model are represented as arrows drawn to a scale that contrasts regions of greater and lesser relative movement. This presentation format not only enables the reader to identify those regions where the ground displaces slightly in response to the excavation (regions colored in red, purple and blue), but also allows the reader to assimilate readily those regions of the model that are apparently unaffected by the presence of the tunnel excavation (regions colored in gray). For all intents and purposes those points that experience imperceptible movements are effectively unaware that the void exists.

By studying the strength factor diagram and displacement diagram we can derive several important conclusions. The first is that the stresses induced in the tunnel invert are greater, relatively to material strength, than in the roof. This occurs because the flat bottom shape is not able to redistribute gravitational stresses as effectively as the circular arch. Correspondingly, the upward displacements in the invert are larger than the downward displacements in the arch, which is seldom recognized. The second conclusion is that the state of stress in regions located approximately a tunnel diameter or so away from the excavation appears to be unchanged by the presence of the tunnel excavation. This indicates that the status quo throughout a considerable proportion of the hillside is unaffected by the presence of the tunnel, and thus the conditions affecting safety of the highways at the ground surface are unchanged by the presence of the tunnel. The third conclusion is that lining fulfils an important and necessary function by enhancing the ability of the rock mass to distribute imposed stresses without becoming overstressed.

The strength factor diagram and the displacement diagram for the integral model incorporating the concrete arch are presented in Figures 10 and 11 respectively. These results clearly show that when the lining is present the displacements in the rock mass are considerably lower and the bulb of highly stressed rock (low strength:stress ratio) above the crown is smaller than had the arch not been present. Furthermore, the results show that the factor of safety in the strength of the arch materials is high even under "low" strength assumptions. This suggests that the arch is experiencing loads that are well within its operating capacity, and that the strength would have to decrease appreciably to precipitate a failure of the lining.

The likelihood of an appreciable decrease in strength occurring in the foreseeable future at points so far from either portal is considered extremely small. The most obvious process that can reduce the concrete strength appreciably is ice deterioration. No evidence of ice deterioration is present at the points in the tunnel below the highways that cross over, as the high concrete strength from the test core attests. The only sections of tunnel where ice deterioration is observed are at the north portal and a small region inside the south portal. Nevertheless, an analysis of the integral model was carried out using the test strengths for ice-deteriorated concrete in Big Savage Mountain Tunnel. This is considered to represent a "worst-case" scenario. In Figure 12, the results of this analysis are compared with the results of the analysis carried out using the "low" estimate for concrete strength. Both analyses assumed the "low" value of strength for the surrounding "disturbed" and "undisturbed" rock materials. Even under these significantly reduced strength conditions, the minimum stress-factor in the concrete lining is greater than 1.1. Hence these calculations support the conclusion that even under the remote possibility of an appreciable deterioration of the concrete lining, the arch would still be stable.

The foregoing discussion of results is for the conditions at Station 8+14, beneath SR 2023. The overburden stress at Station 16+40, beneath SR68, is slightly higher. The worst case run for the arch support at 8+14 was re-run using the overburden stress for Station 16+40. The results of the

two sets of runs are shown in Figure 13. As would be expected, the results show that the stresses induced in the crown increase, and the strength factor reduces below 2.0, but remains above 1.0, over a substantial portion of the arch. If the assumptions used in this analysis closely represented in situ conditions, one might expect to see some signs of arch distress at Station 16+40. However, the condition of the arch at 16+40 is excellent, and one might conclude that the "low" strength assumptions used in these analyses are conservative.

Free-Standing Arch Model

The results for the analyses using the Free-Standing Arch Model are presented in Figure 14. These results apply to both Station 8+14 and 16+40 since the loading is derived only from a debris pile and is independent of the overburden stress in the rock mass. A different computer program was necessarily used to run the Free-Standing Arch Model, and as a consequence the format of the program output is different from that shown previously. For the purposes of this discussion, the most critical strength factors were those associated with the shear stress in the concrete. Generally, the shear strength factors were found to be in excess of 3, as illustrated in Figure 14, when using the "high" concrete strength value, and in excess of 1.9 when using the "low" concrete strength value. The regions experiencing the high shear stress are on the inside surface at the "quarter-points" of the arch, and on the outside surface above the crown. In general the shear strength factor within the body of the arch is considerably higher than at the critical locations, even when the assumed height of the debris pile is 14-ft. These results confirm the overall conclusion made earlier for the Integral Model that the arch is experiencing loads that are well within its operating capacity.

The vertical displacements calculated using the Free-Standing Arch Model are comparable with, although slightly smaller than, those calculated for the Integral Model. However, whereas the movement of the arch is uniformly inward by the Integral Model, by the Free-Standing Model, only the central two quarters" of the arch move inward, the lower two quarters of the arch move outward. This apparent outward movement of the arch is calculated because the boundary condition of no horizontal movement is applied only up to the spring line. Above the spring line the calculation knows of no constraint on arch movement, whereas in practice these movements would likely be confined by the rock mass. The high shear strengths computed on the inside of the liner at the quarter points are directly attributable to the outward movement that is allowed above the spring line in the calculation. This difference illustrates the weakness and limitations of the Free-Standing Model and suggests that the results from the Free-Standing Arch model herein are conservative.

CONCLUSIONS

The evidence collected during this study indicates that the original tunnel was driven through solid rock and was lined with an unreinforced concrete lining varying in thickness between 3-ft in the crown and greater than 6-ft in the walls. The concrete was made using river gravel (i.e. aggregates are largely rounded rather than angular) with the coarse fraction ranging in size up to approximately 2-in or 2.5-in in diameter. The apparent absence of a significant gap between the outside of the concrete and the rock suggests that concrete was pumped into place, thereby filling the gap on the outside of the form. Consequently, at the time of construction, little void remained into which rock could displace, and little "air slack" existed for precipitating significant weathering of the freshly exposed rock units. Consequently, relatively little disturbance of the rock was observed during the site investigation immediately outside the concrete lining.

The results of the engineering analyses of the tunnel and its lining suggest that within the broad range of strengths typically associated with materials of the type found at the site, the concrete lining serves an important function by preventing overstressing of the claystone materials above the crown. By providing confinement to the surrounding rock materials the concrete lining enhances the ability of the rock mass to distribute imposed stresses around the opening, and leave the status quo unchanged throughout much of the hillside. The factor of safety of strength of the concrete materials generally exceeds 3 under the most likely representation of existing conditions. This applies even under the worst case scenario of a completely disintegrated zone of rock extending 14-ft above the crown of the tunnel, relying entirely on the strength and rigidity of the concrete lining for its support. Consequently, under current and foreseeable conditions, there would appear to be little likelihood of a complete collapse of the lining.

With respect to mechanisms that could alter this conclusion, one focusing on weathering and ice deterioration has been identified. Concrete is a material that is remarkably resistant to air temperature, and deterioration by weathering generally requires the assistance of a chemical agent, such as salt, and water. The tunnel lining now exists in a benign environment, and little deterioration through weathering is expected. Ice deterioration has already been a factor for 85 years, and the only areas that have been seriously affected are located at the north portal. Consequently, it is highly unlikely that weathering and ice deterioration could precipitate a complete collapse of the concrete lining beneath the highways in the foreseeable future.

In the unlikely event that a collapse does occur, the initial volume of rock that would fall into the tunnel would be limited to the weathered claystones and shales immediately above the crown. Our calculations have shown that the strength factor of the sandstone overlying the disturbed claystone unit exceeds 5 even under the most conservative of assumptions. A collapse of the claystone materials would unlikely propagate immediately upward beyond the first competent sandstone unit since this would act as a stable beam that would "bridge" the collapse zone.

But let us suppose that the more competent rocks do not "bridge" and material continues to fall, the collapse process will eventually stabilize itself for the following reason. The falling material will "bulk" when it lands (i.e. the volume of broken material would be greater than the original volume of unbroken material as a result of increased porosity), and a point will be reached when the volume of the "bulked" material equals the available void space. At this point, the displacement of additional broken material is prevented. Typical bulking factors used in the mining industry range between 1.4 and 1.5. If we consider the very conservative and unlikely scenario that a cylinder of rock, 20-ft in diameter (i.e. approximately 2/3 of the diameter of the tunnel) collapses through the tunnel lining, and the collapse propagates upward through the competent sandstone units, it can be shown that the collapse would stabilize itself between 237-ft and 286-ft beneath SR-68 and between 97-ft and 146-ft below SR-2023. This is illustrated in Figure 15. Consequently, this calculation would further indicate that even under an extreme and unlikely scenario, the tunnel does not pose a threat to the highways that cross over it.

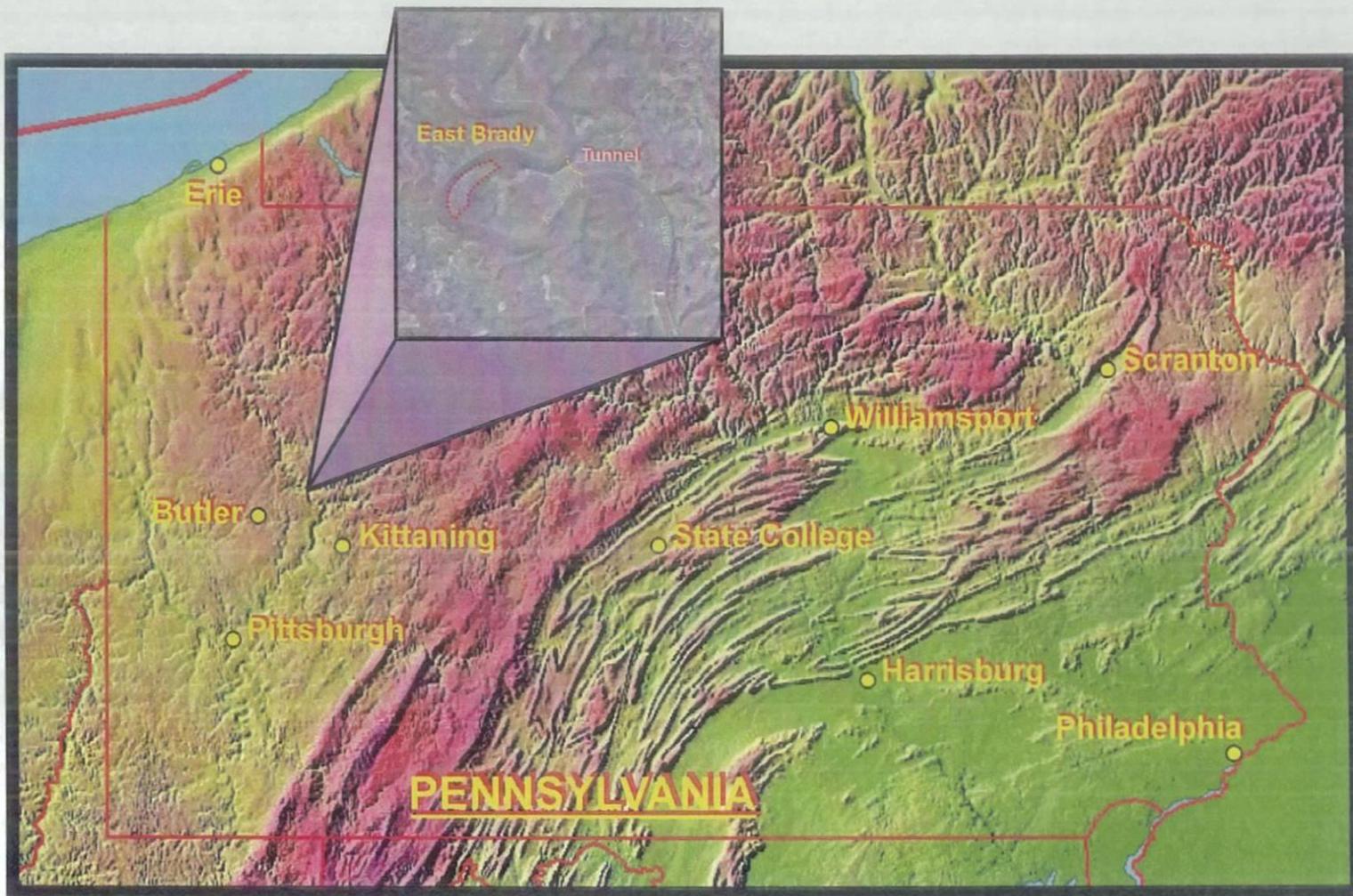
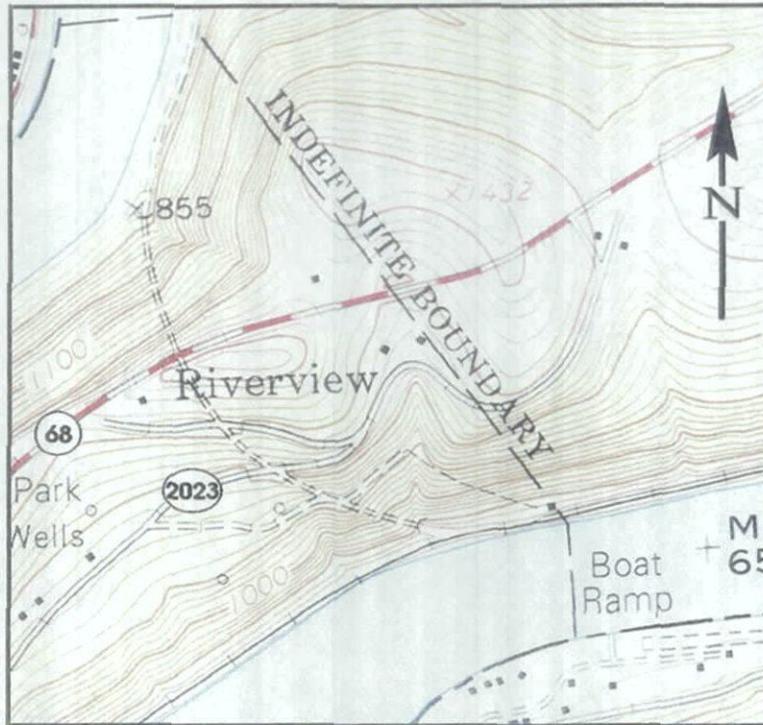


Figure 1
 Location of the East Brady Tunnel
 East Brady Tunnel, PA





Map taken from USGS
7.5 minutes series
East Brady Quadrangle



USGS aerial photograph
courtesy of Microsoft
Terraserver. Taken
April 11, 1993.

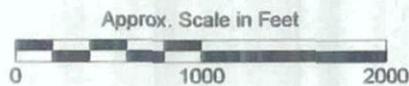
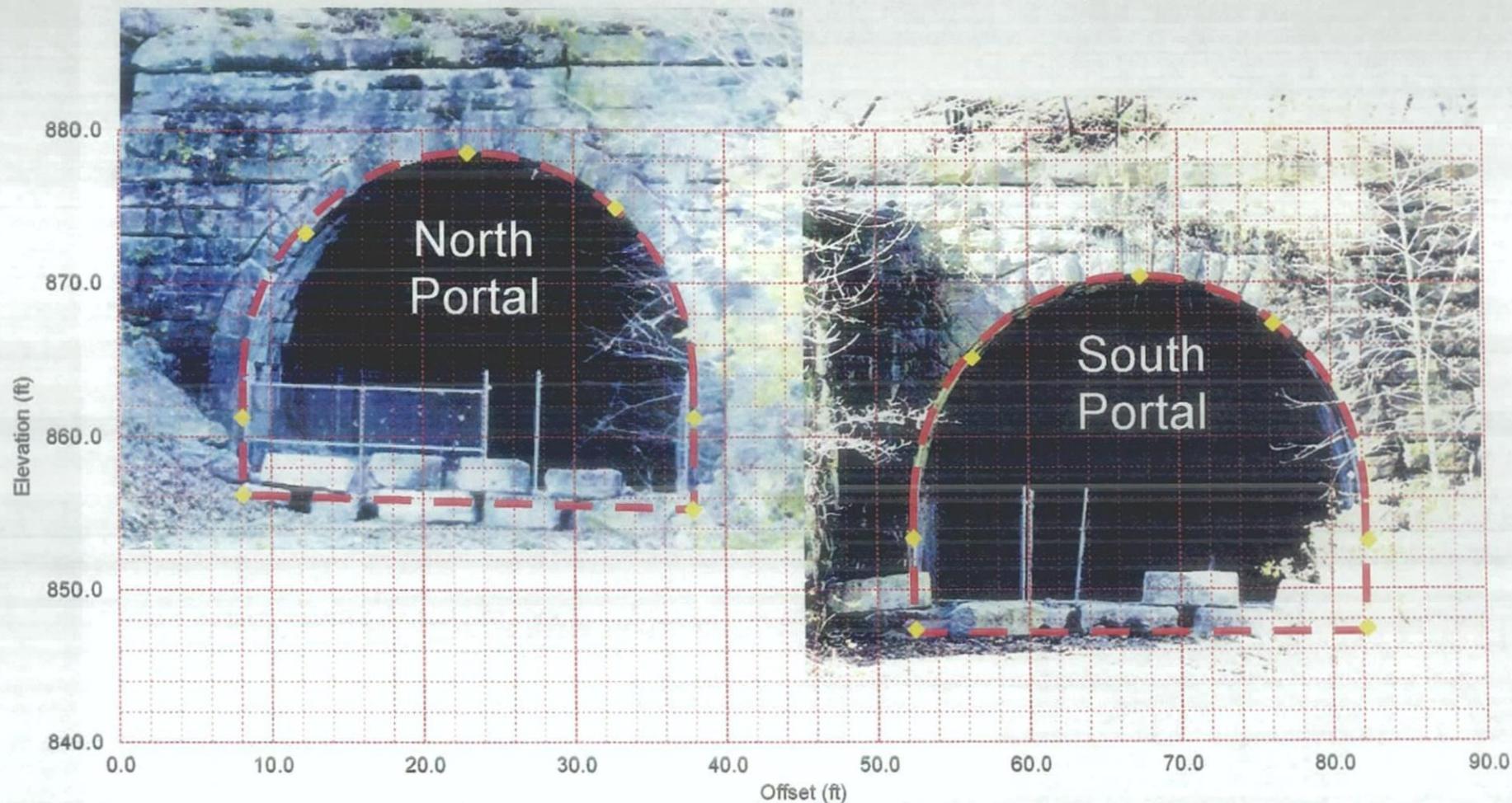


Figure 2
Site Map and Aerial View
East Brady Tunnel, PA



LEGEND

- ◆ Surveyed location
- - - Idealized cross section.
29.8-ft wide by 23.3-ft high

NOTES

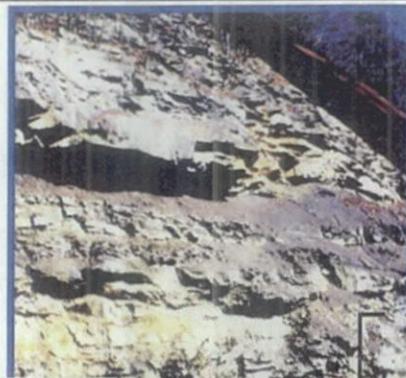
1. Tunnel length approximately 2,464 ft (reported by others)
2. Invert elevation at south portal approximately 847 ft
3. Invert elevation shown at north portal is inferred from East Brady Quadrangle map



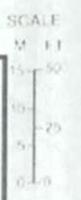
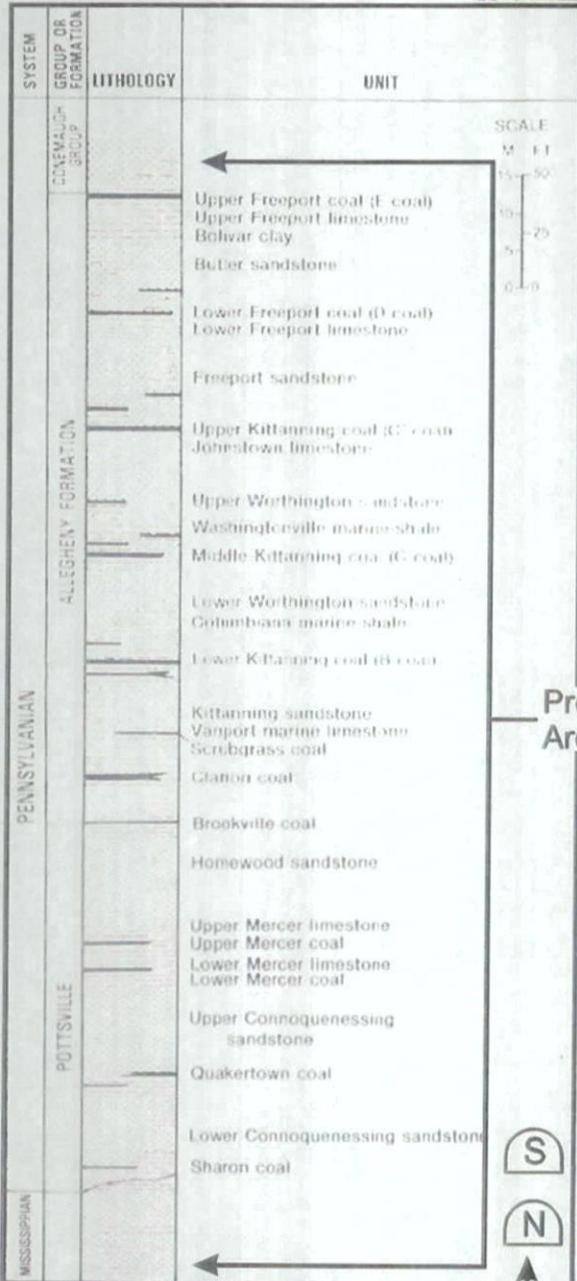
Figure 3
Scaled View of Each Portal
East Brady Tunnel, PA

Upper Connoquenessing sandstone

Quaker Town coal underlain by shale and sandstone interbeds



GEOLOGIC STRATA AT SOUTH PORTAL

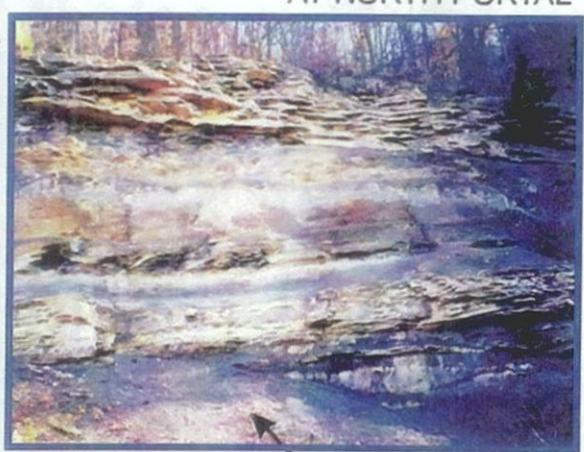


Project Area

Lower Connoquenessing sandstone

Lower Connoquenessing sandstone

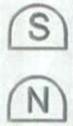
GEOLOGIC STRATA AT NORTH PORTAL



Mauch Chunk formation. Shale and claystone interbedded with sandstone and limestone

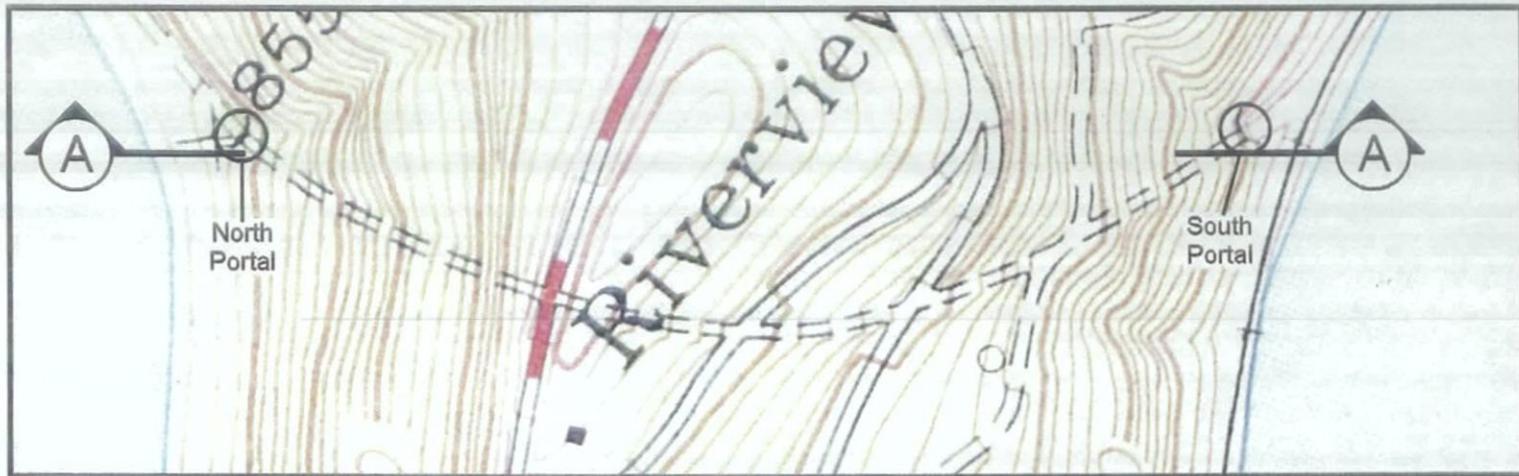
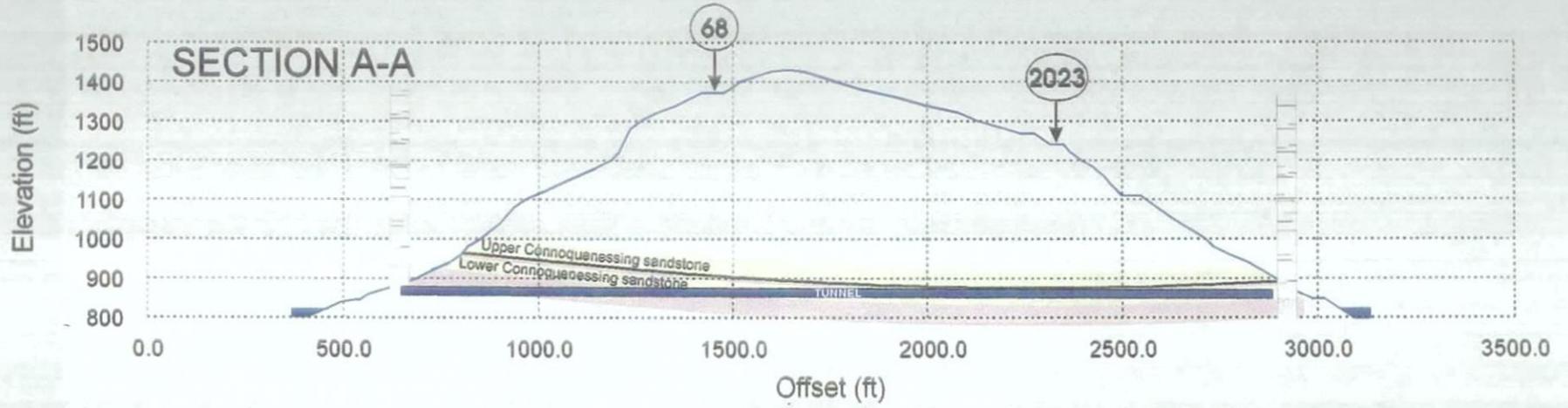
Talus accumulation at toe of slope

Relative setting of North and South Portals



Stratigraphic column taken from: "The Geology of Pennsylvania", Charles H. Shultz, ed.

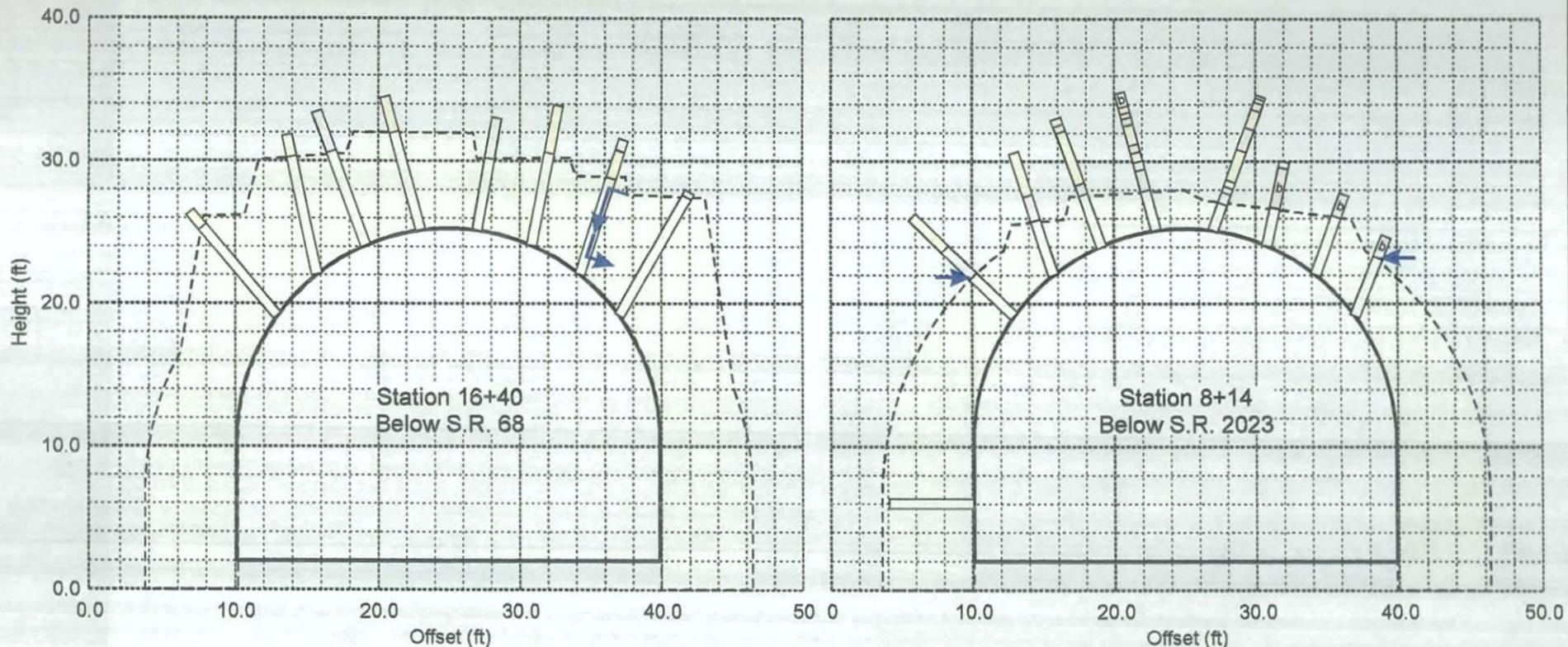
Figure 4
Geologic Setting
East Brady Tunnel, PA



Map taken from USGS 7.5 minute series
East Brady Quadrangle



Figure 5
Geologic Section
East Brady Tunnel, PA



LEGEND

- | | | | |
|--|--|--|---|
| | Smooth hole indicative of hard material, and patches of color indicative of aggregate (Concrete) | | Irregular oversized hole caused by highly broken hard rock, containing possible separations |
| | Smooth hole indicative of hard material, smooth and grainy texture (sandstone or hard shale) | | Void, indicative of separation |
| | Irregular oversized hole indicative of soft and/or weathered or decomposed rock (shale, claystone, or weathered rock debris) | | Observed running water |



Figure 6
Interpretation of Probe Hole Surveys
East Brady Tunnel, PA

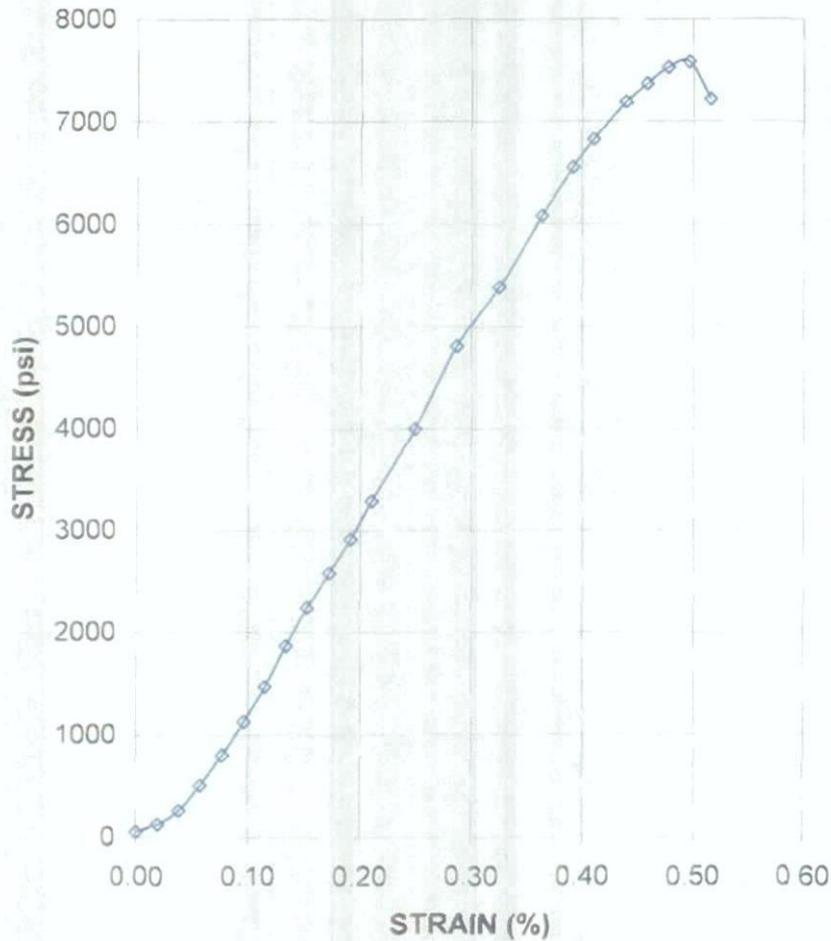


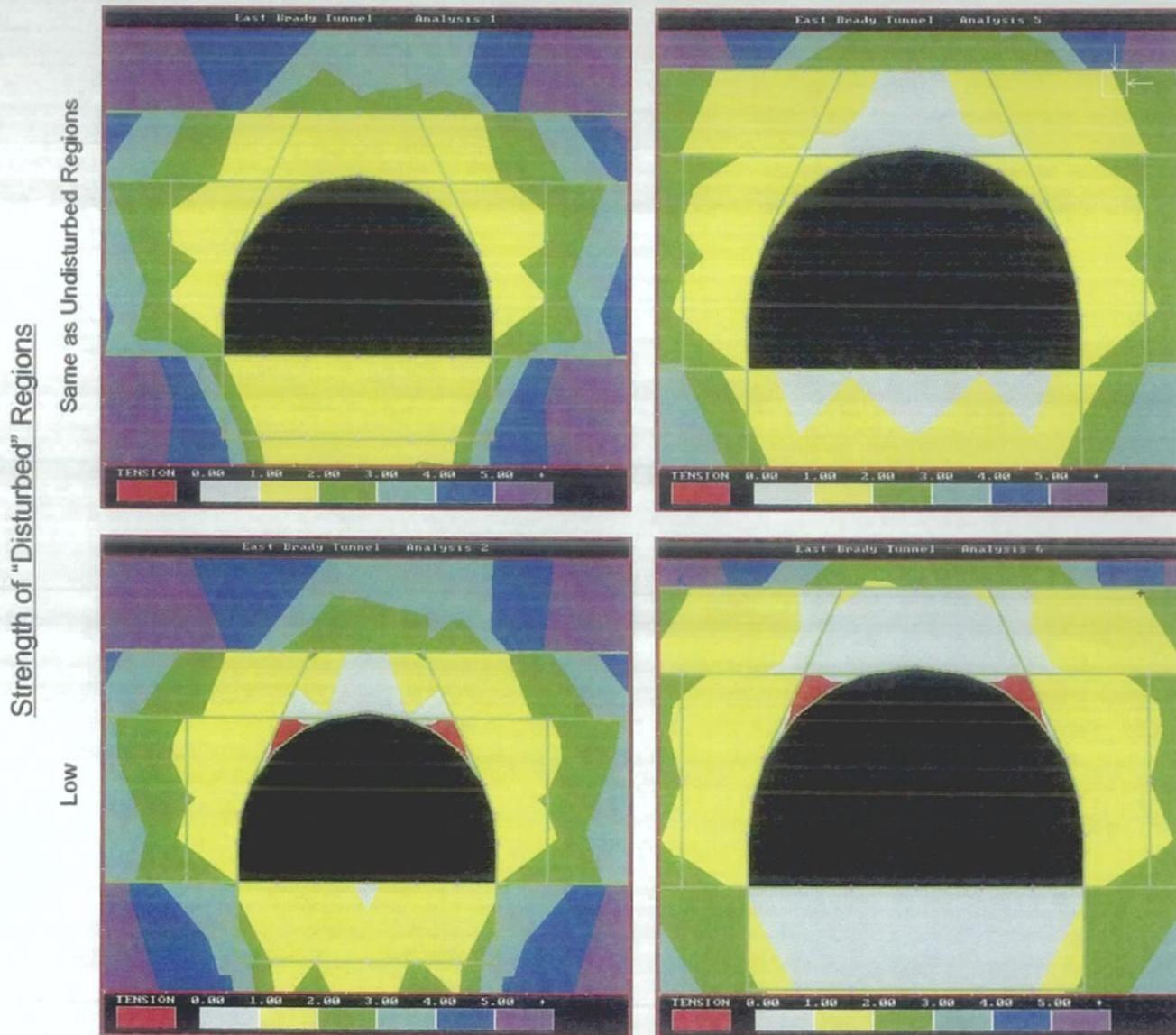
Figure 7
Unconfined Compressive Strength Test
Results for Concrete Core from Sta. 8+14
East Brady Tunnel, PA



Strength of "Undisturbed" Regions

High

Low



Strength of "Disturbed" Regions

Same as Undisturbed Regions

Low

Figure 8
Strength Factor for Tunnel Excavation
East Brady Tunnel, PA



Strength of "Undisturbed" Regions

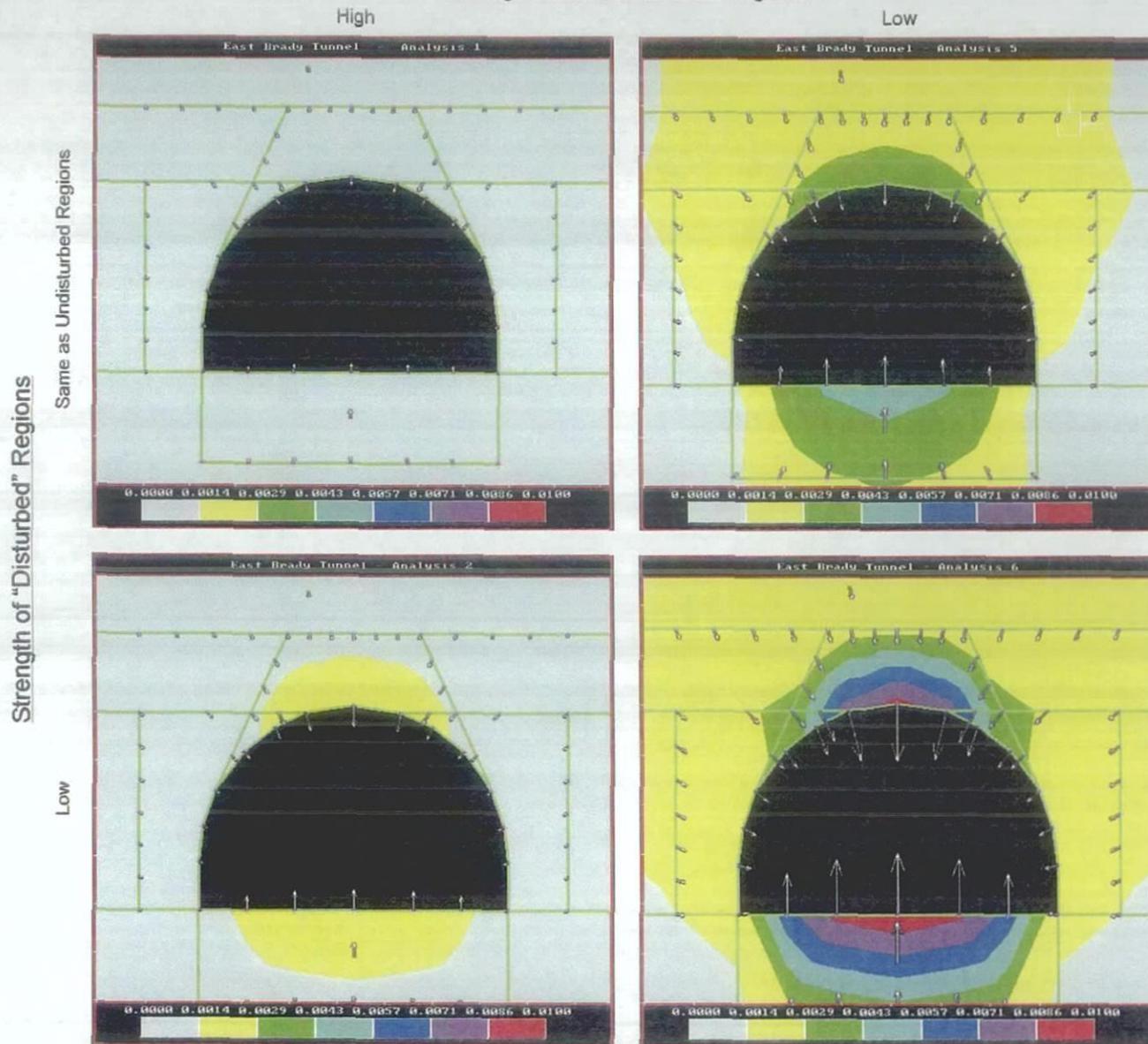


Figure 9
Displacement Diagrams for Tunnel Excavation
East Brady Tunnel, PA



TABLE 1

Material Properties for Finite Element Analysis

Material	Unit Weight kN/m ³	Rock Mass State	Hi/Lo Range	Hoek-Brown Strength Parameters						Young's Modulus MPa	Poisson Ratio
				m _i	UCS MPa	GSI	m _b	a	s		
Sandstone	26.1	Undisturbed	Hi	19	95	70	6.508	0.5	0.0357	30822	0.2
			Lo	19	30	60	4.553	0.5	0.0117	9740	0.2
	26.1	Disturbed	Hi	19	95	55	3.809	0.5	0.00674	12998	0.2
			Lo	19	30	45	2.665	0.5	0.00222	4107	0.2
Siltstone	26.0	Undisturbed	Hi	9	55	70	3.083	0.5	0.0357	23452	0.2
			Lo	9	14	60	2.157	0.5	0.0117	6653	0.2
	26.0	Disturbed	Hi	9	55	55	1.804	0.5	0.00674	9890	0.2
			Lo	9	14	45	1.262	0.5	0.00222	2806	0.2
Shale	24.7	Undisturbed	Hi	5	35	70	1.713	0.5	0.0357	18708	0.2
			Lo	5	7	60	1.198	0.5	0.0117	4705	0.2
	24.7	Disturbed	Hi	4	15	55	1.002	0.5	0.00674	5165	0.2
			Lo	4	4	45	0.701	0.5	0.00222	1500	0.2
Claystone	25.3	Undisturbed	Hi	4	10	70	1.37	0.5	0.0357	10000	0.2
			Lo	4	7	60	0.959	0.5	0.0117	4705	0.2
	25.3	Disturbed	Hi	3	5	55	0.802	0.5	0.00674	2982	0.2
			Lo	3	1.4	45	0.561	0.5	0.00222	887	0.2
Concrete	23.6	Test sample	Hi		52					34218	0.1
		ACI (4000 psi)	Lo		23.44					24856	0.1
		Big Savage Lo	V.Lo		11.38					15964	0.1

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TABLE 2

Summary of Material Property Combinations Used in Finite element Analyses

Material	Rock Mass State	Hi/Lo Range	EBTRUN1	EBTRUN2	EBTRUN3	EBTRUN4	EBTRUN5	EBTRUN6	EBTRUN7	EBTRUN8	EBTRUN9	EBTRUN10	EBTRUN11	EBTRUN12	EBTRUN13	EBTRUN14	EBTRUN15	EBTRUN16	EBTRUN17	EBTRUN18	EBTRUN19	EBTRUN20	EBTRUN21	EBTRUN22	EBTRUN23
			Sandstone	Undisturbed	Hi	X	X	X	X						X	X	X			X	X				
	Disturbed	Lo					X	X	X	X	X									X	X				
		Hi		X		X																			
		Lo						X		X	X	X	X	X				X	X						
Shale	Undisturbed	Hi	X	X	X	X						X	X	X			X	X							
	Disturbed	Lo					X	X	X	X	X										X	X			
		Hi		X		X									NOT USED	NOT USED									
		Lo						X		X	X	X	X	X			X	X	NOT USED		X	X			
Claystone	Undisturbed	Hi	X	X	X	X						X	X	X			X	X							
	Disturbed	Lo					X	X	X	X	X										X	X			
		Hi																							
		Lo		X		X		X		X	X	X	X	X			X	X			X	X			
Concrete	Test sample	Hi			X	X							X	X											
	ACI (4000 psi)	Lo							X	X							X	X							
	Big Savage Lo	V.Lo									X										X				
Maximum displacement (mm)			1.38	2.83	1.03	2.2	5	9.8	3.86	7.73		7.62	1.03	5.84			1.05	5.84		1.34	1.05				

Note: Runs EBTRUN18 and EBTRUN19 used uniform surcharge equivalent to Station 16+40, beneath SR 68.
 The remainder used the geometry configuration for Station 8+14, beneath SR 2023

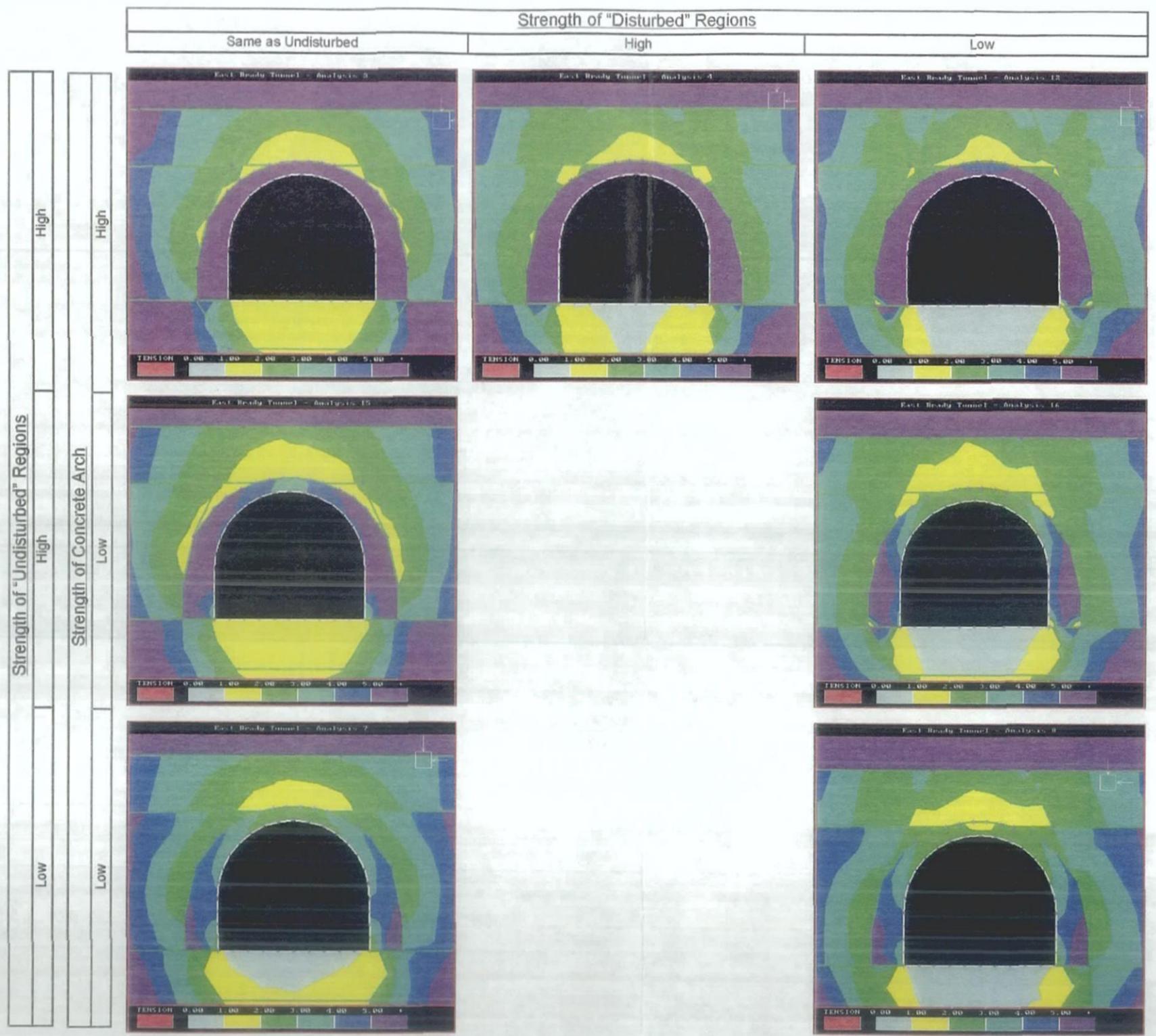


Figure 12.CDR

Figure 10
Strength Factors for Integral Arch Model
East Brady Tunnel, PA

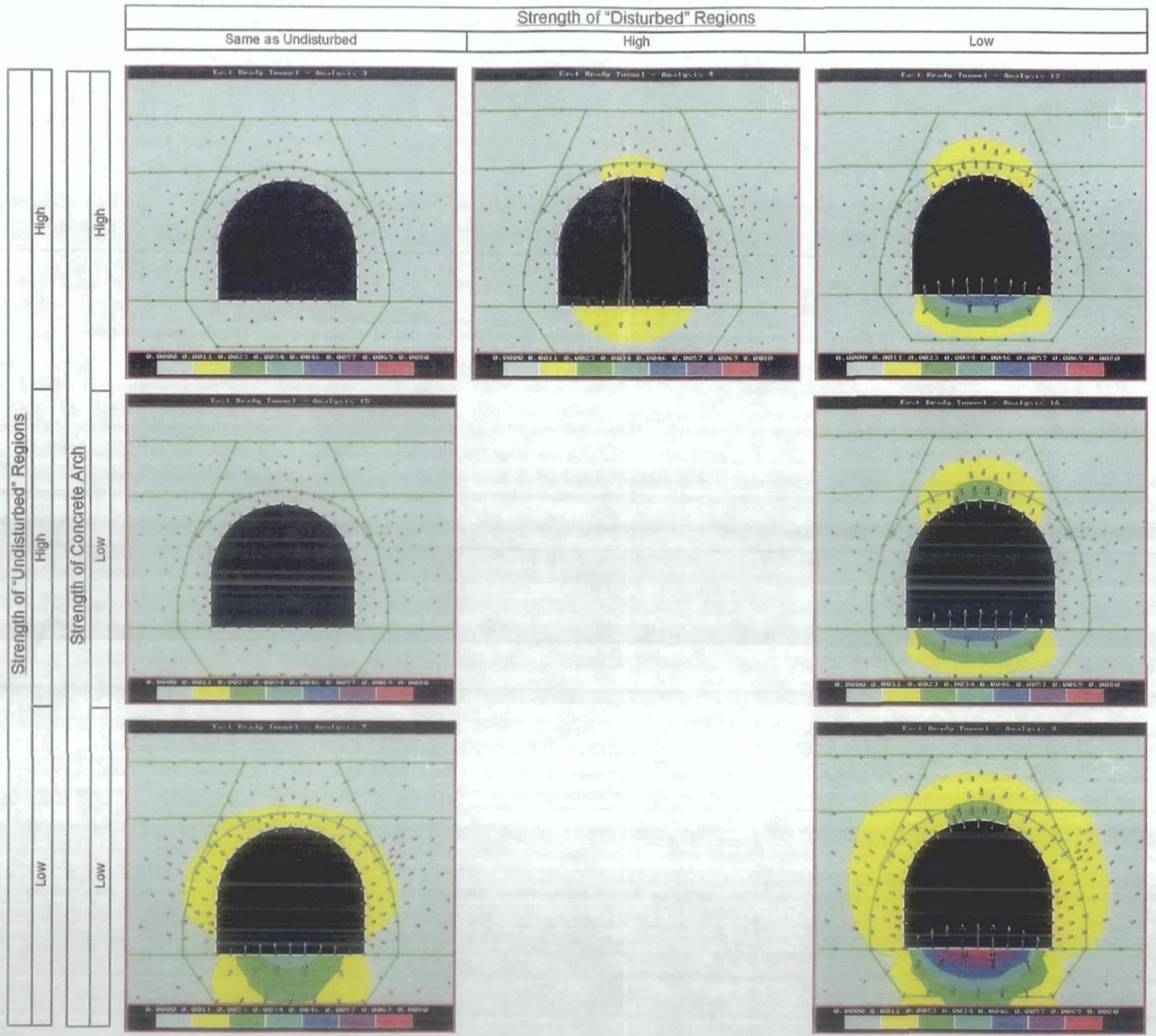
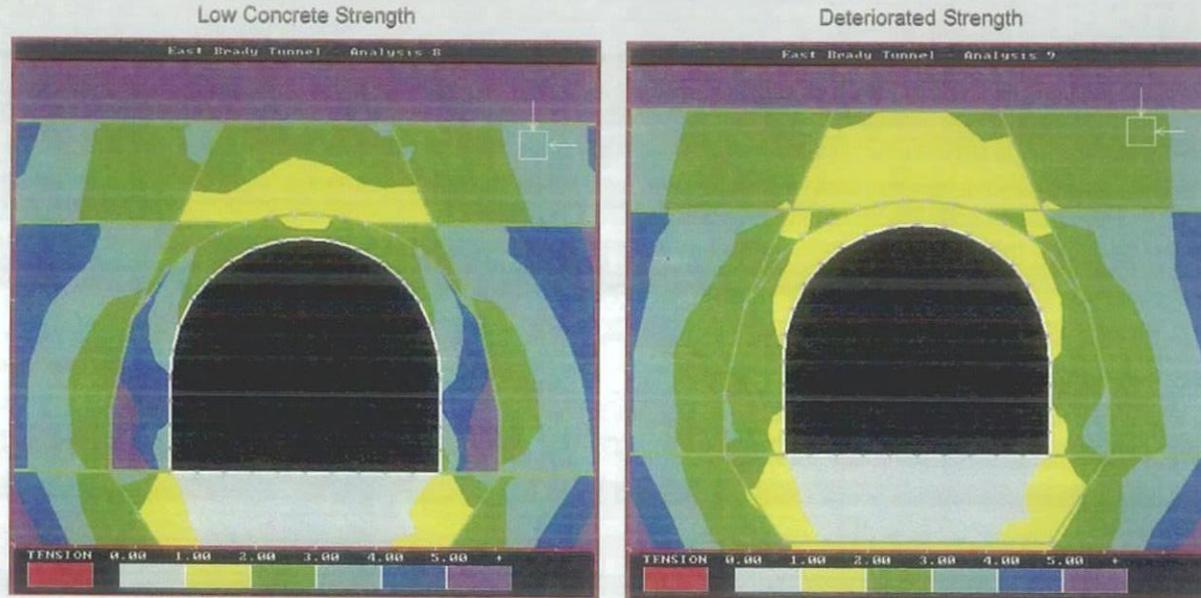


Figure 13.CDR

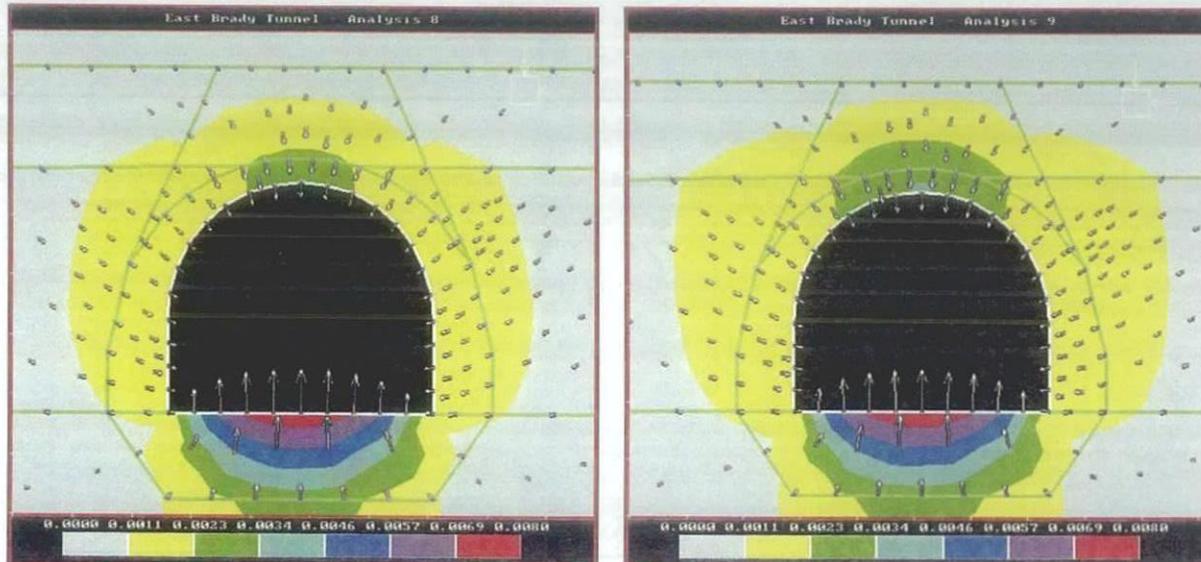
Figure 11
Displacement Diagrams for Integral Arch Model
East Brady Tunnel, PA

Strength of Concrete Lining

STRENGTH FACTORS



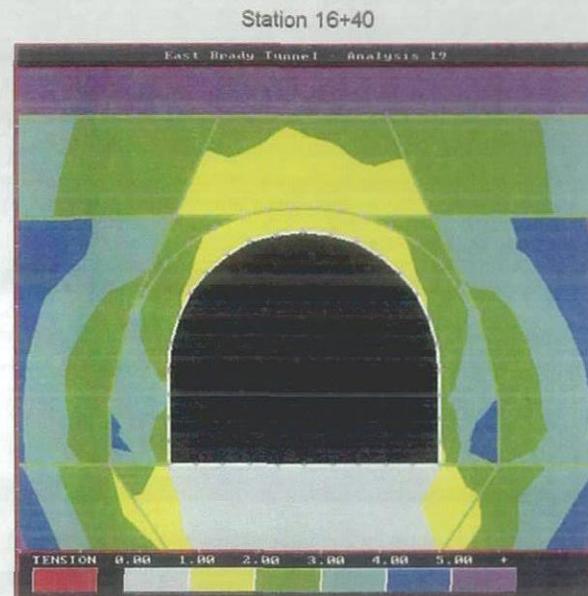
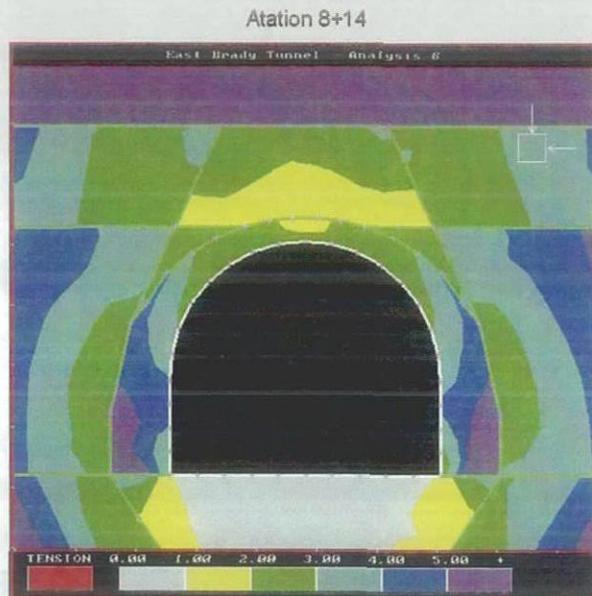
DISPLACEMENT DIAGRAM



NOTE
Analyses using "low" rock strength in "disturbed" and "undisturbed" regions

Figure 12
Comparison of Liner Stability
under Deteriorated Strength
East Brady Tunnel, PA

STRENGTH
FACTORS



DISPLACEMENT
DIAGRAM

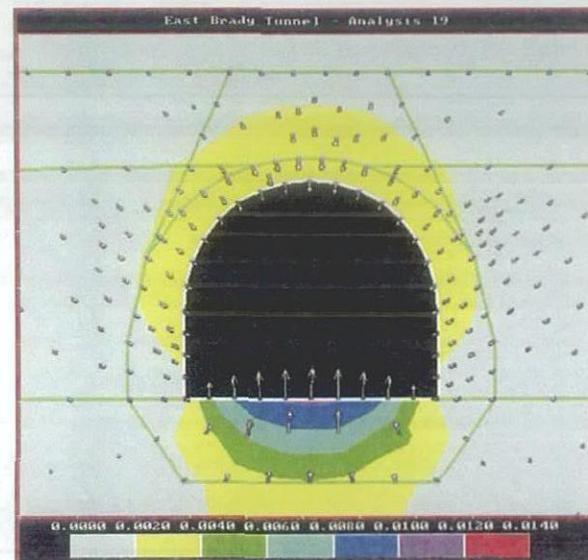
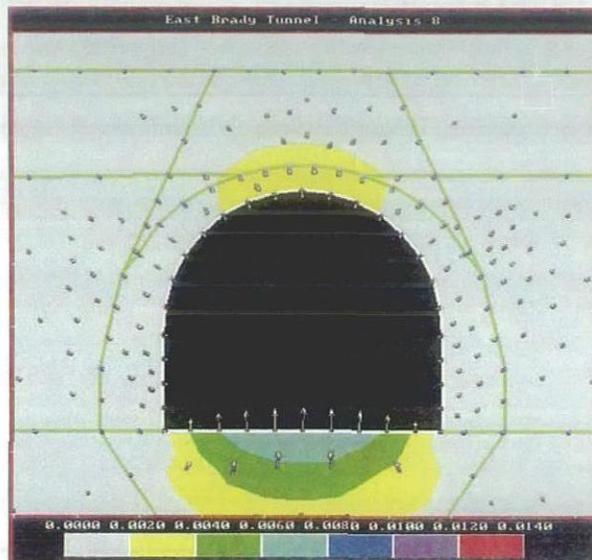


Figure 13
Comparison of Liner Performance
at Stations 8+14 and 16+40
East Brady Tunnel, PA



NOTE
Analyses using "low" rock strength in
"disturbed" and "undisturbed" regions

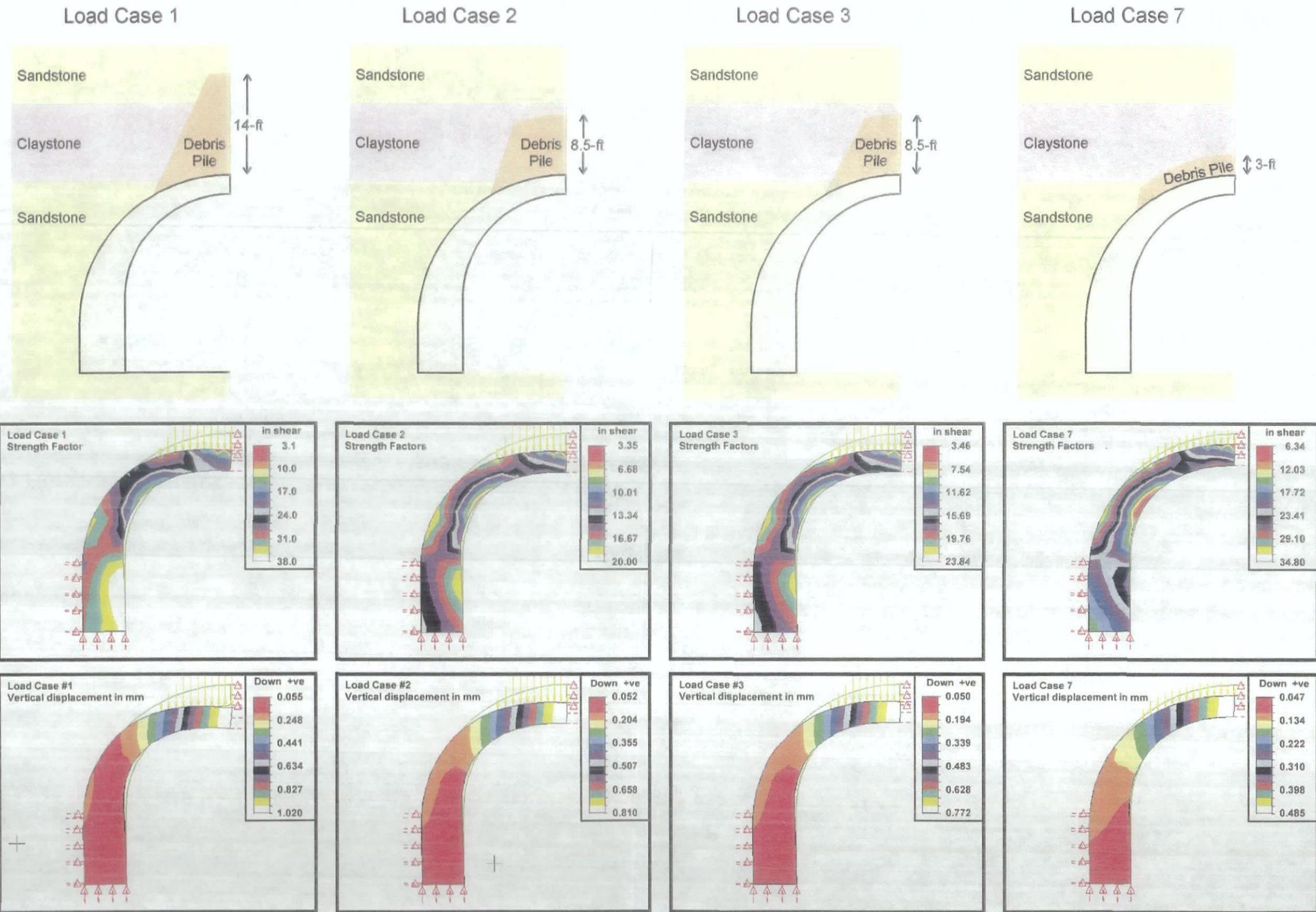


Figure 14
 Summary of Finite Element Results for Free-Standing Arch
 East Brady Tunnel, PA

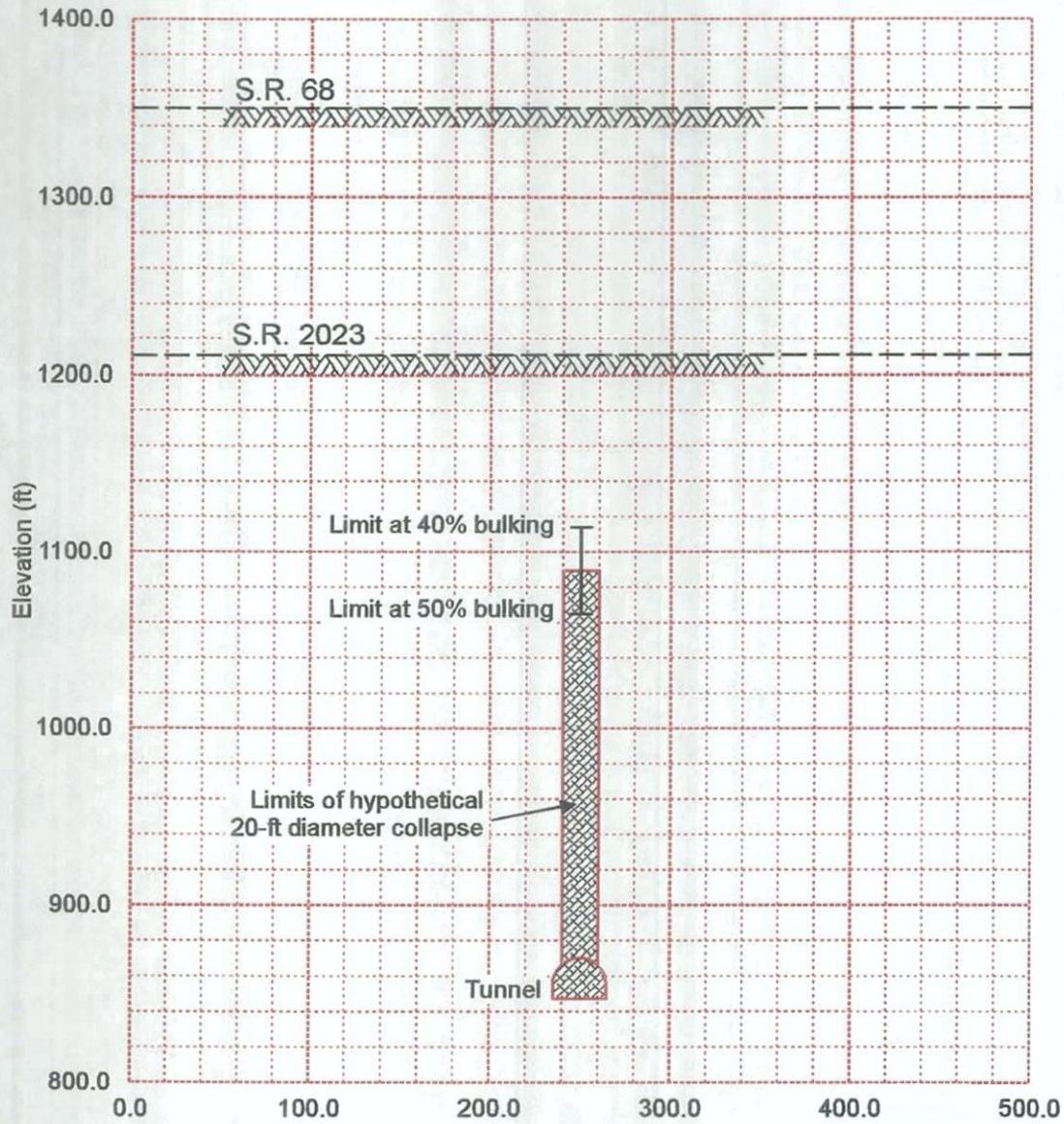


Figure 15
 Limits of Hypothetical
 Tunnel Collapse
 East Brady Tunnel, PA



E. Brady Tunnel
Drainage Investigation
C-00981956
A-00116297

DOT # 3
LW 6/1/01 B/L

On December 1, 2000 I, John S. Blose, did traverse from the area at Station 171+00 on State Route 68 to the north portal of the East Brady Tunnel. A logging road goes from the above location to the southwest about 400 feet where a staging area was located for storing and loading logs that were harvested from the hillside. A road then went north from the staging area curving down and around the area logged. All this activity was well above the tunnel and extended only about 1/4 of the way down the slope of the mountain. From the point of the logging road the slope was very steep. At the north end of the area two small gullies descended very steeply, to an area of much less slope. The flatter area extended around the mountain to the south and west

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DOCKETED DOCUMENT FOLDER

As I moved downward and to the south there were small coal lumps and also browning of surface areas where water had flowed. Also present was a very flat cartway at the most southern end there were coal lumps, and soft shale, where possible mining activity once occurred.

DOCUMENT FOLDER

At the end of the gully was one of the drainage sluices. The sluice consisted of a bituminous coated galvanized steel half pipe. The sluice was nearly filled with debris, leaves, soil, and gravel for about 35 feet. Then it was about 1/4 to 1/3 full for remainder of its length and 2 sections were separated and partly down the slope toward the southwest (river). Drainage would not affect the tunnel as is, if the sluice were functional. A deadfall (tree) has probably depressed the flow line or even broken the connectors. This location is likely

h. correct ish

where collected water escapes and seeps or flows under the lower sluice where it could go into the tunnel through the hole in the top (arch).

At the point where the top sluice begins there is timber cribbing as a support and so that any water flow would be diverted into the sluice. Since the sluice is full of debris, very little water gets into it. Water seeps along the outside of the sluice pipe and down the slope to the gully that contains the lower sluice pipe.

Another small channel adds water from springs and from former mining activity. Water flows to and under the top sluice pipe about 20 feet from the origin. This drainage then flows down the steep slope and under the lower sluice pipe toward the hole in the tunnel or into the bottom sluice pipe.

The bottom sluice has timber cribbing at the point where it starts. A boulder is directly in front of the sluice opening (same type of half round metal pipe as top sluice). Probably only a small amount of flow gets into the sluice, most is diverted due to debris (likely from heavy rains and flooding in August 1980 East Brady flood). About $\frac{1}{4}$ of the depth of the sluice has debris along its length. No flow exits at the end. Most of the flow/seepage is below or under the sluice. This drainage goes into the hole in the tunnel.

To make the drainage pipe (sluice) functional several things need to be done. The first is to clean the sluices from end to end. The next is to stop any leakage from the pipe by repairing and sealing. Then raise

low (spots) areas and support the pipe. Clean debris from cribbings. Extended the sluices so drainage cannot flow toward the tunnel.

The top sluice should also be altered to enable the drainage flow from the spring area to enter by a similar pipe from the right side at the point where it now flows/seeps under the existing sluice. Cribbing could be installed to direct the flow into the connector pipe that would be supported by cribbing at either end. This would prevent any settlement from causing a possible problem at the location where flow would enter into the original sluice.

At the bottom sluice clean out the pipe, level and repair where needed, then seal joints, etc. Clean out the cribbing dam at the origin of the sluice and contour to provide proper drainage into the sluice. Provide support along bottom of sluice across the opening that now goes into the tunnel. Put a heavy duty polyvinyl liner inside the existing sluice if it cannot be repaired. Extend the sluice to beyond the point where drainage would affect the tunnel. This would require about 40 feet of new pipe. Support the new pipe across the tunnel to the slope toward the river.

The effort achieved from doing what is described would take care of a majority of water that now enters into the tunnel. As there is rock strata exposed above the hole location and it is very likely that some water flow would continue.

There are no drainage ditches or pipes that outlet toward the north portal of the tunnel from State Route 68.

Since the drainage sluices were constructed to divert water from the portal area, it is very likely that the work was done by or under direction of the railroad (Conrail or its predecessor Pennsylvania Railroad Co.).

From this scenario the presumption is that the railroad (or successor) or other party under an agreement be assigned the responsibility of repairing, extending, and maintaining the subject drainage sluices.

The cost of performing the needed repairs is difficult to determine since only approximate quantities are calculated.

Clean pipes - 120 feet at \$20.00/foot = \$2400.00

Repair- Place liner - 200 LF x 4 feet at \$8.00/LF = \$1600.00

New pipe - 80 LF at 24" $\frac{1}{2}$ round ACCM pipe at \$50.00/foot including bedding, support, placement = \$4,000.00

Cribbing - 80 LF 6" x 6" treated timber or tie at \$4.00/LF = \$320.00

80 LF support timber (treated) = \$960.00

((4) 10" x 4" x 20" at \$12.00/LF)

Clean existing cribbing timber diversions (including contour drainage)

(2) at \$500.00 each = \$1000.00

Total = \$10,280.00

There has been no positive contact with either Conrail or Norfolk Southern Corp. to determine who constructed the drainage system at the north portal of the East Brady Tunnel.

The investigation continues and will continue as it is the understanding of the Department of Transportation that the railroad will investigate further the drainage and responsibility of such.

To stop all drainage including subsurface water, the hole in the tunnel must be repaired. The cost could approach \$30,000.00 to accomplish.

It is not in the interest of the Department of Transportation to do the work or to pay for such work.

East Brady Tunnel
Drainage Study
C-00981956
A-00116297

A review of construction plans was made on the following roadway construction plan/right-of-way condemnation plan for LR 214, Section 8, located in Clarion County, East Brady Borough, Brady and Madison Townships. The construction stations are station 80+00 to station 321+80 and the length is 22770.71 feet or 4.31 miles. The plan was signed June 9, 1965 by Governor William W. Scranton. Of 27 plans sheets the following sheets are attached: 1,2,3,10,11,12,13, and 14.

The investigation began at the scenic look out at construction station 146+50 left. All drainage before this location flows away and is below any elevation that could flow toward the tunnel portal.

Drainage on left side is channeled to a cross pipe at construction station 159+30 from a parallel ditch station 148+30 to station 162+70. This picks up all drainage that could flow to the tunnel.

The roadway begins a horizontal curve to the right at construction station 157+66.32 to station 177+89.10. This puts the roadway into a superelevation that directs water away from the north side of the roadway. Both superelevation and an upgrade through a cut direct water away from the north side of the highway. In addition a storm sewer from station 162+92 right side to station 168+98 right collects water from crosspipes that pick up drainage from drop inlets on the left side (north).

The roadway grade changes to a downgrade at about station 172+50, so to this point the water would flow to back stations. A turnwater ditch begins at station 174+50 per the plan; however, the location is a drive into a private residence at the present time. The area was altered by filling in station 175+50 to 182+50 left. The end wall at station 180+04 left was replaced by a drop inlet. The structure was built by placing pre-cast sections on top. The depth is approximately 25 feet to 30 feet from top of inlet to flow line. The drainage from station 174+50 flows toward the inlet left of station 180+04 and then across and below the roadway on the right side.

The area that could possibly drain toward the tunnel is from station 173+00 to station 174+00 left and would be the shoulder and the graded area to the left of this location. The area is about 100 feet in length and average width of 30 feet. It should drain to the tail ditch and away from the tunnel.

The conclusion of the afore mentioned information is that no drainage was designed to exit to the north side of the highway in the area of the north portal of the East Brady Tunnel.

Field investigation done on December 1, 2000 verifies that in fact no drainage was altered to exit or flow toward the north portal of the railroad tunnel.

Only a very small area of drainage could even flow toward the north portal, this is because of a private drive at station 174+25 left side.

It is the consensus of the Department of Transportation that no drainage from the roadway of SR 68 affects any condition of the railroad tunnel at its north portal.

John S. Blose
District 10-0 Grade Crossing Administrator

East Brady Tunnel
Drainage Study
C-00981956
A-00116297
Photo Description

The following is a summary explanation of photographs taken December 1, 2000 while making a drainage study of the area of East Brady Tunnel in Brady Township, Clarion County.

- No. 1 The photo first presented is one picturing the area along the Allegheny River Northward from a scenic lookout on State Route 68 (Legislative Route 214) at construction station 146+50 LT. The railroad area is in the center right of the photo. It is indicated by the break in trees that then is out of view as it is obscured by trees on the down slope from the viewing area as it follows closer to bottom of photo. The photo was taken 3-18-99.
- No. 2 The photo is from State Route 68 at construction station 175+00 looking back on left side of highway. Looking at end of cut and curve of highway. Drainage flows away from flat area of center of photo to direction of station 175+00 (ahead).
- No. 3 Photo from State Route 68 station 173+50 left ahead. Drainage flows ahead to an inlet 130 feet left of station 180+04. Area filled in to contour for dwelling. Changed after roadway was built.

- No. 4 Photo of gully above tunnel toward top of mountain. Rock is at area where drainage sluice is constructed, At end of sluice where drainage starts.
- No. 5 Photo of gully above tunnel to right of top sluice toward area that has coal lumps, clay, shale, etc. indicating coal activity in past (long past). The horizon center is about location where tunnel is located below State Route 68 (station 166+10 left) Segment 0100, Offset 0444.
- No. 6 Photo of top sluice. Sluice start at wood timber cribbing (top center). Sluice is full of washed in debris for 30 feet to about location of deadfall (center of photo). Water seeps around or under sluice. Area beyond cribbing has coal lumps, etc., indications of coal mining.
- No. 7 Photo of cut at north portal shows a coal seam in structure of mountain above the tunnel along the cut for the railroad.
- No. 8 Photo of bottom (lower) sluice from just beyond area of hole in tunnel roof. Note large rock just at end of sluice at its beginning. Also visible is cribbing to left of sluice. End of sluice is beyond hole. Also no flow out of sluice. Seepage under and along sluice pipe into hole in tunnel.
- No. 9 Photo looking down into hole in tunnel roof from sluice side.

- No. 10 Photo up through hole in roof. Note water falling over rock into tunnel (bottom center right) see also on (bottom left) edge of stone arch and vertical cracking in stone. Rock out crop visible through hole.
- No. 11 Photo above fencing posts to top of arch and through hole just inside north portal. Timber cribbing (three rows) are visible and top edge of sluice is visible. Water flow drips into tunnel from rock or concrete edges around hole in top of arch.
- No. 12 Photo of damage to fencing and gate at north portal. Wheel tracks of hi-lift are visible as marks on material and bucket edge marks also are visible. Large stone along entrance of portal were not traversed. Vehicle came through tunnel from south end of tunnel.

COMMONWEALTH OF PENNSYLVANIA



PROJECT NO. S-797(2)

DISTRICT	COUNTY	TOWNSHIP	BOROUGH	ROUTE	SECT.	APPL.	TOTAL SHEETS
FAS	10-0	CLARION	BRADY MADISON	EAST BRADY	214	8	* 27

* ALSO TWO (2) SHEETS OF ROADSIDE DEVELOPMENT AND THIRTEEN (13) SHEETS OF SOIL PROFILE

OVER SIZED DOCUMENTS

DEPARTMENT OF HIGHWAYS

DRAWINGS DESIGNATING FUTURE LOCATION AND WIDTH AND AUTHORIZING CONDEMNATION OF RIGHT-OF-WAY AND CONSTRUCTION

OF

LEG. ROUTE NO. 214 SECTION NO. 8

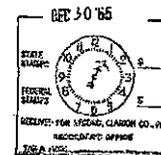
IN CLARION COUNTY

CONSTRUCTION From Sta. 80+00.00 To Sta. 321+80.00 Length 22,770.71 Ft. 4.31 Mi.
CONDEMNATION From Sta. 78+75.00 To Sta. 321+80.00

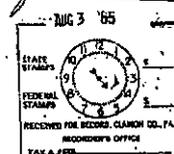
ALSO

CONSTRUCTION & CONDEMNATION LEG. ROUTE NO. 16061 SECTION NO. 1 FROM STA. 0+00.00 TO STA. 12+00.00

THIS PLAN PREPARED PURSUANT TO SECTIONS 219 AND 210 OF THE ACT OF JUNE 1, 1945, P.L. 1242, AND SECTION 402 (b) (3) OF THE EMINENT DOMAIN CODE OF 1964



Re-recorded in the office for the recording of deeds, etc. in CLARION County, Pennsylvania, in Plan Book 5 Page 4 to show revisions on sheets 1-12-18 and 25.
Witness my hand and seal of office July 26th Day of December A.D. 1965
Richard S. Miller Recorder



COMMONWEALTH OF PENNSYLVANIA
COUNTY OF DAUPHIN SS
Before me, a notary public, personally came Henry D. Lacey, Secretary of Highways, who acknowledged that the within official plan was duly revised on Dec. 14, 1965 and desired that the revised Site Plan be re-recorded.
Witness my hand and notarial seal Dec. 14, 1965
Henry D. Lacey Notary Public
My commission expires May 14, 1969

COMMONWEALTH OF PENNSYLVANIA
COUNTY OF DAUPHIN, SS
On this 7th day of June, A.D. 1965, before me, a Notary Public, personally came Richard S. Miller, Secretary of the Department of Highways of the Commonwealth of Pennsylvania, who, in due form of law, acknowledged the within plan, comprising Twenty-Seven (27) separate sheets or sections to be an official plan of the Department of Highways of the Commonwealth of Pennsylvania and desired that the same be recorded as such, in accordance with the provisions of Sections 219 and 210 of the Act of June 1, 1945, P.L. 1242, as amended.
Witness my hand and notarial seal this 7th day of June, 1965

DESIGN CRITERIA
HIGHWAY CLASSIFICATION — CLASS 2
DESIGN SPEED — 60 M.P.H.
PAVEMENT SECTION — 2-12' LANES
SECTION WIDTH — MIN. 44' MAX. 48'

Revised to show the changes of:
Addition of Temporary Required Right of Way Line on Right of Sta. 179+05 to Sta. 179+67, original Position of House, Add Property Lines and General etc.; also to change Required Right of Way Line for Connection to East L.R. 214 (Right of Sta. 243+10 on Left from Sta. 115 to 242), and show Condemnation of House; also shown on sheets 12-18 and 25
Approved Dec. 14, 1965
Henry D. Lacey ASSESSOR CHIEF ENGINEER

THE ESTATE TO BE ACQUIRED BY THE COMMONWEALTH ENCOMPASSES A SURFACE EASEMENT UNLIMITED IN VERTICAL DIMENSION, EXCEPT IN THOSE AREAS, IF ANY, WHERE THE DETAIL PLAN DESIGNATES AN EXCEPTION.
RECOMMENDED April 30, 1965
Walter Stephens DISTRICT ENGINEER
RECOMMENDED June 7th, 1965
Richard S. Miller ASST. CHIEF ENGINEER
APPROVED June 7th, 1965

Recorded in the office for the recording of deeds, etc. in and for the County of CLARION Pennsylvania, in Condemnation Book No. 4 Vol. 4, Page 5.
Witness my hand and Seal of office this 3rd day of August, A.D. 1965.
Richard S. Miller Recorder
Received from Richard S. Miller

PHOTO NO 1



PHOTO NO 2



PHOTO NO 3



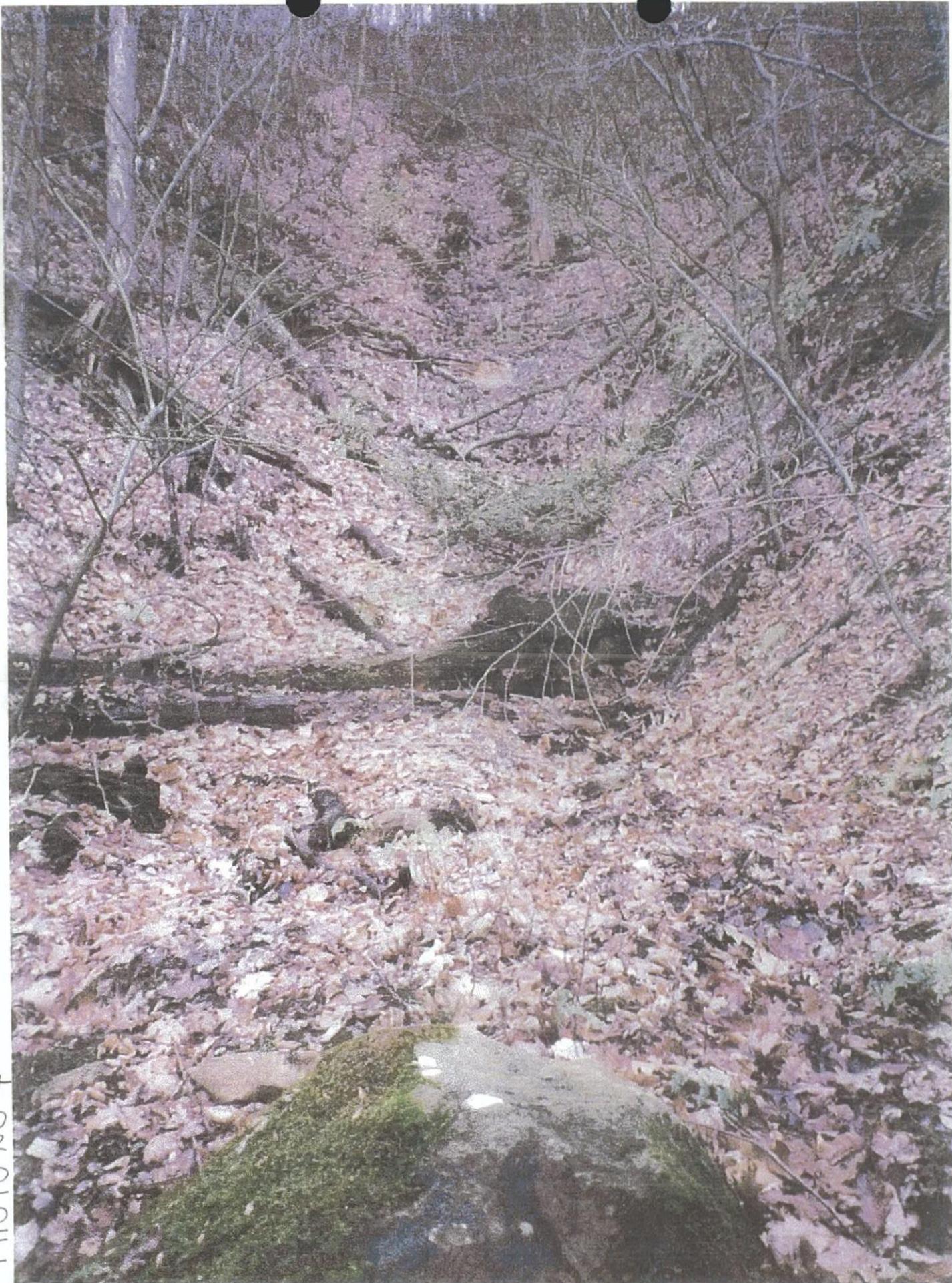


PHOTO NO 4

PHOTO NO 5



PHOTO NO 6



PHOTO NO 7



PHOTO NO 8



PHOTO NO 9



PHOTO NO 10



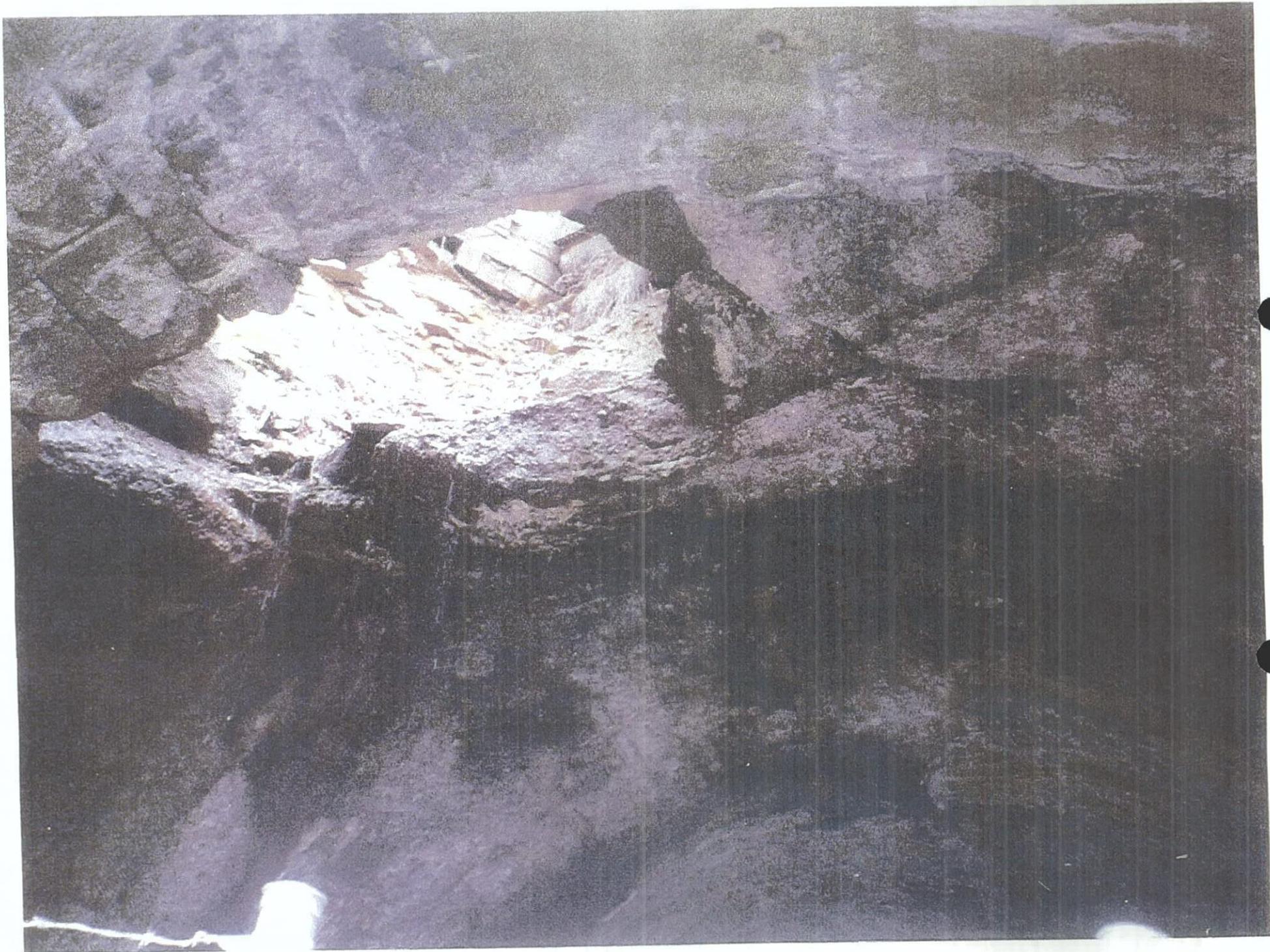


PHOTO NO 11

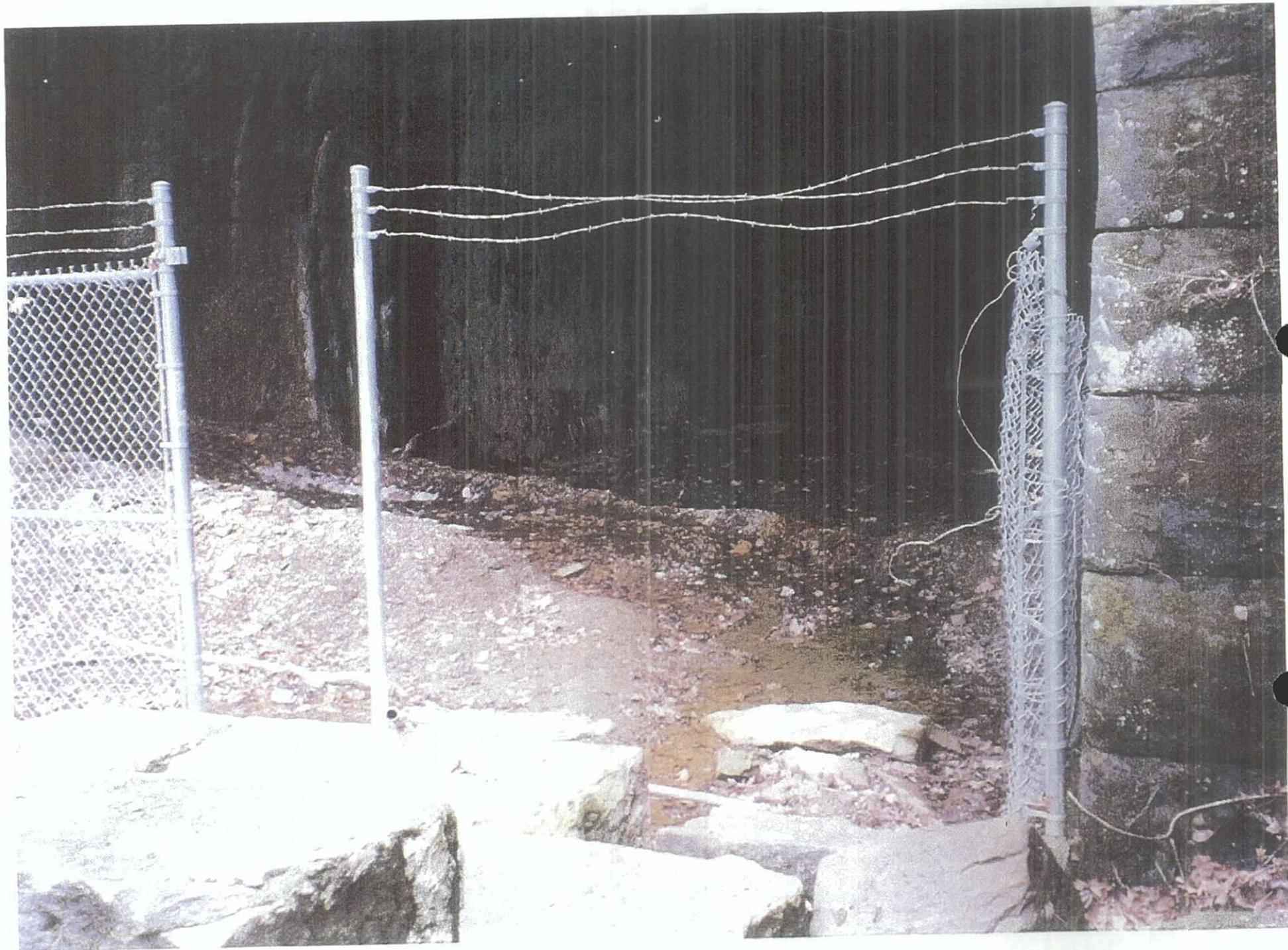


PHOTO NO 12

C-00981956 BRADY TWP CLARION COUNTY
EAST BRADY TUNNEL PHOTOS - MAY 9, 2001



NORTH PORTAL
LARGE CONCRETE
BLOCKS PUT ON TOP
OF LARGE STONE

DOCUMENT
FOLDER

DOCKETED

NORTH PORTAL
AREA OF CUT JUN 29 2001



NORTH PORTAL EXPANDED
AREA OF VIEW

Handwritten signature
6/7/01

C-00981956
A-00116257

DOT#4

C-00981956 EAST BRADY
TUNNEL PHOTOS MAY 9, 2001

SOUTH PORTAL
LARGE CONCRETE BLOCKS
PLACED ATOP LARGE STONE

DOCUMENT
FOLDER



SOUTH PORTAL
WOODEN SLUICE
ABOVE TUNNEL

DOCKETED

JUN 29 2001



SOUTH PORTAL
VIEW FROM INSIDE

RPO Bch 6/7/01

A-00116297

C-00581956

DOT# 5



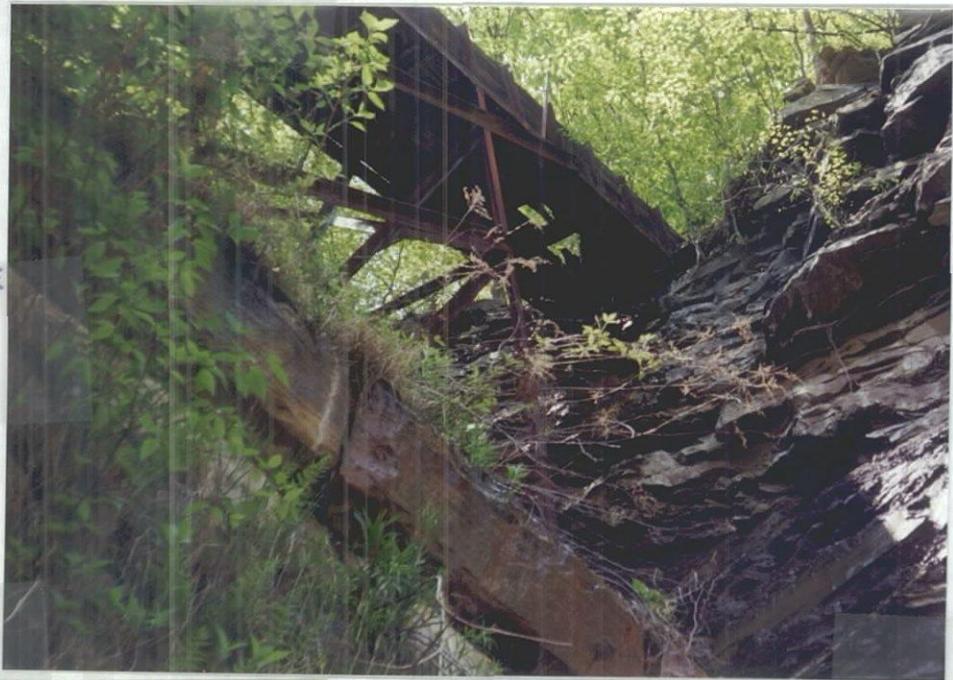


C-00981956 EAST BRADY
TUNNEL BRADY TWP
CLARION CO. PHOTOS
MAY 9, 2001

SOUTH PORTAL
SLUICE - STEEL SUPPORTS
ROCK CUT ABOVE TUNNEL

DOCUMENT
FOLDER

SOUTH PORTAL
VIEW UNDER SLUICE
NOTE WOODEN PLANKS
ROTTED - HOLES VISIBLE



DOCKETED

JUN 29 2001



SOUTH PORTAL
VIEW FROM TURNOUT
TOWARD SOUTH PORTAL
MORE LARGE STONES
AND CONCRETE BLOCKS

RNB PSH 6/7/01
A-00 116397 C-00981956

DOT # 6