3.0 GEOLOGY

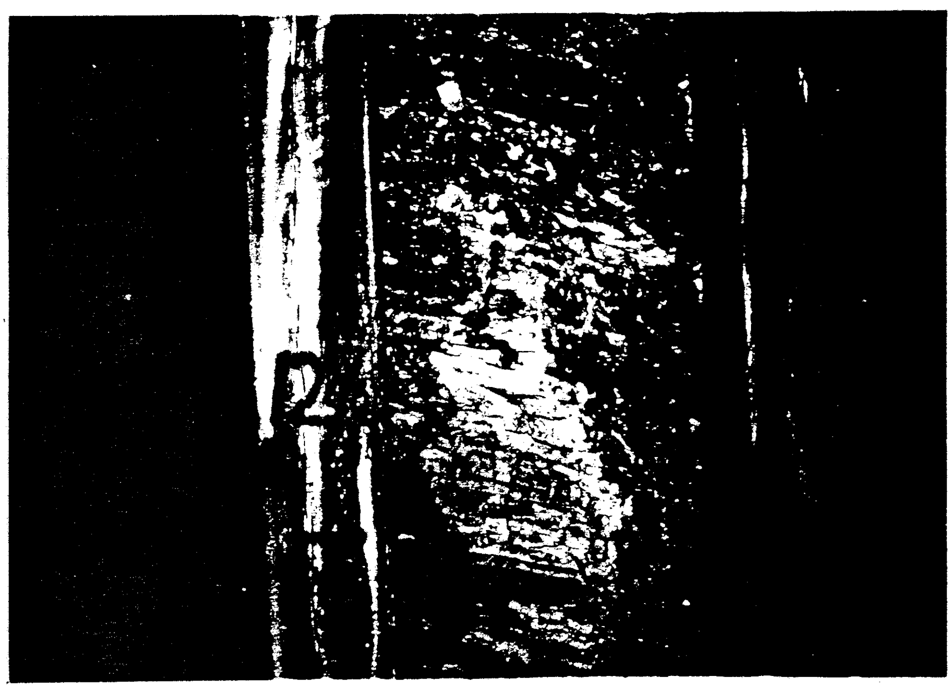
The tunnel is constructed in the Sequatchie and Rockwood Formations. These units are described in the literature as a calcareous silty shale and an interbedded sandstone and shale respectively. The contact between these formations occurs in the brick lined section of the tunnel and thus was not observed.

Exposed rock (Sequatchie Formation) in the tunnel sidewalls is primarily limestone with occasional calcareous shale interbeds. Photos of typical rock exposure are shown in Figure 5. The limestone is grey, hard, fine grained, and unweathered. Joints are infrequent and no weathering was observed. Bedding dips measured in the tunnel range from 17° to 20° and dip directions range from 312° to 315°. The dip direction is approximately parallel to the tunnel alignment. Water inflows are generally small (<5 gpm) and in some areas have resulted in a precipitate forming (presumably $CaCO_3$) on some rock surfaces and tunnel support. The interbedded shale is primarily dark red in color and is locally calcareous. The shale weathers on exposure and small/minor rockfalls have occurred as a result.

Thicknesses of the beds in the limestone range from 1/2 in. up to several feet. Typically beds are about 8 in. thick and can be traced for ten's of feet. A Rock Mass Rating (Reference 1) of 50 was estimated for the limestone exposed in the tunnel.

Rock exposed in slopes at the east portal (Sequatchie Fm) is moderately to highly weathered calcareous shale with shale interbeds. Localized sloughing has occurred due to weathering. Slope angles range from 80° to 85° for the rock cuts and from 30° to 45° for the more weathered rock/soil slopes.

Figure 5
Typical rock exposed
in tunnel



The slopes at the west portal are composed of interbedded sandstone and shale of the Rockwood Formation. Weathering of the shales has resulted in minor sloughing of the near vertical slopes.

Slope stability at the portals is not expected to be significantly influenced by rock structure. Existing and proposed cut slopes are oriented such that the adverse condition, where bedding dips into the excavation, does not occur.

Generally, two failure modes must be considered in the evaluation of underground openings in rock. One involves sliding or fall out of rock blocks bounded by existing structural features, such as bedding and joints. The other involves stress failures in intact rock. With the shallow overburden and high strength rock at Little Tunnel, stress failures can be eliminated from further consideration. Consideration needs to be given only to support loads arising due to the movement of rock blocks bounded by existing structural features.

4.0 EVALUATION OF TUNNEL STABILITY

4.1 Existing Conditions

The existing conditions in the tunnel can be grouped as follows:

1. Sections where the supports are overloaded and have distorted as a result;
2. Sections where the supports have proved adequate to carry the imposed rock loads;
3. Sections where the rock is self-supporting and the condition of the supports is not important for tunnel stability.

The first condition exists at three locations in the tunnel. At Bays 8 through 12 the sets and lagging are bulged inward in the right haunch (Figure 6a). At Set 105-106 the right haunch member has been pushed inward and barely remains in contact with the crown member (Figure 6b). At Bays 183 through 187 the sets and lagging are bulged inward at the left haunch (Figure 7a).

Bulging has also occurred at Bays 181 and 182 (Figure 7b). It appears that high loads distorted and damaged the original timber sets and that additional sets were placed to take the load. The additional sets are undamaged and are providing the required support. Bays 181 and 182 are examples of the second condition.

Sets 18-19 and 23-24 are examples of the third condition where the rock is self-supporting. The right legs of these sets are so severely deteriorated that if any appreciable rock load existed, collapse would have already occurred.

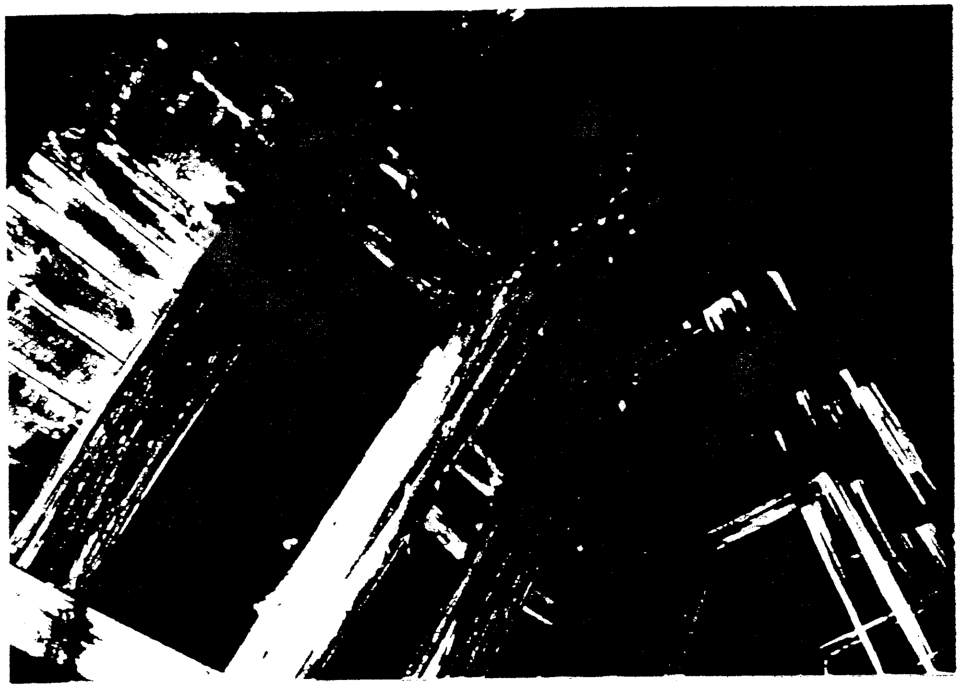


Figure 6a
Distortion of right haunch at Bays 8 through, 12

Figure 6b
Distortion of right haunch at Set 105-106

Figure 7a
Distortion of
left haunch at
Bays 183 through 186

Figure 7b
Distortion of
right haunch at
Bays 181 and 182

More than 80 years of adequate performance indicates that the remaining sections of the tunnel fall into one of the latter two categories. That is, they either have supports adequate to carry the imposed loads, or the rock is self-supporting. The good quality of the rock mass where exposed in the timber section, also suggests that the rock is self-supporting over much of the tunnel. However, some gradual loosening and spalling may have taken place in localized areas of poorer quality rock. Such rock would be more suspectible to freeze-thaw action, joint weathering mechanisms, and stress-redistribution phenomena, over the long term.

4.2 Effect of Highway Construction

Highway relocation work around Little Tunnel will involve the construction of embankments up to about 16 ft. high over the tunnel centerline. Blasting for cuts on U.S. 25E will be done within about 40 ft. horizontally from tunnel centerline and at an elevation about 60 ft. above the tunnel crown. The location of Little Tunnel in relation to the highway construction is shown in Figure 1.

Embankments for South Cumberland Drive and Ramp D will be 11 ft. and 16 ft. high respectively and will be built over the brick lined section of the tunnel. These embankments are expected to have a minimal effect on the brick lining. As discussed previously, our observations and the excellent performance of the brick section suggest that it was built reasonably tight against the rock. This means that the interaction of the ground and the liner will prevent appreciable moments from developing and any additional load exerted by the embankments will be carried almost totally in thrust. The brick lining is estimated to have sufficient thrust capacity to support more than 150 ft. of overburden. Thus, the addition of 16 ft. of fill, which brings the total overburden thickness to about 75 ft., is expected to be carried by the brick lining with a wide margin of safety.

The embankment where U.S. 58 will cross the tunnel is about 16 ft. high and passes over one of the problem areas in the timber supported section. Additional support will be required in the problem area between Bays 183 and 187 before construction of the U.S. 58 embankment. Recommendations for the remedial work are provided in Section 5 of this report.

Ramp A will also pass over the timber section of tunnel at about tunnel Station 14+00. At that location, Ramp A is approximately at existing grade and thus no appreciable change in load should occur in the tunnel supports.

For control of blasting during excavations for U.S. 25E near Little Tunnel, we recommend measuring peak particle velocities during blasting and limiting these to 2 in. per sec at the tunnel crown.

Two in. per sec. is the safe blasting criteria recommended by the U.S. Bureau of Miners for residential structures. It is recommended in this application because the vibration level that will cause damage in the brick tunnel lining is expected to be approximately the same as in a residential structure.

Measurements should be made in the rock at about Bay 184 and in the brick lining near its eastern end. Peak particle velocity for a blast at a given distance from a sensitive structure can be controlled by limiting the charge weight detonated per delay. Approximate charge weights per delay to limit peak particle velocity to 2 in. per sec. are estimated as follows (Reference 2):

Distance (ft.)	Charge Wt. per Delay (lb.)
50	8
75	25
100	60
200	450

These should be used only to guide bidders, as measured attenuation of vibration could change the allowable charge weights substantially.

As an added precaution, we recommend that measures be taken to prevent anyone, other than project personnel from entering the tunnel during the highway construction period.

4.3 Stability During Utility Installation

It is recommended that this report be provided to contractors bidding on the utility work so that they are fully aware of conditions throughout the tunnel before undertaking the work. Utility contractors should be aware that if their operations disturb the existing supports, there is the danger of rock falls from the crown and haunch.

If the project schedule will permit, the remedial measures recommended below should be completed before utilities are installed through the tunnel. Failure is not believed to be imminent at the three problem sections, but, depending on the nature of the utility construction, temporary supports may be required in these areas if the utility work is done before the permanent remedial work.

If remedial measures are completed before utility installation, and the utility contractor is careful not to disturb the tunnel supports, the tunnel is not expected to present any unusual hazard to utility installation.

4.4 Long-Term Stability

The three locations described in Section 4.1 with overloaded supports present the greatest threats to long-term stability. Remedial work at Bays 183 through 187 is required before construction of the U.S. 58 embankment. If the tunnel is to be used as a utility corridor, or if the Park Service becomes the owner, remedial work is also recommended at Set 105-106 and at the east portal. Recommended remedial measures are provided in the next section.

For the remainder of the tunnel, some further deterioration of the timber supports should be expected. Minor rockfalls may occur where no lagging is present. As the sets and the timber cribbing between the sets and the rock deteriorate, some additional sets may become distorted. Additional support may be required in such areas from time to time.

Considering the quality of the rock and the generally good performance of the timbers over more than 80 years in this environment, future remedial work is not expected to be a major problem. In particular, it is considered that a significant reduction in air circulation in the tunnel (through bulkheads and/or access control doors installed at the portals) would have a beneficial effect on its future stability, from the stand point of freeze-thaw action on the rock. Periodic inspection (annually or bi-annually) and maintenance of the tunnel supports is recommended. On this basis, the probability of a major collapse that might result in surface subsidence is considered very small.

5.0 RECOMMENDED REMEDIAL MEASURES

5.1 General

Remedial measures are recommended for three sections of the tunnel. These are as follows:

1. East portal through Bay 12
2. Set 105-106
3. Bays 183 through 187 near the west end of the timber-supported section.

Remedial work at the east portal will be more extensive than elsewhere. Therefore, the east portal will be considered first. Three options have been considered for stabilizing the east portal. The first option involves installation of steel sets. The second option involves installation of a reinforced concrete liner. The third option involves open cut excavation at the east end of the tunnel, thereby, shortening the tunnel and eliminating the problem section.

5.2 Steel-Set Supports (Option 1) for Stabilizing East Portal

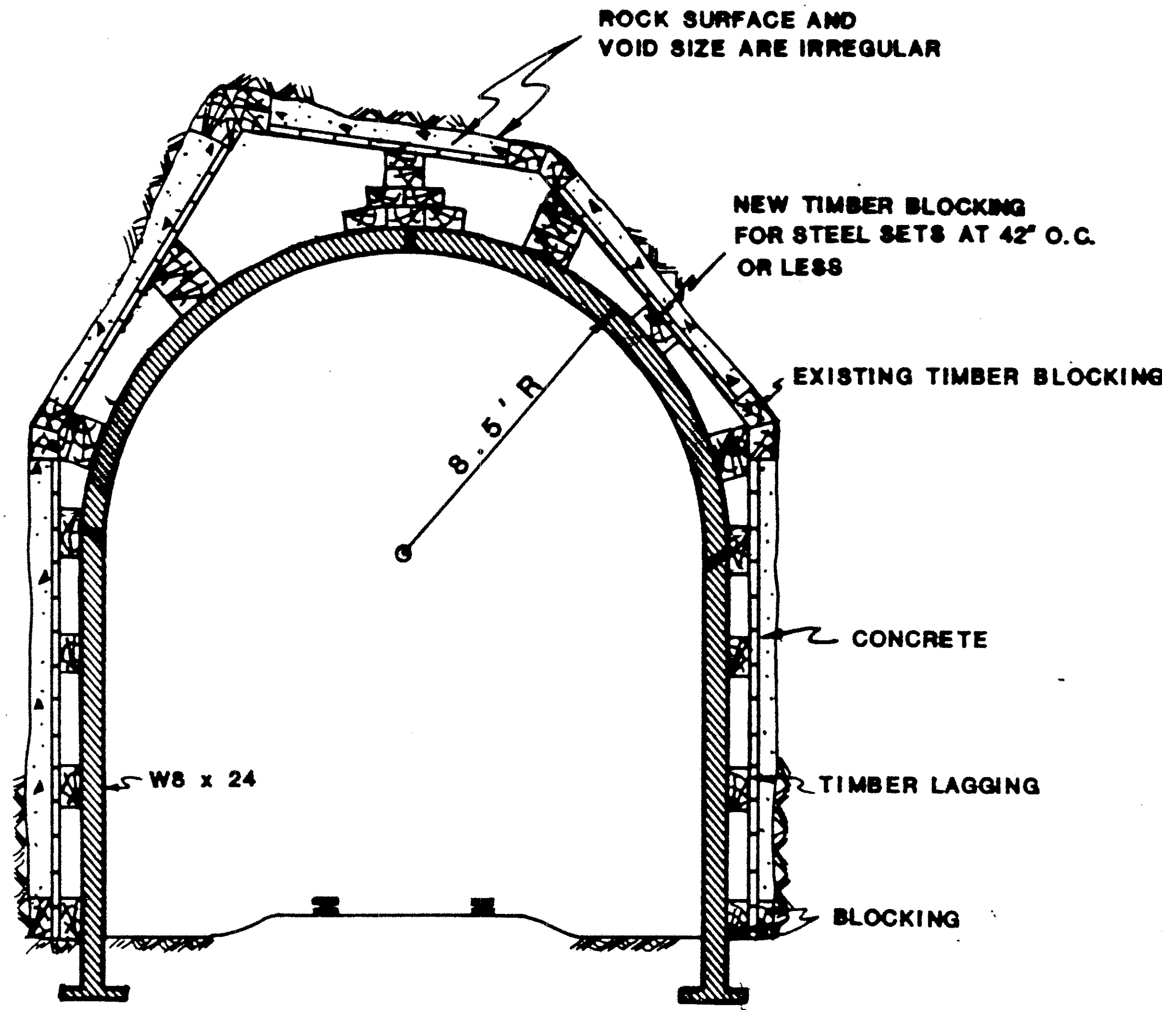
Two criteria need to be satisfied in designing tunnel stabilization measures at the east portal. First, the large displacements that have occurred suggest that the ground has little or no arching capability remaining and that supports must be designed to carry the full overburden load. Second, the design must prevent ravelling of the type that probably led to the current surface subsidences.

The maximum overburden height in the problem area at the east portal is 25 ft. Medium weight steel sets blocked tightly to the rock will support 25 ft. of overburden. However, with this design, the potential for ravelling and surface subsidence would remain.

Ravelling can be prevented and the full overburden load supported with a design that employs steel sets to support the load and pumped-in concrete to fill the void space. The concrete serves the additional purpose of blocking the supports to the rock. The construction sequence envisioned for this option at Bays 1 through 12 is as follows:

1. Replace the severely deteriorated lagging.
2. Erect the steel sets midway between the existing timber sets and block them tightly against the lagging. The steel should be blocked at the crown and springline and elsewhere at a spacing not to exceed 42 in.
3. Construct bulkheads, as required, and pump concrete to fill the void to the maximum level possible. This typically leaves about a one foot high void in the crown.
4. Drill through the lagging and concrete, and pump grout to fill the remaining void.
5. Shotcrete over the steel sets and timber blocking to provide corrosion protection. A minimum covering of 2 in. is recommended.
6. Backfill and re-grade the subsidence pits at the surface.

Size W8 X 24 steel sets are recommended. The recommended configuration is shown in Figure 8. This size set, spaced at 4 ft. on centers, will support the full 25 ft. of overburden without any reliance on the existing timbers. The span outside-to-outside of the existing timber sets is consistently about 18 ft. However, the geometry of the haunch and crown varies considerably. The length of the timber



JOB NO. 853-3256	SCALE 1" - 5'	TYPICAL STEEL SUPPORTED SECTION AT EAST PORTAL
DRAWN SKB	DATE 2-14-86	
CHECKED	DWG. NO.	
Golder Associates		LEE WAN & ASSOCIATES — FIGURE 8

legs (on the outside) from the foot block up to the haunch was measured at several locations to be about 14.5 ft., but this dimension is also expected to vary somewhat.

5.3 Reinforced Concrete Liner (Option 2) for Stabilizing East Portal

An additional option for stabilizing the East Portal, is the construction of a reinforced concrete liner immediately below the existing timber set system. The conceptual design for this solution is shown in Figure 9.

This concept relies on casting a concrete liner tight against the existing timber lagging, in the manner shown on the figure, such that the internal support profile is essentially maintained.

The reinforcement design allows for support of the full 25 feet of overburden,, as is the case of the steel sets; without any reliance on the existing timber sets. A nominal footing, as illustrated in the figure, should be provided on the rock surface which is expected to be generally at 1 to 1.5 foot depth below existing ground surface, along the sidewalls of the tunnel.

As far as the prevention of further ravelling and subsidence above the tunnel crown with this concept, it is recommended that following construction of the concrete liner, probing be carried out from above to try to detect any remaining large voids. Filling of voids with a vertical dimension of greater than 2 feet, should then be carried out, from ground surface, by pumping a sand slurry through appropriate holes for that purpose. Finally, when the major underground voids have been substantially filled, grading and filling of the ground surface should be carried out.

5.4 Open Cut (Option 3) for Stabilizing East Portal

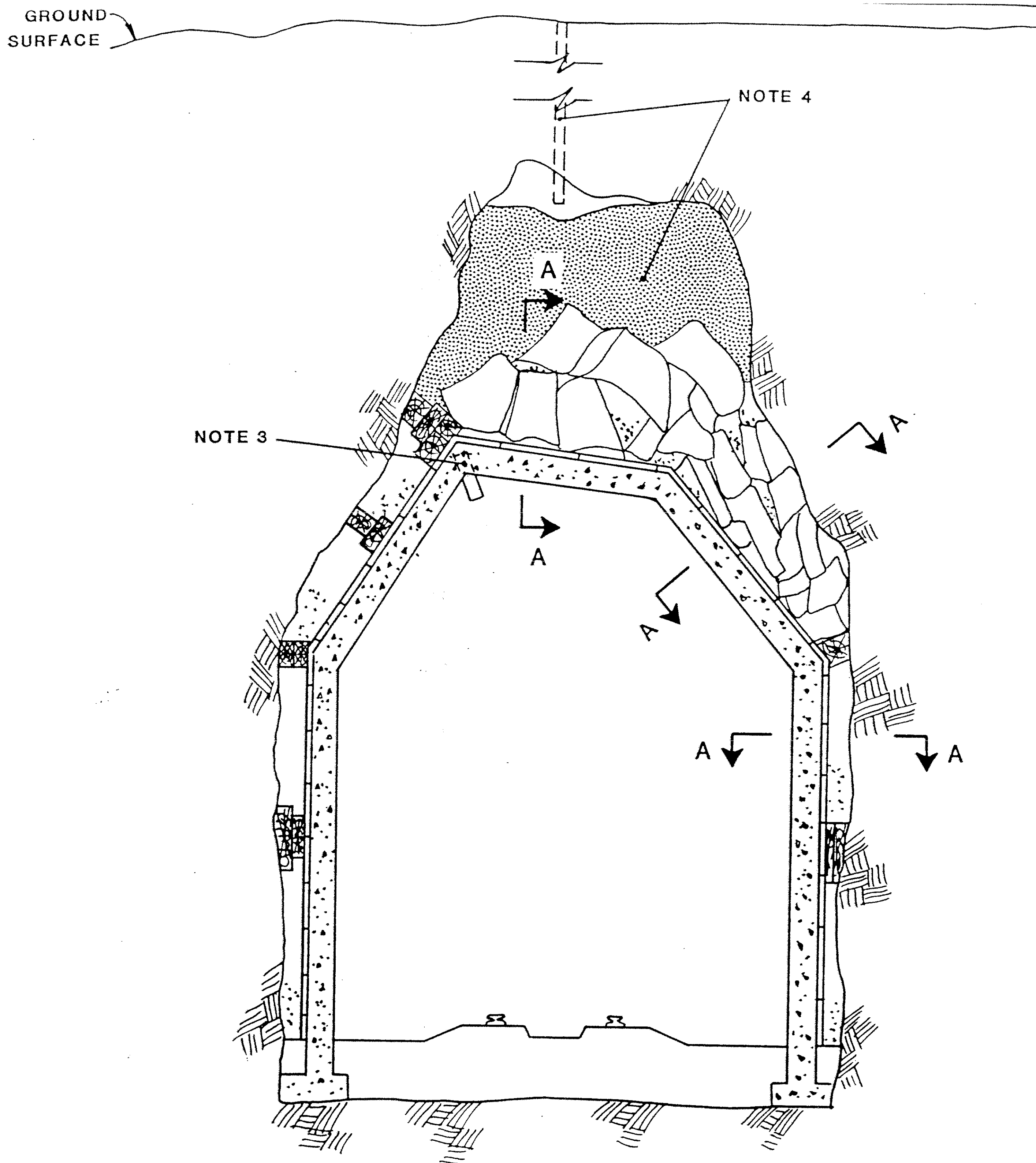
A conceptual design is shown in Figure 10 that involves making an open cut and moving the east portal back 50 ft. With this option, the problem section of the tunnel near the east portal would be eliminated. A conceptual layout of the open cut option has been prepared so that a cost comparison can be made between it and the tunnel stabilization option presented above.

The layout is based on assumed subsurface conditions and if this option is selected, an investigation of subsurface conditions will be required before final design. A soil depth of about 6 ft. and a depth to unweathered rock of about 20 ft. have been assumed. Additionally, it was necessary to estimate topography in the vicinity of the existing wall along U.S. 25E beside the portal. Apparently this wall was constructed after the project topographic mapping was done.

The conceptual design relies on steep, reinforced slopes in unweathered and weathered rock. Rockbolting on about a 5 ft. X 5 ft. pattern and mesh reinforced shotcrete are expected to be required in weathered rock. Spot bolting without shotcrete may be adequate in the fresh rock in the lower part of the cut slopes.

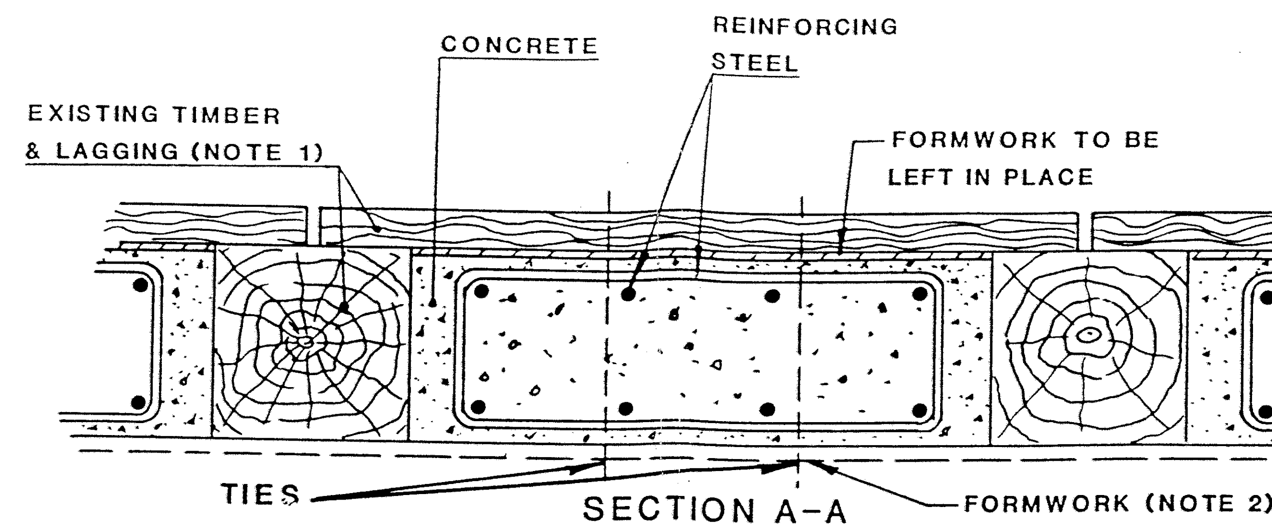
On the north side of the excavation the overburden can be laid back at a slope of 2.0 hor.:1.0 ver. On the west side, the rising topography suggests a retaining wall or soil nailing wall would be desirable in the overburden.

On the south side, the excavation encroaches on the existing retaining wall along U.S. 25E. The cross section in Figure 9 shows the constraint presented by the existing wall. The temporary excavation required to build a conven-



TYPICAL SECTION
(NOTE 5)

SCALE 1'- 5'



SECTION A-A
BAYS 1 TO 12 AND 183 TO 187

SCALE 1'- 1'

NOTES

1. EXISTING TIMBER SETS AND LAGGING TO BE LEFT IN PLACE. WHERE LAGGING IS MISSING OR WEAK IT SHOULD BE REPLACED OR EQUIVALENT MEANS SHOULD BE PROVIDED OF FORMING THE BACK FACE OF THE CONCRETE.

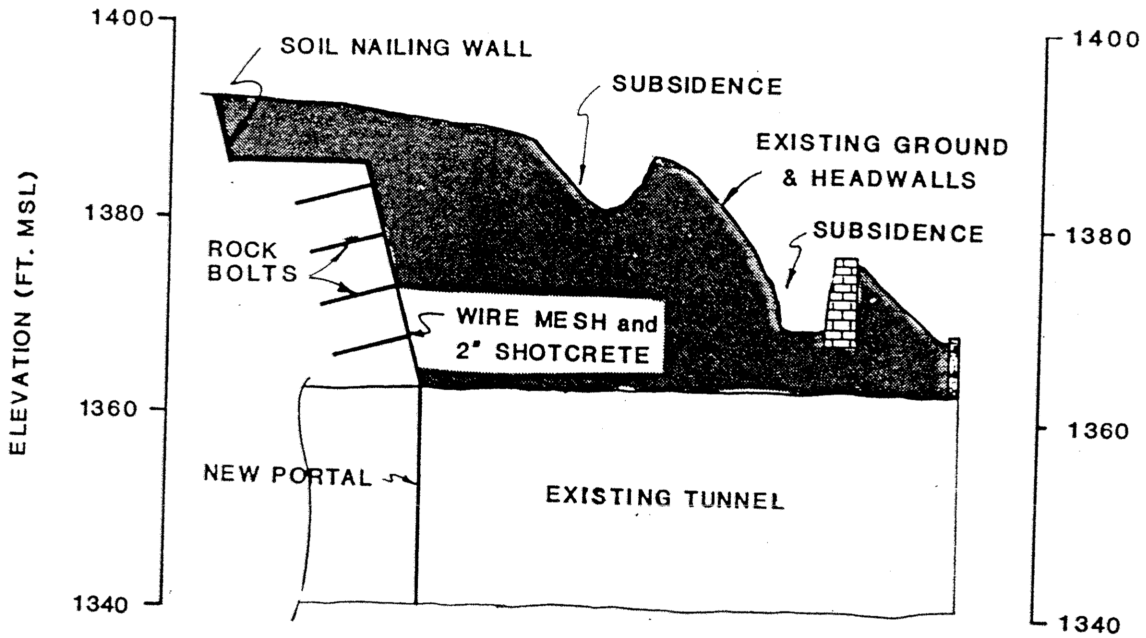
2. IN THE SIDE WALL SECTIONS THE FORMWORK SHOULD CONSIST OF APPROVED PLYWOOD AND TIED, AS SHOWN TO THE EXISTING LAGGING. IN ADDITION, IN THE HAUNCHES AND CROWN AREAS, THE FORMWORK SHOULD BE SUPPORTED BY APPROVED FALSEWORK, PRIOR TO CONCRETING. THE SIDEWALL TIE HOLES SHOULD BE LEFT OPEN, TO ACT AS WEEPHOLES.

3. CONCRETING OF THE HAUNCHES AND CROWN AREAS SHOULD BE DONE BY PUMPING INTO THE HIGHEST POINT IN EACH BAY. THE PUMPED CONCRETE SHOULD CONTAIN A SUPER PLASTICIZER ADDITIVE, TO IMPROVE ITS FLOW CHARACTERISTICS, AND THE VOLUME TO BE PUMPED SHOULD BE CAREFULLY MONITORED TO ENSURE COMPLETE FILLING OF THE CROWN SECTION.

4. THE GROUND ABOVE THE EAST PORTAL AREA (BAYS 1 TO 12) SHOULD BE PROBED FROM SURFACE, AT 5' INTERVALS ALONG THE TUNNEL AXIS, FOLLOWING COMPLETION OF THE INTERNAL REINFORCED CONCRETE WORK. VOIDS LARGER THAN 2 FT. (VERTICALLY) SHOULD SUBSTANTIALLY BE FILLED BY PUMPING A SAND SLURRY FROM GROUND SURFACE, AS SHOWN.

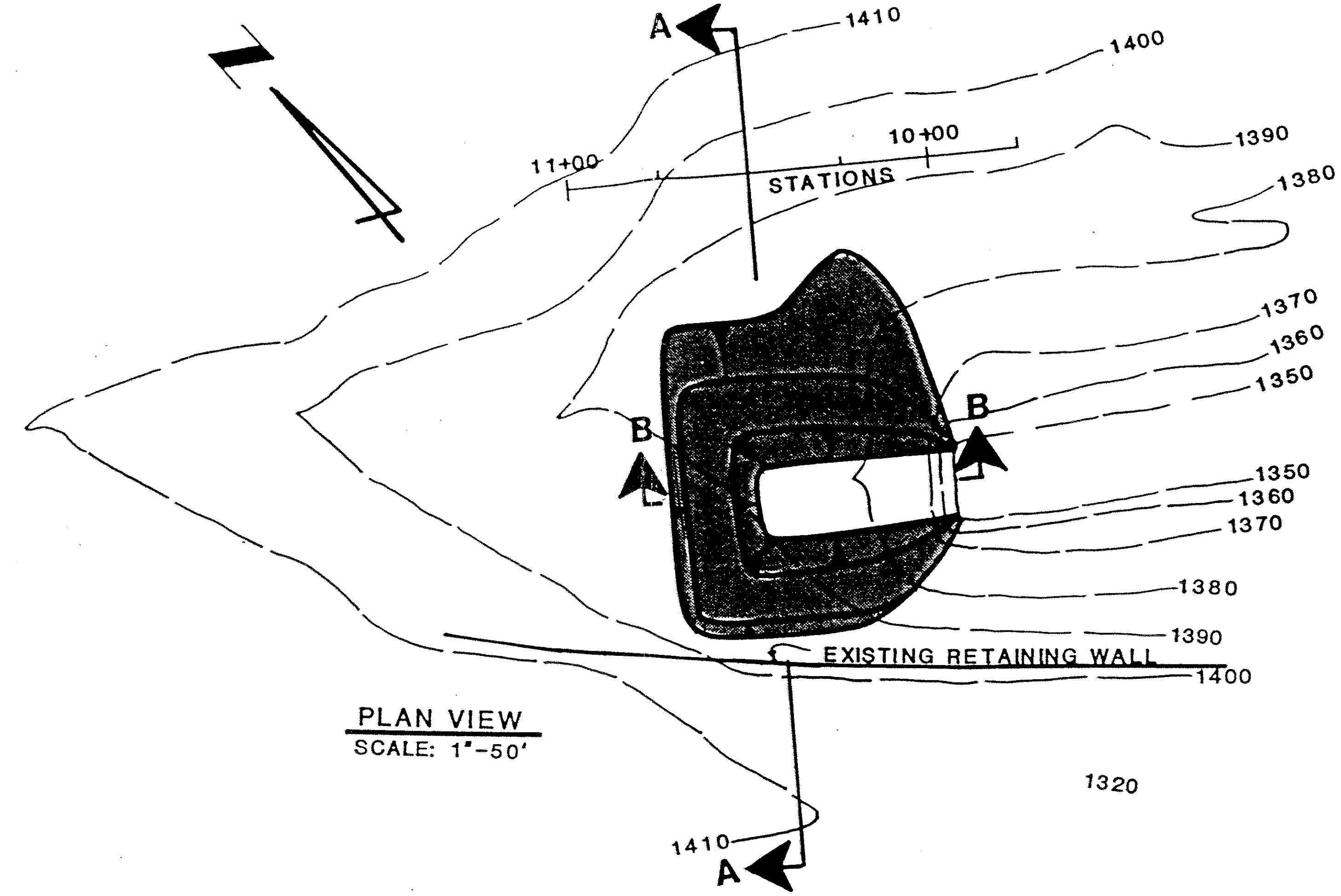
5. THE VOID PROBING AND FILLING TREATMENT ILLUSTRATED IN THE TYPICAL SECTION, AND DESCRIBED IN NOTE 4, WILL NOT BE REQUIRED AT BAYS 183 TO 187.

JOB NO. 853-3256	SCALE AS SHOWN	TYPICAL REINFORCED CONCRETE SUPPORT SYSTEM
DRAWN T.s.R.	DATE 5/20/86	
CHECKED	DWG. NO.	
Golder Associates		LEE WAN AND ASSOCIATES — FIGURE 9



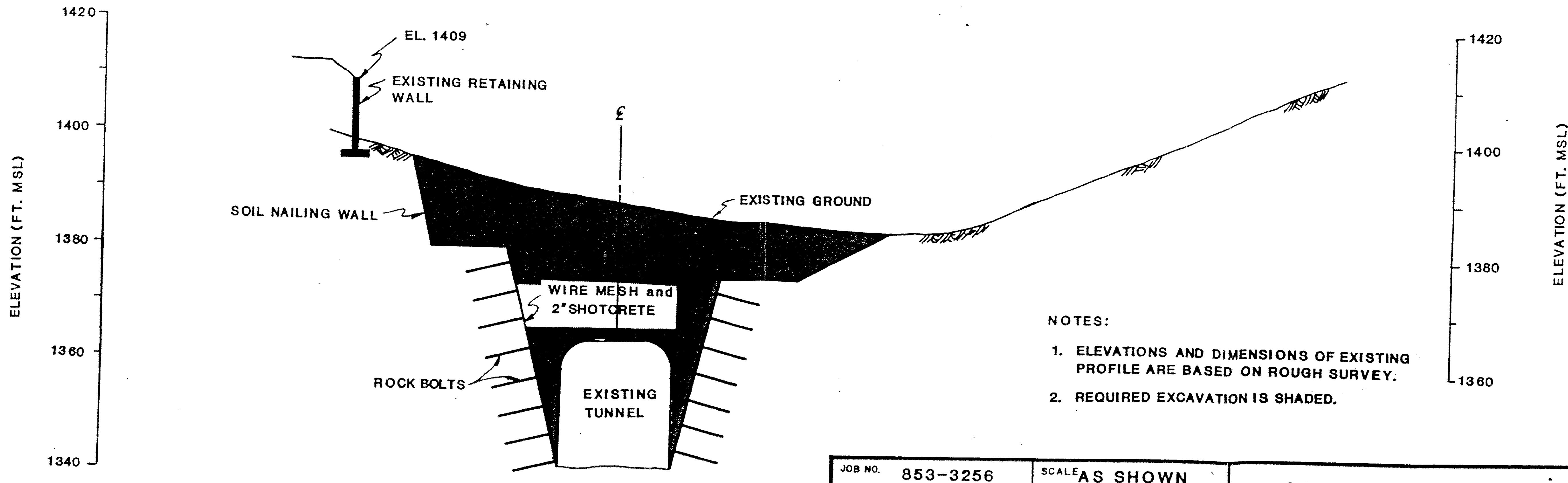
SECTION B-B at East Portal of Little Tunnel

SCALE: 1"-20'



PLAN VIEW

SCALE: 1"-50'



SECTION A-A at Station 10+50

SCALE: 1"-20'

NOTES:

1. ELEVATIONS AND DIMENSIONS OF EXISTING PROFILE ARE BASED ON ROUGH SURVEY.
2. REQUIRED EXCAVATION IS SHADED.

JOB NO. 853-3256	SCALE AS SHOWN	CONCEPTUAL DESIGN FOR OPEN CUT OPTION AT EAST PORTAL
DRAWN SKB	DATE 2-14-86	
CHECKED	DWG. NO.	
Golder Associates		LEE WAN & ASSOCIATES
		FIGURE 10

tional retaining wall founded at the estimated top of rock would threaten the stability of the existing wall. This constraint means that the open cut option is technically feasible only if a special technique, such as a tie-back wall or soil nailing wall, is used to support the overburden on the south side of the cut. The feasibility of tie-backs or soil nailing depend on subsurface conditions and on the foundation details of the existing wall.

Carefully controlled blasting will be required in making the excavation. Pre-reinforcement of the rock in the tunnel crown near the new portal is also advisable. This could be done with grouted, vertical dowels installed before blasting from the bench in the overburden above the new portal.

5.5 Remedial Measures at Set 105-106

A photo of the distorted right haunch at Set 105-106 is shown in Figure 6b. No other distortion is evident in the vicinity. Other than the poor contact between the haunch and crown member the set is in good condition.

It is recommended that another 12 in. X 12 in. crown member be bolted onto the present one and that it be mitered to fit tightly against both haunches. This repair is expected improve the capacity of the set to near its original level.

5.6 Remedial Measures at Bays 183 through 187

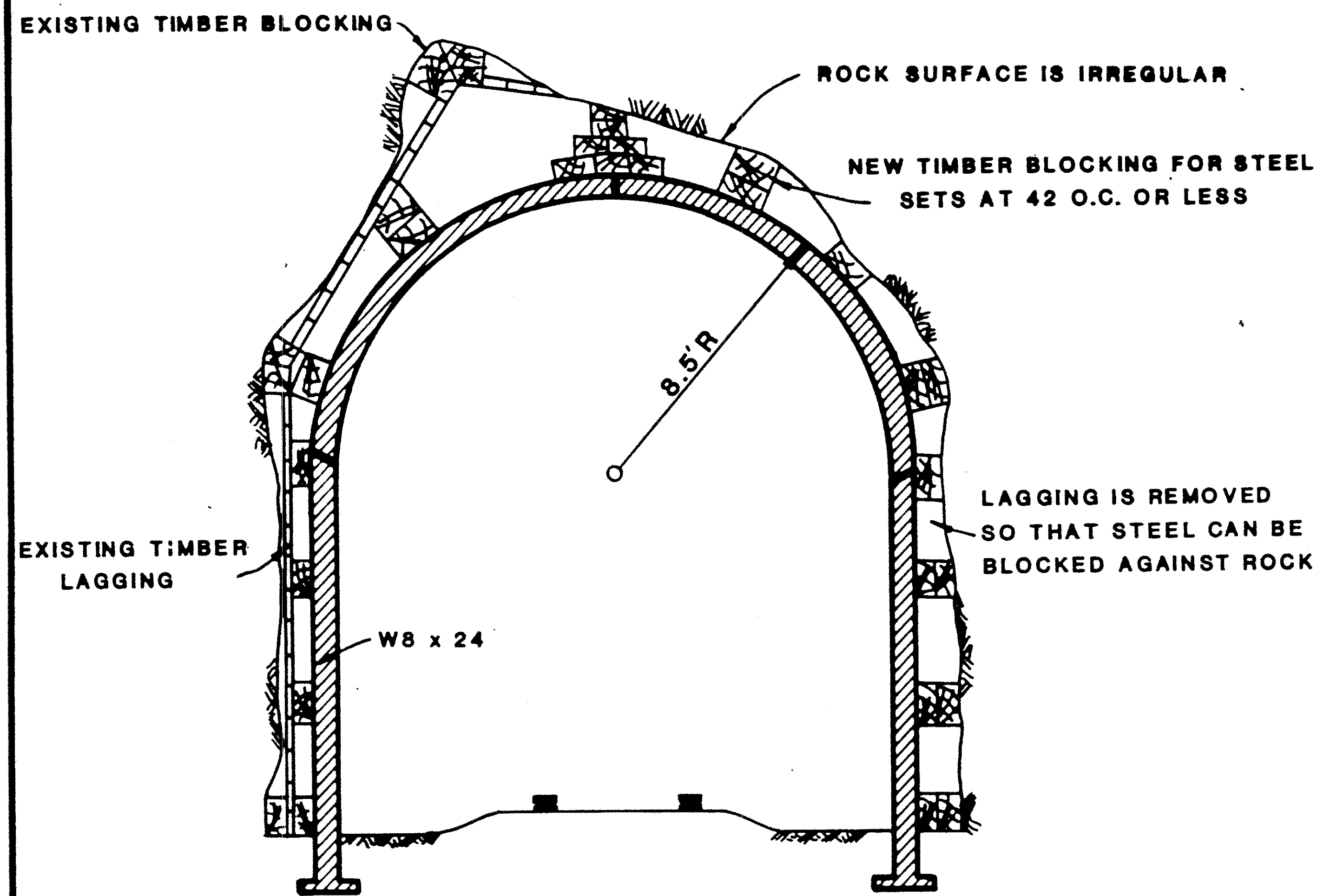
A photo of the distortion at Bays 183 through 187 is shown in Figure 7a. In addition to the distortion, the left haunch members in these bays are severely deteriorated. Thus, either steel sets or reinforced concrete are necessary to stabilize this section of the tunnel. The choice will depend on cost considerations, which are discussed in the following section of the report.

The reinforced concrete option would be the same in all respects, as already described for the East Portal and as illustrated on Figure 9, except for the probing and filling of crown void spaces. It is considered that in this case, in view of the much greater cover, there will be no need to fill existing voids to prevent ground subsidence.

For the steel sets support option, the sets would be of the same size as those recommended for the east portal work. The steel sets should be placed midway between the existing timber sets in Bays 183 through 187. They should be blocked tightly against the existing lagging in the left haunch and left portion of the crown, where the rock is clearly in contact with the lagging. Elsewhere, enough lagging should be removed so that access can be gained to block the steel tightly to the rock (see Figure 11). The steel should be blocked at the crown and springline and elsewhere at spacings not greater than 42 in. The steel sets, timber blocking, and any exposed rock should then be covered with a minimum of 2 in. of shotcrete.

5.7 Cost Estimate

Quantity and cost estimates are provided in Tables 1, 2, and 3 for the three options at the east portal, plus the repairs recommended at Set 105-106 and Bays 183 through 187. Unit prices used in the estimates were based primarily on unit prices from the pilot tunnel (E3/E6) bidding and input from FHWA senior staff at Cumberland Gap. On a project such as this where quantities are small, the estimated unit prices should be considered to have an accuracy not greater than 50 percent.



JOB NO. 853-3256	SCALE 1"-5'	TYPICAL STEEL SUPPORTED SECTION AT BAYS 183 - 187	
DRAWN JLW	DATE 4/9/86		
CHECKED WDM	DWG. NO.		
Golder Associates		LEE WAN & ASSOCIATES	FIGURE 11

For the "Option 1" tunnel stabilization work, the weight of steel shown is based on using 17 steel sets, each weighing 1300 lb. Shotcrete quantities are based on an assumption that 2 cubic yards will be required to cover one set and its blocking. The concrete-grout quantity was arrived at by assuming a 2 ft. average thickness will be required around the entire perimeter. Locally, our observations indicate this thickness will vary from near zero to 4 ft. or more. Two feet is considered a reasonable average thickness but it should be remembered that there is insufficient data to actually estimate the average thickness. If an reasonable upper limit on the concrete-grout quantity is desired, we recommend using a 3 ft. average thickness.

For the "Option 2" stabilization work, the volume of concrete is based on a 1 foot thick arch of the same configuration as the existing timber sets. The void space to be filled, above the crown of the East Portal Section, was assumed to be an average of 4 feet high and 10 feet wide, over a distance of about 50 feet. The quantity of sand required to significantly fill this void was estimated at 2/3 of this volume.

For the "Option 3" (open cut) stabilization work, the excavation volume was estimated using the 1 in. = 50 ft. mapping, with a 10 ft. contour interval. Thus, for an excavation of this size the quantities should be considered fairly crude. It was assumed that the entire excavation surface below the over-burden bench will be covered with mesh reinforced shotcrete and that 8 ft. rock bolts will be installed at 5 ft. on centers. A permanent soil nailing wall was assumed to extend along the overburden cut slope on the south and west sides of the excavation. It was assumed that 20 ft. long vertical dowels will be installed at 5 ft. on centers over a 15 ft. by 20 ft. area above the new portal.

5.8 Recommendations

Our cost estimates suggest that Option 2, i.e. stabilization with a reinforced concrete liner, is significantly lower in overall cost than the other two options considered, and it is therefore our recommended solution. In addition to the lower cost, this solution involves the least specialized construction work, which is an added advantage in terms of letting contracts for the work.

The recommended repairs do not permanently eliminate all potential problems in the tunnel as some further deterioration of the timbers should be expected. The repairs discussed in this report have been recommended as a cost effective means of stabilizing the tunnel for installing the services. Regular inspections of the tunnel will be required in the future.

Very truly yours,

GOLDER ASSOCIATES

W. Randall Sullivan, P.E.
Associate

Richard W. Humphries, P.Eng.
Associate

WRS/RWH:cee

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2. Hendron, A. J., Jr., "Engineering of Rock Blasting on Civil Projects," from Structural and Geotechnical Mechanics, W. J. Hall (ed.), 1977

TABLE 1

COST ESTIMATE FOR TUNNEL STABILIZATION OPTION 1

Item	Quantity	Units	Unit Price	Price
1. Replacement of lagging	500	sf	6.00	$3,000
2. Steel Sets	22,100	lb.	1.50	33,150
3. Set 105-106 Repair	L.S.	-	-	2,000
4. Concrete and Grout	200	cy	400.00	80,000
5. Shotcrete on Steel Sets & Blocking	17	set	1000.00	17,000
6. Grading & Filling Surface Subsidence	L.S.	-	-	5,000
Subtotal 1				$140,150
7. Mob. & Demob.	L.S.	-	-	20,000
Subtotal 2				$160,150
Contingency @ 20% of Subtotal 2				32,030
TOTAL ESTIMATED COST				$192,180

TABLE 2

COST ESTIMATE FOR TUNNEL STABILIZATION OPTION 2

Item	Quantity	Units	Unit Price	Price
1. Concrete	100	cy	400	$40,000
2. Reinforcing Steel	19,000	lb.	0.50	9,500
3. Formwork and Falsework	L.S.	-	-	3,000
4. Void Probing from Ground Surface	L.S.	-	-	2,000
5. Void Filling	50	cy	150	7,500
6. Set 105-106 Repair	L.S.	-	-	1,000
7. Grading and Filling Surface Subsidence	L.S.	-	-	$ 2,000
Subtotal 1:				$65,000
8. Mob & Demob.	L.S.	-	-	$15,000
Subtotal 2				$80,000
9. Contingency @ 20% of Subtotal 2				$16,000
TOTAL ESTIMATED COST				$96,000

TABLE 3

COST ESTIMATE FOR TUNNEL STABILIZATION - OPTION 3

Item	Quanity	Units	Unit Price	Total Price
1. Excavating	3,100	cy	$ 20.00	$ 62,000
2. 2 in. Mesh Rein. Shotcrete	440	sy	45.00	19,800
3. 8 ft. Resin-grouted Rock Bolts	150	ea.	40.00	6,000
4. 20 ft. Cement-grouted Dowels	20	ea.	100.00	2,000
5. Soil Nailing Wall	1,600	sf	50.00	80,000
6. Set 105-106 Repair	L.S.	-	-	2,000
7. Steel Sets - Bays 183-187	6,500	lb.	3.00	19,500
8. Shotcrete on Steel Sets & Blocking	5	set	1000.00	5,000
Subtotal 1				$196,300
9. Mob. & Demob.	L.S.	-	-	30,000
Subtotal 2				$226,300
Contingency @ 20% of subtotal 2				45,260
TOTAL				$271,560