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1. PURPOSE AND SCOPE

The purpose of the analysis is to provide a basis for ensuring safe conditions in the Case VI area.

The scope of this analysis is to analyze and recommend future action for a section of the South Ramp with a deviated alignment, here referred to as Case VI 3.01.X area. This area is located between Stations 71+07.10 to 71+44.10 in the South Ramp of the ESF (Exploratory Studies Facility) Main Loop tunnel.

The scope of the analysis includes accomplishing the following goals:

- 1) Document the as-built configuration including existing voids and installed ground support
- 2) Document the geotechnical properties of the rock mass and structural features
- 3) Evaluate the stability and develop recommendations for future action.

This analysis is performed under the *Development Plan: South Ramp Deviated Alignment 3.01.X Area Ground Support Analysis (STATION 71+07.10 to 71+44.10)* (Reference 8.1). This analysis also supplements the work conducted for 3.01.X areas utilizing NLP-3-35, *Exploratory Studies Facility Walk-Down for Quality Affecting Ground Support*.

2. QUALITY ASSURANCE

This document has been prepared according to AP-3.10Q, *Analyses and Models*. Classification of Items was determined in accordance with QAP-2-3, and the Activity Evaluation, per QAP-2-0, has determined that the QA Program applies to this analysis. The classifications for ground support items discussed in this analysis are presented in *QA Classification Analysis of Ground Support Systems* (Reference 8.2). The work control evaluation for ESF design, *Resolve Q Ground Support Issues* (Reference 8.3) has determined that the QA program applies to this analysis. This work activity has been evaluated in accordance with AP-SV.1Q, *Control of the Electronic Management of Information* and will be implemented per applicable methods detailed in Section 5.2 of AP-SV.1Q that include; process controls relating to personal computers, servers electronic transmission of data and storage of documents. The permanent function ground support installed in the ESF Main Loop is classified as QA-1 and QA-5 in Reference 8.2 and is therefore subject to QA controls. Any temporary function ground support installed by the constructor for reasons of personnel safety does not require classification (Reference 8.2).

3. COMPUTER SOFTWARE

Microsoft Excel 97 was used in this analysis. It was used to perform support activities that are not the controlled source of information in this analysis, and thus not subject to qualification per AP-SI.1Q.

Microsoft Excel is a commercial spreadsheet program designed to assist in performing the routine calculations. The program provides built-in mathematical functions together with user-defined formulae to automate the calculation process. Output formulae are updated automatically as input data are added or changed. *Excel* also includes a graphical package to

assist in data presentation. *Excel* was used to tabulate and to provide graphical presentation of data. The Computer Aided Design (CAD) system was used to produce drawings of the ESF Main Loop tunnel cross-sections at various locations as shown in Attachment I. The function MEASURE is an integral part of the CAD software. It was used to calculate the area of these tunnel cross-sections. In order to verify the correctness of results provided by this function, manual crosscheck of calculations was performed as documented in *North Ramp 3.01.X Areas Ground Support Analysis* (Reference 8.27). The results of these manual calculations are in close agreement with the results of calculations obtained using the CAD function MEASURE.

4. INPUTS

4.1 DATA AND PARAMETERS

A separate analysis was not done for this analysis since data and parameters are similar to the other 3.01.X areas already analyzed (References 8.25 and 8.27). Results from those analysis are used here-in. The input data listed in those reports are appropriate for this analysis.

Surveying data DTN; MO9907DVSABESF.000, were used to plot the trace of the Deviation of the South Ramp alignment at the location of Case VI 3.01.X Area, *Design Versus As-Built of the Exploratory Studies Facility (ESF) South Ramp*, (Reference 8.15). The input for this plot is appropriate for this analysis.

4.1.1 Tunnel Dimensions (Design Input):

For the ESF Main Loop a tunnel diameter of 7.62 meters was used in the original analysis (References 8.25 and 8.27), and as noted in the *Exploratory Studies Facility Design Requirements* (Reference 8.13). The input data is appropriate for this analysis.

4.2 CRITERIA

The *ESF Ground Support Design Analysis* (Reference 8.10) followed criteria that were based on the requirements contained in the Exploratory Studies Facility Design Requirements (ESFDR) document (Reference 8.13). The listing of all design requirements applicable to ESF ground support design extracted from ESFDR is provided in Table 1 along with the corresponding design criteria.

Table 1. List of ESFDR-Defined Design Requirements and Corresponding Design Criteria Related to ESF Ground Support for 3.01.X Area (Reference 8.13). (Continued on pages 8 and 9)

No.	ESFDR Requirement	Design Criteria
1	The permanent and temporary items of the ESF shall be designed to withstand the applicable seismic environment specified in Appendix A. (ESFDR 3.2.1.2.1.2.A)	Seismic input data from Reference 8.13 are used in numerical calculations such that the resulting design is capable of accommodating design conditions.
2	The ESF subsurface facilities and equipment shall be designed to withstand and operate in temperatures ranging from a low of 50 degrees F to a high of 70 degrees F. (ESFDR 3.2.1.2.1.2.C).	The ESF ground support system is designed to perform its intended function within the range of temperatures specified.

Table 1. List of ESFDR-Defined Design Requirements and Corresponding Design Criteria Related to ESF Ground Support for 3.01.X Area (Reference 8.13). (Continued on pages 8 and 9)

No.	ESFDR Requirement	Design Criteria
3	The ESF subsurface facilities and equipment shall be designed to withstand and operate in a relative humidity environment of 13 to 71%. (ESFDR 3.2.1.2.1.2.D).	The ESF ground support system is designed to perform its intended function within the range of relative humidity levels specified.
4	The ESF non-permanent items shall be designed for a 25-year maintainable service life. (ESFDR 3.2.1.2.2.A).	A 25-year maintainable service life of the non-permanent items is provided by designing items that from past experience are durable within the required time.
5	The ESF permanent items shall be designed for a 150-year maintainable service life. (ESFDR 3.2.1.2.2.B).	The 3.01.X area permanent function ground support system is designed for a 150-year maintainable service life.
6	The ESF underground openings CIs design, construction, and in situ testing shall be planned and coordinated with the repository design. (ESFDR 3.7.2.1.2.B).	The repository design and other affected participating organizations' interfaces will be handled during the normal course of design activities. The interfaces will be formalized by using these organizations as participants in the design review process for this analysis.
7	Permanent ESF items shall be designed, constructed, and maintained consistent with the quality controls and record keeping requirements expected for permanent items that are part of a potential repository. (ESFDR 3.7.2.1.2.C).	QA records generated by this design will be handled in accordance with appropriate QA procedures for permanent ESF items.
8	The ground support system shall be designed to include provisions for installation of instrumentation and data collection units as specified by testing requirements. (ESFDR 3.7.3.1.A).	The 3.01.X area ground support system is designed such that it accommodates installation of instrumentation and data collection units.
9	The ground support system design shall limit the use of pressure grouting and shall obtain prior approval from the Test Coordination Office before usage. (ESFDR 3.7.3.1.B).	The 3.01.X area ground support does not incorporate pressure grouting. If pressure grouting in these locations is deemed to be necessary in the field, Test Coordination Office (TCO) approval shall be obtained prior to its application.
10	The ground support system shall be compatible with the excavation methods and equipment. (ESFDR 3.7.3.1.D).	The 3.01.X area ground support system is designed to be compatible with the excavation method and available equipment.
11	The ground support system shall incorporate the use of noncombustible and heat resistant materials in the design. (ESFDR 3.7.3.1.E).	The 3.01.X area ground support system is designed to incorporate the use of noncombustible and heat resistant materials.
12	The ground support system shall be designed to permit periodic inspection, monitoring, testing, and maintenance, as necessary, to evaluate their readiness and to ensure their continued function. (ESFDR 3.7.3.1.F).	The 3.01.X area ground support system is designed to permit inspection and monitoring as needed to evaluate their readiness and to ensure their continued function. The 3.01.X area ground support is designed such that it provides for routine maintenance.

Table 1. List of ESFDR-Defined Design Requirements and Corresponding Design Criteria Related to ESF Ground Support for 3.01.X Area (Reference 8.13). (Continued on pages 8 and 9)

No.	ESFDR Requirement	Design Criteria
13	The ground support system shall be designed and installed throughout the main access openings and all alcove transition zones to reduce the potential for deleterious rock movement or fracturing. (ESFDR 3.7.3.1.G).	Ground support system is designed to be installed throughout the 3.01.X areas to reduce the potential for deleterious rock movement or fracturing.
14	The ground support system shall be designed to accommodate the anticipated ground conditions at the main access opening, the operations support areas, and the test support areas, utilizing the available site data at that time. (ESFDR 3.7.3.1.I).	The 3.01.X area ground support system is designed utilizing available site data. The ground support is designed to accommodate anticipated ground conditions in these areas.
15	The ground support system shall be designed to have the capability to be supplemented as required when identified through additional site characterization data and analyses. (ESFDR 3.7.3.1.J).	The 3.01.X area ground support system is designed to have the capability to be supplemented. The 3.01.X area ground support system provides capability for installation of additional support to supplement it as required.
16	The ground support system shall be designed with sufficient flexibility to allow adjustments where necessary to accommodate specific site conditions encountered during excavation or identified through in situ monitoring and testing. (ESFDR 3.7.3.1.K).	The ground support is designed with the flexibility needed to allow adjustments and to accommodate specific site conditions encountered during excavation or identified through in situ monitoring and testing.
17	The main access opening shall be designed for a 7.62 meter (25 foot) diameter opening constructed by mechanical excavation. (ESFDR 3.7.2.3.1.D)	The main access opening of 7.62 meter excavated diameter is assumed and used in the analyses.

4.3 CODES AND STANDARDS

American Concrete Institute 1995. *Building Code Requirements for Structural Concrete (ACI-95) and Commentary (ACI 318R-95)*. ACI 318-95/318R-95 Farmington Hills, Michigan: American Concrete Institute. TIC: 233584. *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary* (Reference 8.19)

5. ASSUMPTIONS

5.1 INPUT PROPERTIES FOR CONCRETE/SHOTCRETE

- Specific properties for concrete/shotcrete noted in section 6.6 were previously determined (Reference 8.25 and 8.27). In general, concrete/shotcrete shall be designed to ACI 318 (Reference 8.19 and Reference 8.13)

The assumptions listed below are used throughout Section 6.7.

5.2 SEISMIC INPUTS (TBV-193)

Although indirectly, an effect of a seismic event characterized by 23 cm/sec peak ground velocity (PGV) seismic wave velocity was considered in the analysis based on Reference 8.13. A wave frequency of 5 Hz with duration value of 1 second was assumed and used in the analysis. The previous analysis of the ESF ground support system (Reference 8.10) was used as a basis for these assumptions. All seismic parameters will be verified upon resolution of TBV-193.

5.3 IN SITU STRESS ESTIMATES

Numerical analysis was not performed as a part of this study. However, the initial stresses were evaluated to justify the use of the results of previous analyses *South Ramp 3.01.X Area Ground Support Analysis* (Reference 8.25) and *North Ramp 3.01.X Areas Ground Support Analysis* (Reference 8.27) as a complementary part of this analysis. Initial stresses are estimated using the gravitational stresses induced by the weight of the overburden. The vertical stress $\sigma_v = \sum h_i \rho_i g$, where h_i and ρ_i are thickness and density for each rock mass unit and g is the acceleration due to gravity. The stresses used here are evaluated at the steel set #1525 located at Station 71+08.48. Depth of the tunnel at the centerline for this steel set location is equal to 101.47 meters and is estimated from the best available geologic model, as presented in *Geology of the Exploratory Studies Facility Topopah Spring Loop* (Reference 8.17). The in situ horizontal-to-vertical stress ratios are in the range of 0.3 and 1.0.

A comparison of depths used to calculate stresses in the previous analyses (References 8.25 and 8.27) reveals that the stress magnitude within the Case VI 3.01.X area is equal to approximately one half of the stress used in the analysis of ground support at South Ramp (Reference 8.25). Since the rock strata are located within the same thermal mechanical unit (TSw2) and the tunnel geometry is the same, it was concluded that the results of the previous analysis are applicable in the current study. The results of the numerical portion of the previous analysis are used here in support of recommendations discussed later in this Report.

6. ANALYSIS

This section presents details pertaining to the analysis performed for the Case VI 3.01.X location along the South Ramp Main Loop tunnel. This analysis was performed utilizing various sources of input, including the results of previous analyses, field observations and in situ measurements. The results of numerical analyses performed on other ESF Main Loop Tunnel sections (References 8.25 and 8.27) are utilized to examine the impact of seismic event on various ground support configurations.

The analysis is assigned an importance level 3 as per the Screening Criteria for Grading Data Attachment in AP-3.15Q. The analysis does not provide estimates of any of the Factors or Potentially Disruptive Events listed in the Attachment of AP-3.15Q. A combination of methods, here applied to analyze the ground support system in the South Ramp (Case VI) of the ESF Main Loop tunnel is appropriate for this design analysis.

6.1 BACKGROUND

The 3.01.X areas refer to areas of the ESF Main Loop tunnel that were constructed under Section 3.01.X of the Specification, *Subsurface General Construction* (Reference 8.4). This specification indicates that special actions may be necessary to continue excavation should the adverse ground conditions prevent normal Tunnel Boring Machine (TBM) operations and jeopardize continued tunnel advance. The specification allows the Constructor and A/E to develop jointly, a plan based on the actual ground conditions in accordance with Technical Document Preparation Plan (TDPP) for *Technical Document Preparation Plan for Plans for Continuing TBM Advance* (Reference 8.14). Upon completion of the work or actions specified by the plan, an evaluation of the impact of the plan to specifications and drawings is required. During the ESF Main Loop construction, eight tunnel sections were constructed utilizing provisions of the 3.01.X section of the subsurface general construction specification.

Figure 1 shows locations of the 3.01.X areas existing along the ESF Main Loop tunnel. For the purpose of analysis, the eight 3.01.X areas located along the sections of the North Ramp and South Ramp were further divided into the three groups. The first group includes Cases I, II and III located along the North Ramp, these were analyzed and reported in Reference 8.27. The second group includes the Cases IV, V, VII and VIII. Located in the South Ramp these cases were analyzed and reported in Reference 8.25. The third group includes Case VI which is the subject of this analysis.

Analysis of the Safety of Case VI is the objective of this report. This analysis complements the work reported earlier (References 8.25 and 8.27) and addresses the 3.01.X area located along the deviated section of the South Ramp of the ESF Main Loop tunnel. The Case VI area extends from Station 71+07.10 to Station 71+44.10 and is located at the depth below the surface of approximately 101.47 meters (at the lower/deeper end). Within this area the tunnel deviates from the designed alignment in both horizontal and vertical directions (Attachment 3). The TDDP (Reference 8.14) that was developed for this section of tunnel requires an evaluation of the impact of the action to the specifications and drawings. The impact is primarily to the alignment but also has minor impact to the specifications and drawings as to the tolerance specified for the size and configuration of some steel set components and use of timber support.

The as-built survey information is presented in, *3.01.X Areas Survey Information (Steel Set Stations and Steel Set Numbers) and Type of Support/Materials* (Reference 8.5).

The "Q" values (as discussed later in Section 6.3) were used as a guide to ground conditions in the ESF main loop. At the beginning of this 3.01.X section "Q" values fell to 0.14 which is considered "very poor" rock in the NGI "Q" Value and Classification System, as shown in the *ESF Ground Support Design Analysis* (Reference 8.10). As ground conditions deteriorated further into the section, the "Q" value could not be determined and were noted as "NR" or not determined as shown on the Department of Interior map, *ESF - South Ramp Station 71+00.00 to Station 72+00.00, As Built Geology and Geotechnical Data* (Reference 8.22). Within the first 5 meters of this section the fallout averaged from 1 to 3 meters in the crown and occurred from springline to springline (Attachment I). In this interval (steel sets #1524 to 1527) the lighter W6x20 steel sets were installed on nominal 1.22 meter spacing. This area was stabilized with timber cribbing and tight lagging and under these in situ conditions has shown no sign of instability since construction. However, because this type of support is considered light support (Reference 8.10), and because the existing timber support in the crown cannot be directly inspected nor are instrumentation results available that would indicate continued stability, the adequacy of the ground support in its present configuration cannot be reliably determined, especially under dynamic loading as analyzed for similar conditions in the North Ramp 3.01.X areas (Reference 8.27).

Within the fault zone (from steel set #1528 to 1547), no "Q" values could be obtained due to the extensive fallout. The fallout was measured to a maximum of 4.5 meters deep at steel set #1542 and fallout occurred around the tunnel circumference to well below springline from set #1535 to set #1545 (Attachment I). In this section the heavier W8x31 steel sets were used on nominal 1.22 meters spacing with timber cribbing and lagging. Except for minor raveling of the wall rock, the in situ conditions for this 24 meter interval has shown no sign of instability. However; due to the unknown rock mass properties of the faulted material that may affect the analysis, the lack of instrumented data indicating continued stability, and the lack of a practical method of monitoring the timber support, the adequacy of the ground support in its present configuration cannot be reliably determined, especially under dynamic loading.

Beyond steel set #1547 to the end of the section at set #1554, better ground conditions were encountered ("Q" values from 1.4 to 4.5 were determined) and the W6x20 steel set were used. The void survey of this section indicates substantially less fallout and the support appears more appropriate for the conditions encountered. This is also the section of maximum alignment deviation due to loss of adequate gripper support back of the TBM in the zone of maximum fallout.

Previous numerical analysis of ground performance in similar tunnel conditions under dynamic loading (Reference 8.25 and 8.27) indicate stability can be improved if the voids surrounding the opening are backfilled as shown in Stage 1 remediation (Figure 5) and would be considered stable if the entire void were backfilled with concrete as shown in Stage 2 (Figure 5). Since no numerical analysis was done specifically for remediation of this area, additional ground support and steel set analysis will be required to determine if the existing steel supports as configured can withstand the additional weight that will be imposed by backfill placed in the crown during Stage 2 (figure 5).

Due to the alignment deviation and drop in elevation of the invert, the steel sets were stepped down and sideways to follow the excavation. As a result of the difficult geometry, minor tolerance variations occurred in steel set spacing and steel construction but would not be expected to materially impact the function of the ground support. Difficulty maintaining grade with the TBM in this area resulted in over-excavation of the invert as well. As with the other minor variance items, this is also not expected to materially impact the function of the ground support.

These construction features depart from the design and distinguish Case VI from the other 3.01.X areas that were grouped together because they encompass a dissimilar range of difficult ground conditions encountered elsewhere along the North and South Ramps.

6.2 AS-BUILT CONFIGURATION OF 3.01.X AREAS IN SOUTH RAMP

In general, the first step in evaluation of the stability of all 3.01.X areas was aimed at establishing the as-built configurations in these tunnel sections. Ground control consists of steel sets with partial steel lagging installed throughout these areas. The approximate configuration and the depth of the voids were essential to conduct this analysis. However, the information on the nature of the voids was limited. To complete this task, surveying was performed to obtain the as-built configuration of the voids along the 3.01.X sections of the ESF Main Loop tunnel. The survey data (Reference 8.23) identifies eight 3.01.X areas along the tunnel alignment, referred to as Case I to Case VIII as shown in Figure 1. Figure 2 shows an example of the tunnel and the associated void cross section documented at steel set #1542 in this 3.01.X area that was developed as a result of this tunnel survey.

6.2.1 Case VI - 3.01.X Area from Station 71+07.10 to Station 71+44.10

The Case VI 3.01.X area is approximately 37 meters long. The tunnel is supported by W8x31 and W6x20 steel sets with partial steel lagging installed to below springline. Wood blocking was used for cribbing to provide support in areas of large voids. This area is located in the section of the South Ramp with the beginning at steel set #1524 and ending at steel set #1554. At this location, the ESF Main Loop tunnel transects a series of faults referred to as imbricate faults. The tunnel section is located within the TSw2 thermal mechanical unit, classified as the middle nonlithophysal zone (Tptmn) lithostratigraphic unit. The fault zone crossing the tunnel in this section puts the TSw2 rock unit in fault contact with the normally overlying TSw1 thermal mechanical unit near the end of this 3.01.X area at about station 71+44.10.

During the construction of this portion of the South Ramp, large voids, in some cases as high as 4.5 meters above the crown (References 8.23), as well as on the right and the left tunnel ribs, were created before ground control measures could be introduced to stabilize the rock mass surrounding the tunnel. The contractor used shotcrete and timber cribbing to stabilize the ground during construction. In most of the Case VI 3.01.X area, lagging extends to well below springline on both sides of the tunnel. The extent and geometry of voids were determined after construction using a combination of information obtained from the voids survey and construction records (Reference 8.23).

The results of the voids survey are presented in Attachment I showing cross sections at steel set locations including approximate voids dimensions. Wood blocking was used for cribbing to

provide support for the main roof. The deepest void in case VI was recorded in the tunnel crown at steel set #1542 location, and a cross section is reproduced as Figure 2.

During excavation, the voids in the crown and tunnel ribs caused conditions such that the gripping mechanisms used to control the TBM direction could not be used effectively. Figure 3 shows the plot of the offset between the designed and the as-built South Ramp alignment. The positive horizontal offset indicates tunnel alignment deviation to the right. The negative vertical offset indicates the excavated tunnel invert was located below the designed grade. After deviation occurred, it was decided to correct the tunnel deviation gradually. The deviated portion of the South Ramp was successfully corrected approximately 150 meters before the tunnel end at the South Portal. At this location, both horizontal and vertical offsets were reduced to acceptable (minor) deviations from the designed values. The maximum horizontal and vertical offsets of the deviated section are equal to 0.9 m and 1.8 m, respectively. The as-built tunnel alignment offset survey data are listed in Attachment III (and Reference 8.15). The data resulting from this survey is designated as Qualified-Verification Level 2 data since it does not produce a technical document that provides estimates for any of the Principal Factors or Potentially Disruptive Processes and Events and no TBV is assigned to this data.

Table 2 provided in Section 6.2.2 summarize the tunnel status and the volume of voids for the Case VI 3.01.X area.

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Figure 1. Locations of the Case VI 3.01.X Area Addressed in this Analysis. Case Numbers and Associated Stations as Defined in Reference 8.5.

Figure 1. Locations of the Case VI 3.01.X Area Addressed in this Analysis. Case Numbers and Associated Stations as Defined in Reference 8.5.

STATION: 71+29.40

STEEL SETS: 1542

AREA(m²): 39.995

Figure 2. Estimate of Void Dimensions at Steel Set #1542 Location Based on Field Survey Data - Case VI, 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (Reference 8.5 and 8.23). All measurements are in the metric system.

Horizontal And Vertical Offsets in a Deviated Section of the Main Loop Tunnel - Case VI

Figure 3. Plot of the Offset between the Designed and the As-Built ESF Main Loop Tunnel Section. Offset Starts at the Case VI 3.01.X Area - South Ramp Location. See *Design versus As-built of the ESF, South Ramp* (Reference 8.15).

6.2.2 South Ramp Case VI 3.01.X Area Tunnel Status

The tunnel walk-downs, periodic field inspections and 3.01.X areas void survey (Reference 8.23), provided the basis for assessment of the void locations and their dimensions. The tunnel and associated void section details for Case VI are presented in Attachment I. Table 2 provides a summary of the tunnel status and estimated void dimensions. The planar void cross-sections

were used to calculate the area of each individual cross section. Using the spacing between the subsequent steel sets and an average between two adjacent cross section areas, the approximate volume of voids was calculated. The last column in Table 2 provides a general volume of the voids as obtained from the field records (Reference 8.23).

Table 3 shows the quantity of concrete and/or shotcrete that will be necessary to fill these voids (the volume of stage 1 plus the volume of stage 2 equals the calculated void volume shown in Table 2). The work is anticipated to be completed in two stages as indicated.

Because timber was used in the Case VI 3.01.X area ground support system, requirement #11 listed in Table 1 of the ESFDR-Defined Design Requirements is not met. The use of timber as a temporary ground support has a beneficial ground-stabilizing effect, but because its performance over an extended timeframe is uncertain encasement during remediation should be considered.

The time factor involved in resolving the "timber issue" allows the designer to consider a two-stage approach. The recommended Stage 1 remedial repair (Figure 5) addresses the minimum seismic safety-related concerns. The recommended Stage 2 remedial repair (Figure 5) could be delayed and allow for a full evaluation and selection of the method suitable to resolve the potential future problems related to the use of timber in the current ESF ground support.

The as-built ground support system can be brought into compliance with the current requirement for heat resistant and noncombustible materials (Requirement 11) by pursuing one of the three options. The first option is to remove the wood from the void and backfill the void with non-combustible materials. The second option is to provide appropriate treatment of the timber. This treatment may include encasing the existing timber within lightweight backfill concrete, rendering the materials presence non-combustible. The function of concrete encasement/backfill is to fulfill the 150 years design life requirement. This option is acceptable for the ESF ground support system but may require further evaluation before incorporation into the final repository. The third option is to obtain a change of the current ESFDR requirement. In this case, M&O could pursue obtaining a variance from DOE to complete Stage 1 which includes very little timber. Completion of Stage 2 would then require resolution with the NRC of the issue associated with the use of combustible materials (timber) underground.

Another aspect, particularly relevant in this South Ramp section is associated with the offset from the designed tunnel alignment. The offsets (horizontal and vertical) deviate from their design values. The offset magnitude does not appear to impact the safety of the tunnel providing at least Stage 1 remedial repair is completed or for the short term, additional blocking of steel sets is conducted.

The tunnel alignment along the South Ramp was designed considering the access and transportation needs along this portion of the Main Loop tunnel. The tunnel grade combined with the finished tunnel diameter were designed to accommodate certain sizes of equipment and other types of hardware that must be transported from the surface to the target repository horizon. A separate study is therefore required to evaluate the impact of this deviated as-built tunnel configuration. The outcome of this study should provide a recommendation of the type of remedial work that should be done to alleviate any potentially negative effects of the tunnel deviations.

Table 2. South Ramp 3.01.X Area, Case VI- Estimated Void Locations and Volumes

Case VI - Station 71+07.10 to 71+44.10 - Steel Set Numbers 1524 through 1554

Case #	Steel Set #	Station #	Steel Set Size	Surveyed Void Maximum Depth, m			(A) Calculated Void Cross Sectional Area, m ²	(B) Tunnel Interval m	(C) Calculated Void Volume m ³
VI				Left	Center	Right	Totals		
	1524	71+07.10	W6x20	No Data					
	1525	71+08.48	W6x20	0.60	1.20	3.11	9.81	1.23	12.1
	1526E	71+09.71	W6x20	0.60	1.20	3.42	16.52	1.84	30.4
	1527		W6x20	No Data					
	1528	71+12.16	W8x31	0.80	1.50	2.90	19.85	1.84	36.5
	1529	71+13.39	W8x31	1.00	1.20	2.30	16.77	1.86	31.1
	1530		W8x31	No Data					
	1531	71+15.87	W8x31	0.70	1.60	1.20	14.53	2.47	35.9
	1532		W8x31	No Data					
	1533	71+18.33	W8x31	0.80	1.50	1.10	13.17	2.45	32.3
	1534		W8x31	No Data					
	1535	71+20.78	W8x31	0.75	1.00	0.80	10.80	1.84	19.8
	1536	71+22.00	W8x31	1.20	1.75	2.30	19.77	1.84	36.4
	1537		W8x31	No Data					
	1538	71+24.46	W8x31	0.70	2.17	0.70	21.59	2.47	53.2
	1539		W8x31	No Data					
	1540	71+26.93	W8x31	0.70	3.15	2.50	29.98	2.47	74.1
	1541		W8x31	No Data					
	1542E	71+29.40	W8x31	1.40	4.30	1.40	39.99	2.47	99.0
	1543		W8x31	No Data					
	1544	71+31.88	W8x31	1.30	3.00	1.20	33.50	1.91	64.0
	1545	71+33.22	W8x31	1.00	1.20	0.80	12.64	1.95	24.6
	1546		W8x31	No Data					
	1547	71+35.78	W6x20	0.70	1.30	0.80	8.34	2.51	21.0
	1548		W6x20	No Data					
	1549	71+38.25	W6x20	0.00	0.00	0.60	1.62	2.47	4.0
	1550		W6x20	No Data					
	1551	71+40.71	W6x20	No Data			0.00	2.03	0.0
	1552		W6x20	No Data					
	1553		W6x20	No Data					
	1554	71+44.10	W6x20	No Data					
Total		37 m	A x B = C						574.4m ³

"E" - Existing steel set instrumented with strain gages.

Calculated Void Cross Sectional Area = Area computed in CAD that spans from half the distance before the steel set listed to half the distance beyond the next surveyed steel set (Ref. 8.23).

CAD Area - Routine calculation verified manually.

All steel sets are spaced on nominal 1.22 meter intervals. (Not all steel sets were surveyed)

Cross Sectional Area (A) times Tunnel Interval (B) = Void Volume (C)

Table 3. South Ramp 3.01.X Area, Case VI - Summary of Proposed Shotcrete/Formwork Locations and Estimated Concrete Backfill Volumes

Case Number	Station Number		Steel Set Number		Stage 1 - Void Volume Estimate m ³	Stage 2 - Void Volume Estimate m ³	Stage 1 Shotcrete/Concrete	
	From	To	From	To			Left	Right
VI	71+08.48	71+40.71	1525	1551	217.55	356.66	Y	Y

Data presented in Table 3 is based on best available information. This data is provided to facilitate an approximate estimate of the volume and location of shotcrete and concrete to be placed during the remedial work discussed in this analysis. The estimates for volumes and locations of voids are provided to aid in estimating construction cost and schedule. The data does not contain elements critical to the design.

6.3 ROCK MASS PROPERTIES AND STRUCTURAL FEATURES

The structural features of the rock mass in the ESF were documented during tunnel excavation by scanline and full peripheral mapping of the exposed rock. The results of the full peripheral mapping of the rock mass structures of the entire ESF are presented in *ESF Ground Support Confirmation* (Reference 8.9). Scanline and full peripheral mapping provided the basis for deriving the rock quality (Q) estimates for the tunnel sections. The steel sets installed and the extent of steel lagging are also shown on the full peripheral maps. The tunnel sections encountering difficult ground conditions such as in Case VI, were supported with steel sets and the crown area was blocked and lagged tightly to provide for personnel safety. Because of the poor rock quality of portions of this area, tight lagging was installed immediately after excavation. In areas of poor rock quality that is intensely fractured, such as in the fault zone of Case VI, the "Q" system does not produce a valid "Q" number. As a result of the poor quality rock and lack of visual access, most of the Case VI area was not adaptable for determination of the scanline and full peripheral rock mass quality values and was not rated. Mapping of this area shows only a limited amount of rock mass quality values but does show the extent of the support and lagging as noted in the *ESF - South Ramp Station 71+00.00 to Station 72+00.00 As Built Geology and Geotechnical Data* (Reference 8.22).

Table 4. Range of Recorded Q Values Based on Scanline and Full Peripheral Mapping in the rated sections of the Case VI 3.01.X Area.

Station	Scanline		Full Peripheral	
71+07.10 to approximately 71+14.00	Q = 6.6 to 0.14	Poor to Very Poor	Q = 2.2 to 0.53	Poor to Very Poor
Approximately 71+40.00 to 71+44.10	Q = 1.4 to NR (Not Recorded)	Poor	Q = NR (Not Recorded)	Very Poor
NOTE: Q values extracted from Reference 8.22. (No values recorded between stations 71+14.00 to 71+40.00)				

The most predominant structural feature of this 3.01.X area is an unnamed fault zone crossing the tunnel at a steep angle (tight lagging and blocking obscure much of the fault). The fault itself appears to be only a few meters wide but activity along the fault has disturbed and fractured the adjacent rock so that the faulted zone is at least 14 meters wide and extends from about Station 71+20.6 to Station 71+34.3 (See Attachment II). The fault has significant offset that results in the thermal mechanical unit TSw2 in near vertical faulted contact with the TSw1 unit at about station 71+34.3. Due to the fractured nature of this area, extensive fallout occurred both in the crown and in the walls of the tunnel during excavation.

6.4 SOUTH RAMP CASE VI 3.01.X AREA INSTRUMENTATION AND MONITORING RESULTS

Monitoring provisions at selected points within the 3.01.X areas and elsewhere within the ESF included steel sets equipped with strain gages and convergence measurement stations. No instrumentation was installed in the Case VI 3.01.X area until near the end of construction. At that time two steel sets were equipped with strain gages.

In the Case VI area, there are two steel sets instrumented with strain gages (steel set #1526 is a W6x20 set at Station 71+09 and steel set #1542 is a W8x31 set at Station 71+29). Readings at these locations include only initial (zero) readings. There are no records of readings taken at a later date. As a result, no quantitative strain gage data are available to allow for observation of the time-dependent rock mass response. There are also no convergence records available for this 3.01.X area as no measurement points were established. As a result no quantitative data are available about either tunnel convergence or stresses in the steel sets from this section of the Main Loop tunnel.

A qualitative assessment of the tunnel status within the Case VI 3.01.X section was performed during the field inspections. The most recent inspection took place on Feb. 8, 2000. The earliest detailed inspection of the same area took place on March 6, 1997. Results of these inspections are summarized and presented in Attachment II. Changes, if any, that may have taken place in the steel sets or invert segments during this period of time were not obvious, however, none of the sets appear to be "taking weight". As indicated in the more recent field report (Attachment II), in the three years that have elapsed since the initial inspection (March 6, 1997), no noticeable visual changes have occurred to the ground support.

6.5 3.01.X AREA CASE VI - GROUND AND GROUND SUPPORT ASSESSMENT

The case VI 3.01.X addressed in this analysis is in a heavily fractured and faulted zone within the TSw2 unit. An analysis of three other 3.01.X areas in the South ramp completed earlier (Reference 8.25) include an assessment of the performance of steel sets located within TCw and TSw2 thermal mechanical units at depths comparable to or greater than those encountered in Case VI along the South Ramp. The rock properties of TCw and TSw2 thermal mechanical units used in the associated numerical analysis indicate relatively strong rock types. The results of this analyses revealed no significant effects of stresses on the rock and steel set performance. To complement these earlier studies, the properties of the weaker PTn thermal mechanical unit was analyzed for the more recent numerical analysis of the three 3.01.X areas of the North ramp (Reference 8.27). The rock properties of the PTn unit are the weakest of the rock units in the ESF and are assumed more like the rock in this Case VI heavily fractured section of the TSw2 unit. The results of that analysis also revealed no significant effects of the in situ stresses on steel set performance, however, some rock loosening was noted in the voids during the project seismic event.

Most of the voids where fallout occurred were generated during construction prior to steel set installation. The exposed surfaces of the voids in some locations were coated with a thin layer of shotcrete. Wood cribbing was installed in the voids above the steel sets. Steel lagging was installed tightly at the crown to well below springline but some of the steel sets were not

adequately blocked where exposed below the lagging. Lack of blocking for the steel sets below the springline and continued minor spalling of the walls decreases the natural stability.

In Case VI 3.01.X area, the wood blocking in large voids over the tunnel crown cannot be inspected due to the closeness of the steel lagging. Therefore, the condition of the support cannot be assessed directly. However, some of the blocking can be seen through cracks in the lagging and along the left side of the tunnel over the conveyor. Loose rock can be seen behind the lagging in some sections and some raveling is occurring. Although it is difficult to determine if any unusual stress is taking place, no broken or crushed timber was observed. It was noted that several pieces of blocking were loose and laying in the voids along the tunnel walls where they had apparently fallen from higher on the wall, behind the lagging. Some shrinkage of the timber could be occurring, however, one must assume the blocking is still effective since there is no evidence of recent rockfalls.

Case VI, ground and ground support appears to behave in a manner similar to that observed at the other 3.01.X areas previously analyzed. The basis for this assessment consists of evidence gathered during the tunnel inspections and results of in situ convergence measurements and the pattern of stresses found in the steel sets elsewhere in other portions of the ESF.

The lack of apparent evidence of progressing ground support deterioration and other corroborating data are consistent with conclusions indicating that the tunnel convergence is in general, low. As discussed in detail in Reference 8.9, after the first 16 months the steel set stress and deformation measurements in other areas show no time-dependent effects, which confirms the overall stability of those sections monitored. Observations of steel set and ground performance within Case VI 3.01.X area appear to provide qualitative evidence of similar behavior. However, results of measurements from the strain gaged steel sets installed within this area are needed to confirm these observations.

The timber blocking is considered a temporary ground support and eventually, for the long-term repository operations, a more permanent support of the crown such as lightweight concrete fill should be considered. The wood blocking could be encased in lightweight concrete making the wood a "temporary function" ground support providing encasement is found to be acceptable. Ground support would therefore no longer rely on the wood support and instead the concrete will evenly distribute the loads on the steel sets. This treatment will stop further loosening of rock blocks and lessen the potential for deleterious rock movement. The filling of the crown area may be deferred at this time until the site is found to be suitable for the repository. An aggressive and routine monitoring program must be utilized if the crown area concrete fill is deferred. Generally, the voids behind steel sets below springline were not adequately blocked but could be re-blocked during normal maintenance. Loose rock fragments and spalling shotcrete were observed at a number of locations. There is not enough evidence to determine when the spalling occurred.

To ensure the integrity of the steel supports and for safety purposes, the voids at the sides of the tunnel must be filled with concrete or shotcrete applied to the rock (Stage 1, Sec. 6.6). This treatment will prevent further raveling and degradation of the tunnel walls and more importantly, provide the lateral support necessary for the steel sets to be effective in providing support for the crown rock. This work will further enhance the stability of the crown area.

6.6 3.01.X AREAS -PROPOSED REMEDIAL MEASURES

Fallout occurred during excavation on the sides of the tunnel and some continued raveling of blocks of rock occur, mainly at the sides of the tunnel. Maintenance in this area consisting of additional blocking installed between the rock and the steel sets in those areas below the lagging would assist in control of raveling of the ribs until remediation is undertaken.

To reinforce both tunnel sides, remedial measures are required (Stage 1, Figure 5). The voids on the sides, both between and behind the steel sets, should be backfilled. This can be accomplished either by applying shotcrete or backfilling the voids with ordinary concrete (Table 3). The large voids in the tunnel roof will require further treatment in order to satisfy requirements pertaining to long-term tunnel stability (Stage 2, Figure 5). The issue of timber cribbing used for blocking in the voids will need to be addressed and possibly, lightweight concrete could be used to encase the timber and fill the voids.

For concrete filling, the installed steel sets could be utilized as a part of the construction forms in a cost-effective manner. Shotcrete application provides another method for filling the voids on the sides of the tunnel. The A/E shall evaluate the construction method and ensure that steel sets will accommodate the dynamic loading due to concrete filling as well as the dead loads of the fill material. Of special concern for additional loading are the light steel sets (W6x20) that were installed in those areas of large voids above the crown (the locations of the light sets are noted on Table 2). Figure 4 illustrates a typical formwork that could be used to fill the voids on the sides of the tunnel. If the constructors plans include backfilling or pressure grouting from the surface or other methods that will create additional stress on the support, the plans shall be submitted to the A/E for approval.

Figure 5 illustrates a proposed two-stage process for filling the voids. Stage 1 is to be completed to enhance safety and to supplement the existing ground support system. Stage 2 can be deferred at this time, until the site is found to be suitable, at which time Stage 2 should also be completed. However, if Stages 1 and 2 are both deferred, a more rigorous monitoring program must be implemented. This program should incorporate installation of strain gages and programmatic recording and evaluation of readings complemented by routine visual inspection and documentation of the tunnel status. The remedial measures proposed here are consistent with those recommended for the South and North Ramp 3.01.X areas as reported in References 8.25 and 8.27.

The deviation of the tunnel alignment from the designed alignment at the Case VI 3.01.X area, adds another element that requires examination and recommendation regarding any potential impact these deviations may have on the future functionality of this portion of the ESF tunnel. Until such a study is performed, attention must be directed at maintaining the safety of the tunnel in its present alignment.

Existing Steel Sets
(W8 and W6), Use as form bracing.

Suggested Forms:
Existing Lagging/ Voids Behind Lagging- Cover spaces with fine steel mesh or fine WWF and block to inside web of steel set. Shotcrete over mesh and lagging where necessary to seal gaps to create backfill form.

Suggested Forms:
For Existing Empty Voids/No Lagging. Install thin membrane as form and shotcrete over for strength (see detail), or install removable plywood form and block plywood inside web of steel set. Add grout injection ports and vents in deep voids.

Figure 4. Typical Construction Form that Can Be Used in 3.01.X Areas for Shotcrete/Concrete Backfill (Reference 8.25, page 78).

First Stage (Backfill from Invert Segment up to approximately 2.0 m above the springline) Apply minimum 2 inch **Shotcrete** from Invert Segment up to lowest void, prepare **form** and **backfill** void with concrete/shotcrete. Backfill can be placed in several lifts if necessary to lessen pressure on forms and reduce leakage.

Backfill in Stage 1, should not exceed an elevation above crown level.
(voids in crown to remain visible after completion of Stage 1 for monitoring purposes if Stage 2 is deferred).

Second Stage (Lightweight materials - type to be selected) Install lightweight materials through grout pipes or vent pipes to entirely fill cavities and encase existing wood blocking.

Figure 5. Typical Method Applicable to Remedy 3.01.X Area Shotcrete/Concrete Backfill (Reference 8.25, page 79).

6.7 NUMERICAL ANALYSIS OF 3.01.X AREA GROUND SUPPORT

6.7.1 General

The numerical analyses of the ground support in the ESF was performed in the previous two analyses of the other seven 3.01.X areas along the North and South Ramps (References 8.25 and 8.27). Three separate rock units (TSw2, TCw and PTn) were used in those analyses and cover the range of conditions existing in all eight of the 3.01.X areas.

The installed ground support system was subjected to in situ stresses and seismic loads in the initial analysis (Reference 8.25 and 8.27). Then the ground support system was subjected to the increased load (steel sets plus the concrete/shotcrete fill, Reference 8.19) and the stability was evaluated.

The previous analyses (Reference 8.25) was performed on two of the thermal mechanical units encountered along the South Ramp of the ESF Main Loop tunnel. One location was selected as representative of stress and rock performance for the structures located within the TSw2 thermal mechanical unit at the depth of 182.9 meters. The second location analyzed was chosen to represent the performance of steel set-supported strata, located at relatively shallow depth (40.22 m) within the TCw thermal mechanical unit. In the North Ramp, the depth of the tunnel crown within the 3.01.X areas range from 23.43 meters to 146.88 meters, and are overlain by the TSw1, TCw, PTn and UO, thermal mechanical units as noted in the *Yucca Mountain Site Geotechnical Report* (Reference 8.24). Among the four thermal mechanical units encountered along the North Ramp, the PTn unit, characterized by the lowest strength parameters, was used in the analysis reported in Reference 8.27. Since the type of rock encountered at the Case VI 3.01.X area location and the magnitude of stress is within the range of the analyses performed previously, performing similar analysis for this tunnel section was considered unnecessary.

6.7.2 ANALYTICAL PROCEDURE - SEISMIC LOADING

Note: The analysis (all parameters used for seismic analysis) needs to be re-evaluated when the seismic criteria for ESF is finalized and more data is available (removal of TBV-193). The ground support designed here does not preclude the option to supplement the installed ground support to satisfy more stringent seismic criteria (higher mean peak velocities).

Analysis of the impact of an earthquake remains an integral part of the ground support design, as the dynamic loads may affect the tunnel performance. In the previous analyses (References 8.25 and 8.27), evaluation of the rock/ground support performance was also performed at selected representative locations. These locations were chosen because they represent the largest void areas within the three 3.01.X areas encountered in the North and South Ramps. The results and associated conclusions obtained from these analyses are considered valid for the current Case VI study as well.

The seismic loads were superimposed on in situ loads by applying a sinusoidal velocity wave at the bottom boundary and allowing it to propagate upward. The amplitude of the sine wave corresponds to the maximum allowable velocity of 23 cm/sec (Reference 8.13, Appendix A). For added conservatism, attenuation with depth was not considered in the analyses. A frequency of 5

Hz was assumed and used in the dynamic analysis with a duration value of 1.0 second (see Section 5.2).

The results of the dynamic analysis at the designated steel set locations were analyzed considering K_0 values of 0.3 and 1.0, respectively. By comparing the results to those obtained under the in situ loading conditions, it was observed that the yield zone around the opening is slightly larger for the seismic case. This was considered indicative of an increased potential for failure and loosening of the rock mass at the opening boundaries under seismic loading conditions. The analysis was then reworked with voids in the walls assumed to be backfilled to above spring line (Stage 1 of Figure 5). Using this additional criteria the minimum seismic stability was attained. Considering the poor ground conditions of this area it is likely that at least Stage 1 remediation will need to be completed to attain minimum seismic stability. Until then an intense visual and instrumented monitoring program, to ensure the blocking in the voids remains intact, will be needed.

In general, the results of this analysis show a higher failure probability for those areas with unfilled cavities and a beneficial effect with the concrete backfill. These concrete-backfilled portions of the void remain stable after the seismic event.

The numerical results obtained for the selected steel set locations indicating stress factors due to bending, compressive axial loads, and combined bending and axial loads are presented in detail in Reference 8.13, Appendix A. The combined stress calculations were based on the methods prescribed in *Manual of Steel Construction, Allowable Stress Design*, (Reference 8.11).

6.8 MONITORING

Monitoring of ground support performance is the top order priority in maintaining safe underground working conditions. It provides both quantitative and qualitative measure of the tunnel ground support performance. Monitoring is especially important in the 3.01.X areas as the tunnel sections show evidence of rock deterioration. This evidence can be observed in the form of raveling, presence of loose fragments of rock on wire mesh and lagging and presence of loose, possibly deteriorated rock material on the exposed portions of the tunnel ribs below the springline

The results obtained from the monitoring of instrumented (strain gages) steel sets at the other 3.01.X locations along the North and South Ramps (References 8.27 and 8.25) and ground deformation measurements indicate the following:

- The steel set stresses vary little with time, indicating no significant time-dependent effects, i.e., there is relative equilibrium between the rock mass and the interacting ground support.
- The ground deformation monitoring results (convergence data) indicate consistent, relatively small, time-dependent effects. This technique although more sensitive than steel set stress monitoring, is also more complex to use in the field. It is especially difficult to apply in the areas where raveling rock required the extensive use of steel lagging, thus limiting the access to the rock to set measurement pins.

Monitoring of this area could be by several methods that when applied in combination should provide the evidence needed to assess the ground support performance. The simplest, most cost

effective method of monitoring is to maintain a clean tunnel invert and adjacent exposed portions of the tunnel ribs. This provides a method of accounting for the rockfalls and spalled shotcrete fragments accumulating over a period of time. It is also a means of tracking tunnel performance over time. In addition, the 3.01.X area inspections should be performed according to specifically designed procedures that include both visual and instrumented data. The objective of these frequent and regular inspections is to gather information about the tunnel ground support performance on a regular basis while directing attention to specific items within the tunnel that should be checked and their satisfactory performance verified.

The second method includes installation of additional monitoring stations within Case VI 3.01.X area, currently not covered by the existing monitoring stations. This will require instrumentation of additional steel sets with strain gages or some other method that would provide a broad indication of stability within this zone. Once strain gages are installed, the periodic readings can be accomplished with relative ease. Since the measurements do not require direct access to the rock, this steel set stress monitoring technique is preferred for monitoring changes in steel set loads. Load variations caused by shifting weight of backfill or an increase in loads imposed by caving rock are important to identify for safety.

As a minimum, it is recommended that the two existing strain gage instrumented steel sets shown in Table 2 with an (E) designation be monitored on a regular basis. Based on gathered field evidence, the number of instrumented steel sets may increase or the frequency of reading may be adjusted to match the observed ground support performance. The new designated locations should correspond to the large voids and consequently should provide data representative of the performance of steel sets at these critical locations. A regular schedule of instrument readings and interpretation must be initiated as soon as possible.

Proper monitoring is key to supporting informed decisions regarding implementation of the necessary supplementary ground support or assessment of the safety of the tunnel.

6.9 3.01.X AREA PERMANENT FUNCTION GROUND SUPPORT

The existing ground support system in the 3.01.X area addressed in this analyses must be supplemented to meet the 150 year maintainable life requirement for permanent function items. Treatment of the temporary material (wood blocking), upgrading of some blocking on the tunnel side walls, and providing a means to prevent raveling or deterioration of the tunnel side walls are included in the necessary supplemental items.

The task of ground support remediation is designed as a flexible two-stage process, as shown in Figure 5. During Stage 1 the voids are repaired from the invert segment up to approximately 2.0 meters above the springline. As an option, Stage 1 includes the construction of a 5 cm (2 in.) thick layer of shotcrete to be emplaced in areas without deep voids and behind the steel sets. Shotcrete can also be used to reinforce the existing lagging and to create a shell or form, behind which the concrete backfill could be placed in several lifts to lessen the effect of hydrostatic pressure buildup during backfill. The top level of backfill should not exceed the elevation of the tunnel crown at any particular location so that if Stage 2 is deferred, inspection of the timber

support would still be possible. Completion of Stage 1 but not Stage 2 will not meet the 150-year maintainable life but would allow for continued monitoring of the timber support in the crown. The general purpose of Stage 1 is to stabilize the tunnel walls by sealing the voids and fractures and to build abutments on both sides of the tunnel, upon which the lightweight concrete could be placed. Additional benefit of the Stage 1 work is that the steel sets will be adequately supported laterally which will assure ground support compliance with the minimum seismic safety requirements.

The second Stage involves placement of backfill above the tunnel crown. Lightweight materials would preferably consist of lightweight concrete with minimum uniaxial compressive strength equal to 3.4 MPa (500 psi) capable of filling the voids. The material would be placed through a system of grout pipes to fill the voids entirely. This backfill will encase the temporary wood blocking that was installed to stabilize the rock mass around the voids and additionally will bridge between the steel sets to more evenly spread the rock loads. The backfill will also upgrade the wood blocking to "temporary function" status.

The two-stage arrangement provides the project with flexibility either to complete the ground support remedial tasks by finishing both stages or to perform one stage at a time. Stage 1 defines the minimum required to stabilize these 3.01.X sections and to meet the minimum seismic safety requirements. Completing the second stage would bring the ground support into compliance with the required 150 years maintainable opening. Stage 2 can be accomplished at a later date and until the site is found suitable for a repository. Completing these remedial tasks will be cost effective and will not have any significant long-term impact on the total cost. A systematic and rigorous monitoring of the crown section and the steel sets must be provided if the second stage is deferred. The ground support remediation system recommended for this 3.01.X areas minimizes the continued maintenance problem in these sections and eliminates the potential for deleterious rock movement in the immediate future.

Tables 2 and 3 present a summary of information pertaining to quantities of materials required during remediation of Case VI 3.01.X area. The tables include information on void volumes (table 1) and quantities of backfill required for stages 1 and 2 (table 2).

7.0 CONCLUSIONS

In general, no new data were developed as a result of this analysis. Therefore, no new information requiring Data Tracking Numbers (DTNs) were generated.

The Case VI 3.01.X area extends from Station 71+07.10 to Station 71+44.10, a distance of 37 meters. This tunnel section was constructed in accordance with Section 3.01.X of the Subsurface General Construction specification (Reference 8.4). The constructor and the A/E developed a joint plan to advance the TBM in this section in accordance with the TDPP for documentation of Plans For Continuing the TBM Advance (Reference 8.14). As a result, the as-built construction of the 3.01.X areas may vary from the recommended design of the tunnel.

The main objectives of this analyses was to document the as-built configuration of the 3.01.X area including existing voids and installed ground support. The objective was also to recommend ground support system enhancements that would ensure personnel safety and tunnel stability to meet ESF permanent function status. The as built configurations showing the void areas at cross sections along the 3.01.X areas were developed and presented in Attachment I, with a detailed description provided in Section 6.2.1.

The existing ground conditions and the status of the installed ground support system were evaluated during numerous visits to the ESF site as part of the ESF quality affecting ground support walk-downs. The concerns at these locations were identified to be the continued raveling of the highly fractured rock on the sides of the tunnel and the timber cribbing used in the large voids in the crown above the steel sets. The rock in this section is highly fractured creating unstable conditions with an expectation of continued raveling unless an effective means of support is incorporated. An earthquake could cause further loosening and instability of highly fractured rock on the tunnel sides or loosening of the support of the crown.

The steel sets were installed at nominal 1.22 meters spacing with lagging on the tunnel sides extending below springline. Presently, there is limited ground support at the tunnel sides to stop the raveling and control the potential for rock movement behind the lagging. Some of the steel sets are not adequately blocked where they can be observed below the lagging. The voids at the crown area, which are relatively large, and range from less than 1 meter up to 4.5 meters deep, were blocked with over 33,000 lineal feet of timber (according to the DIE database records of timber used between stations 70+00 and 72+00). Timber cribbing is not considered a permanent function ground support material but minimal use of timber blocking was allowed in the Plan to Continue TBM Advance. Considerably more than a minimal amount of timber was needed to continue however, and will need to be treated to make it a "temporary function" item since it probably cannot be easily or safely removed.

Although this section is within the TSw2 thermal mechanical unit, faulting has disturbed and intensely fractured the rock mass so that for purposes of analysis, the rock mass in general is expected to react more comparably to the properties of Category 1 used in the analysis of the North Ramp 3.01.X areas. This Case VI 3.01.X area was stabilized with steel support, timber cribbing and tight lagging and for these in situ conditions has shown no sign of instability since construction. However, because the existing timber support in the crown cannot be directly inspected, and no instrumentation results are available that would indicate continued stability, the

adequacy of the ground support in its present configuration cannot be reliably determined, especially under dynamic loading.

The results of numerical analyses performed on other 3.01.X areas (References 8.25 and 8.27) are adopted in this analysis as valid for the existing support in Case VI 3.01.X area. The main objective of the numerical analysis was to evaluate the reinforced ground support system's performance under seismic loading conditions. Results indicate that stability is decreased due to voids outside of the steel reinforcement but that minimum seismic safety can be attained if the voids are backfilled on the sides of the tunnel to above springline (as shown in Stage 1). In general, it was found that the stress factors, considering combined bending and axial loads, remain well below the border value of 1.0. It should be noted that the analyses represented the rock as a continuum media with rock mass properties incorporating the worst assumed conditions found in the field (Category 1 rock mass) which in general appears to be appropriate for this Case VI area. Prior to implementing Stage 2, additional analysis will be necessary to determine if the installed support is adequate to withstand the increased load due to backfilling the voids.

Evaluation of the existing conditions and the installed ground support (steel sets) indicates that requirements 1, 5, 11, 12, and 13 in Table 1 showing the ESFDR-Defined Design Requirements are not being fully addressed with the existing ground support system. The support would have to be supplemented, as prescribed in this analysis, to address these requirements. The existing ground support system does not meet the seismic requirement (requirement 1) and the 150-year long-term performance requirement (requirement 5). Use of timber results in noncompliance with requirement 11. The monitoring program would have to be upgraded as prescribed in this analysis to satisfy requirement 12 in Table 1. Finally, the supplemental ground support recommendations prescribed in this analysis would mitigate the potential for deleterious rock movement (requirement 13 in Table 1), and would provide a beneficial stabilizing effect in case of a seismic event.

Stage 1 enhancements to the existing ground support system will stabilize the rock mass surrounding the tunnel as well as provide additional support for the steel sets and will bring the system into compliance with the minimum seismic safety requirement. This will satisfy two requirements listed in Table 1 as Items #1 and #13. Completion of the Stage 2 supplementary ground support system will resolve noncompliance with the required 150-year maintainable service life (Item #5 in Table 1).

The use of timber (combustible material) makes the system non-compliant with the requirement #11 listed in Table 1. The as-built ground support system can be brought into compliance with the current requirement pursuing one of the three options. The first option is to remove the wood from the void and backfill the void with non-combustible materials (this method is possible but is considered more dangerous to personnel than the others). The second option is to accept the use of timber in 3.01.X areas, provided appropriate treatment to the timber is applied. This treatment may include encasing the existing timber within the lightweight concrete backfill, rendering the material fire retarded. The function of the concrete backfill encasement will be to fulfill the 150-year design life requirement for permanent support. (This option is acceptable for the ESF ground support system but may require further evaluation before incorporation into the final repository). The third option is to obtain change of the current ESFDR requirement, which may satisfy Item 11 but does nothing for satisfying the integrity of the support. Completion of

Stage 2 would require resolution of the issue associated with the use of combustible materials (timber) underground.

A ground support monitoring and inspection program, specifically directed toward documenting the performance of the ground support in the 3.01.X areas using both visual and instrumented data is required to fulfill requirement #12 listed in Table 1.

TBV-193 applies to the seismic inputs used in this analysis. The results of this analysis must be reevaluated for impacts on these conclusions at the time of the closure of the TBV. "This document may be affected by technical product input information that requires confirmation. Any changes to the document that may occur, as a result of completing the confirmation activities will be reflected in subsequent revisions. The status of the input information quality may be confirmed by review of the Document Input Reference System database."

The Case VI 3.01.X area addressed in this analysis is located in the rock mass represented by TSw2 thermal mechanical unit. Computer analysis using FLAC software, *FLAC, Fast Lagrangian Analysis of Continua* (Reference 8.7), were utilized in previous analyses to analyze the existing ground support system at designated steel set locations. The data from the field survey was used to incorporate a representative void depth in the analyses, subjected to in situ and seismic loading conditions. The shotcrete/concrete backfill recommended for Stage 1 remedial work was also incorporated and analyzed.

Based on the results of detailed observations of the ground conditions at these 3.01.X areas in the South and North Ramps, and the results of computer analyses, it is recommended to supplement the existing steel set support with additional support to be installed in two stages as illustrated in Figure 5.

For Stage 1, a minimum 50 mm (2 in.) thick layer of shotcrete would be applied from the invert segment up to the lowest void. Then, it is recommended that the void be filled either by shotcrete application or by preparing a construction form and using ordinary concrete for backfill. Also, for Stage 1 the concrete/shotcrete shall have a minimum compressive strength value of 2.76×10^7 Pa (4000 psi) at 28 days, with placement slump of 100 mm (4 in.) $\pm$ 25 mm (1 in.). If used, grout shall meet the same requirements for strength as concrete.

The constructor would determine the suitable backfill method and obtain A/E and Test Coordination Office (TCO) concurrence prior to construction. Coordination of the work will be necessary with testing and other potential site characterization activities.

The voids between the steel set and the rock should be filled to a height of 2 meters ($\pm$ 300 mm) above springline (Figure 5). The extent of the height of fill above 2 meters would be determined in the field by the constructor, with A/E concurrence based on the field conditions. If the decision is made to defer Stage 2, Stage 1 supplementary support should not exceed the recommended fill height. The plans for method of construction including the forms to be used shall be submitted to A/E for review and approval prior to start of construction.

For Stage 2, lightweight materials, preferably lightweight concrete would be installed to fill the cavities entirely and encase the existing wood blocking. The Stage 2 backfill material (e.g. lightweight concrete) shall have a minimum compressive strength of 3.45×10^6 Pa (500 psi) to account for tributary rock loads with an adequate safety factor. The material to be used in Stage

2 construction shall be submitted to the A/E for approval prior to the start of construction. Depending on the manner in which the backfill in the crown is placed, it may be necessary for the A/E to perform an additional analysis to evaluate the adequacy of the installed steel sets to ensure they can accommodate the loads due to dynamic loading caused by concrete backfill and dead loads of the fill material.

To ensure maximum filling of the voids during Stage 2, vent/top off return pipes should be installed at the highest points of the fallout area. The return material will indicate proper filling of these voids. Concrete used in both stages shall follow the recommendations of applicable ACI standards.

The original calculations for stability considered blocking points approximately 500 mm apart which was considered appropriate for the analysis. (References 8.25 and 8.27). A situation where no rock to steel contact exists, such as in the fallout areas at most steel sets in the Case VI area, the assumption is that the wood blocking is mostly uniform. Some wood blocking below the lagging was noted to be missing and can be installed as part of normal maintenance. Completion of Stage 1 and 2 would ensure the proper blocking and satisfy this assumption.

It should be noted that this method of providing supplementary support (Stage 1 and 2) is valid for the majority of all 3.01.X areas analyzed. Because of the deviated alignment of Case VI 3.01.X area, this procedure may require modifications in terms of design and extent of additional remedial tunnel work. A separate study is therefore required to evaluate the impact of this deviated as-built tunnel configuration. This study cannot be undertaken until the repository design has progressed to the point where it is practical to determine what the impact of remediation of this area will have on repository operation or construction. The outcome of this study should provide a recommendation of the type of remedial work that should be done to alleviate any potentially negative effects of the tunnel deviations

As shown in Table 2, two steel sets (shown with E designation) are instrumented with strain gages and monitoring on a regular basis should begin as soon as possible. The existing instrumented steel sets do not presently provide adequate information needed to assess the performance of the ground support system at these large-void locations due to lack of measurements. The two existing strain gage locations correspond to some of the largest voids in this area; however, supplemental strain gages installed on adjacent steel sets would be helpful in providing more coverage of the monitored area. The monitoring program for the existing strain gage locations should be implemented as soon as possible. The frequency of monitoring should follow the schedule defined in a procedure developed specifically for this purpose.

It is recognized that potential uncertainties exist. Those are associated with the accuracy of the estimate of void dimensions (volumes), variations in properties of the rock mass at different locations along the tunnel and inaccuracies of numerical representation of the void shapes. Also, the magnitudes of the seismic inputs used in this analysis will require verification. A more precise confirmation of the design input may lead to reduction of uncertainties, however, any potential delays associated with acquisition of such data is considered of no advantage in comparison to the implementation as recommended in this analysis. Therefore, it is stressed that the timely implementation of the recommended ground support remedial measures will have a positive effect on the overall stability of the ESF Main Loop tunnel.

Although it is recommended in this and previous analyses (References 8.25 & 8.27) that the remedial work should be completed to fulfill all project requirements, compliance with the recommendations and various items noted above and shown in Table 1 have been deferred by the project as noted in the "*System Safety Evaluation of Title III Evaluation Reports Recommended Work*" (Reference 8.28). The report indicates that "A design basis seismic event is improbable within the next seven years". It also states that "The 3.01.X locations are primarily transit areas that personnel pass through with few tasks performed in these sections". It concludes that "there is no increased or decreased risk to personnel by deferral of this work. Routine inspection of the 3.01.X areas was noted as a requirement however, to monitor the ground support and perform necessary maintenance."

The final design of the subsurface repository has not been finalized. Remediation of the Case VI 3.01.X area may adversely impact the use of this area as a future construction access. Work performed today may have major consequences on future activities requiring, under worst case scenarios, major rework to meet still unidentified requirements. It may be in the project's best long-term interests to delay remediation of this area pending resolution of SR issues and adoption of LA layout, as long as the recommended visual observation and instrumented monitoring programs, do not indicate more immediate remedial actions.

If a decision is made to delay remediation, the tunnel crown area must be monitored regularly to detect any raveling or changes in the status of wood blocking. The instruments should be read and the results analyzed according to the schedule defined in the designated procedure. Periodic review of the field measurement results and observations and associated records shall be used to rationalize decisions regarding the additional ground support installation schedule. Any sudden changes in ground conditions or unusual readings of the instruments shall be reported to the A/E and their cause, any potential impact and the necessary remedial measures shall be evaluated immediately. If Stage 1 construction is deferred for some time, a more rigorous monitoring of Case VI 3.01.X area must be conducted utilizing weekly visual observations and regular instrument monitoring and analysis of the data.

8.0 INPUT AND REFERENCES

The following is a complete list of references used in this analysis. Data Tracking Numbers (DTN), Record Processing Center Accession numbers (ACC), and Technical Information Center catalog numbers (TIC) are shown as appropriate for each reference. Legacy data from the following references were not used as direct inputs in the analyses and therefore are not assigned a TBV number. The changes in these data will not impact the conclusions of this analysis.

- 8.1** CRWMS M&O 2000. *South Ramp Deviated Alignment 3.01.X Area Ground Support Analysis (Station 71+07.10 to 71+44.10)*. Development Plan BAB000000-01717-4600-00041 REV 00. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.20000817.0506.
- 8.2** CRWMS M&O 1996. *QA Classification Analysis of Ground Support Systems*. BABEE0000-01717-2200-00001 REV 05. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19970508.0371.

- 8.3 CRWMS M&O 1998. *SC50560 Resolve Q Ground Support Issues*. Activity Evaluation: November 14, 1998. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19981229.0130.
- 8.4 CRWMS M&O 1997. *Subsurface General Construction*. BAB000000-01717-6300-01501 REV 05. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19980127.0685.
- 8.5 CRWMS M&O 1999. *3.01.X Areas Survey Information (Steel Set Stations and Steel Set Numbers) and Type of Support/Materials*. Input Transmittal TFD-FED-99193.Ta. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19990823.0188.
- 8.6 (Not Used).
- 8.7 Itasca Consulting Group 1996. *FLAC, Fast Lagrangian Analysis of Continua*, Version 3.3. Volume I. Minneapolis, Minnesota: Itasca Consulting Group. TIC: 236418
- 8.8 Not Used
- 8.9 CRWMS M&O 1998. *ESF Ground Support Confirmation*. BABEE0000-01717-5705-00003 REV 00. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19980727.0308.
- 8.10 CRWMS M&O 1999. *ESF Ground Support Design Analysis*. BABEE0000-01717-0200-00002 REV 01. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.20000119.0135.
- 8.11 AISC (American Institute of Steel Construction Inc.) 1991. *Manual of Steel Construction, Allowable Stress Design*, 9th Edition. Chicago, Illinois: American Institute of Steel Construction Inc. TIC: 4254.
- 8.12 Merritt, F.S., ed. 1983. *Standard Handbook for Civil Engineers*. 3rd Edition. New York, New York: McGraw-Hill. TIC: 206892.
- 8.13 YMP (Yucca Mountain Site Characterization Project) 1997. *Exploratory Studies Facility Design Requirements*. YMP/CM-0019, Rev. 2, ICN 1. Las Vegas, Nevada: Yucca Mountain Site Characterization Office. ACC: MOL.19980107.0544; MOL.19960926.0065.
- 8.14 CRWMS M&O 1998. *Technical Document Preparation Plan for Plans for Continuing TBM Advance*. BAB000000-01717-4600-00024 REV 01. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19980410.0591.
- 8.15 MO9907DVSABESF.000 *Design Versus As-Built of the Exploratory Studies Facility (ESF) South Ramp*. Submittal Date: 07/30/1999.
- 8.16 (Not Used)
- 8.17 CRWMS M&O 1998. *Geology of the Exploratory Studies Facility Topopah Spring Loop*. BAB000000-01717-0200-00002 REV 01. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19980415.0283.
- 8.18 CRWMS M&O 1995. *Cast-In-Place Concrete-Surface*, BAB000000-01717-6300-03300 REV 02. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19960514.0331.

- 8.19 ACI 318-95 1995. *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)*. Farmington Hills, Michigan: American Concrete Institute. TIC: 233584.
- 8.20 (Not Used)
- 8.21 (Not Used)
- 8.22 DOI (U.S. Department of the Interior) 1997. *ESF - South Ramp Station 71+00.00 to Station 72+00.00 As Built Geology and Geotechnical Data*. OA-46-277. Mercury, Nevada: U.S. Department of the Interior. ACC: MOL.19980127.0365.
- 8.23 MO0002GSC00075.000. 3.01.X Void Survey, As-Built Steel Sets Used as Control, North Ramp and South Ramp. Submittal date: 02/15/2000.
- 8.24 CRWMS M&O 1997. *Yucca Mountain Site Geotechnical Report*. B00000000-01717-5705-00043 REV 01. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19980212.0354.
- 8.25 CRWMS M&O 1999. *South Ramp 3.01.X Area Ground Support Analysis*. BAB000000-01717-0200-00023 REV 00. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.19990908.0318.
- 8.26 Not Used
- 8.27 CRWMS M&O 2000. *North Ramp 3.01.X Areas Ground Support Analysis*. BAB000000-01717-0200-00154 REV 00. Las Vegas, Nevada: CRWMS M&O. ACC: MOL.20000627.0253.
- 8.28 Gwyn, D.W. 1999. "System Safety Evaluation of Title III Evaluation Reports Recommended Work." Interoffice correspondence from D.W. Gwyn (CRWMS M&O) to W. Kennedy, August 11, 1999, LV.SA.WWP.08/99-077, with attachment. ACC: MOL.19990830.0320; MOL.19990830.0321.

9. ATTACHMENTS

Attachment I – Estimates of Void Dimensions Based on Field Survey Data - Case VI, 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel

Pages I-1 to I-17

Attachment II - Report of Field Tunnel Inspection Case VI 3.01.X Area

Pages II-1 to II-6

Attachment III –The As-Built Tunnel Alignment Survey Data, Case VI 3.01.X Area

Pages III-1 to III-5

ATTACHMENT I

Estimates of Void Dimensions Based on Field Survey Data - Case VI, 3.01.X Area, South Ramp
of the ESF Main Loop Tunnel (References 8.5 and 8.23)

Figure I - 1. Locations of the Case VI 3.01.X Area Addressed in this Analysis. Case Numbers and Associated Stations as Defined in Reference 8.5.

STATION: 71+08.48

STEEL SETS: 1525

AREA(m²): 9.814

Figure I-2. Estimate of Void Dimensions at Steel Set #1525 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+09.71

STEEL SETS: 1526

AREA(m²): 16.515

Figure I-3. Estimate of Void Dimensions at Steel Set #1526 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+12.16

STEEL SETS: 1528

AREA(m²): 19.846

Figure I-4. Estimate of Void Dimensions at Steel Set #1528 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+13.39

STEEL SETS: 1529

AREA(m²): 16.767

Figure I-5. Estimate of Void Dimensions at Steel Set #1529 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+15.87

STEEL SETS: 1531

AREA(m²): 14.528

Figure I-6. Estimate of Void Dimensions at Steel Set #1531 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+18.33

STEEL SETS: 1533

AREA(m²): 13.166

Figure I-7. Estimate of Void Dimensions at Steel Set #1533 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+20.78

STEEL SETS: 1535

AREA(m²): 10.803

Figure I-8. Estimate of Void Dimensions at Steel Set #1535 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+22.00

STEEL SETS: 1536

AREA(m²): 19.771

Figure I-9. Estimate of Void Dimensions at Steel Set #1536 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

Figure I-10. Estimate of Void Dimensions at Steel Set #1538 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

Figure I-11. Estimate of Void Dimensions at Steel Set #1540 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+29.40

STEEL SETS: 1542

AREA(m²): 39.995

Figure I-12. Estimate of Void Dimensions at Steel Set #1542 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+31.88

STEEL SETS: 1544

AREA(m²): 33.502

Figure I-13. Estimate of Void Dimensions at Steel Set #1544 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+33.22

STEEL SETS: 1545

AREA(m²): 12.640

Figure I-14. Estimate of Void Dimensions at Steel Set #1545 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+35.78

STEEL SETS: 1685

AREA(m²): 8.345

Figure I-15. Estimate of Void Dimensions at Steel Set #1547 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

STATION: 71+38.25

STEEL SETS: 1549

AREA(m²): 1.62

Figure I-16. Estimate of Void Dimensions at Steel Set #1549 Location Based on Field Survey Data - Case VI 3.01.X Area, South Ramp Section of the ESF Main Loop Tunnel (References 8.5 and 8.23). All dimensions are in the metric system.

ATTACHMENT II

Report of Field Tunnel Inspection Case VI 3.01.X Area

Attachment II

MEMO

To: Marek Mrugala
From: Ralph C. Dow
Date: Feb. 10, 2000
Subject: Report of Field Inspection of Case VI, 3.01.X Area With the Deviated Alignment.

On Feb. 8, 2000 this section of tunnel was again visited in preparation for the initiation of an analysis of the existing ground support. The team included Marek Mrugala, Agustin R. Passalacqua, and myself.

This section of tunnel was first inspected by me in March 6, 1997 and the report prepared at that time appears to be still valid. A portion of the instrumentation recommended at that time has been installed (strain gages on two steel sets, #1526 & 1542). (That report appears at the end of this memo).

In the three years that has elapsed since the first inspection, no noticeable visual changes have occurred to the ground support. The strain gages that were installed in 1997 are not currently read and no zero readings or calibration data has not yet been located.

Basically the wood blocking in the large voids over the tunnel cannot be inspected due to the closeness of the steel lagging, therefore the condition of the support cannot be assessed. Along the left side of the tunnel over the conveyor, some of the blocking can be seen and was inspected. Much of the blocking was installed initially in a haphazard manner so it is difficult to determine if any unusual stress is taking place. No broken or crushed timber was seen but several pieces of blocking were loose and laying in the voids as if fallen from somewhere higher behind the lagging. One must assume the blocking is still effective since there is no evidence of recent rockfalls.

A spreadsheet showing details of the support, surveyed void volumes, deviation of the crown and invert segments from that designed, and scanline and instrumentation data was reviewed and confirmed during this visit. This spreadsheet is attached to this memo and titled: Walk-Down Record Sheet, 3.01.X Area, Case VI. The notes on the bottom indicate details on how the data was obtained for the record. (*) Note: This spreadsheet is not attached to this document.

As with the recommendations made for the other 3.01.X areas, similar remedial measures need to be taken to maintain the safety of the opening. This includes:

1. Immediate monitoring of the existing instrumentation.
2. An analysis of the adequacy of the steel sets, especially the W6 x 20 Light steel support and the W8 x 31 steel that is erected on a steep grade and is fitted with extra spacers along with sloping

rather than horizontal set ties. The analysis also needs to take into account that some of the Invert Segments rest on a sand fill rather than rock. At the moment, the Factor of Safety is at least 1 or greater. However, if the blocking were to fail locally or the weight increases over the crown when the voids are backfilled, residual strength will be required in the steel. The present state of stress on the blocking or on the steel is unknown.

3. This section needs to be backfilled, (staged), on the sides and crown to encase the wood and provide uniform load to the steel sets. Little can be done with the Invert Segments short of grouting which may not be acceptable at this time.

It is argued that nothing should be done to this area until the Repository staff provides their input to the deviated alignment. This is not necessarily so because:

Removing the existing steel sets to mine along the crown to make the tunnel more level obviously, cannot be done safely, due to the known unstable ground conditions and large areas of voids above the crown. For this reason alone, remedial stabilization in the present configuration should not be delayed as it would enhance the tunnel stability and would not affect any future decisions to realign this section. The reason is that any re-alignment will necessarily include deepening and flattening of the existing grade and will involve removing the invert segments, possibly cutting off the lower portion of the existing steel sets, securing the cutoff steel sets to the wall temporarily to allow for deepening and leveling, then lining with concrete or other support.

My recommendation would be to proceed with remediation in the current alignment using techniques similar to the other 3.01.X area analysis providing the steel set analysis is positive with the added weight of the backfill. Monitoring of the installed instrumentation would be mandatory.

To Files

From: R.C. Dow

Subject: Tunnel Inspection Report

March 6, 1997

Bob Skorseth and I performed a preliminary inspection on the portion of tunnel that was constructed under the terms of section 3.01.x of specification 01501, between approximately Station 70+96.24 and Station 71 + 44.1 m. The "Contractors Plan For Continuing TBM advance" with the cutter head at Station 71+20 and when the cutter head was at Station 71+43 was reviewed along with a cross-section sketch (Sketch No. 97-01, dated 3/5/97) based partially on survey data, but in some sections, survey data is still forthcoming. There are basically three zones of concern within this section. 1)the first portion with W6x20 steel sets, 2)the portion with W8x31 steel sets, and 3)the portion where the tunnel dips down that also consists of W6x20 steel sets.

Existing Conditions Observed

The zone from **70+96.24 to 71+20.6** consists of W6x20 steel sets spaced on nominal 4 foot centers with partial lagging which is considered Class 3a support. Large voids exist above the steel lagging in at least the easterly 28 feet of this section. Wood blocking can be seen behind the lagging in the crown and around to spring line and below. Spalled rock fragments can be seen resting on the lagging around the crown. This section is relatively on line and grade.

Concerns are :

1. Even though the fallout area was coated with a skin of shotcrete above the crown, there is no guarantee of future stable conditions.

2. Considerable point load blocking exists on the steel sets from the crown around to spring line, the blocking around spring line in some sections does not appear adequate and could be seriously reducing the load the steel is required to carry.

Possible alternatives (or combinations of alternatives) leading to acceptance of this section of tunnel:

1. Instrument at least one steel set in this section with strain gages.

2. Install one MPBX in the crown to monitor both rock relaxation above the fallout area and steel set settlement (Sets taking weight).

3. Backfill the voids above and on the sides of the steel sets to stabilize and spread the load. Use of a light weight concrete would be preferable. This alternative needs to be reanalyzed to make sure the dead weight of the backfill does not exceed the required safety factor for the support.

Staging the backfill , by first backfilling the tunnel sides and allowing this to set properly will aid the support.

4. Three or four jump sets may be necessary in this section where W6x20 sets were installed. This also needs to be re-analyzed.

5. Construct a permanent fix -- Install a cast-in-place concrete liner, backfill the existing voids and grout the crown.

The zone from **71+20.6 to 71+34.3** is within the fault zone and has W8x31 steel sets installed on nominal 4 foot centers with considerable lagging and wood blocking installed. Large voids exist above the crown and around to the spring line and below. Some of these sets have extra Inserts and spacers installed resulting in non- symmetrical support. Most of this section declines slightly from design grade a maximum of 0.707 m.

Concerns are :

1. Include all of those concerns listed above.
2. Also existing grade may not be acceptable.

Possible Alternatives for acceptance :

1. Include all of the possible alternatives listed above.
2. Adjust the invert grade (by adding select backfill or other temporary smoothing or leveling method) providing subsequent repository operations are not impacted by the reduction in inside diameter as a result.
3. Adjust the grade to an acceptable smooth transition by lowering the invert rather than removing any of the existing steel support.

The zone from Station **71+34.3 to 71+44.1** is the zone that is basically beyond the fault zone and rock can be seen exposed in many sections just above the partial lagging of the W6x20 steel sets. (Fewer voids appear to exist in the crown in this section). This is the section however, that has declined in grade the most due probably to the inability to effectively use the top thrust pad on the rear of the TBM and keep the machine level. The soft nature of the invert could have also have been a factor when driving through the fault zone. This section is below design grade from about 0.707 m to a maximum of 1.417 m. Some invert fill (possibly 1 foot or so) has apparently been used to smooth the grade in this transition section already. Beyond about Station 71+44.1 the contractor has resumed excavation on about a 2.88% upward grade to be able to reach the designed vertical curve at the required point near the portal.

Concerns are:

1. Basically the grade is low by about 1.4 m and the alignment is off by possibly 0.6 m or so.
2. The rock in this section appears much more stable and fewer voids exist above the crown.
3. The W6x20 steel sets appear adequate for support even though mis-aligned.

Possible Alternatives for acceptance are:

1. Smooth the grade with select backfill or other method if acceptable to repository as noted above.
2. Backfill the voids above the steel where it exists, especially where wood blocking was used.
3. In the easterly portion of this section it may be possible to replace some of the steel sets with rockbolts, channel and mesh if it is deemed important that the vertical diameter is not going to be adequate when the invert is smoothed.

ATTACHMENT III

Location data from Design versus As-Built analyses of the ESF South Ramp

Case VI 3.01.X Area

Data Source: DTN: MO9907DVSABESF.000 (Reference 8.15)

DTN: MO9907DVSABESF.000

Row #	Location	Date	Reference	Reference	Offset	Offset
					Horizontal (+=Right) (-=Left)	Vertical (+=Above) (-=Below)
1	69+99.67	2/09/99	YMP	109/74	0.04	-0.02
2	70+04.54	2/09/99	YMP	109/74	0.03	-0.03
3	70+10.64	2/09/99	YMP	109/74	0.05	-0.03
4	70+15.53	2/09/99	YMP	109/74	-0.01	-0.03
5	70+20.40	2/09/99	YMP	109/74	-0.01	-0.01
6	70+25.27	2/09/99	YMP	109/74	-0.01	-0.01
7	70+30.15	2/09/99	YMP	109/74	-0.03	-0.02
8	70+35.02	2/09/99	YMP	109/74	-0.01	-0.01
9	70+39.91	2/09/99	YMP	109/74	-0.01	0.01
10	70+46.01	2/09/99	YMP	109/74	-0.03	0.01
11	70+49.67	2/09/99	YMP	109/74	-0.01	-0.01
12	70+54.54	2/09/99	YMP	109/74	-0.02	-0.03
13	70+60.64	2/09/99	YMP	109/74	0.03	0.01
14	70+64.29	2/09/99	YMP	109/74	0.01	0.06
15	70+70.39	2/09/99	YMP	109/74	-0.03	0.12
16	70+76.48	2/09/99	YMP	109/74	-0.01	-0.04
17	70+81.39	2/09/99	YMP	109/74	0.00	-0.06
18	70+86.30	2/09/99	YMP	109/74	-0.01	-0.01
19	70+92.39	2/09/99	YMP	109/74	-0.01	0.00
20	70+99.74	2/09/99	YMP	109/74	-0.05	-0.01
21	71+07.09	2/09/99	YMP	109/74	0.00	-0.04
22	71+11.99	2/09/99	YMP	109/74	0.00	0.04
23	71+15.69	2/09/99	YMP	109/74	-0.01	-0.04
24	71+19.40	2/09/99	YMP	109/74	0.01	-0.08
25	71+24.33	2/09/99	YMP	109/74	-0.06	-0.10
26	71+29.27	2/09/99	YMP	109/74	-0.08	-0.48
27	71+34.33	2/09/99	YMP	109/74	0.04	-0.75
28	71+39.19	2/09/99	YMP	109/74	0.43	-1.32
29	71+44.09	2/09/99	YMP	109/74	0.51	-1.39
30	71+49.88	2/09/99	YMP	109/74	0.62	-1.47
31	71+52.27	2/09/99	YMP	109/74	0.65	-1.50
32	71+57.66	2/09/99	YMP	109/74	0.69	-1.57
33	71+63.05	2/09/99	YMP	109/74	0.74	-1.62
34	71+68.44	2/09/99	YMP	109/74	0.73	-1.66
35	71+73.83	2/09/99	YMP	109/74	0.78	-1.79
36	71+79.24	2/09/99	YMP	109/74	0.74	-1.71
37	71+84.63	2/09/99	YMP	109/74	0.75	-1.82
38	71+89.62	2/09/99	YMP	109/74	0.76	-1.78
39	71+95.42	2/09/99	YMP	109/74	0.83	-1.80
40	72+00.83	2/09/99	YMP	109/74	0.76	-1.73
41	72+06.20	2/09/99	YMP	109/74	0.79	-1.71
42	72+11.60	2/09/99	YMP	109/74	0.80	-1.67
43	72+16.99	2/09/99	YMP	109/74	0.79	-1.64
44	72+22.40	2/09/99	YMP	109/74	0.79	-1.62
45	72+27.78	2/09/99	YMP	109/74	0.81	-1.55
46	72+33.42	2/09/99	YMP	109/74	0.78	-1.54
47	72+38.82	2/09/99	YMP	109/74	0.82	-1.71
48	72+44.97	2/09/99	YMP	109/74	0.84	-1.49
49	72+49.59	2/09/99	YMP	109/74	0.89	-1.55
50	72+54.97	2/09/99	YMP	109/74	0.85	-1.53
51	72+59.79	2/09/99	YMP	109/74	0.82	-1.56
52	72+64.65	2/16/99	YMP	109/78	0.77	-1.57
53	72+69.53	2/16/99	YMP	109/78	0.79	-1.58
54	72+76.80	2/16/99	YMP	109/78	0.75	-1.59
55	72+82.92	2/16/99	YMP	109/78	0.76	-1.52
56	72+87.79	2/16/99	YMP	109/78	0.74	-1.49
57	72+92.67	2/16/99	YMP	109/78	0.73	-1.48
58	72+98.76	2/16/99	YMP	109/78	0.70	-1.46
59	73+03.57	2/16/99	YMP	109/78	0.68	-1.34
60	73+08.96	2/16/99	YMP	109/78	0.67	-1.20
61	73+14.38	2/16/99	YMP	109/78	0.66	-1.19
62	73+19.77	2/16/99	YMP	109/78	0.65	-1.19

DTN: MO9907DVSABESF.000

Row #	Location	Date	Reference	Reference	Offset	Offset
					Horizontal (+=Right) (-=Left)	Vertical (+=Above) (-=Below)
63	73+25.16	2/16/99	YMP	109/78	0.61	-1.22
64	73+30.55	2/16/99	YMP	109/78	0.66	-1.17
65	73+35.94	2/16/99	YMP	109/78	0.61	-1.17
66	73+41.34	2/16/99	YMP	109/78	0.62	-1.20
67	73+46.74	2/16/99	YMP	109/78	0.58	-1.23
68	73+52.13	2/16/99	YMP	109/78	0.60	-1.22
69	73+57.53	2/16/99	YMP	109/78	0.60	-1.22
70	73+62.92	2/16/99	YMP	109/78	0.60	-1.16
71	73+68.32	2/16/99	YMP	109/78	0.63	-1.11
72	73+73.70	2/16/99	YMP	109/78	0.58	-1.06
73	73+79.08	2/16/99	YMP	109/78	0.58	-1.05
74	73+84.50	2/16/99	YMP	109/78	0.57	-1.03
75	73+89.90	2/16/99	YMP	109/78	0.55	-1.02
76	73+95.28	2/16/99	YMP	109/78	0.55	-1.01
77	74+00.67	2/16/99	YMP	109/78	0.50	-1.02
78	74+06.06	2/16/99	YMP	109/78	0.49	-0.99
79	74+11.45	2/16/99	YMP	109/78	0.47	-0.95
80	74+18.36	2/16/99	YMP	109/78	0.48	-0.95
81	74+22.23	2/16/99	YMP	109/78	0.47	-0.95
82	74+27.63	2/16/99	YMP	109/78	0.46	-0.96
83	74+32.32	2/16/99	YMP	109/78	0.44	-0.95
84	74+38.60	2/16/99	YMP	109/78	0.44	-0.96
85	74+43.99	2/16/99	YMP	109/78	0.43	-0.95
86	74+49.40	2/16/99	YMP	109/78	0.42	-0.95
87	74+54.78	2/16/99	YMP	109/78	0.40	-0.91
88	74+60.18	2/16/99	YMP	109/78	0.41	-0.87
89	74+65.57	2/16/99	YMP	109/78	0.40	-0.87
90	74+70.95	2/16/99	YMP	109/78	0.38	-0.87
91	74+76.76	2/16/99	YMP	109/78	0.37	-0.84
92	74+81.63	2/16/99	YMP	109/78	0.36	-0.82
93	74+86.53	2/16/99	YMP	109/78	0.35	-0.82
94	74+92.66	2/16/99	YMP	109/78	0.34	-0.78
95	74+97.56	2/16/99	YMP	109/78	0.33	-0.75
96	75+03.67	2/16/99	YMP	109/78	0.31	-0.76
97	75+08.80	2/16/99	YMP	109/78	0.28	-0.81
98	75+13.72	2/16/99	YMP	109/78	0.29	-0.76
99	75+19.82	2/16/99	YMP	109/78	0.27	-0.77
100	75+24.71	2/16/99	YMP	109/78	0.27	-0.79
101	75+29.47	2/16/99	YMP	109/78	0.24	-0.74
102	75+35.75	2/16/99	YMP	109/78	0.31	-0.74
103	75+39.99	5/27/99	YMP	123/72	0.25	-0.73
104	75+45.01	5/27/99	YMP	123/72	0.23	-0.64
105	75+49.75	5/27/99	YMP	123/72	0.23	-0.59
106	75+51.98	5/27/99	YMP	123/72	0.21	-0.58
107	75+57.36	5/27/99	YMP	123/72	0.20	-0.58
108	75+62.75	5/27/99	YMP	123/72	0.19	-0.55
109	75+68.23	5/27/99	YMP	123/72	0.18	-0.53
110	75+73.62	5/27/99	YMP	123/72	0.16	-0.54
111	75+79.01	5/27/99	YMP	123/72	0.16	-0.53
112	75+84.40	5/27/99	YMP	123/72	0.16	-0.53
113	75+89.82	5/27/99	YMP	123/72	0.16	-0.52
114	75+95.20	5/27/99	YMP	123/72	0.16	-0.46
115	76+00.59	5/27/99	YMP	123/72	0.14	-0.45
116	76+05.97	5/27/99	YMP	123/72	0.11	-0.43
117	76+11.36	5/27/99	YMP	123/72	0.11	-0.41
118	76+16.75	5/27/99	YMP	123/72	0.08	-0.38

DTN: MO9907DVSABESF.000

Row #	Location	Date	Reference	Reference	Offset	Offset
					Horizontal (+=Right) (-=Left)	Vertical (+=Above) (-=Below)
119	76+22.15	5/27/99	YMP	123/72	0.08	-0.32
120	76+27.54	5/27/99	YMP	123/72	0.08	-0.32
121	76+32.96	5/27/99	YMP	123/72	0.08	-0.31
122	76+38.42	5/27/99	YMP	123/72	0.05	-0.30
123	76+43.58	5/27/99	YMP	123/72	0.06	-0.26
124	76+49.67	5/27/99	YMP	123/72	0.04	-0.30
125	76+54.58	5/27/99	YMP	123/72	0.06	-0.25
126	76+59.97	5/27/99	YMP	123/72	0.05	-0.22
127	76+65.53	5/27/99	YMP	123/72	0.04	-0.22
128	76+71.66	5/27/99	YMP	123/72	0.04	-0.23
129	76+76.53	5/27/99	YMP	123/72	0.06	-0.15
130	76+81.41	5/27/99	YMP	123/72	0.04	-0.20
131	76+87.52	5/27/99	YMP	123/72	0.05	-0.21
132	76+92.42	5/27/99	YMP	123/72	0.05	-0.18
133	76+97.25	5/27/99	YMP	123/72	0.06	-0.13
134	77+00.90	5/27/99	YMP	123/72	0.05	-0.11
135	77+08.22	5/27/99	YMP	123/72	0.05	-0.05
136	77+13.08	5/27/99	YMP	123/72	0.04	-0.07
137	77+19.15	5/27/99	YMP	123/72	0.08	-0.03
138	77+24.05	5/27/99	YMP	123/72	0.05	-0.05
139	77+30.24	5/27/99	YMP	123/72	0.06	-0.02
140	77+36.32	5/27/99	YMP	123/72	0.06	-0.05
141	77+41.24	5/27/99	YMP	123/72	0.08	-0.03
142	77+46.16	5/27/99	YMP	123/72	0.09	-0.05
143	77+51.06	5/27/99	YMP	123/72	0.06	0.04
144	77+57.22	5/27/99	YMP	123/72	0.11	0.03
145	77+62.62	5/27/99	YMP	123/72	0.09	0.05
146	77+67.85	5/27/99	YMP	123/72	0.16	0.01
147	77+74.28	5/27/99	YMP	123/72	0.15	0.08
148	77+80.17	5/27/99	YMP	123/72	0.16	0.09
149	77+86.40	5/27/99	YMP	123/72	0.16	0.00
150	77+90.06	5/27/99	YMP	123/72	0.15	0.06
151	77+94.92	5/27/99	YMP	123/72	0.16	0.01
152	77+99.80	5/27/99	YMP	123/72	0.17	0.04
153	78+05.89	5/27/99	YMP	123/72	0.17	0.04
154	78+11.98	5/27/99	YMP	123/72	0.16	0.04
155	78+16.84	5/27/99	YMP	123/72	0.16	0.04
156	78+21.72	5/27/99	YMP	123/72	0.17	0.12
157	78+27.82	5/27/99	YMP	123/72	0.16	0.06
158	78+32.70	5/27/99	YMP	123/72	0.17	0.08
159	78+37.47	5/27/99	YMP	123/72	0.15	0.05
160	78+43.68	5/27/99	YMP	123/72	0.14	0.02
161	78+49.78	5/27/99	YMP	123/72	0.17	0.03
162	78+54.68	5/27/99	YMP	123/72	0.17	-0.02
163	78+59.60	5/27/99	YMP	123/72	0.14	0.02
164	78+65.69	5/27/99	YMP	123/72	0.16	0.02
165	78+70.60	5/27/99	YMP	123/72	0.19	-0.05
166	78+76.76	5/27/99	YMP	123/72	0.28	0.07

Figure III-1. Plot of the Offset Between the Designed and the As-Built ESF Main Loop Tunnel Section. Offset Starts at the Case VI 3.01.X Area - South Ramp Location.