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CHPRC-03364
Revision 0

PUREX Tunnel 1 Engineering Evaluation

Prepared for the U.S. Department of Energy
Assistant Secretary for Environmental Management

Contractor for the U.S. Department of Energy
under Contract DE-AC06-08RL14788

P.O. Box 1600
Richland, Washington 99352

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CH2M HILL Plateau Remediation Company

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By Lana Perry at 2:54 pm, Jun 27, 2017

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Title: *PUREX TUNNEL 1 ENGINEERING EVALUATION*
Project Number 693839

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Date

ch2m	Calculation Title: PUREX Tunnel 1 Engineering Evaluation	Project Number: 693839.BS Project: PUREX Tunnel 1 Client: CHPRC

1. Purpose

This engineering evaluation of PUREX Tunnel 1 was conducted to:

- Determine the cause of partial roof collapse in PUREX Tunnel 1
- Provide a structural evaluation for PUREX Tunnel 1
- Assess if there is an immediate risk of further failures in PUREX Tunnel 1

It will be submitted to the State of Washington Department of Ecology to fulfill Administrative Order Docket #14156, dated May 10, 2017, Corrective Action 1.

2. Introduction

On May 9, 2017, a portion of the PUREX Tunnel 1 wood timber roof structure was observed to have collapsed into the tunnel resulting in a hole approximately 19 ft wide by 17 ft long. Actual time of the collapse and cause of the failure has not been determined. Potential factors contributing to the collapse are speculated to include heavy rainfall on May 4 and 5, 2017, deterioration of tunnel wood timber structural support members due to prolonged exposure to high levels of radioactivity, and influence of low vibration sources near the site such as local thunderstorms or distant low-magnitude seismic activity. Due to the uncertainty in the condition and structural integrity of the remaining roof and wall timber supports, measures were taken to prevent additional roof loads or personnel from being placed over top of Tunnel 1 and within the roof load zone of influence until permanent stabilization measures can be taken.

Fifty-three truckloads of uncompacted soil fill were placed through the roof opening at the collapsed area to stabilize the tunnel support walls and to cap and seal off the tunnel interior space from further exposure to the atmosphere. A temporary protective cover was installed over the full length of Tunnel 1 to reduce soil loading over the tunnel by minimizing or eliminating rainfall water infiltration into the 8 ft high soil berm over the tunnel timber roof structure. The protective cover consists of water resistant tarpaulin material which has an expected design life on the order of months.

Construction of Tunnel 1 was completed in 1956 as part of the PUREX Plant construction project. The Tunnel 1 consists of three sections, a water-fillable door, a storage area, and a ventilation shaft. The water-fillable door located at the north end of the tunnel is housed in a concrete structure. The water-fillable door is 24.5 ft high, 22 ft wide, and 7 ft deep, constructed of 0.5 inch thick steel plate and hollow to permit filling with water for radiation shielding. The storage area, which is the main portion of Tunnel 1, extends from the water-fillable door south 358 ft to the ventilation shaft and is 22 ft high by 19 ft wide with a 1% grade downward slope from north to south. The roof and walls are constructed of 12 inch by 14 inch rough sawn creosote pressure treated Douglas-Fir wood timbers with the exception that the first 100 ft of the east wall was constructed with 3 ft thick reinforced concrete. Timber wall supports bear on a 1 ft thick by 3 ft wide continuous unreinforced concrete footing. The timber structure is covered with 90 lbs mineral surface roofing material. A minimum depth of 8 ft of uncompacted soil fill was placed over the top of the tunnel. The tunnel floor consists of two railroad track rails supported by 7 inch by 9 inch rough sawn creosote pressure treated Douglas-Fir wood timber railroad ties that extend between the wall footings to brace and support the base of the tunnel walls. Railroad ties are laid on a gravel bed with spaces between ties filled with gravel ballast to the top of members.

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The ventilation shaft is located at the south end of the tunnel. The shaft is approximately 5 ft by 5 ft in cross section and constructed of reinforced concrete. The ventilation shaft protrudes approximately 1 ft above grade and is capped with a single-stage high-efficiency particulate air filter, an exhaust fan, and a 20 ft tall stack. The ventilation system is not in operation. The tunnel was filled to its capacity in 1965 with eight rail cars, each of 40 to 42 ft in length, containing radioactive process equipment. Tunnel 1 remains in the aforementioned state while the structural evaluation and corrective actions for Tunnel 1 are completed.

3. Design Input

Design input used in the structural evaluation consists of the following:

A. Tunnel 1 Drawings and Specifications
   - H-2-55587 (drawing), Structural Floor Plan & Section, 1955 Tunnel 1
   - H-2-55588 (drawing), Structural Sections & Details, 1955 Tunnel 1
   - H-2-55589 (drawing), Structural Sections & Details, 1955 Tunnel 1
   - HWS-5638, *Specifications for Disposal Facility for Failed Equipment Project CA-513-A*, 1955 Tunnel 1

B. Structure Performance Category = PC-1, General Service in accordance with PRC-PRO-EN-097, *Engineering Design and Evaluation (Natural Phenomena Hazard)*, Rev. 2

C. Soil Design Parameters (see Appendix B):
   - Soil Density = 110 pcf (moist condition)
   - Lateral Earth Pressures (at-rest condition)
     a) At-Rest Lateral Earth Pressure Coefficients
        — Horizontal Ground Surface: $K_O = 0.50$
        — Sloping Ground Surface: $K_{Oi} = K_O*[\ 1+\sin(\theta)\ ]$ where θ = slope of ground surface in degrees
     b) Equivalent fluid pressure at Timber Walls based on geometry and depth of sloped soil backfill, see Appendix B Attachment C for supporting calculations.
        — 44 psf/ft maximum (STA 3+00 East Side) – controlling design evaluation load
        — 34 psf/ft minimum (STA 3+00 West Side)
   - Allowable Bearing Capacity (estimated based on standard practice 3.0 Factor-Of-Safety)
     a) 4,400 psf for Timber Wall concrete footing (3 ft wide)
     b) 5,500 psf for Concrete Wall footing (5 ft wide, east wall at north end of tunnel)

D. Ground Snow Load = 15 psf (PRC-PRO-EN-097)

E. Live Load: None Permitted (includes personnel and equipment)

F. Load and Resistance Factor Design (LRFD) Load Factors: 1.2 Dead (self-weight + vertical soil weight)
   1.6 Snow
   1.6 Lateral Earth Pressures

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G. Wood Timber Properties: Douglas-Fir No. 1, rough sawn (NDS, 2012, *NDS Supplement: Design Values for Wood Construction*, Table 4D)

- Specific Gravity: 0.50
- Modulus of Elasticity (E_w): 1600 ksi (deflection check)
- Modulus of Elasticity (E_{min}): 580 ksi (strength check)
- Bending Stress (F_b): 1350 psi
- Shear Stress (F_v): 170 psi
- Tensile Stress (F_t): 675 psi
- Parallel Compressive Stress (F_c): 1100 psi
- Perpendicular Compressive Stress (F_{cperp}): 625 psi

H. Concrete: 3,000 psi compressive strength at 28 days (per Specification HWS-5638)

I. Reinforcing: Fy = 40 ksi for ASTM A15-52T Intermediate Grade (per Specification HWS-5638)

4. Methodology

Structural evaluation of the Tunnel 1 wood timber structure is based on 2012 International Building Code design standards using LRFD methods when subjected to soil and snow loading conditions. Evaluation did not include potential for structural degradation of wood timber due to long term exposure to high levels of radioactivity and effects of wood decay and insect attack.

The 2008 Light, Data, and Ranging (LIDAR) topographic survey data was utilized to provide an initial estimate of existing grade elevations associated with the Tunnel 1 structure. This information was compared to 1955 design drawing tunnel geometry to determine depth of soil fill and slope configuration for determination of lateral earth pressure load conditions used in the structural evaluation. Field surveys of finish grade and tunnel structure elevations would be required to determine actual soil depth and soil loading conditions that are applied to the structure.

Based on information provided in RHO-RH-34-3, *Geologic and Seismic Investigation of the PUREX Building Site*, there are no known geotechnical investigation or design reports associated with the PUREX facility including Tunnel 1. Estimated geotechnical soil properties for undisturbed native soil and for soil fill placed over the tunnel used in this engineering evaluation were based on best available geotechnical information further described in Appendix B. Sampling and testing of existing soils at or near Tunnel 1 would be required to determine actual soil design properties of soil fill and native soils used in the construction of Tunnel 1.

5. Computations

Calculations are performed in U.S. customary units and are included in the attachments.

6. Computer Software

None

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7. Assumptions

Subsurface soil conditions are similar to those reported by others in the project vicinity. Topography of existing cover soil over Tunnel 1 is consistent with the 2008 LIDAR topographic survey and 1955 Tunnel 1 design drawings listed in Section 12.

8. Results

Comparison of structural support member demand to capacity summarized by individual Design-to-Capacity Ratio (DCR) for each design check are as follows (DCR greater than 1.0 exceed design code requirements). For detailed calculations, see Appendix A.

A. Timber Roof Beams (12 inch x 14 inch rough sawn, Douglas-Fir No. 1):

DCR	Design Check
0.06	Axial Compression
0.78	Bending
0.81	Combined Bending and Axial Compression
0.22	Shear
0.21	Bearing

Deflection (vertical): 0.66 inches long-term, 0.99 inches long-term with creep factor (upper bound)

B. Timber Wall Supports (12 inch x 14 inch rough sawn, Douglas-Fir No. 1):

DCR	Design Check
0.06	Axial Compression
1.43	Bending – EXCEEDS DESIGN CODE LIMITS
1.49	Combined Bending and Axial Compression – EXCEEDS DESIGN CODE LIMITS
0.40	Shear
0.21	Bearing

Deflection (horizontal): 1.34 inches long-term, 2.01 inches long-term with creep factor (upper bound)

C. Railroad Ties (7 inch x 9 inch rough sawn laid flat, Douglas-Fir No. 1, transverse supports along base on tunnel):

DCR	Design Check
0.79	Axial Compression
0.40	Bearing

D. Timber Wall Concrete Footing (unreinforced 12 inch thick x 3 ft wide footing supporting timber walls):

DCR	Design Check
0.15	Bending
0.33	Shear (for plain unreinforced concrete)

E. Foundation Bearing Capacity

DCR	Design Check
0.81	Soil Bearing Capacity At Timber Walls
0.81	Soil Bearing Capacity At Concrete Wall

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9. Cause of Partial Roof Collapse

Due to the potential risk for exposure to high levels of radiation and urgency to close off and seal the roof breach area, insufficient information was obtained to determine the cause of the localized failure and partial collapse of a 17 ft long segment of the Tunnel 1 wood timber roof system. Recent LIDAR topographic survey data analyzed at 6 cross section stations along the length of tunnel indicates the depth of soil berm over the tunnel varied from approximately 7.7 ft to 9.5 ft in depth (compared to 8 ft depth specified on design drawings) with an average depth of 8.2 ft. Structural evaluation calculations included herein, based on a 8 ft soil depth, indicate that the roof timber beams were within design limits when subjected to loading conditions that occurred prior to the collapse. Therefore, potential causes of the partial roof collapse were narrowed down as follows:

- loss of roof beam end bearing (at one or both ends) due to decay and/or deterioration; this reason could be the result of prolonged water infiltration and pooling along a 17 ft length of concrete ledge support that was formed and cast-in-place at the top of concrete wall support along the east side of tunnel; reason is only moderately plausible when considering the tunnel is located within and subjected to overall arid and dry climate conditions.
- wood defects in roof beam timbers near maximum bending stress locations (at or near mid-span); this reason could explain why a small number of roof timbers failed but does not explain the contiguous loss of 17 individual roof timber support members.
- loss of roof beam structural capacity due to thru-roof core drilled holes used to install monitor standpipes and to obtain wood core samples taken in 1980 for testing purpose; this reason could explain why a small number of roof timbers failed but does not explain the contiguous loss of 17 individual roof timber support members.

10. Conclusions

This structural evaluation of Tunnel 1 indicates that structural wood and concrete support members of the size and configuration specified in the 1955 tunnel design drawing are within building code design requirements when subjected to ground snow and estimated soil loading conditions with the exception that vertical timber wall members which support long duration at-rest lateral earth pressures are up to 49% overstressed. Successful structural performance of Tunnel 1 vertical timber wall supports over the last 61-years indicates that the actual bending strength of wood timbers used during original construction are well above industry average for that wood species and are closer to results from the 1980 in-situ testing based on information provided in RHO-CD-1079, *Structural Evaluation of the PUREX No. 1 Burial Tunnel*.

Neither the results from the structural evaluation nor potential increases in bending strength of the timber based on 1980 in-situ wood core sample testing provides any explanation for why there was a 17 ft long partial roof collapse of Tunnel 1 timber roof beams and why the vertical timber wall supports in this area did not immediately or shortly thereafter collapse inward.

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11. Risk of Future Failure

The Tunnel 1 wood timber structure has been in service for more than 60 years which is beyond the typical design life for similar structures. The risk of future failure of the tunnel (partial or global collapse) is considered high based on significant design overstress of timber wall supports noted in the structural evaluation herein and on the recent partial roof collapse. As a result, the existing Tunnel 1 structure presents an extreme collapse hazard until such time that physical evaluation of remaining timber members and their supports can be performed. For safety purposes (e.g. avoid potential collapse, avoid exposure to high levels of radiation, etc.), placement of personnel and equipment on top of the tunnel and within the roof load zone of influence is not recommended without further evaluation.

12. References

H-2-55587 (drawing), 1980, *Structural Floor Plan & Section, Tunnel No. 1,* Rev. 7, General Electric Company, Richland, Washington.

H-2-55588 (drawing), 1955, *Structural Sections & Details,* Rev. 1, General Electric Company, Richland, Washington.

H-2-55589 (drawing), 1955, *Structural Sections & Details,* Rev. 2, General Electric Company, Richland, Washington.

HWS-5638, 1955, *Specifications for Disposal Facility for Failed Equipment Project CA-513-A*, General Electric Company, Richland, Washington.

PRC-PRO-EN-097, *Engineering Design and Evaluation (Natural Phenomena Hazard)*, Rev 2, Change 0, CH2M HILL Plateau Remediation Company, Richland, Washington.

NDS, 2011, *Supplement National Design Specification, Design Values for Wood Construction*, 2012 Edition, American Wood Council, Leesburg, Virginia. Available at: http://www.awc.org/pdf/codes-standards/publications/nds/AWC-NDS2012-Supplement-ViewOnly-1402.pdf

ASTM A15, 1952, *Standard Specification for Billet-Steel Bars for Concrete Reinforcing,* ASTM International, West Conshohocken, Pennsylvania.

2012 International Building Code (IBC), International Code Council.

FLUOR Hanford LIDAR Digital Elevation Model(s) & Topographic Contour Maps, 2008, prepared by Aero-Metric, Seattle, Washington.

RHO-R-34-3, 1981, *Geologic and Seismic Investigation of the PUREX Building Site*, URS/John A. Blume & Associates, San Francisco, California.

RHO-CD-1079, 1980, *Structural Evaluation of the PUREX No. 1 Burial Tunnel*, Rockwell Hanford Operations, Richland, Washington.

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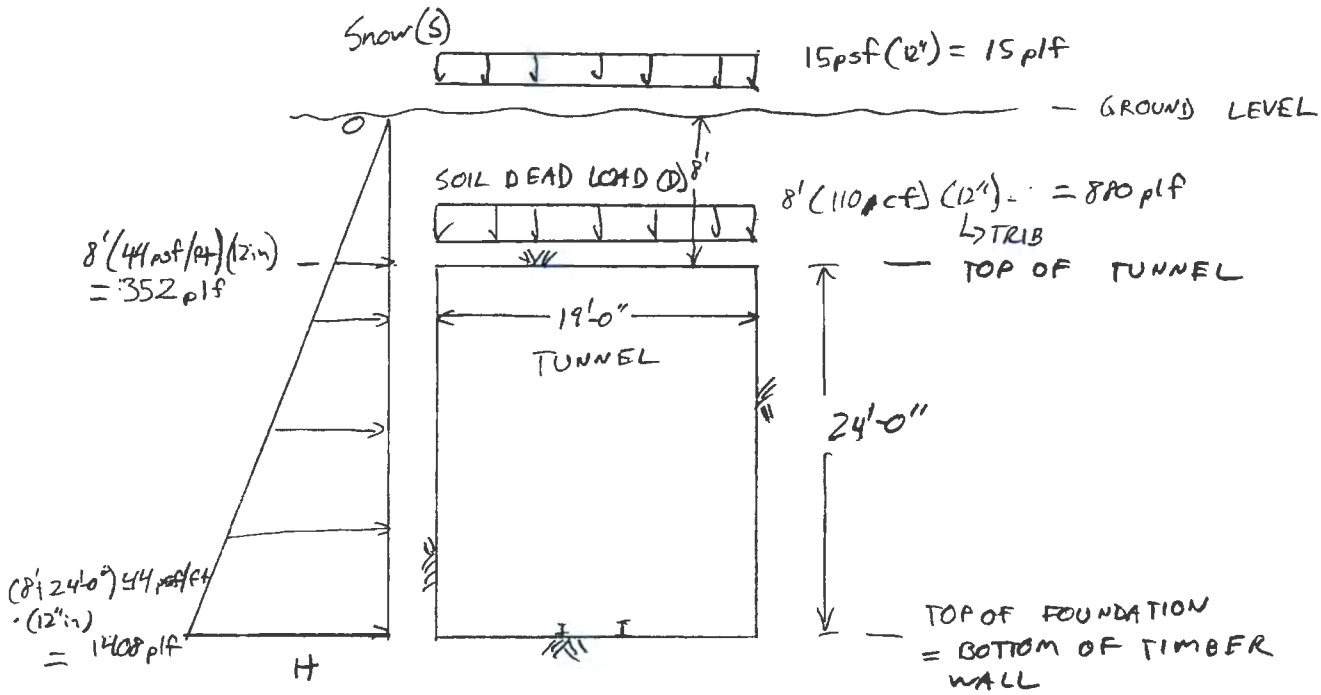
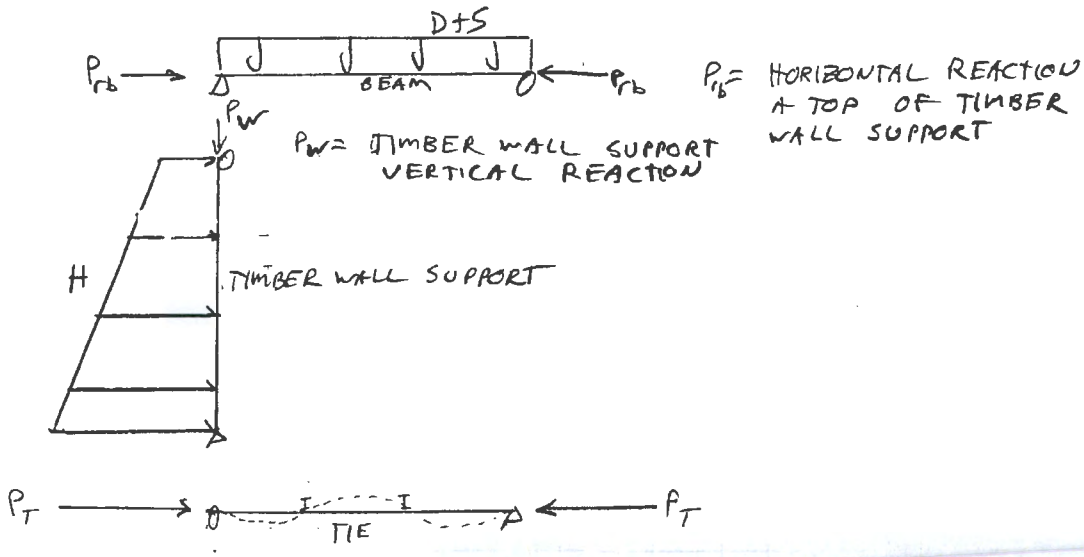
Appendix A

Calculations

PUREX Tunnel 1
Loading Diagram





Timber Roof Beam Supports

References:

1. 2012 NDS "National Design Specifications for Wood Construction" U.O.N.
2. 2012 NDS Supplement

Notes:

1. LRFD design used for wood members per NDS Section 4.3 including lambda, K.f and phi values.

Design Information

Length of roof beam: $L_{rb} := 19.33\text{ft}$

Effective length factor for compression: $K_{ex.rb} := 1.0$ *(Table G1)*

Effective length for bending: (Fully braced)

Tributary width: $trib_{rb} := 12\text{in}$

Depth of overburden soil: $h_s := 8\text{ft}$

Overburden Loads:

Moist soil unit weight: $\gamma_s := 110\text{pcf}$

Dead Load (soil weight): $w_{DL} := \gamma_s \cdot h_s \cdot trib_{rb} = 880 \cdot \text{plf}$ (Moist Soil)

Live Load (ground snow load): $w_{LL} := 15\text{psf} \cdot trib_{rb} = 15 \cdot \text{plf}$ (Snow)

Lateral Earth Pressures at top of wall:

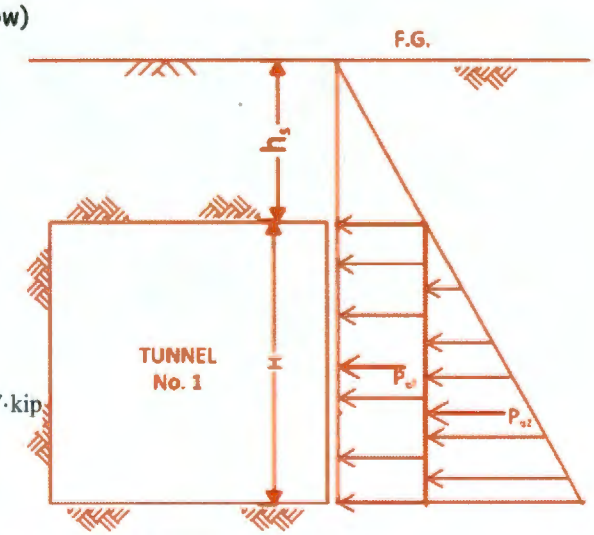
Height of tunnel: $H := 24\text{ft}$

Soil (at-rest): $q := 44 \frac{\text{psf}}{\text{ft}}$

Horizontal loads on tunnel from lateral earth pressure:

$P_{q1} := H \cdot (q \cdot h_s) \cdot trib_{rb} = 8.45 \cdot \text{kip}$

$P_{q2} := 0.5 \cdot H \cdot [(H + h_s) \cdot q - h_s \cdot q] \cdot trib_{rb} = 12.67 \cdot \text{kip}$



Wood Specific Gravity: $\gamma_{wood} := 0.5$ Douglas-Fir Larch per NDS Supplement Table 4D

Self weight of roof beam: $sw_{rb} := 12\text{in} \cdot 14\text{in} \cdot 62.4\text{pcf} \cdot \gamma_{wood} = 36.4 \cdot \text{plf}$ $sw_{u.rb} := 1.2 \cdot sw_{rb} = 44 \cdot \text{plf}$

(Factored self weight)

Axial compression force on beam
(reaction of horizontal pressures at top of wall member): $P_{rb} := \frac{P_{q1}}{2} + \frac{P_{q2} \cdot \left(\frac{1}{3} H\right)}{H} = 8.45 \cdot kip$

Factored Axial: $P_{u.rb} := 1.6 \cdot P_{rb} = 14 \cdot kip$

Service Moment: $M_{rb} := \frac{\left[\left(w_{DL} + sw_{rb}\right) + w_{LL}\right] \cdot L_{rb}^2}{8} = 44 \cdot kip \cdot ft$

Factored Moment: $M_{ux.rb} := \frac{\left[1.2 \cdot \left(w_{DL} + sw_{rb}\right) + 1.6 \cdot w_{LL}\right] \cdot L_{rb}^2}{8} = 52 \cdot kip \cdot ft$

Service Shear: $V_{rb} := \frac{\left[\left(w_{DL} + sw_{rb}\right) + w_{LL}\right] \cdot L_{rb}}{2} = 9 \cdot kip$

Factored Shear: $V_{u.rb} := \frac{\left[1.2 \cdot \left(w_{DL} + sw_{rb}\right) + 1.6 \cdot w_{LL}\right] \cdot L_{rb}}{2} = 10.9 \cdot kip$

Wood Species: Doug-Fir Larch ***No. 1 Beam and Stringers*** *(Supplement Table 4D)*

Beams are 12x14 rough sawn.

Section Width: $b_{rb} := 12in$ Section Depth: $d_{rb} := 14 \cdot in$

Bearing length: $l_{b.rb} := 4in$

Material properties:

Modulus of Elasticity:	$E_w := 1600000 \cdot psi$	$E_{min} := 580000 \cdot psi$	
Design Bending Stress:	$F_b := 1350 \cdot psi$	Design Shear Stress:	$F_v := 170 \cdot psi$
Design Tensile Stress:	$F_t := 675psi$	Parallel Compressive Stress:	$F_c := 925psi$
Perp. Compressive Stress:	$F_{cperp} := 625 \cdot psi$	Wood Specific Gravity:	$\gamma_{wood} := 0.5$

Douglas-Fir Larch per NDS Supplement Table 4D

Load Duration Factor: $C_D := 0.9$ Permanent dead load *(Table 2.3.2)*

Wet Service Factor: $C_M := 1.0$ Dry service condition *(Supplement Table 4D)*

Size Factor: $C_{F.rb} := \left(\frac{12in}{d_{rb}}\right)^{\frac{1}{9}}$ $C_{F.rb} = 0.98$ < or = 1.0 *(Supplement Table 4D)*

Time Effect: $\lambda := 0.8$ (1.2D+1.6L)
(Table N.3.3)

Resistance Factors: $\phi_b := 0.85$ $\phi_v := 0.75$ $\phi_c := 0.9$ $\phi_s := \phi_b$ $\phi_t := 0.8$
(Table N.3.2)

Format Conversion Factors: $K_{F_Fb} := 2.54$ $K_{F_Fc} := 2.40$
(Table N.3.1)

$K_{F_Fv} := 2.88$ $K_{F_E.min} := 1.76$ $K_{F_Ft} := 2.70$

Design Checks

Compression check (Section 3.6 and 3.7)

$F_{cn} := F_c \cdot K_{F_Fc}$ $F_{cn} = 2220 \cdot psi$ $E'_{min} := E_{min} \cdot K_{F_E.min} \cdot \phi_s \cdot C_M$ $E'_{min} = 867.68 \cdot ksi$

$$F_{cE1.rb} := \frac{0.822 \cdot E'_{min}}{\left(\frac{L_{rb} \cdot K_{ex.rb}}{d_{rb}}\right)^2} \quad F_{cE1.rb} = 2.6 \cdot ksi$$

$F_{cn.star} := F_{cn} \cdot \phi_c \cdot \lambda \cdot C_D \cdot C_M$ $F_{cn.star} = 1439 \cdot psi$ $\alpha := \frac{F_{cE1.rb}}{F_{cn.star}}$ $c := 0.85$ (Rough sawn) (Sec. 3.7.1)

Column Stability Factor: $C_{P.rb} := \frac{1 + (\alpha)}{2 \cdot c} - \sqrt{\left[\frac{1 + (\alpha)}{2 \cdot c}\right]^2 - \frac{\alpha}{c}}$ $C_{P.rb} = 0.88$ *[Eqn. 3.7-1]*

$C_{P.rb} := \min(C_{P.rb}, C_{F.rb}) = 0.88$

Compressive stress: $f_{c.rb} := \frac{P_{u.rb}}{b_{rb} \cdot d_{rb}} = 80 psi$

Compressive strength: $F'_{c.rb} := \lambda \cdot K_{F_Fc} \cdot \phi_c \cdot F_c \cdot C_D \cdot C_M \cdot C_{P.rb}$ $F'_{c.rb} = 1.26 \cdot ksi$

Compression only interaction ratio: $\boxed{DCR_{comp.rb} := \frac{f_{c.rb}}{F'_{c.rb}} = 0.06}$

$Check := if(DCR_{comp.rb} < 1.0, ok, ng)$ $Check = "OK"$

Flexure check

Compression edge supported: $C_{L.rb} := 1.0$

Flexure stress: $f_{b.rb} := \dfrac{M_{ux.rb}}{\left(\dfrac{b_{rb} \cdot d_{rb}^2}{6}\right)}$ $\quad f_{b.rb} = 1606.6 \cdot psi$

$F'_{b.rb} := \lambda \cdot K_{F_Fb} \cdot \phi_b \cdot F_b \cdot C_M \cdot C_{F.rb} \cdot C_D$ $\quad F'_{b.rb} = 2.06 \cdot ksi$

Flexure only interaction ratio: $\boxed{DCR_{flex.rb} := \dfrac{f_{b.rb}}{F'_{b.rb}} = 0.78}$

$Check := if\left(DCR_{flex.rb} < 1.0, ok, ng\right)$ $\quad Check = "OK"$

Combined Bending and Axial Compression Check (Section 3.9.2 Bending and Axial Compression)

$$\boxed{DCR_{comb.rb} := \underbrace{\left(\frac{f_{c.rb}}{F'_{c.rb}}\right)^2}_{\text{Compression component of demand}} + \underbrace{\frac{f_{b.rb}}{F'_{b.rb} \cdot \left[1 - \left(\frac{f_{c.rb}}{F_{cE1.rb}}\right)\right]}}_{\text{Bending component of demand}} = 0.81}$$

[NDS Eqn. 3.9-3]

$Check := if\left(DCR_{comb.rb} \le 1.0, ok, ng\right)$ $\quad Check = "OK"$

Shear Check

Shear Strength: $F'_{v.rb} := \lambda \cdot K_{F_Fv} \cdot \phi_v \cdot F_v \cdot C_M$ $\quad F'_{v.rb} = 294\ psi$

For chord to top cord connections $\quad V'_{r.rb} := F'_{v.rb} \cdot b_{rb} \cdot d_{rb} = 49 \cdot kip$

Stress interaction ratio: $\boxed{DCR_{shear.rb} := \dfrac{V_{u.rb}}{V'_{r.rb}} = 0.22}$

$Check := if\left(DCR_{shear.rb} < 1.0, ok, ng\right)$ $\quad Check = "OK"$

Bearing Check

Bearing factor: $C_{b.rb} := 1.1$ *[Sec. 3.10.4]*

Shear Strength: $F'_{c.rb} := \lambda \cdot K_{F_Fc} \cdot \phi_c \cdot F_{cperp} \cdot C_M \cdot C_{b.rb} \cdot C_D$ $F'_{c.rb} = 1069\,\text{psi}$

Bearing Stress: $f_{c.rb} := \dfrac{V_{u.rb}}{b_{rb} \cdot l_{b.rb}}$ $f_{c.rb} = 226\,\text{psi}$

Bearing interaction ratio: $\boxed{DCR_{bear.rb} := \dfrac{f_{c.rb}}{F'_{c.rb}} = 0.21}$

$Check := \text{if}\left(DCR_{bear.rb} < 1.0, \text{ok}, \text{ng}\right)$ $Check = \text{"OK"}$

Deflection Check

NDS suggest using a creep factor in deflection calculations when long-term loading <u>must be limited.</u> Long-term deflection limitations were not an original design criteria. However, due to the life span of the project, the deflections calculated incorporate the creep factor. They have only been provided to serve as a higher range value of what could be observed and are only provided as a reference.

Moment of inertia of roof beam: $I_{x.rb} := \dfrac{b_{rb} \cdot d_{rb}^3}{12} = 2744 \cdot \text{in}^4$

Short term loading deflection: (Snow) $\Delta_{st.rb} := \dfrac{5\left(w_{LL}\right) \cdot L_{rb}^4}{384 \cdot E_w \cdot I_{x.rb}} = 0.01 \cdot \text{in}$

Long term deflection of roof beam: (soil loading) $\Delta_{lt.rb} := \dfrac{5\left(w_{DL} + sw_{rb}\right) \cdot L_{rb}^4}{384 \cdot E_w \cdot I_{x.rb}} = 0.66 \cdot \text{in}$

Creep factor: $K_{cr} := 1.5$ *[Section 3.5.2]*

Total deflection: $\Delta_{T.rb} := \Delta_{lt.rb} \cdot K_{cr} + \Delta_{st.rb} = 0.99 \cdot \text{in}$ *[Eqn. 3.5-1]*

Timber Wall Supports

Design Information

Length of member: $L_w := 24ft - \frac{14in}{2} - \frac{4in}{2} = 23.25\,ft$

Effective length factor for compression: $K_{ex.w} := 1.0$ *(Table G1)*

Effective length for bending: (Fully braced)

Tributary width: $trib_w := 12in$

Wood Specific Gravity: $\gamma_{wood} := 0.5$ Douglas-Fir Larch per NDS Supplement Table 4D

Self weight of timber wall support: $sw_w := 12in \cdot 14in \cdot 62.4pcf \cdot \gamma_{wood} \cdot H = 0.87 \cdot kip$ $sw_{u.w} := 1.2 \cdot sw_w = 1048\,ft \cdot plf$ (Factored self weight)

Service Moment: $M_w := 60.03kip \cdot ft$ Loads by analysis, Ref. Loading Diagram

Factored Moment: $M_{ux.w} := 96.04kip \cdot ft$

Service Shear: $V_w := 12.3kip$

Factored Shear: $V_{u.w} := 19.64kip$

Service Axial at mid height (beam reaction): $P_w := V_{rb} + \frac{sw_w}{2} + h_s \cdot \gamma_s \cdot \left(14in - \frac{4in}{2}\right) \cdot trib_w = 10.32 \cdot kip$

Factored Axial at mid height (beam reaction): $P_{u.w} := V_{u.rb} + 1.2 \cdot \frac{sw_w}{2} + 1.2 \cdot \left[h_s \cdot \gamma_s \cdot \left(14in - \frac{4in}{2}\right) \cdot trib_w\right] = 12.4 \cdot kip$

Wood Species: Doug-Fir Larch *Dense No. 1 Beam and Stringers* *(Supplement Table 4D)*

Wall Supports are 12x14 rough sawn.

Section Width: $b_w := 12in$ Section Depth: $d_w := 14 \cdot in$

Bearing length: $l_{b.w} := 4in$ at bottom L4x3 support (LLV)

Size Factor: $C_{F.w} := \left(\frac{12in}{d_w}\right)^{\frac{1}{9}} = 0.98$ < or = 1.0 *(Supplement Table 4D)*

Design Checks

Compression check (Section 3.6 and 3.7)

$$F_{cE1.w} := \frac{0.822 \cdot E'_{min}}{\left(\frac{L_w \cdot K_{ex.w}}{d_w}\right)^2} \quad F_{cE1.w} = 1.8 \cdot ksi \qquad \alpha := \frac{F_{cE1.w}}{F_{cn.star}} = 1.25$$

Column Stability Factor: $C_{P.w} := \frac{1+(\alpha)}{2 \cdot c} - \sqrt{\left[\frac{1+(\alpha)}{2 \cdot c}\right]^2 - \frac{\alpha}{c}} \quad C_{P.w} = 0.79$ *[Eqn. 3.7-1]*

$$C_{P.w} := \min(C_{P.w}, C_{F.w}) = 0.79$$

Compressive stress: $f_{c.w} := \frac{P_{u.w}}{b_w \cdot d_w} = 74 psi$

Compression strength: $F'_{c.w} := \lambda \cdot K_{F_Fc} \cdot \phi_c \cdot F_c \cdot C_D \cdot C_M \cdot C_{P.w} \quad F'_{c.w} = 1.14 \cdot ksi$

Compression only interaction ratio: $\boxed{DCR_{comp.w} := \frac{f_{c.w}}{F'_{c.w}} = 0.06}$

$Check := if(DCR_{comp.w} < 1.0, ok, ng) \quad Check = "OK"$

Flexure check

Compression edge supported: $C_{L.w} := 1.0$ Compression zone of member is continuously braced by the adjacent members. Members are placed side-by-side and continuous along length of tunnel.

Flexure stress: $f_{b.w} := \frac{M_{ux.w}}{\left(\frac{b_w \cdot d_w^2}{6}\right)} \quad f_{b.w} = 2940 \cdot psi$

$F'_{b.w} := \lambda \cdot K_{F_Fb} \cdot \phi_b \cdot F_b \cdot C_M \cdot C_{F.w} \cdot C_D \quad F'_{b.w} = 2.06 \cdot ksi$

Flexure only interaction ratio: $\boxed{DCR_{flex.w} := \frac{f_{b.w}}{F'_{b.w}} = 1.43}$

$Check := if(DCR_{flex.w} < 1.0, ok, ng) \quad Check = "No\ Good"$

Using in-situ test results of F_b= 1,980 psi (Silvan 1980), DCR would be... $DCR_{flex.w} \cdot \frac{F_b}{1980 psi} = 0.97$

Compression/bending ratio

$$DCR_{comb.w} := \left(\frac{f_{c.w}}{F'_{c.w}}\right)^2 + \frac{f_{b.w}}{F'_{b.w}\cdot\left[1-\left(\frac{f_{c.w}}{F_{cE1.w}}\right)\right]} = 1.49 \qquad [3.9\text{-}3]$$

$Check := if(DCR_{comb.w} \le 1.0, ok, ng)$ $\quad Check = "No\ Good"$

Using in-situ test results of F_b= 1,980 psi (Silvan 1980), DCR would be... $\quad DCR_{comb.w}\cdot\frac{F_b}{1980psi} = 1.02$

Shear Check

Shear Strength: $\quad F'_{v.w} := \lambda\cdot K_{F_Fv}\cdot\phi_v\cdot F_v\cdot C_M \qquad F'_{v.w} = 294\,psi$

For chord to top cord connections $\quad V'_{r.w} := F'_{v.w}\cdot b_w\cdot d_w = 49\cdot kip$

Shear Stress: $\quad DCR_{shear.w} := \frac{V_{u.w}}{V'_{r.w}} = 0.4$

$Check := if(DCR_{shear.w} < 1.0, ok, ng)$ $\quad Check = "OK"$

Bearing Check

Bearing factor: $\quad C_{b.w} := 1.13$ *[Sec. 3.10.4]*

Shear Strength: $\quad F'_{c.w} := \lambda\cdot K_{F_Fc}\cdot\phi_c\cdot F_{cperp}\cdot C_M\cdot C_{b.w}\cdot C_D \qquad F'_{c.w} = 1098\,psi$

Bearing Stress: $\quad f_{c.w} := \frac{V_{u.w}}{b_w\cdot l_{b.w}} \qquad f_{c.w} = 409\,psi$

Bearing interaction ratio: $\quad DCR_{bear.rb} := \frac{f_{c.rb}}{F'_{c.rb}} = 0.21$

$Check := if(f_{c.w} < F'_{c.w}, ok, ng)$ $\quad Check = "OK"$

Deflection Check

NDS suggest using a creep factor in deflection calculations when long-term loading must be limited. Long-term deflection limitations were not an original design criteria. However, due to the life span of the project, the deflections calculated incorporate the creep factor. They have only been provided to serve as a higher range value of what could be observed and are only provided as a reference.

Moment of inertia of member: $I_{x.w} := \frac{b_w \cdot d_w^3}{12} = 2744 \cdot in^4$

Short term loading deflection: $\Delta_{st.w} := 0in$ No short term horizontal loading is applied to vertical wall supports

Long term deflection: (soil loading)

$$\Delta_{lt.w} := \frac{5(h_s \cdot q \cdot trib_w) \cdot L_w^4}{384 \cdot E_w \cdot I_{x.w}} + \frac{0.013 \cdot \left[0.5 \cdot H \cdot \left[\left[(H + h_s) \cdot q - h_s \cdot q\right] \cdot trib_w\right]\right] \cdot L_w^3}{E_w \cdot I_{x.w}} = 1.34 \cdot in$$

Total deflection: $\Delta_{T.w} := \Delta_{lt.w} \cdot K_{cr} + \Delta_{st.w} = 2.01 \cdot in$ *[Eqn. 3.5-1]*

Timber Railroad Ties

Design Information

Length of tie: $L_t := 19ft - 1.92ft = 17.08\,ft$

Effective length factor for compression: $L_{x.t} := 6ft$ $K_{ex.t} := 1$ (Table G1)

Car Rails brace beam in the horizontal direction and weight of cars braces beam in the vertical direction

Effective length for bending: $l_{ex.t} := 6ft$

Tributary width: $trib_t := 1.83ft$

Factored Moment: $M_{ux.t} := 0kip \cdot ft$ Bending due to car weight is resisted by at-grade soil below

Factored Shear: $V_{u.t} := 0kip$

Factored Axial (horizontal reaction at bottom of timber wall support): $P_{u.t} := V_{u.w} \cdot \frac{trib_t}{1ft} = 36 \cdot kip$

24 kips is load per foot of length at bottom of timber wall support

Wood Species: resisted Larch Dense No. 1 Beam and Stringers (Supplement Table 4D)

Ties are 9x7 rough sawn laid with 9" dimension horizontal

Section Width: $b_t := 9in$ Section Depth: $d_t := 7 \cdot in$

Size Factor: $C_{F.t} := 1.0$ (Supplement Table 4D)

Design Checks

Compression check (Section 3.6 and 3.7)

$$F_{cE1.t} := \frac{0.822 \cdot E'_{min}}{\left(\frac{L_t \cdot K_{ex.t}}{d_t}\right)^2} \qquad F_{cE1.t} = 0.83 \cdot ksi \qquad \alpha := \frac{F_{cE1.t}}{F_{cn.star}} = 0.58$$

Column Stability Factor: $$C_{P.t} := \frac{1 + (\alpha)}{2 \cdot c} - \sqrt{\left[\frac{1 + (\alpha)}{2 \cdot c}\right]^2 - \frac{\alpha}{c}} \qquad C_{P.t} = 0.5$$ [Eqn. 3.7-1]

$$C_{P.t} := \min(C_{P.t}, C_{F.t}) = 0.5$$

Compressive stress: $$f_{c.t} := \frac{P_{u.t}}{b_t \cdot d_t} = 570\,psi$$

Compression strength: $F'_{c.t} := \lambda \cdot K_{F_Fc} \cdot \phi_c \cdot F_c \cdot C_D \cdot C_M \cdot C_{P.t}$ $F'_{c.t} = 0.72 \cdot ksi$

Compression interaction ratio: $\boxed{DCR_{comp.t} := \frac{f_{c.t}}{F'_{c.t}} = 0.79}$

$Check := if(DCR_{comp.t} < 1, ok, ng)$ $Check = "OK"$

Bearing Check

Shear Strength: $F'_{c.t} := \lambda \cdot K_{F_Fc} \cdot \phi_c \cdot F_c \cdot C_M \cdot C_D \cdot C_{F.t}$ $F'_{c.t} = 1439\,psi$

Bearing Stress: $f_{c.t} := \frac{P_{u.t}}{b_t \cdot d_t} = 570.5\,psi$

Bearing interaction ratio: $\boxed{DCR_{bear.t} := \frac{f_{c.t}}{F'_{c.t}} = 0.4}$

$Check := if(DCR_{bear.t} < 1, ok, ng)$ $Check = "OK"$

Wall Footing Design

References:

1. ACI 318-11 "Building Code Requirements for Structural Concrete"

Assumptions:

1. Analysis is per foot of wall length
2. Eccentricity of loads are measured from the center of footing

Design Information

Dimensional Information

Thickness of foundation: $t_f := 1ft$

Heel of foundation: $L_{toe} := \frac{L_f - t_w}{2} = 11 \cdot in$

Thickness of wall: $t_w := 14in$

Total length of footing: $L_f := 3ft$

Density of Concrete: $\gamma_c := 145pcf$

Density of Water: $\gamma_w := 62.4pcf$

Soil unit weight: $\gamma_s := 115pcf$

Loading

Service Load: $P_f := P_w + \frac{sw_w}{2} = 10.76 \cdot kip$

Includes the lower half of the wall timber support self weight

Factored Load: $P_{u.f} := P_{u.w} + 1.2 \cdot \frac{sw_w}{2} = 12.96 \cdot kip$

No applied moment at base as the post hast a pin reaction: $M_f := 0kip \cdot ft$

Allowable bearing pressure: $q_a := 4400psf$

Concrete compressive strength: $f'_c := 3000psi$

Bearing Pressure Analysis

Applied bearing pressure: $q_f := \frac{P_f}{1ft \cdot L_f} = 3585 \cdot psf$

Demand capacity ratio: $\boxed{DCR_{q.f} := \frac{q_f}{q_a} = 0.81}$

$Check := if(DCR_{bear.t} < 1, ok, ng)$ $\quad Check = "OK"$

Flexure in footing analysis
(Analysis is done for plain concrete as the original plans do not indicate any reinforcement)

Moment from bearing pressure: $M_f := q_f \cdot \left[1ft \cdot \frac{\left(\frac{L_f}{2} - \frac{t_w}{2}\right)^2}{2}\right]$ $\quad M_f = 1.51\ ft \cdot kip$

Cracking moment of section: $M_{cr} := 7.5 \cdot \sqrt{f'_c \cdot psi} \cdot \frac{1ft \cdot t_f^2}{6}$ $\quad M_{cr} = 9.86 \cdot kip \cdot ft$ *[Ref. Eqn. 9-10]*

Footing does not contain any rebar. Therefore, the moment capacity is the cracking moment

Demand capacity ratio: $\boxed{DCR_{f.m} := \frac{M_f}{M_{cr}} = 0.15}$

$Check := if(DCR_{f.m} < 1, ok, ng)$ $\quad Check = "OK"$

One Way Shear Check

$V_{u.f} := 1.2 q_f \cdot 12in \cdot L_{toe} = 3.94 \cdot kip$ **(1.2 factor for controlling load case)**

Reduction factor for shear: $\phi_v := 0.75$ *[Section 9.3.2.3]*

Capacity of concrete: $\phi V_{c.f} := \phi_v \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot 12in \cdot t_f = 11.83 \cdot kip$ *[Eqn. 11-3]*

Demand capacity ratio: $\boxed{DCR_{f.s} := \frac{V_{u.f}}{\phi V_{c.f}} = 0.33}$

$Check := if(DCR_{f.s} < 1, ok, ng)$ $\quad Check = "OK"$

Concrete Wall Footing Analysis

References:

1. ACI 318-11 "Building Code Requirements for Structural Concrete"

Assumptions:

1. Analysis is per foot of wall length
2. Eccentricity of loads are measured from the center of footing

Design Information

Dimensional Information

Thickness of foundation: $t_{cf} := 1ft$

Thickness of wall: $t_{cw} := 3ft$

Total length of footing: $L_{cf} := 5ft$

Loading

Service Load: $P_{cf} := V_{rb} + t_{cw} \cdot h_s \cdot \gamma_s \cdot ft + t_{cw} \cdot H \cdot \gamma_c \cdot 1ft = 22.2 \cdot kip$

Roof beam reaction + soil above wall+wall sw

Factored Load: $P_{u.cf} := V_{u.rb} + 1.2t_{cw} \cdot h_s \cdot \gamma_s \cdot ft + 1.2t_{cw} \cdot H \cdot \gamma_c \cdot 1ft = 26.7 \cdot kip$

No applied moment at base as the post hast a pin reaction: $M_{cf} := 0kip \cdot ft$

Allowable bearing pressure: $q_a := 5500psf$

Bearing Pressure Analysis

Applied bearing pressure: $q_{cf} := \frac{P_{cf}}{1ft \cdot L_{cf}} = 4440 \cdot psf$

Ultimate bearing pressure: $q_{u.cf} := \frac{P_{u.cf}}{1ft \cdot L_{cf}} = 5340 \cdot psf$

Demand capacity ratio: $\boxed{DCR_{q.cf} := \frac{q_{cf}}{q_a} = 0.81}$

$Check := if(DCR_{bear.t} < 1, ok, ng)$ $\quad Check = "OK"$

Flexure in footing analysis

Moment from bearing pressure: $M_{cf} := q_{cf} \cdot \left[1ft \cdot \frac{\left(\frac{L_{cf}}{2} - \frac{t_{cw}}{2}\right)^2}{2} \right]$ $M_{cf} = 2.22\,ft \cdot kip$

Footing has #5@12" o.c. transverse bars. Moment capacity of footing is adequate by inspection. Reference wall footing design.

Check for shear capacity

Design 'd': $d := t_{cf} - 3in - 0.5 \cdot \frac{5}{8}in = 8.69 \cdot in$

Critical Shear: $V_{u.cf} := q_{u.cf} \cdot \left(\frac{L_{cf}}{2} - \frac{t_{cw}}{2} - d \right) \cdot 1ft$ $V_{u.cf} = 1.47 \cdot kip$

Capacity of concrete: $\phi V_c := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{f'_c \cdot psi} \cdot 1ft \cdot d$ $\phi V_c = 6.85 \cdot kip$

Demand capacity ratio: $DCR_{f.s} := \frac{V_{u.cf}}{\phi V_c} = 0.22$

$Check := if(DCR_{f.s} < 1, ok, ng)$ $Check = "OK"$

ch2m	Calculation Title: PUREX Tunnel 1 Engineering Evaluation	Project Number: 693839.BS Project: PUREX Tunnel 1 Client: CHPRC

Appendix B

Soil Design Parameters

Geotechnical Soil Design Parameters for Engineering Evaluation of PUREX Tunnel 1

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DATE:	June 23, 2017
PROJECT NUMBER:	693839.BS
REVISION NO.:	0
REVIEWED BY:	Nason McCullough/CH2M

Introduction

Geotechnical soil design parameters are presented herein for use in structural evaluation of PUREX Failed Equipment Storage Tunnel No. 1, Facility 218-E-14 (hereinafter described as Tunnel 1). Structure-specific geotechnical data is not available for the Tunnel. This memorandum describes the methodologies employed to develop tunnel soil geometry, geotechnical engineering properties of soils, earth pressures on tunnel roof and walls, and bearing capacity of footings. Geotechnical analysis of the progressive failure of roof timbers is also provided.

Limitations

At the time of issue of this report, site-specific geotechnical data was not available for use in the evaluation of Tunnel 1. Available geotechnical information generally consists of historical photographs, regional geologic studies, or site-specific studies for other facilities in the project vicinity. All evaluations and recommendations provided herein are derived from interpretation of data reported by others. As such, errors or misrepresentation of site information by others would affect our recommendations. In the event that additional geotechnical data is made available, or subsurface investigation programs are performed, analysis results should be updated accordingly.

Geotechnical Properties of Cover Soil

Geotechnical properties of Tunnel 1 cover soils were estimated for use in evaluation of soil loading on the roof and walls of the structure. At this time, there is no known geotechnical investigation data which characterizes the Tunnel 1 cover soil, and original design calculations for the tunnel are not available.

Soil properties were estimated considering review of as-built drawings, construction specifications, construction photographs, previous geotechnical investigation results in the vicinity of Tunnel 1, regional geologic studies, and other investigations performed by others. As can be seen in Exhibit 1, it is

interpreted that the cover soil was not placed in lifts and compacted as specified, but was placed loosely with a bulldozer, and was not compacted in lifts as specified. It is unknown whether the cover soils over the roof of the tunnel were compacted in lifts.

Exhibit 1 – Backfill operations at Tunnel 1; note placement of fill against Tunnel walls during winter conditions using a bulldozer

We recommend cover soil properties for Tunnel 1 as follows, based on our experience in the area, existing nearby information, and typical properties:

- Unified Soil Classification System (USCS) classification: Silty Sand with Gravel (SM)
- Moist unit weight, γ_m = 110 pcf (Expected range: 105 to 115 pcf)
- Internal friction angle, ϕ' = 30 degrees
- Cohesion, c' = 0 psf

The selection of soil parameters for the cover soils is complicated due to the lack of clear documentation of material properties or fill placement methods. Variability in the dry density of the soil, the in-situ moisture content, and material gradation are expected to contribute to variations in moist unit weight of the soil. The soil unit weight may change seasonally in response to seasonal weather patterns, variations with depth and location are likely. Discussion of the selection of these parameters is included in Attachment A.

Cover Soil Geometry

The geometry of the Tunnel 1 cover soil was evaluated by comparing the existing site topography to the as-built geometry of the tunnel. Light, Data, and Ranging (LiDAR) topographic survey data was collected in 2008 for the Central Plateau of the Hanford site by AeroMetric, Inc. of Seattle, Washington.

The collected data for the Central Plateau is arranged in 343 tiles arranged across the site. Tiles 153, 154, 177, and 178 overlap the PUREX Storage Tunnel 1 and Tunnel 2 and were provided to CHPRC by MSA of Richland, Washington for use in the analysis. The LiDAR survey data in the provided tiles consisted of a "bare earth" digital elevation model (DEM) processed by Aero-Metric to remove projections from the ground surface such as structures and vegetation as well as a "first return" DEM which includes all data points regardless of impacted surface. An excerpt from the LiDAR topography developed for the site is shown in Exhibit 2, detailed topography and cross sections are shown in Attachment B.

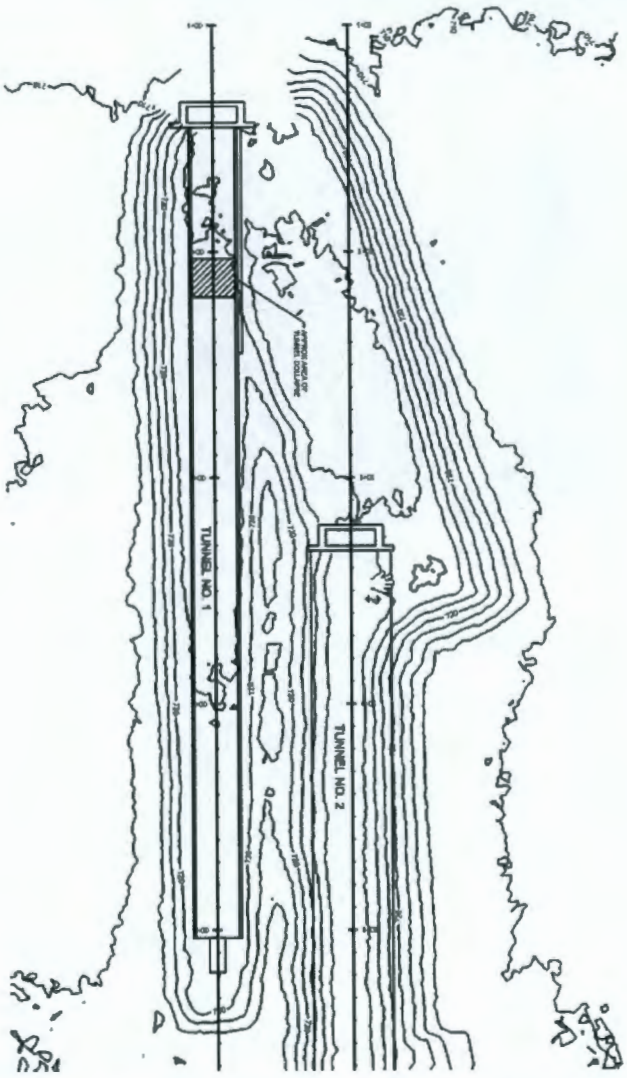

Exhibit 2 – Plan view showing of LIDAR topography and as-built geometry of Tunnel 1. North is up.

The horizontal coordinate system for the LiDAR data is State Plane, NAD83, Washington South Zone, meters. The vertical elevation system was NAVD88, meters. The LiDAR data was converted from the original coordinate system to match the 200E survey datum using survey monument data for brass-cap Monument 2E-41. This conversion allows direct comparison of topographic survey data to the as-built plan and elevation of the Tunnel 1 structure. An example is shown in Exhibit 3. The results of the LiDAR analysis, including site topography, tunnel profile, and tunnel cross sections are included in Attachment B.

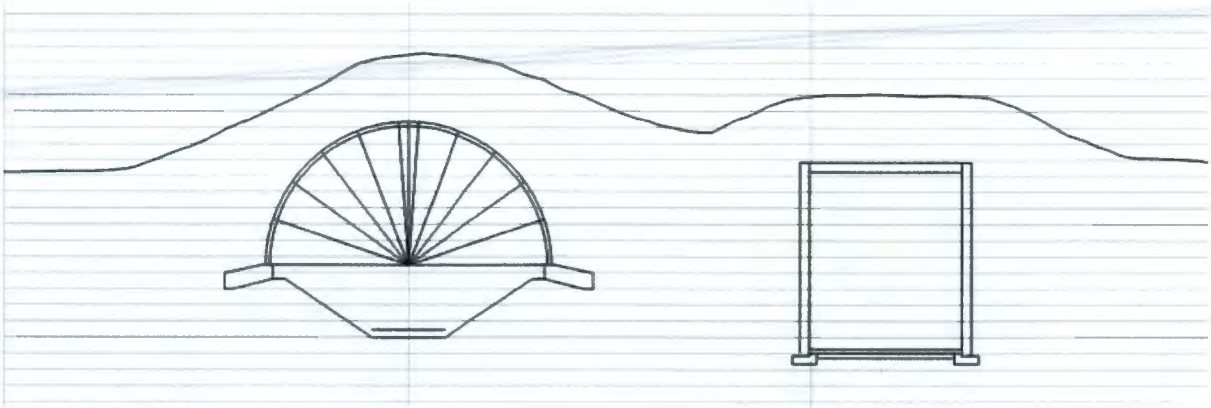

Exhibit 3 – Cross Section at STA 4+00 showing geometry of Tunnel 1, Tunnel 2, and Tunnel Cover Soils

At this time, no known recent confirmation survey data is available to confirm the horizontal or vertical position of the tunnel or the cover soil. The LiDAR data was collected in 2008, and at the time of this analysis was greater than 9 years old. It is possible that changes in the ground surface may have occurred since the data was collected. The geometry of Tunnel 1 was reproduced from horizontal and vertical coordinates shown on the 1955 as-built drawings. Plan, profile, and cross-sections of Tunnel 1 were then generated for evaluation of cover soil geometry as shown in Attachment B. Considering the age of the data, it is emphasized that the elevations and locations developed for this study are approximate.

To investigate the accuracy of the tunnel and cover soil geometries, "first return" LiDAR elevation data for the exposed portions of the Tunnel 1 and Tunnel 2 water-filled door structures was compared to the as-built design elevations of the structures reported in 1955 and 1962, respectively. First-return elevation data was found to generally be within 0.3 feet of the as-built elevation of the water-filled door structures. To our knowledge, the elevation of the water filled door structures have not been recently surveyed to confirm the as-built elevations; some movement of these structures over time is possible.

The LiDAR elevation data is expected to be the best-available topographic data for the site. The agreement between the LiDAR data and as-built water-filled door elevation supports the use of these two data sources to evaluate earth pressures on the tunnel with reasonable accuracy. If improved topography or tunnel geometry data becomes available, the cover soil geometry should be re-evaluated and incorporated into revised earth pressure calculations.

Earth Pressures

Earth pressures were estimated for the tunnel roof and the vertical walls of the tunnel. Earth loads were estimated using the 2008 LiDAR cover soil topography, the as-built tunnel geometry, and the estimated geotechnical properties of the cover soils. Estimation of roof pressures and lateral earth pressures are described below.

Roof Pressure

Soil pressures on the tunnel roof were estimated by taking scaled measurements of the cover soil height over the tunnel roof. Measurements were taken at the eastern edge, the centerline, and the western edge of the tunnel roof. Soil heights over the tunnel roof were found to vary from 7.8 to 9.5 feet, with an average of 8.3 feet. The soil height was multiplied by the estimated moist unit weight of the cover soil. The design snow load (15 psf) was added to the roof pressure. A summary of expected soil and

snow loading on the tunnel roof is shown in Exhibit 4 for 6 cross sections along the tunnel. The locations of these cross sections can be seen in Attachment B.

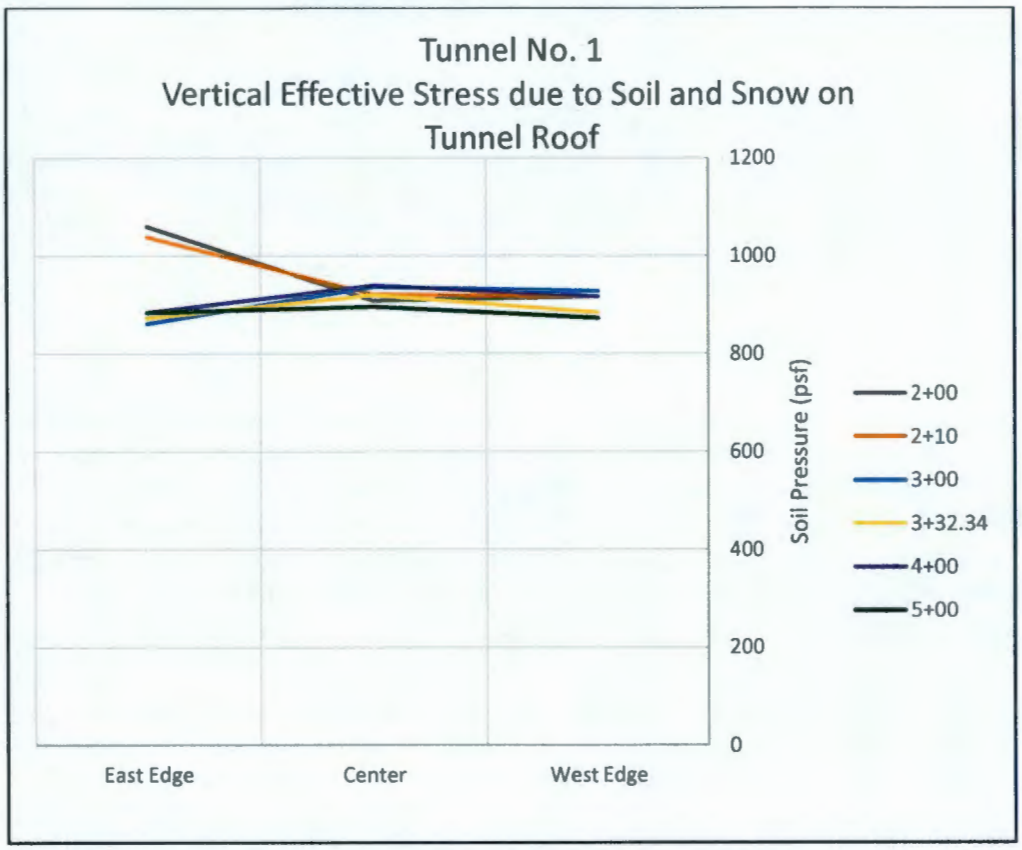

Exhibit 4 – Estimated vertical stress on tunnel roof for 6 cross-sections of Tunnel 1

Lateral Earth Pressures

Lateral earth pressures for the tunnel walls were estimated at the top and bottom of the tunnel walls for 6 cross-sections assuming at-rest lateral earth pressures act against the structure, and that pressures vary linearly along the height of the wall.

The at-rest lateral earth pressure coefficient was estimated using the material properties described herein and adjusted to account for sloping ground surface.

At-rest Earth Pressure Coefficient, $K_O = 1 - \sin(\phi')$

Adjusted At-Rest Earth Pressure Coefficient, $K_{Oi} = K_O \cdot (1 + \sin(i))$

where:

ϕ' is the drained internal friction angle of the soil

i is the slope of the ground surface in degrees

The average slope of the cover soils adjacent to the tunnel walls was estimated using the geometry shown in Exhibit 5. The adjusted lateral earth pressure coefficient was multiplied by the estimated vertical effective stress at the top and bottom of the wall to obtain a lateral earth pressure. The design snow load (15 psf) was included in the lateral earth pressure estimation. The estimated lateral earth pressures for each section are shown in Exhibits 6 and 7. Detailed lateral earth pressure calculations are included in Attachment C.

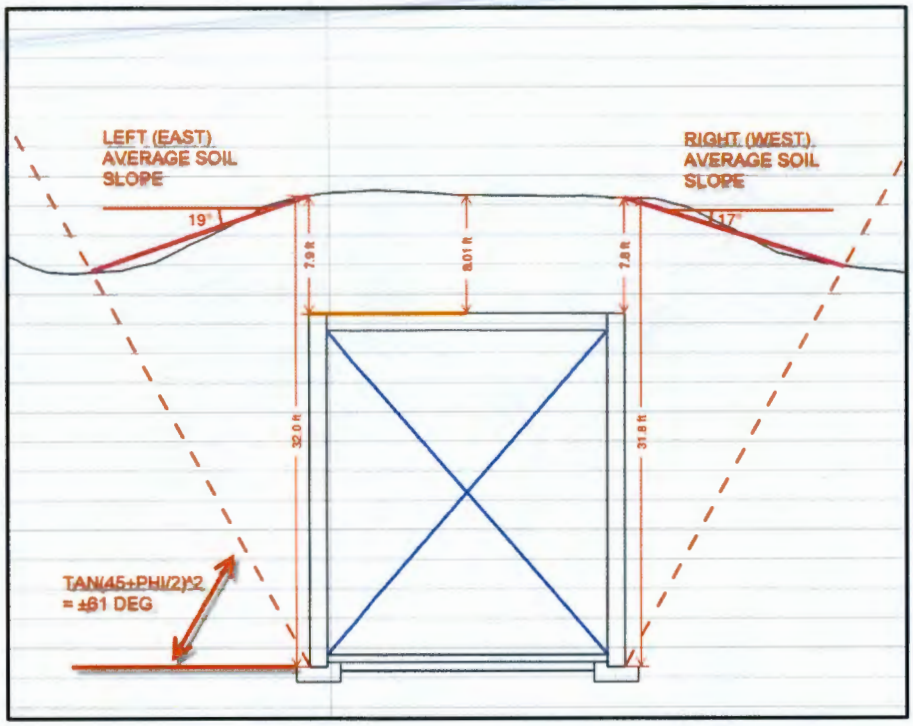

Exhibit 5 – Geometry used for evaluation of lateral earth pressures at Tunnel 1.

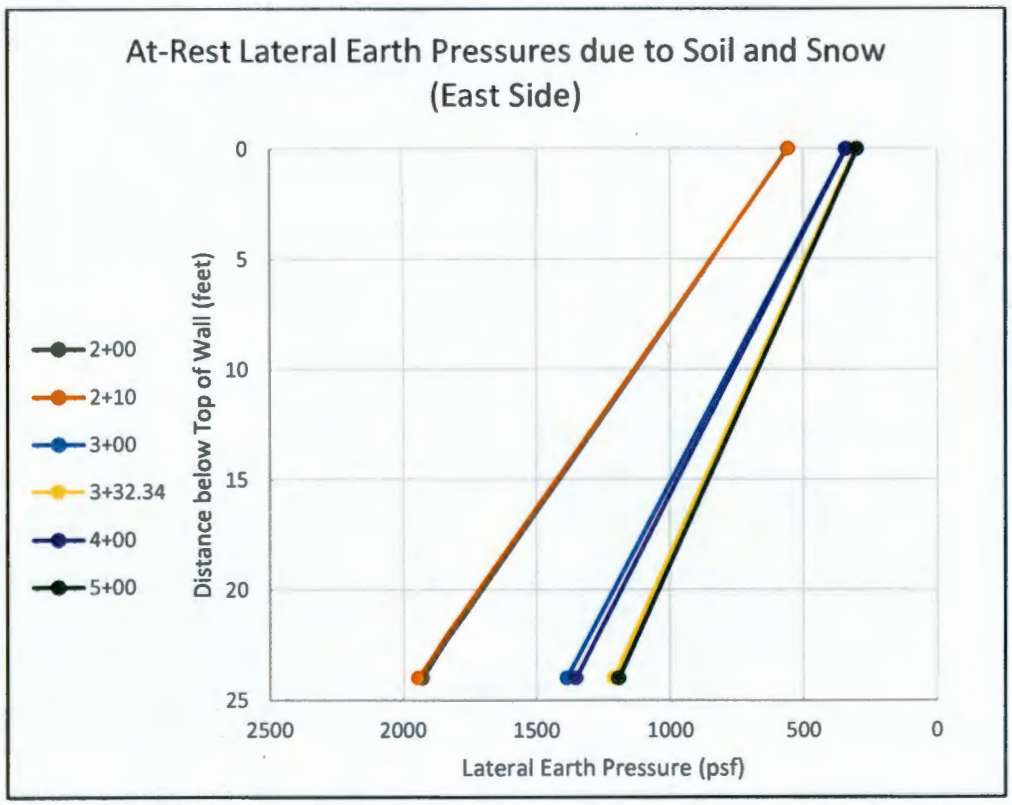

Exhibit 6 – Estimated Lateral Earth Pressures on East wall of Tunnel 1 due to soil and snow loads

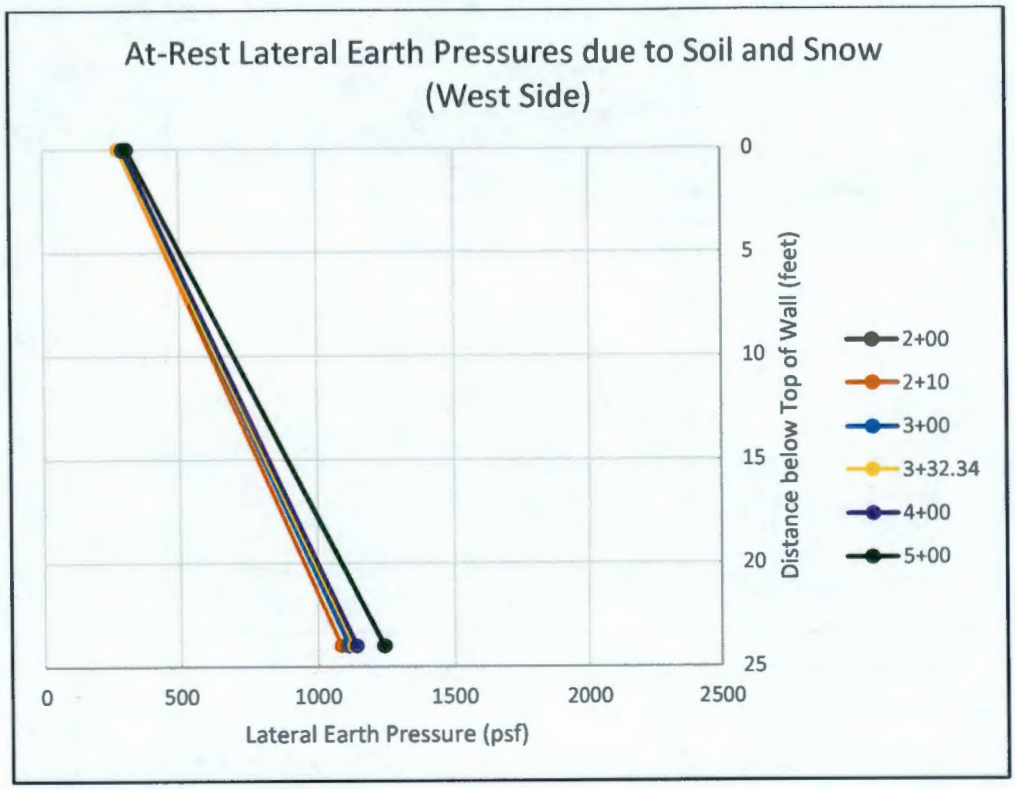

Exhibit 7 - Estimated Lateral Earth Pressures on West wall of Tunnel 1 due to soil and snow loads

Footing Bearing Capacity

The ultimate bearing capacity of the Tunnel 1 footings was estimated using Vesic's extended bearing capacity equation. Ultimate bearing capacity was estimated considering two continuous strip footing geometries: the 5-foot-wide footings beneath the concrete wall on the east side of Tunnel 1, and the 3-foot-wide footings located beneath the timber walls. The footings support vertical loadings and bear upon horizontal ground on the inside of the tunnel.

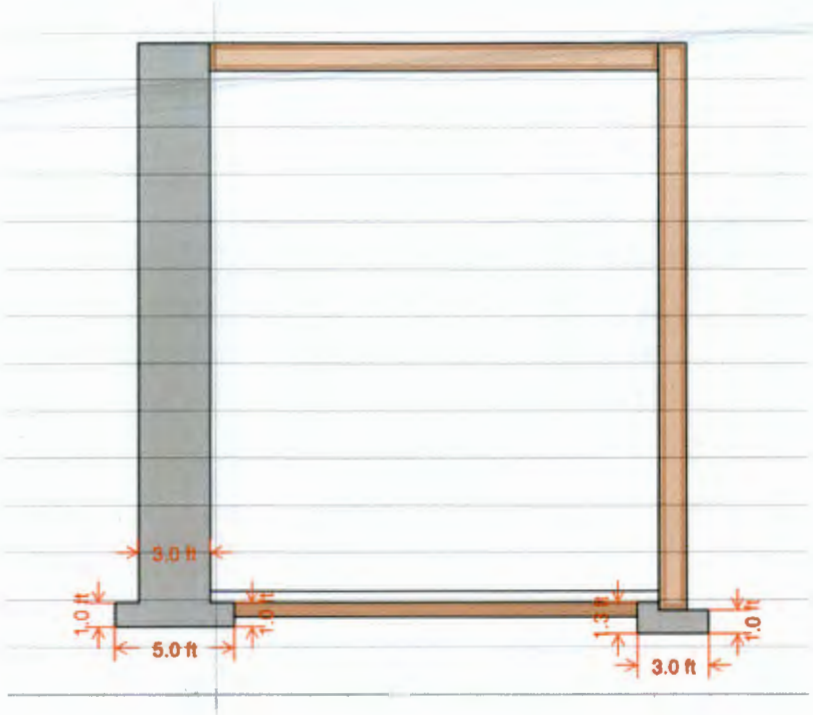

Exhibit 8 –Geometry used in Evaluation of Ultimate Bearing Capacity of Tunnel 1 Footings

The footing subgrade was assumed to be relatively undisturbed native material with an internal friction angle of 34 degrees, a cohesion of 0 psf (recommended by Dames & Moore, 1984), and a unit weight of 125 pcf as recommended by Shannon & Wilson (2015). Groundwater is greater than 300 feet deep below the project site as noted by URS/Blume (1981).

The geotechnical strength parameters adopted from Dames & Moore (1984) were originally prepared for Process Facility Modification Project located approximately 0.25 miles west of the PUREX storage tunnels. These strengths are believed to be conservative considering that Shannon & Wilson later provided a range of recommended strengths for the Hanford Formation as a friction angle of 38 to 54 degrees with zero cohesion. Review of construction photographs of exposed material in the subgrade and experience with similar materials suggests that the strengths provided by Dames & Moore are conservative.

Using the as-built footing geometry and Dames & Moore's (1984) recommended strength results in the following ultimate bearing capacity estimates:

$q_{ultimate,concrete}$ = 16,600 psf (footing on the left in Exhibit 8)

$q_{ultimate,wood}$ = 13,100 psf (footing on the right in Exhibit 8)

The standard of practice for conventional geotechnical design is to incorporate a factor of safety of 3.0 into the development of allowable bearing capacities. Larger factors of safety could be justified considering the uncertainty in subsurface conditions and the consequences of failure.

$q_{allowable,concrete}$ = 5,500 psf

$q_{allowable,wood}$ = 4,600 psf

Factor of Safety = 3.0

These bearing pressures are expected to be conservative considering the relatively low strength used in the calculation, the additional bracing provided by the timbers in the tunnel floor, and the weight of the rail cars and equipment within the tunnel. Soil bearing capacity calculations are included in Attachment D.

Progressive Failure of Roof Timbers

A soil mechanics review was performed to assess the mechanisms of roof collapse for Tunnel 1. The review was conducted to determine if a single roof timber collapse could be attributed to the development of the 17-foot-wide roof collapse observed on May 9, 2017. The premise for the review was the consideration that, if a single timber collapsed, the overburden soil loading on the tunnel roof would be expected to be redistributed to adjacent timbers and cause a corresponding increase in stress. This stress, if sufficient to cause failure of the adjacent timbers, could result in a progressive failure mechanism and additional timber failure. The progressive failure would continue until the soil is unable to arch over the void, and collapses into the opening, thereby reducing the additional weight on the adjacent timbers.

The soil mechanics review was inconclusive; however, the general finding was that the 17-foot-wide collapse cannot be readily explained by soil mechanics principles of a simplified progressive failure mechanism. The loss of a single timber is expected to result in a re-distribution of tunnel stresses which, if sufficient to cause timber failure with 8 feet of cover soil, could open to a width of 4 to 5 feet before the soil arch would collapse into the opening.

Additional factors, such as the apparent cohesion provided by negative pore pressures in the moist cover soils, the "reinforcement" provided by plant roots, three-dimensional effects from the sloping and adjacent cover soils, or disturbance caused by the 1980 RHO investigations of the Tunnel 1 roof timbers, may provide partial explanation for the collapse. Further study may be warranted using numerical methods and by assigning spring constants to individual timbers and investigating the loss of individual timber members.

References

CHPRC-03241 Revision 0. PUREX Burial Tunnels, dated March 2017, prepared for the U.S. Department of Energy Assistant Secretary for Environmental Management, prepared by M. A. Maloof CH2M HILL Plateau Remediation Company.

FLUOR Hanford LiDAR Digital elevation Model(s) & Topographic Contour Maps, prepared by Aero-Metric, Seattle, dated 2008

Geotechnical Engineering Study (Rev2) West Replacement Footings for 324 Building 'B' Cell Excavation Hanford Reservation, Washington. 22-1-03078-010. Submitted to Kurion, Inc., dated February 27, 2015, prepared by Shannon & Wilson.

H-2-55587 (drawing), Structural Floor Plan & Section, 1955 Tunnel 1.

H-2-55588 (drawing), Structural Sections & Details, 1955 Tunnel 1.

H-2-55589 (drawing, Structural Sections & Details, 1955 Tunnel 1.

HWS-5638, Specification for Disposal Facility for Failed Equipment, dated April 25, 1955 (Tunnel 1), Project CA-513-A.

Report of Soils Investigation, Proposed Process Facility Modification Project, PUREX Plant, Hanford, Washington for US Department of Energy, dated January 4, 1984, prepared by Dames & Moore.

RHO-CD-1079, Structural Evaluation of the PUREX No. 1 Burial Tunnel, dated September 1980, by Silvan, G.R.

RHO-R-34-3, Geologic and Seismic Investigation of the PUREX Building Site near Richland, Washington, prepared for Vitro Engineering Corporation, Richland Washington, dated March 1981, prepared by URS/John A. Blume & Associates, Engineers.

ch2m	Calculation Title: PUREX Tunnel 1 Engineering Evaluation	Project Number: 693839.BS Project: PUREX Tunnel 1 Client: CHPRC

Attachment A

Cover Soil Parameters

Geotechnical Properties of Tunnel 1 Cover Soil

PREPARED FOR:	Project File
COPY TO:	Craig Barrett/CH2M
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CHECKED BY:	Youssef Bougataya/CH2M
DATE:	June 23, 2017
PROJECT NUMBER:	693839.BS
REVISION NO.:	0

This memorandum documents the development of recommended geotechnical properties for the Tunnel 1 Cover soils for use in structural evaluation of Tunnel 1. Specifically, this memorandum briefly summarizes the available information and provides recommended strength and unit weight values.

Review of Available Subsurface Information

There is very limited geotechnical information available for Tunnel 1. Collection of site-specific geotechnical data was beyond the scope of this evaluation. Recommendations provided herein are derived from interpretation of geotechnical data reported by others. Much of the available data is fragmentary or otherwise developed for other purposes. No responsibility can be taken for errors or misrepresentation of site information by others.

Construction Drawings (1955)

Original construction drawings are available for Tunnel 1. The drawings include limited information on the cover soils. The following are noted from review of the drawings:

1. Tunnels were to be constructed with minimum 8'-0" of soil cover
2. The width of the soil cover is 28'-0"
3. Side slopes for the soil cover were to be constructed at 2H:1V

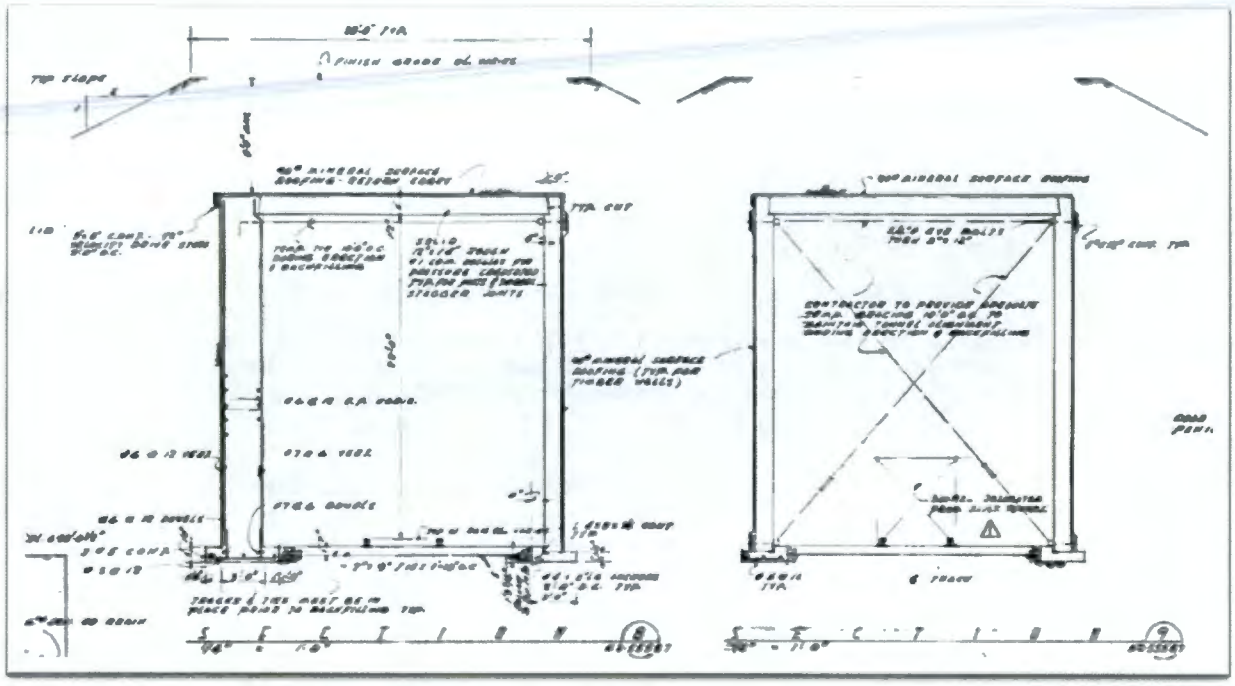

Exhibit 1 – Design Cross section of Tunnel 1 showing proposed finish grade relative to tunnel structure

Construction Specifications (1955)

Original construction specifications for Tunnel 1 were available for review. Review of the available construction photos (discussed hereinafter) makes it difficult to discern the extent to which these specifications were enforced during construction of the tunnel. The following were included in the specifications:

1. Frozen fill materials were not to be used; however placement of fill in wintertime is visible in several photos.
2. Materials were to be screened to remove particles larger than 8 inches; large particles were not discernable in construction photos.
3. Fill materials were to be free from vegetable matter or trash. Tumbleweeds are visible in tunnel excavations, it is unclear whether these materials were removed prior to fill placement.
4. Fill materials were to be placed in maximum 24-inch loose lifts and thoroughly compacted. A compaction specification, method specification, or density was not specified. Construction photos suggest that this material was not placed in horizontal lifts as specified.
5. Compaction methods were not specified, except as to prohibit sluicing or flooding. There is no compaction equipment visible in the photos, it is unclear if cover soils were compacted.
6. Fill placement was to avoid excessive loads on walls, unbalanced loads, or construction of walls out of verticality.
7. The tunnel roof was to receive 6 inches of clean sand fill over the 90# mineral surface roofing. The presence or absence of this material cannot be ascertained from the available photos.

6. **BACKFILL**

a. Additional embankment material, if required in addition to material excavated under these specifications, shall be obtained from the borrow area designated by the Commission.

b. Only selected material free of frozen particles, stones greater than 8 inches in any dimension, vegetable matter and trash shall be used in any backfill.

c. Trash shall not be allowed to accumulate in spaces to be backfilled.

d. Material for load bearing backfill, such as that under stairwell and pump house, shall be placed in level layers not to exceed 12 inches loose measurement, wetted and thoroughly compacted.

-8- EMS-5638

Exhibit 2a – Backfill specifications for Tunnel 1

e. Material for non-load-bearing backfill and embankment shall be placed in layers not to exceed 24 inches loose measurement unless otherwise specified herein, and thoroughly compacted. Compaction by loaded scrapers and/or tractors will be considered adequate for non-load-bearing compaction subject to the restrictions specified in Section 3-c, above.

f. A compacted sand cushion at least 4 inches thick shall be placed under lines in trenches where such lines pass through rocky soil or as directed by the Commission.

g. After necessary line testing has been accomplished, backfill for piping shall be carefully placed around pipe before covering with a minimum of 12 inches of compacted sand. Common backfill material shall then be placed as noted in items above.

h. Care shall be exercised during backfilling to prevent excessive loads on walls and to insure balanced loading on opposite walls.

All necessary precautions shall be exercised to insure vertical walls upon completion of backfilling.

i. Six inches of clean sand free from stones over 1 inch in diameter shall be placed over tunnel roofing material prior to placing embankment material.

j. Backfilling by means of sluicing or flooding will not be permitted.

Exhibit 2b – Backfill specifications for Tunnel 1 (continued)

Design Documentation

At this time, no design documentation, design calculations, design reports, or related materials are known to exist.

Construction Photographs (various)

Tunnel 1 construction photographs documented in CH2M (2017) were reviewed. The following observations were made:

- Tunnel 1 was constructed and backfilled in winter conditions, with visible snow on ground, possible placement of frozen materials.
- Tunnel excavation geometry (i.e. trench) does not appear to allow adequate access for compaction of backfill soils in lifts against tunnel walls. Photos suggest that material may have been pushed into the trench using a bulldozer or similar equipment. It is unknown if small walk-behind compaction equipment was used to compact material in 24-inch lifts as specified. It is unknown how material was placed on the roof of the tunnel.
- Internal guy wires are visible inside tunnel as shown on drawings.
- Wooden cleats are visible on outside of tunnel.
- Backfill material generally appears to be fine or sandy in nature, there are very few cobbles, gravels, or coarse materials visible in the photos.

Exhibit 3 – Construction photo of Tunnel 1. Note stockpiled backfill and timbers, concrete wall, and footing formwork.

Exhibit 4 – Construction of Tunnel 1. Note snow, steep-walled trench adjacent to tunnel walls, internal guy wires, external timber cleats.

Exhibit 3 – Tunnel 1 construction aerial view. Note snow and relatively even placement of fill material in narrow trench against tunnel walls by construction equipment.

Studies by Others

1.0 Rockwell Hanford Operations Structural Evaluation of Tunnel 1 (1980)

Rockwell Hanford Operations (RHO) performed a structural evaluation of Tunnel 1 in 1980. The following items were noted from the archived report:

- **Rockwell International completed a structural evaluation of Tunnel 1, including sampling and evaluation of 4-1/8" diameter cores of roof timbers from 3 locations along the tunnel in May 1980.**
- **"Location Number 2" was approximately 54 feet south of the water filled door and 4 feet west of the tunnel centerline. This location is very close to the observed tunnel roof collapse. This location (similar to the other locations sampled) was selected because it appeared to have received the greatest amount of radiation exposure.**

- To obtain the cores, the cover soil was excavated with a clamshell to expose the roof of the tunnel.
- Although no references or methodology is cited, the following geotechnical parameters for the cover soils were used:
    - Unit weight = 110 pcf
    - Active Earth Pressure Coefficient, *Ka* = 0.4 **Note that the use of *Active* lateral earth pressures in this 1980 evaluation is inappropriate, considering the top of the wall is restrained against rotation away from the backfill by the roof timbers. *At-Rest* earth pressures are more appropriate.**
- Visible in the report photographs are sandy fill material with small to medium gravels. Silt content is not readily apparent. The presence or absence of 6 inches of clean sand above the tunnel roof is not discernable.
- The method for backfilling the investigation holes is not described, although it is noted that the backfilling was delayed approximately 6 days due to equipment delays resulting from ash from the May 18, 1980 eruption of Mt. St. Helens.

Figure 4: Typical Sample Site

Exhibit 4 – View of excavation into Tunnel 1 cover soils for sampling of roof timbers

Figure 6: Drill Motor in Use

Exhibit 5 – Alternate view of sampling of roof timbers, note soil sandy soil with fine to medium gravels visible in background.

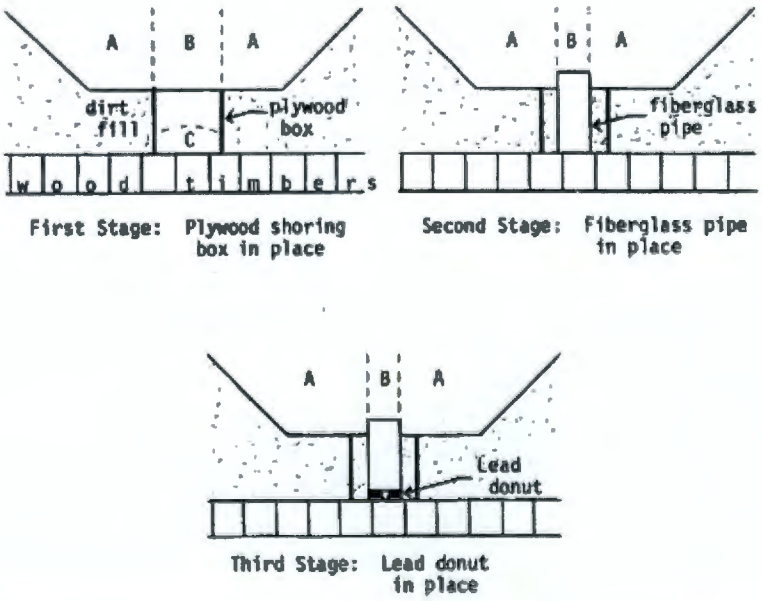

Exhibit 6 – General configuration and sequencing of timber sampling locations on Tunnel 1

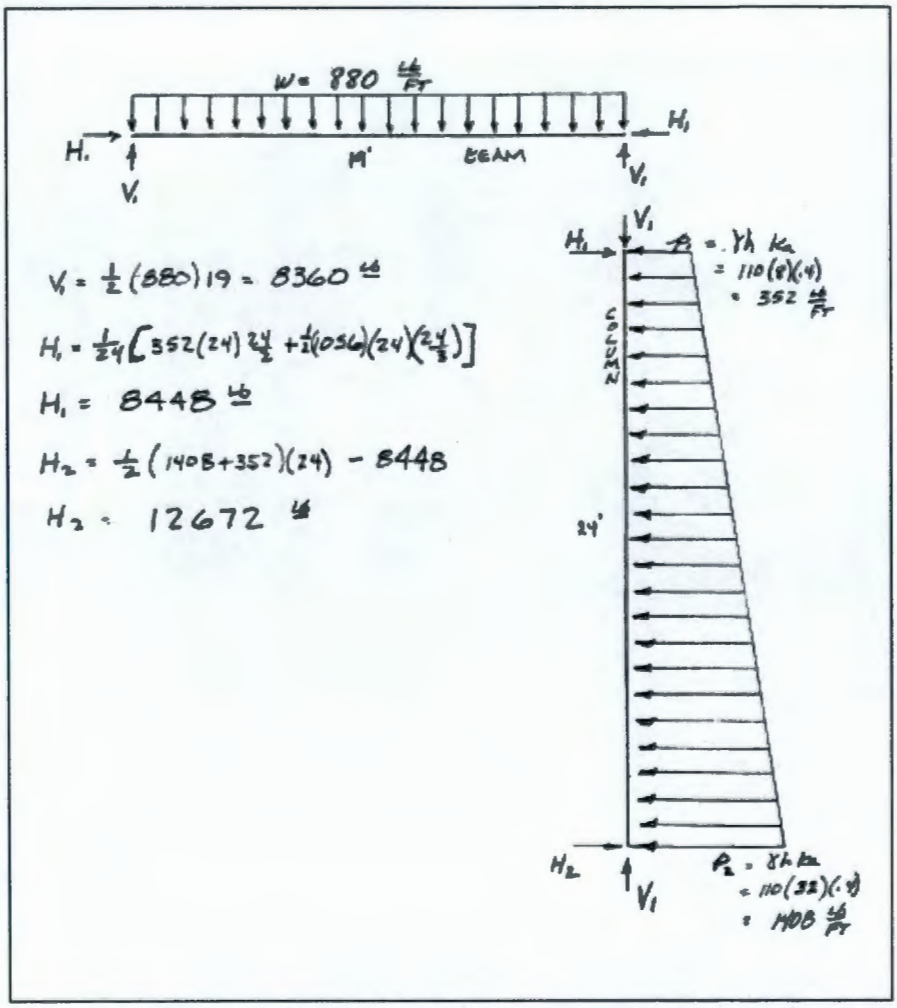

Exhibit 7 – Soil loading used in structural evaluation of Tunnel 1. Note 110 pcf unit weight of soil and Active (Ka) earth pressure coefficient of 0.4 noted. Use of Active lateral earth pressures is inappropriate for this evaluation due to the bracing of the top of the tunnel provided by the roof timbers.

2.0 Report of Soils Investigation for Proposed Process Facility Modification Project (1984)

Dames & Moore submitted a soil investigation report for a proposed Process Facility Modification at the PUREX plant site in January 1984. Four boreholes were advanced for the facility and geotechnical design parameters were developed. The facility was never constructed and the report was never finalized. URS (1981) noted that no known direct subsurface investigations were performed for the design or construction of the PUREX facility. As such, the Dames & Moore (1984) investigation is thought to represent the best available subsurface information in the vicinity of the PUREX Storage Tunnels. It is assumed that subsurface conditions at the PUREX Storage Tunnels are similar to those encountered by Dames & Moore (1984).

The following notes were made from this report:

1. 4 boreholes were advanced for this study. Boreholes were located approximately 1,230 to 1,350 feet west of PUREX Tunnel 1. Borehole were advanced with rotary hollow-stem auger methods and sampled using Dames & Moore proprietary U-type sampler.

   B-1, Total depth = 54.5 feet

B-2, Total depth = 19.5 feet

B-3, Total depth = 59.5 feet

B-4, Total depth = 19.5 feet

2. Field classifications and laboratory testing were performed on selected samples to support evaluating shear strength, moisture content, density, grain-size distributions, resistivity, and pH.

3. The following geotechnical engineering parameters were recommended for use in design of the proposed facility:

   Angle of Internal Friction, ϕ = 34 degrees

   Cohesion, c' = 0 psf

   Mass Density, γ = 110 pcf

   Poisson's ratio, μ = 0.25

   Coefficient of Active Earth Pressure, Ka = 0.28

   Coefficient of At-Rest Earth Pressure, Ko = 0.44

   Coefficient of Passive Earth Pressure, Kp = 3.5

4. Lateral earth pressures:

   Active Equivalent Fluid Unit Weight = 30 pcf

   At-Rest Equivalent Fluid Unit Weight = 50 pcf

   *At-Rest Pressure for relatively rigid structures (psf) = 26*H (where H is the height of the wall in feet)*

   Passive Equivalent Fluid Unit Weight = 250 pcf (includes FS=1.5)

5. General Soil Description:

   The near-surface soils generally consist of a thin fill layer of gravelly sand (SW) to sandy gravel (GW). This fill was probably placed at the time that the subject area was temporarily used during construction of the nearby PUREX plant. Under this fill and down to depths ranging from 3.5 to 6 to 7 feet, all borings encountered a layer of loose silty fine sand (SM) containing variable amounts of coarser sand and fine gravel with depth. In each of the shallow borings (B-2 and B-4), about 4 to 5 feet of medium dense to dense fine to coarse sands with occasional gravel (SW) were encountered below the silty fine sand (SM); however, this well-graded sandy material was not encountered in either of the deeper borings (B-1 and B-3).

   Below a depth ranging from 3.5 to approximately 6 to 7 feet, all borings encountered medium dense to dense poorly graded sands (SP). These soils generally appear to grade back and forth between the limits of a fine to medium sand with a trace of silt to a medium to coarse sand with variable amounts of fine gravel. ... It is believed that this stratum of poorly graded sand with varying amounts of silt and fine gravel can be assumed to extend under the entire project site to depths in excess of 60 feet. This assumption is based on our field engineer's inspection of a 64-foot-deep excavation to elevation 622 that exists approximately 2300 feet to the northeast of the subject area. This tank farm excavation was made in soils consisting of interbedded sands and gravelly sands overlying poorly graded sands that are similar to those encountered in our exploratory borings.

No groundwater was encountered in any of the four borings. Based on previous geologic and seismic investigations at the PUREX building site, it is believed that the water table is at a depth of approximately 300 feet beneath the subject area.

6. Applicability to PUREX Tunnel Evaluation

Although the boreholes are located nearly 0.25 miles west of the PUREX storage tunnels, Dames & Moore (1984) noted that similar materials were observed in a tank farm excavation nearly 2,300 feet northeast of the boreholes and surmised that conditions could be expected to be relatively similar between this area, which approximately includes the PUREX storage tunnels.

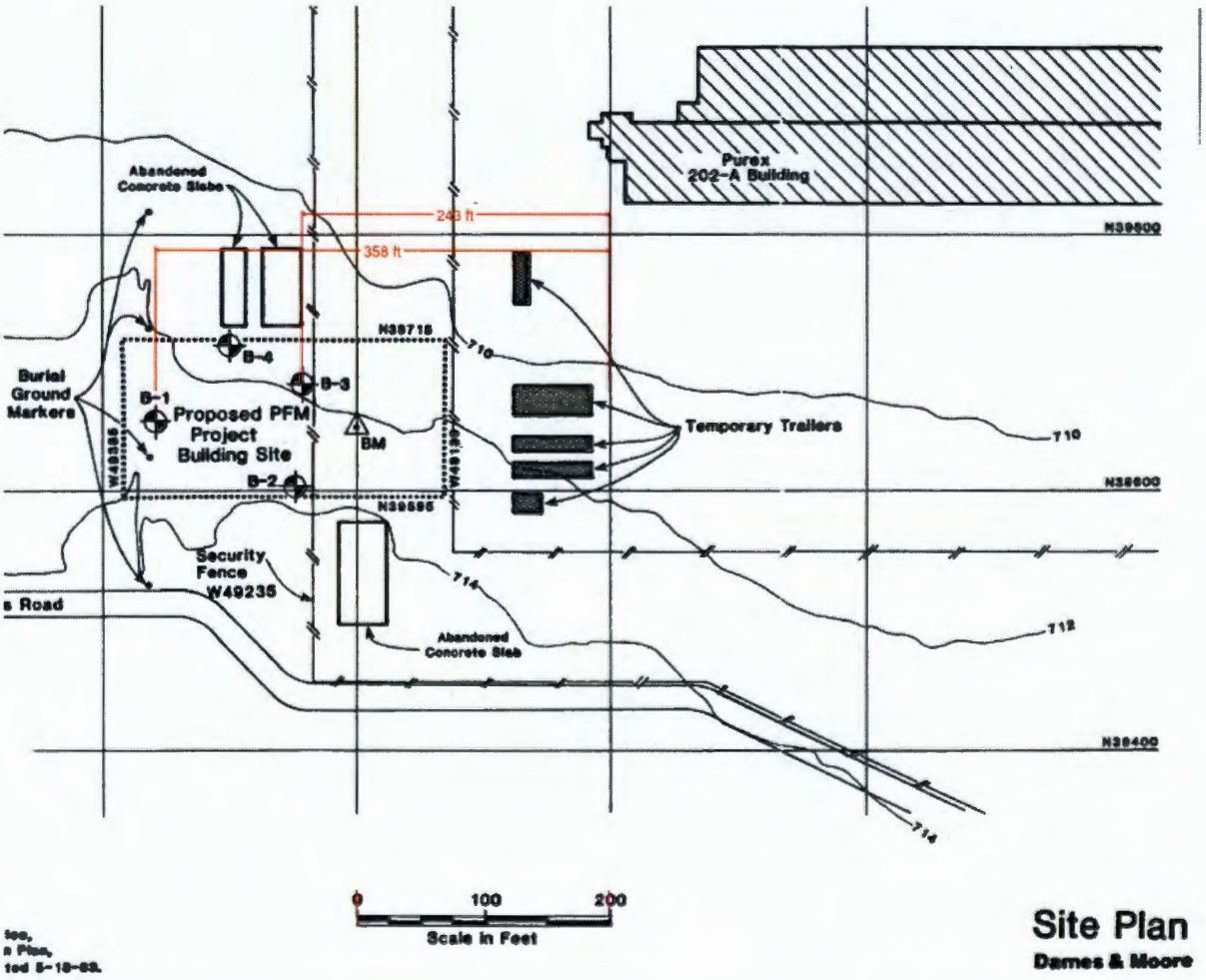

Exhibit 8 – Dames & Moore Site Plan Showing Borehole Locations relative to PUREX Facility. Purex storage tunnels are not shown in this view.

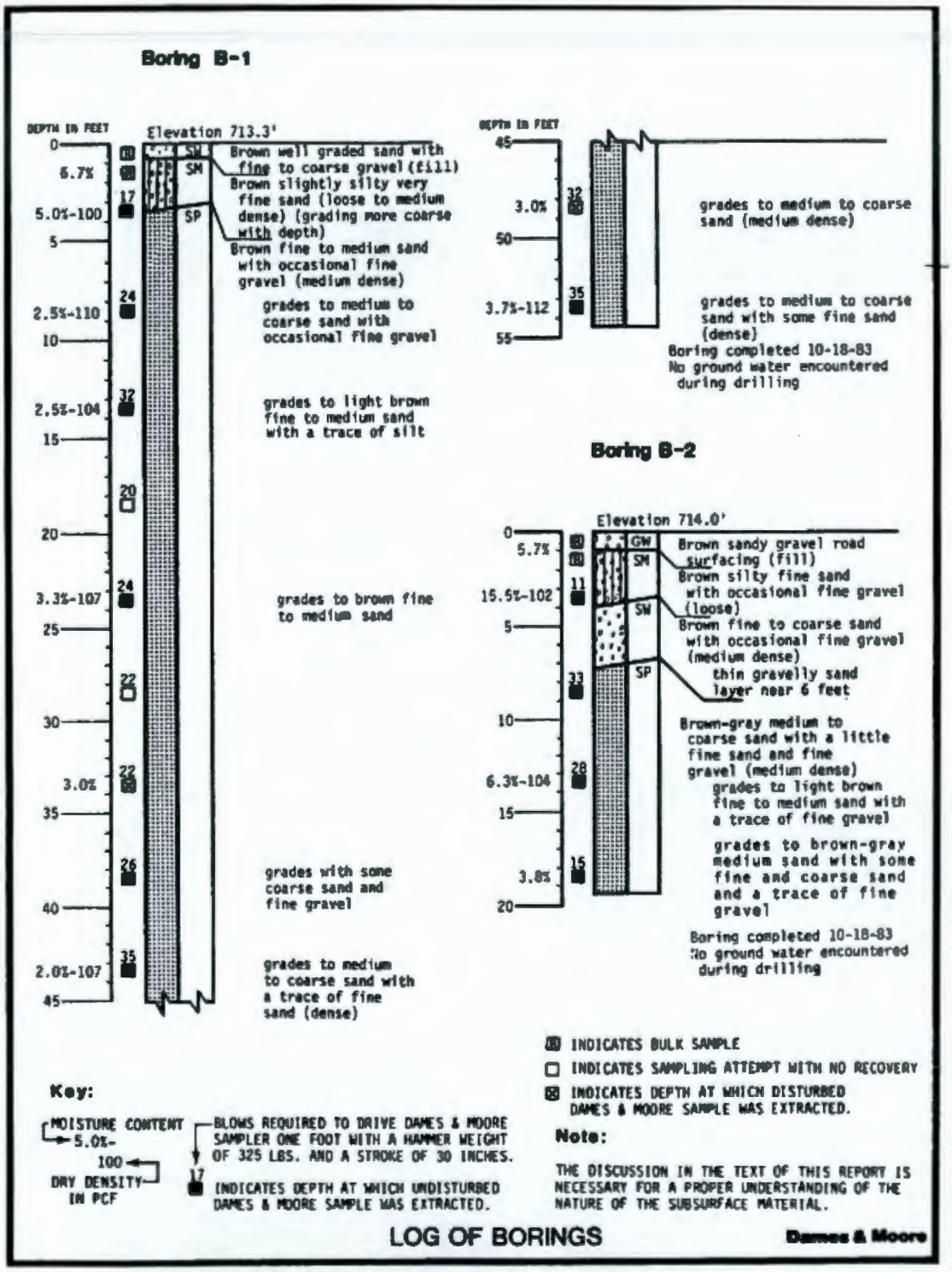

Exhibit 9 – Log of Borings B-1 and B-2

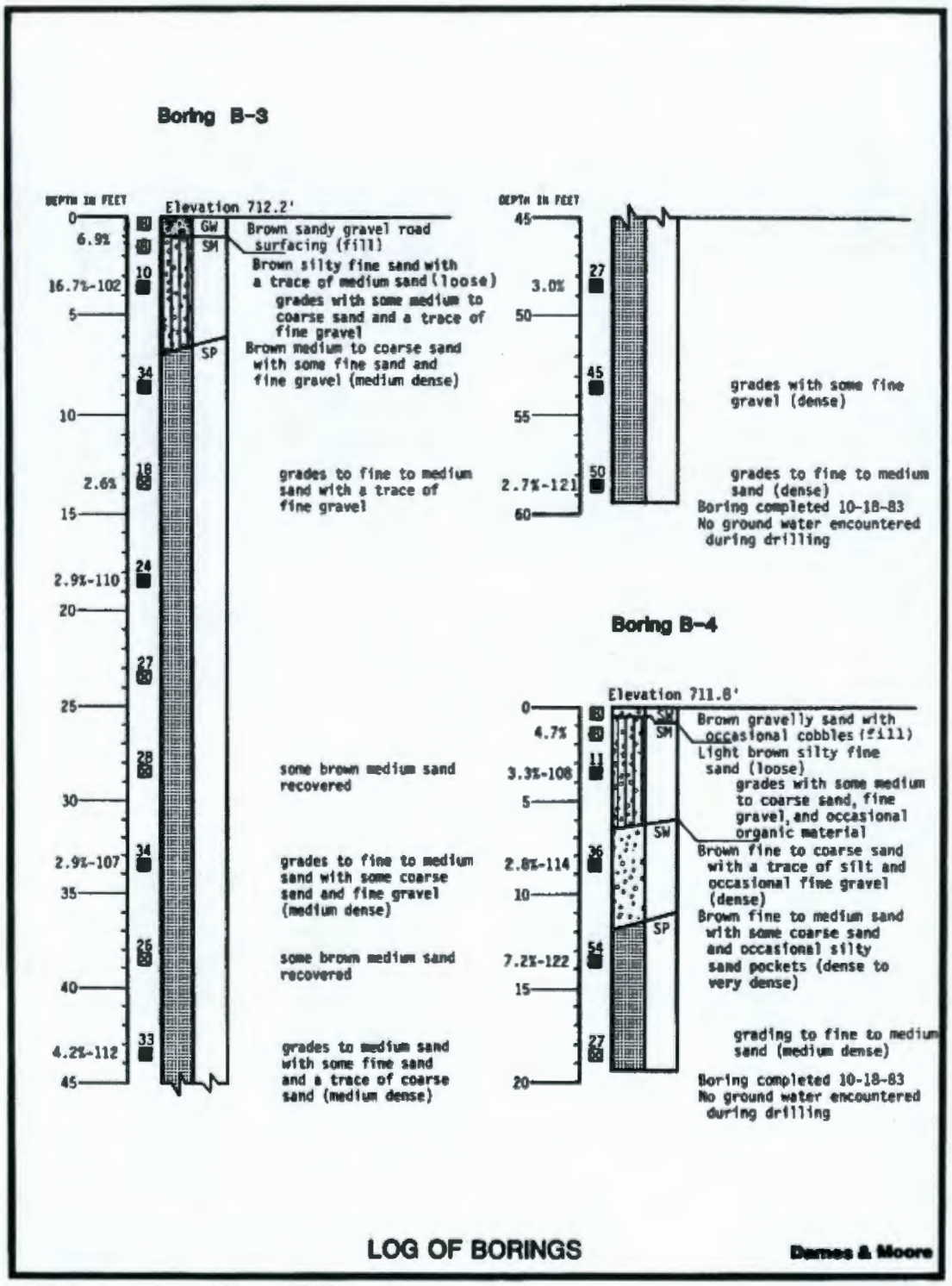

Exhibit 10 – Log of Borings B-3 and B-4

3.0 Geologic and Seismic Investigation of the PUREX Building Site (URS, 1981)

URS (1981) prepared a discussion of the regional geology setting the PUREX facility. This information provides a relevant geologic context for comparison to available geotechnical data, primarily for purposes of confirming expected extents and continuity of site materials.

The project site is underlain by three major geologic units:

1. *The Pasco (glaciofluvial) gravels and associated sediments of the late Pleistocene age at the ground surface* [also known as the Hanford Formation]
2. *Pleistocene-age Ringold formation*
3. *Basaltic lavas and intercalated sediments of the Columbia River basalt group.*

The Pasco basin was formed by downwarping and downfaulting of the basalt flows that underlie the basin. The basin then became the site of deposition of Pliocene sediments of the Ringold Formation and Pleistocene glaciofluvial deposits. The ground surface of the Pasco basin is now largely covered by windblown sand.

Deposition of the Ringold Formation ceased in late Pliocene time, one to two million years ago. Later, ice sheets of the Pleistocene glacial stages advanced from the north but stopped before reaching the Pasco basin. At the close of glaciation, while the ice sheets were retreating, great quantities of water were suddenly released. These huge floods scoured vast areas of basalt terrain in northeastern Washington, swept across the Hanford Reservation area, and crossed the Horse Heaven Hills anticline at Wallula Gap.

...

Locally, within the Pasco Basin, zones of medium-dense to loose sands and gravels, probably resulting from rapid accumulation during glacial floods, are encountered.

The Pasco Gravels are compact, though uncemented, deposits of late Pleistocene and early Recent times. They were laid down by glacial meltwaters and glacial lake floodwaters between about 100,000 and 10,000 years ago. Evidence suggests that in some places the sediments were buried by perhaps an additional 200 feet of gravel that was later swept away.

The Pasco Gravels occur at the surface, or under a thin cover of loessal materials. The water table is controlled by the Columbia River elevation. The Ringold formation occurs near the river level. The [Pasco] gravels, therefore, are typically unsaturated.

Within the 200-East area, where PUREX Building 202-A is located, the uppermost sands and silts were largely reworked during construction activities []. Although the soils beneath the PUREX building have been explored using seismic refraction, other direct exploration methods, such as drilling and sampling, have not been employed.

The water table is at a depth of about 300 feet beneath the plant; there is no liquefaction hazard.

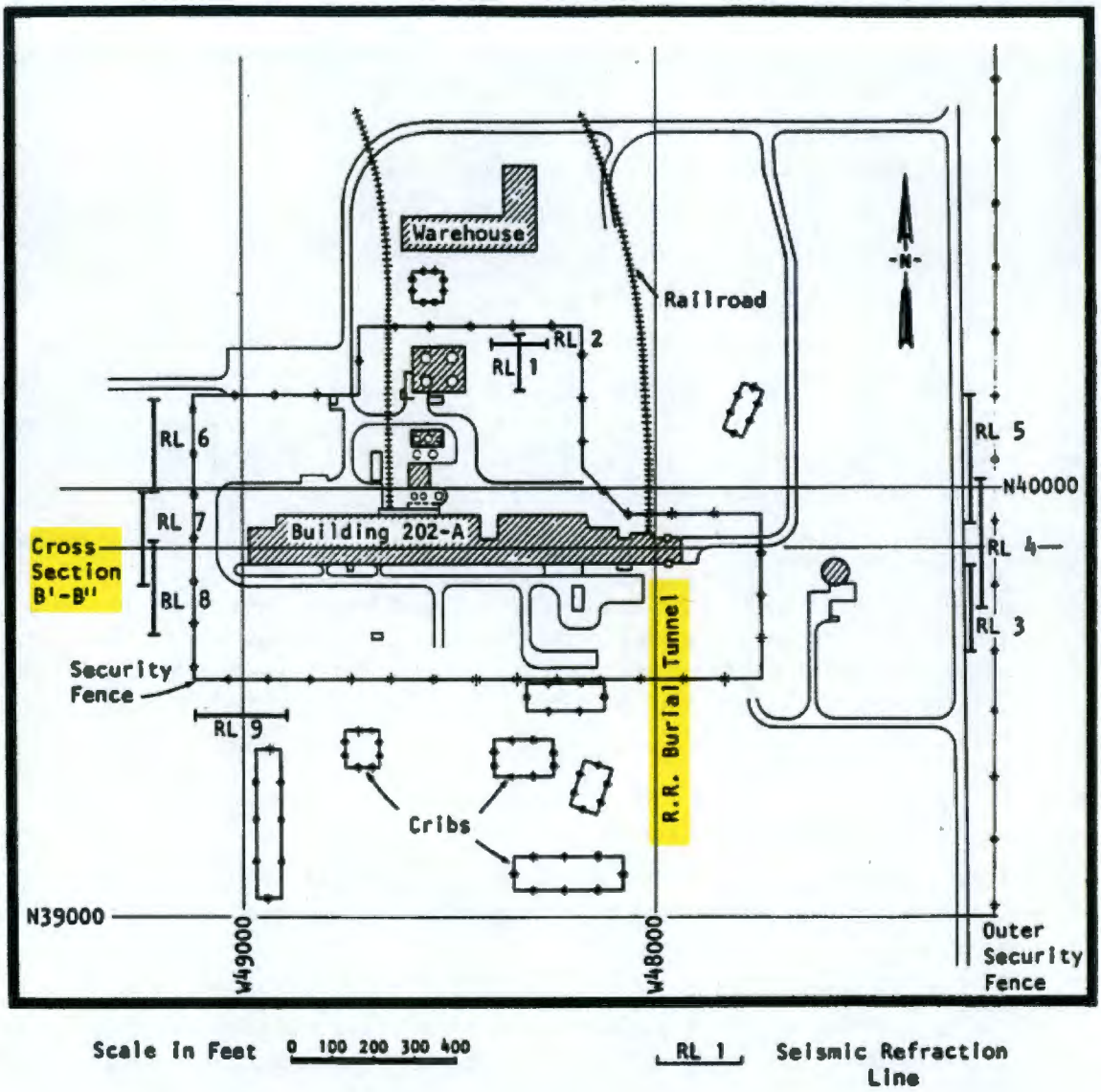

FIGURE 5 PUREX BUILDING SITE PLAN AND SEISMIC REFRACTION LINE LOCATIONS

Exhibit 11 – Site Plan showing location of Cross Section B'-B", located near the northern extent of Tunnel 1

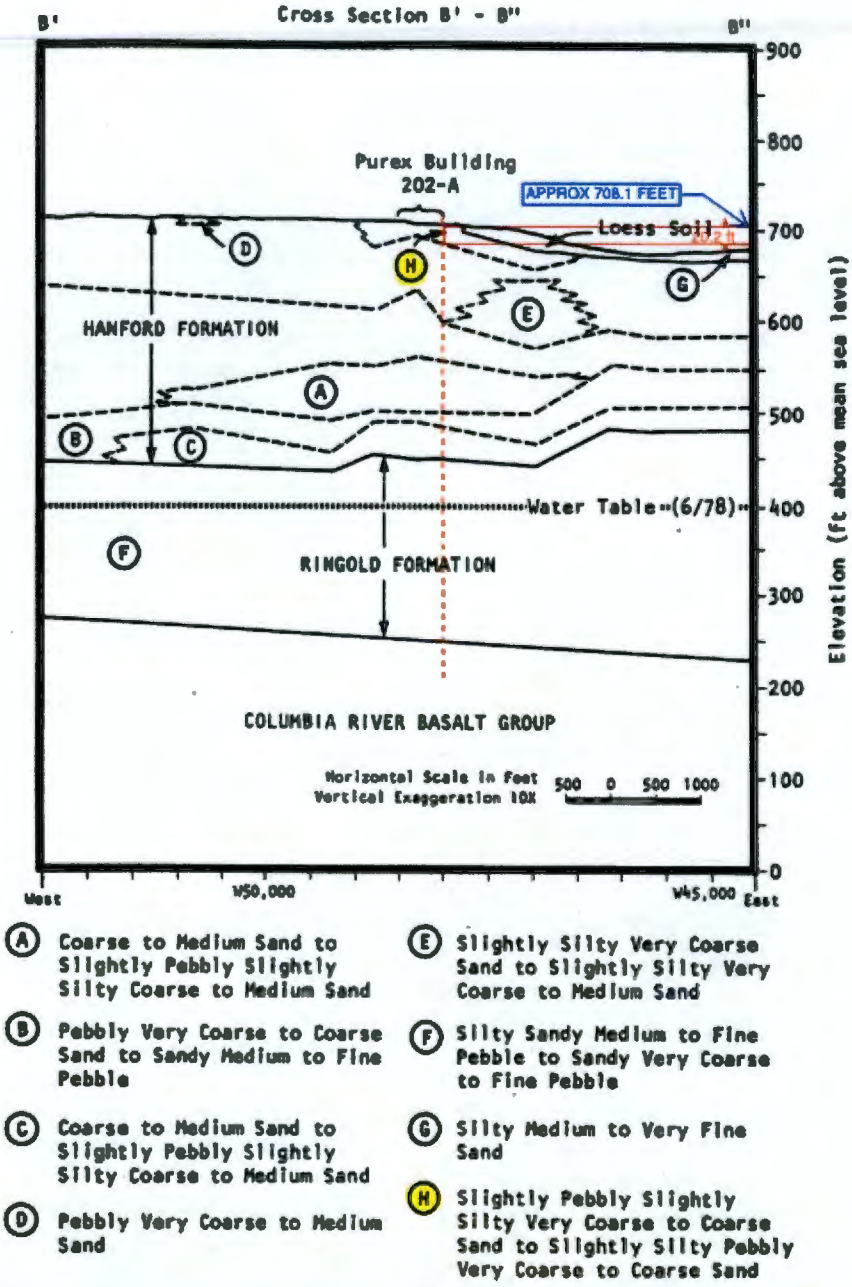

FIGURE 4 GENERALIZED GEOLOGIC CROSS SECTION SHOWING FORMATION NAMES AND SOIL CLASSIFICATION (adapted from Tallman et al., 1979)

Exhibit 12 – Cross Section B'-B". Note descriptions and relative depths of materials and groundwater underlying the PUREX facility. The red dashed line indicates the approximate alignment of the PUREX Storage Tunnels.

Interpretation

Cover soils materials placed over Tunnel 1 were derived from local site soils and processed to remove organic materials and coarse materials larger than 8 inches in diameter.

Project specifications indicate that cover soils were to be "thoroughly compacted," however, no measurable reference density, method specification, moisture condition requirement, or relative compaction is specified. Experience with similar soils suggests that 24-inch lift size does not promote thorough compaction of soil. From the available construction photos, it is believed that the space between the tunnel walls and the tunnel excavation walls was relatively narrow, backfilled from the top using a bulldozer, and not did not permit the use of propelled compaction equipment. Compaction equipment was not visible in the available photos. It appears that the tunnel was constructed from north to south, and backfill was placed behind the advancing tunnel face. It is unknown whether soils placed above the tunnel roof were compacted.

It is assumed that the material placed against the tunnel walls is silty sand material with occasional gravel as described in the boring logs (Dames & Moore, 1984), geologic descriptions of the Hanford Formation / Pasco Gravels (URS, 1981), and construction photographs (CHPRC, 2017), and roof timber sampling photos (Rockwell, 1980). The Tunnel 1 bottom is approximately 20 to 25 feet below existing grade.

Assuming that boring logs from Dames & Moore (1984) are similar to the materials at PUREX Tunnel 1, and that the materials are reasonably well mixed, the fill material is assumed to be as described in the following sections.

4.0 Soil Classification

- The Tunnel 1 cover soils are fill materials obtained from the Tunnel 1 excavation.
- The USCS classification of the soils is Silty Sand with Gravel (SM).
- Fines are expected to be non-plastic.
- Sand is primarily fine to medium size.
- Gravel is fine to medium, subrounded to rounded as visible in photos.

5.0 Specific Gravity

The specific gravity of solid soil particles is assumed to be 2.65.

6.0 Moisture Content

The in-situ moisture content of site soils was estimated by selecting the average in-situ moisture content for 17 samples collected by Dames & Moore (1984) in the top 25 feet of the soil profile. For the reported measurements, the maximum water content observed was 16.7 percent and the minimum was 2.5 percent. The **average in-situ moisture content for the 17 samples was 6 percent** and is recommended for use in tunnel evaluations. Note that actual moisture contents are expected to vary with depth, season, and location along the tunnel alignment.

7.0 Dry Unit Weight

The unit weight of the fill material is difficult to estimate due to the uncertainties in fill placement. Dames & Moore (1984) measured the in-situ dry density of site soils near the PUREX facility. For the 11 field measurements of density taken in the upper 25 feet of the soil profile, the minimum was 100 pcf, the maximum was 122 pcf, and the average was 108 pcf.

Based on a review of site photographs, it is assumed that the cover soils were not well compacted during construction. As such, **the recommended dry unit weight of the loosely placed cover soil is 100 pcf.**

8.0 Moist Unit Weight

The moist unit weight of the Tunnel 1 soil can be estimated using the dry unit weight and the moisture content, using the following fundamental relationship:

$$\gamma_{moist} = \gamma_d \cdot (1 + w\%/100)$$

where:

- γ_m is the moist unit weight,
- γ_d is the dry unit weight,
- w(%) is the in-situ water content, as a percent

Substituting 100 pcf for the dry unit weight, and 6% for the moisture content, gives 106 pcf

Substituting 108 pcf for the dry unit weight and 6% for the moisture content gives 114 pcf.

For structural evaluation of the tunnel, it is recommended that moist unit weights varying from 105 to 115 pcf be considered, with an expected average value of 110 pcf. This recommendation is in general agreement with calculations performed by Rockwell (1980) which used 110 pcf in soil loading calculations.

9.0 Shear Strength

The shear strength of the cover soil was estimated using the published charts in the 1986 Naval Facilities Engineering Command (NAVFAC) Foundations and Earth Structures Design Manual 7.02. Assuming a dry unit weight of 100 pcf and a relative density of 25 to 30 percent for uncompacted soils, **the angle of internal friction, ϕ' is estimated to be approximately 30 degrees. The soil is assumed to be cohesionless (c' = 0psf)**. Drained (effective stress) conditions are expected to govern all loading short term and long term loading scenarios.

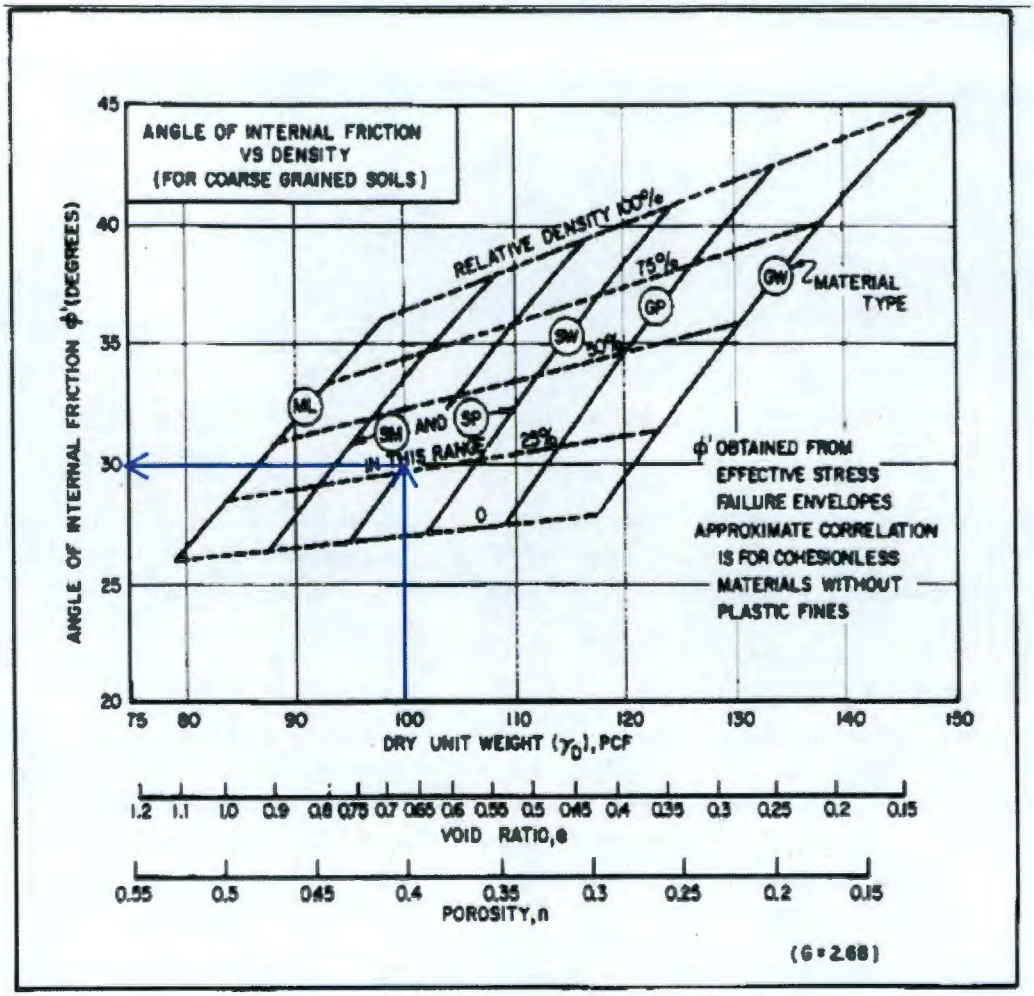

FIGURE 7
Correlations of Strength Characteristics for Granular Soils

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Rockwell Hanford Operations (1980) *Structural Evaluation of the PUREX 1 Burial Tunnel.* Document No. RHO-CD-1076. Prepared by G. R. Silvan. September. (75 pages.)

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URS/John A. Blume & Assoc. (1981) *Geologic and Seismic Investigation of the PUREX Building Site near Richland, Washington.* Prepared for Vitro Engineering Corporation. March.

ch2m	Calculation Title: PUREX Tunnel 1 Engineering Evaluation	Project Number: 693839.BS Project: PUREX Tunnel 1 Client: CHPRC

Attachment B

LIDAR Topography Results

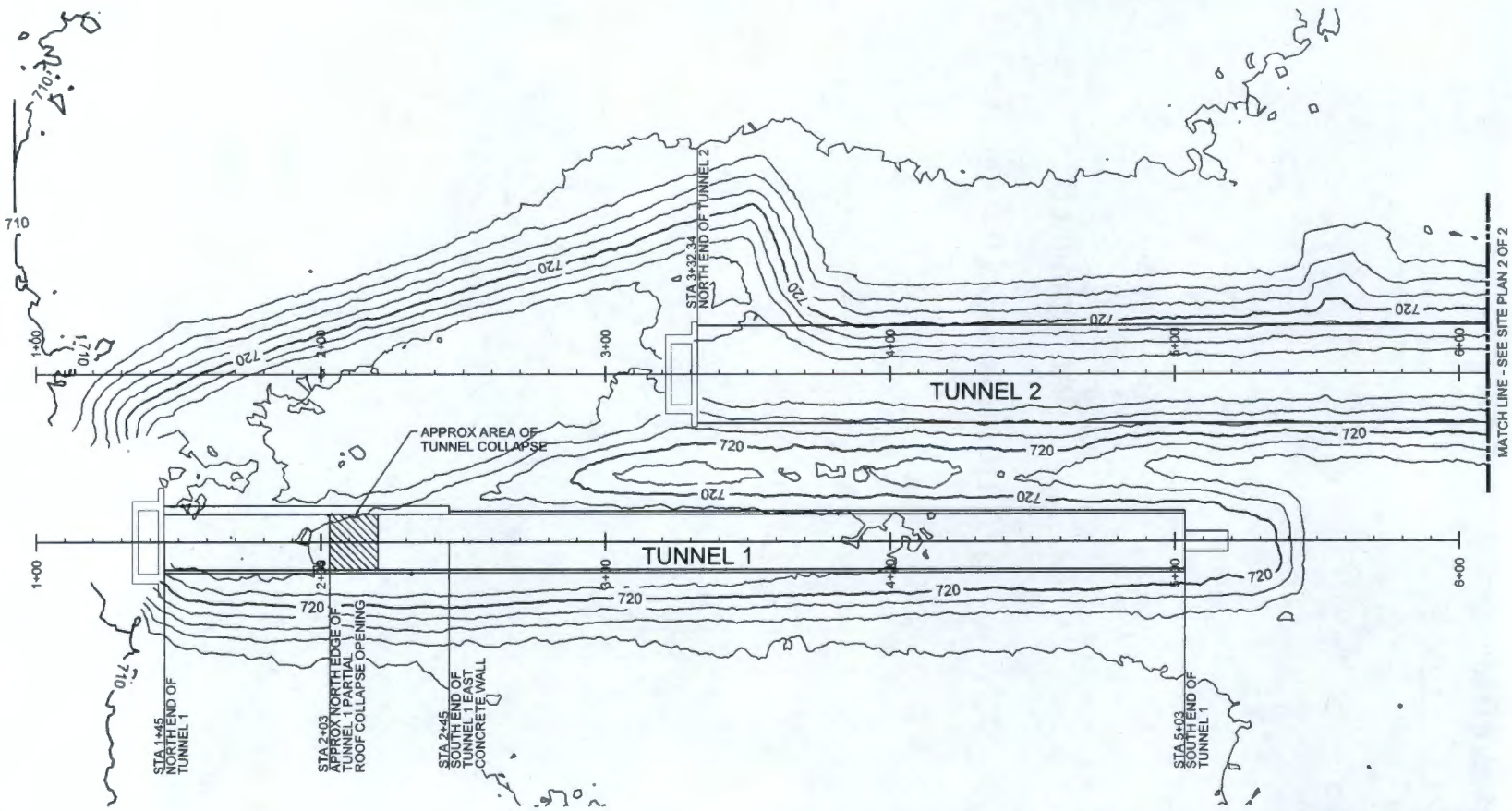
TOPOGRAPHY FROM 2008 *LIDAR CENTRAL PLATEAU REPORT* PREPARED BY AERO-METRIC FOR FLUOR HANFORD. DATA ACQUISITION APRIL 13, 2008. HORIZONTAL DATUM: STATE PLANE NAD83, WASHINGTON SOUTH ZONE, METERS. VERTICAL DATUM: NAVD88, METERS.

HORIZONTAL SYSTEM CONVERTED TO HANFORD SITE DATUM, AREA 200E (PUREX CONTROL), FEET, USING SURVEY MONUMENT COORDINATE DATA FOR MONUMENT 2E-41.

VERTICAL SYSTEM CONVERTED TO HANFORD SITE DATUM, AREA 200E (PUREX CONTROL), FEET, USING SURVEY MONUMENT COORDINATE DATA FOR MONUMENT 2E-41. VERTICAL DATUM SHIFT FROM LIDAR TOPOGRAPHY, -1.100 METERS.

TUNNEL 1 GEOMETRY FROM 1955 AS-BUILT DRAWINGS. FACILITY 218-E-14, DISPOSAL TUNNEL NO. 1 FOR FAILED EQUIPMENT, PROJECT CA-513-A. ISSUED MARCH 10, 1955.

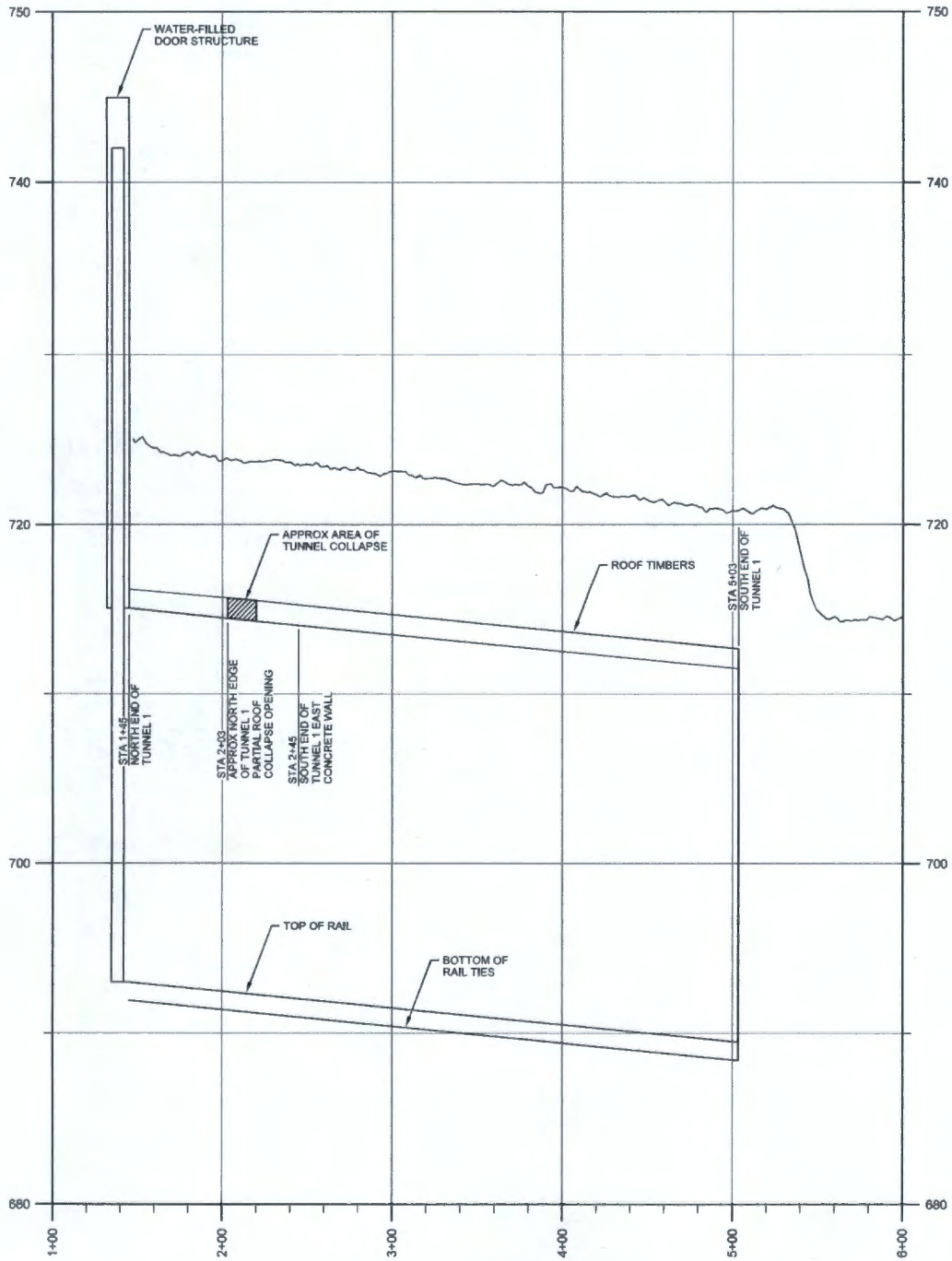
TUNNEL 2 GEOMETRY FROM 1962 AS-BUILT DRAWINGS. FACILITY 218-E-15, DISPOSAL TUNNEL NO. 2 FOR FAILED EQUIPMENT, PROJECT CGC 964. ISSUED SEPTEMBER 1962.

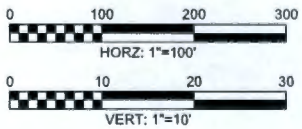


TOPOGRAPHIC SITE PLAN
1 OF 2

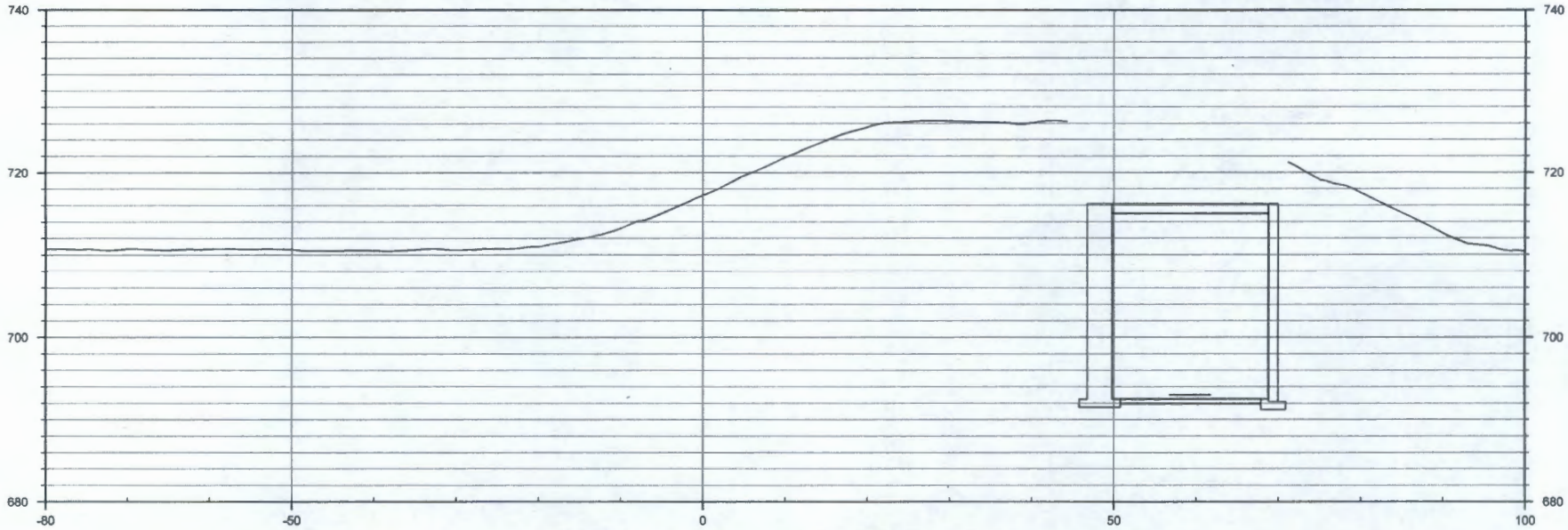
PUREX TUNNEL 1
ENGINEERING EVALUATION

CHPRC-03364, REV. 0

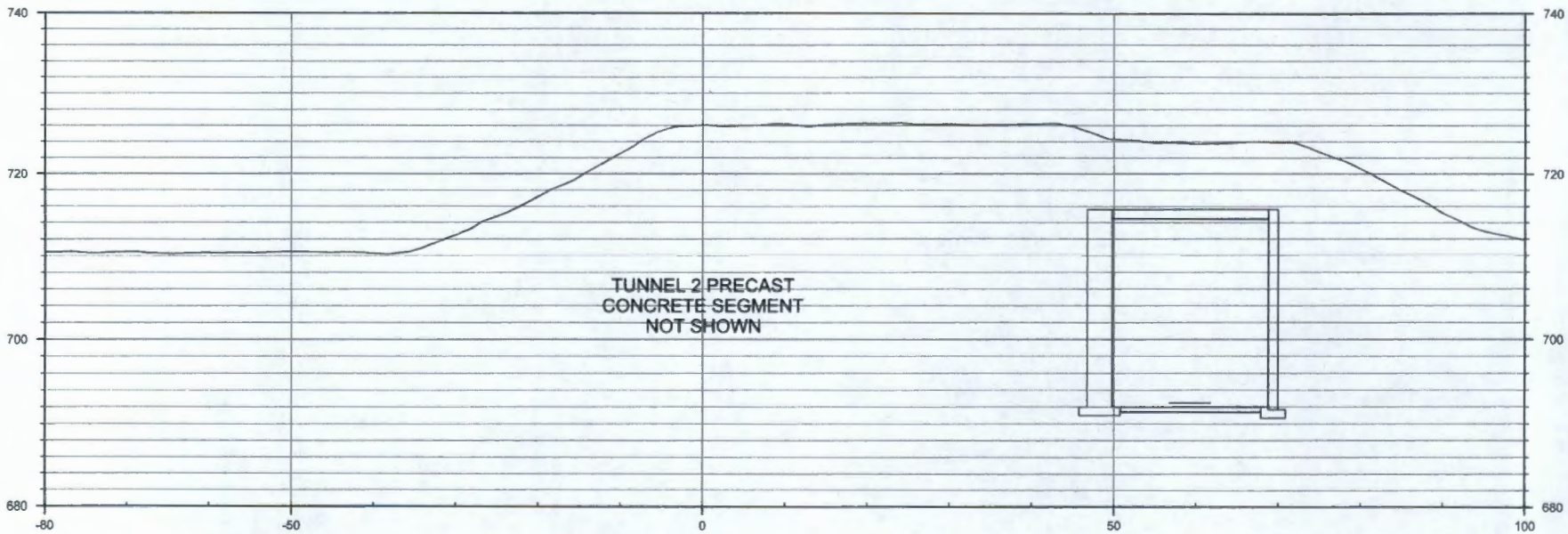




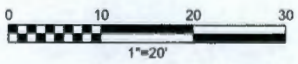
TUNNEL 1 PROFILE
PUREX TUNNEL 1
ENGINEERING EVALUATION



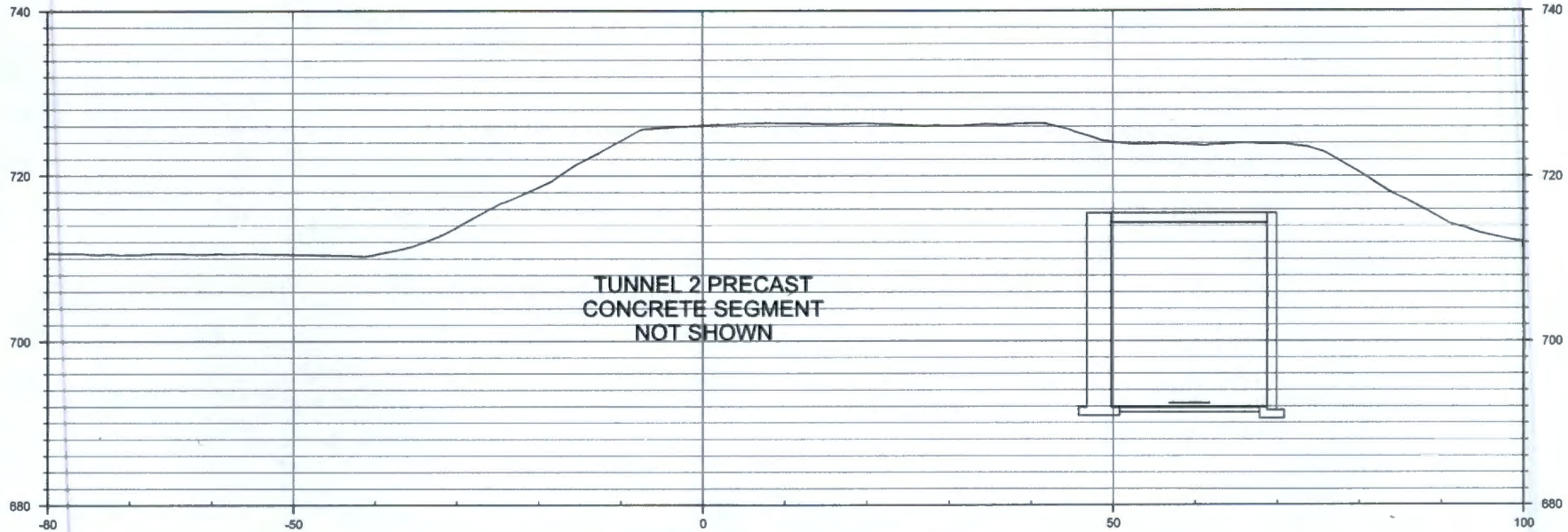
1+45 - NORTH END OF TUNNEL 1



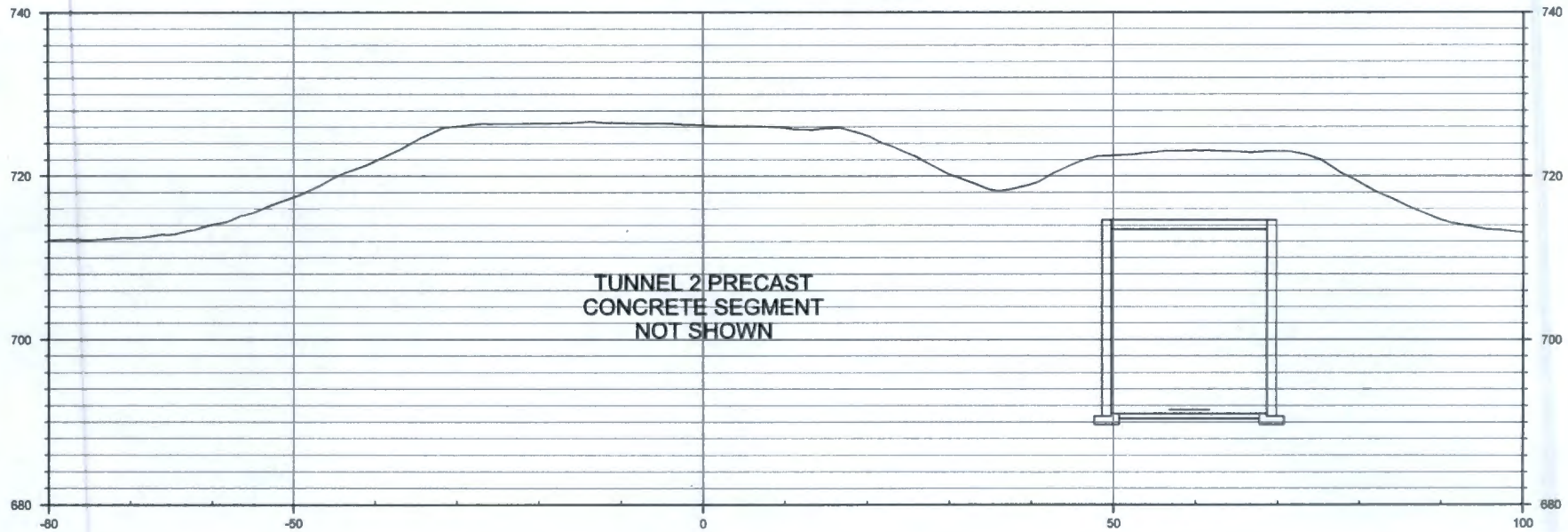
2+00



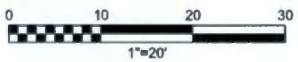
CROSS SECTIONS
1+45 AND 2+00
PUREX TUNNEL 1
ENGINEERING EVALUATION



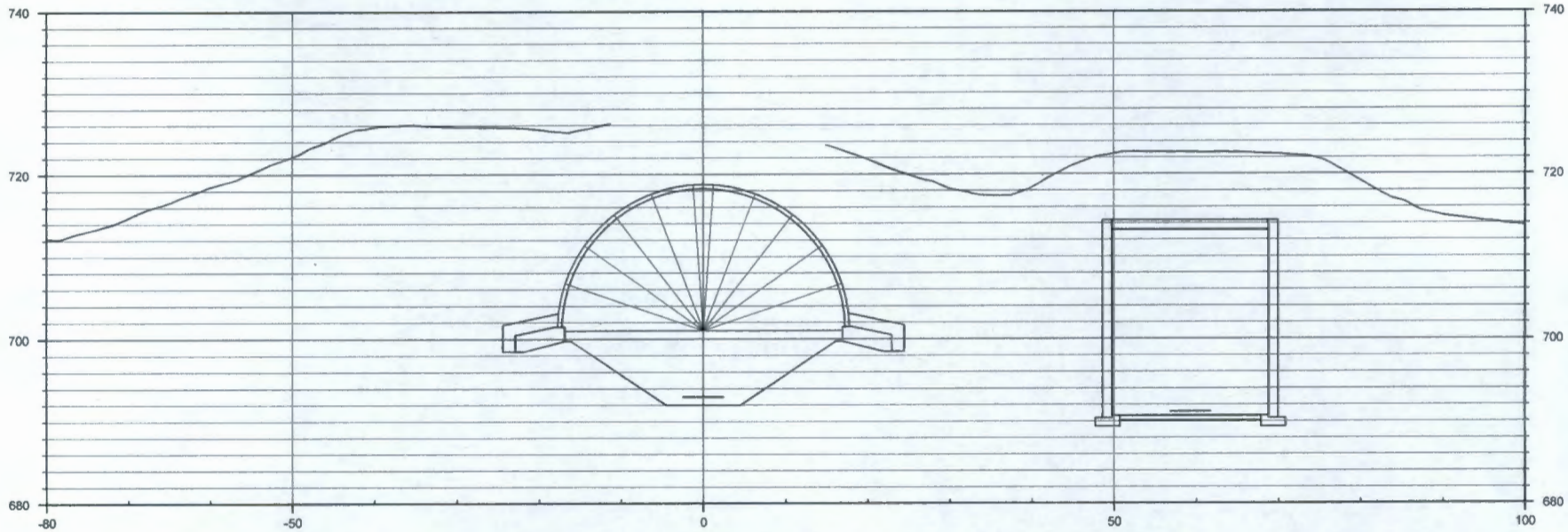
2+10 - TUNNEL 1 PARTIAL ROOF COLLAPSE OPENING



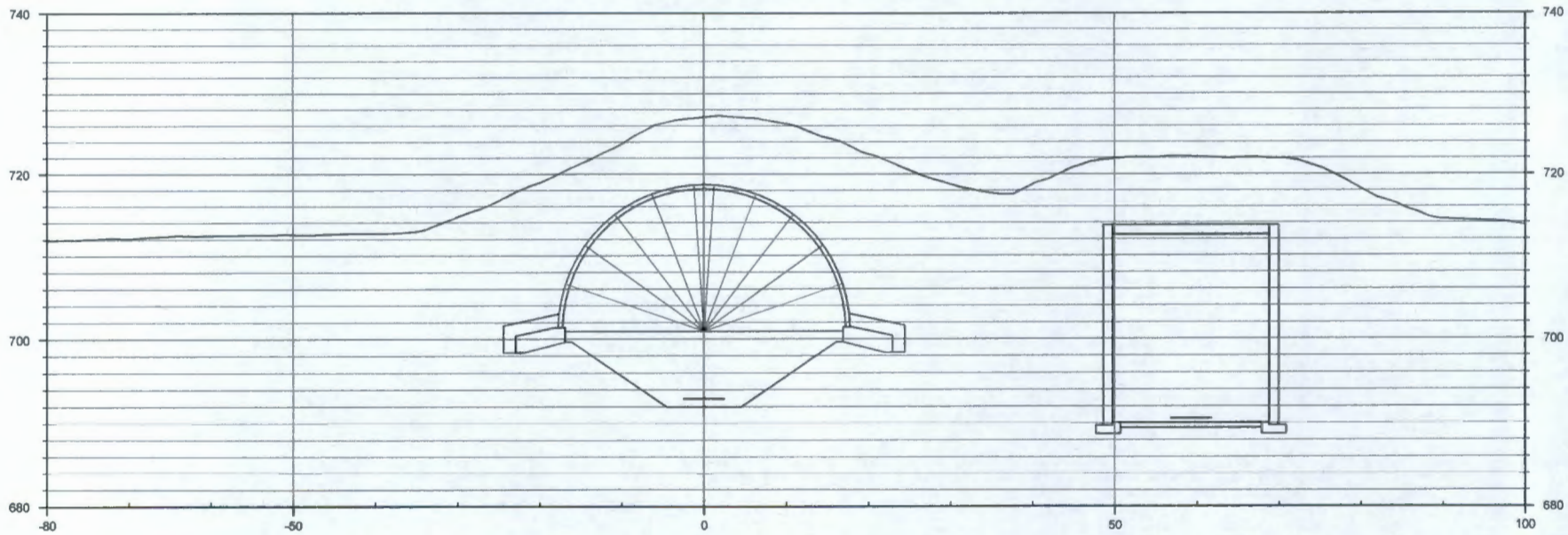
3+00



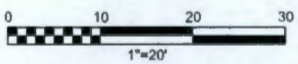
CROSS SECTIONS
2+10 AND 3+00
PUREX TUNNEL 1
ENGINEERING EVALUATION



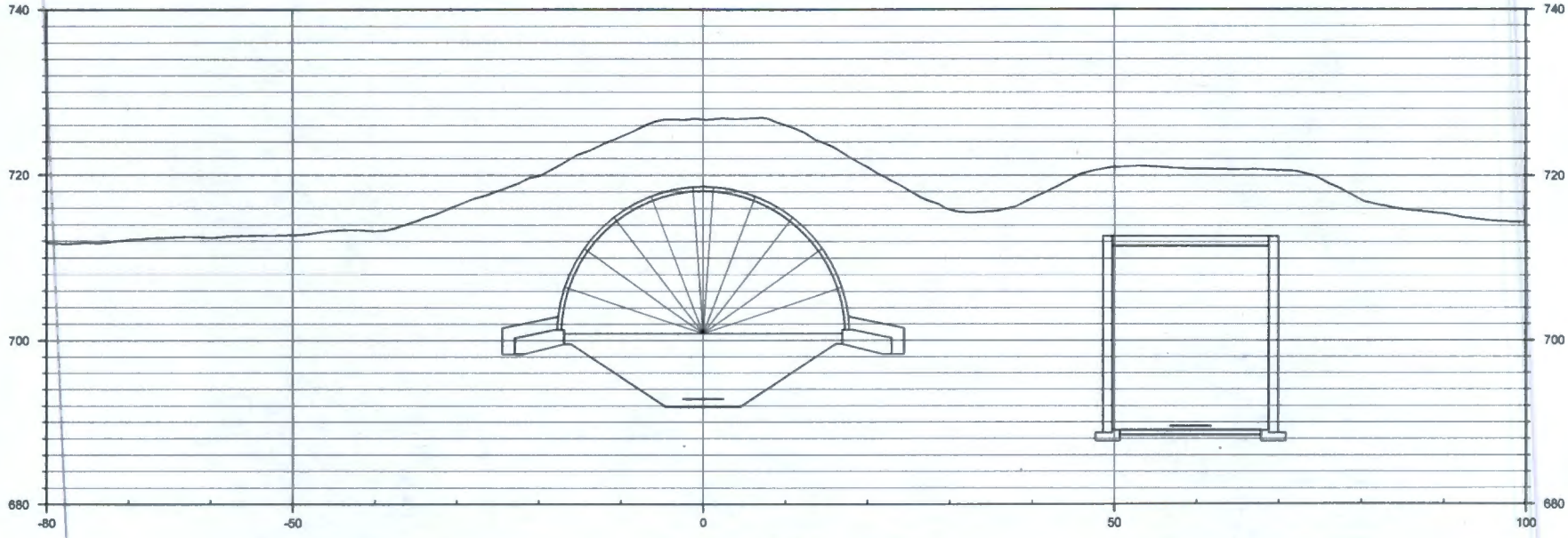
3+32.34 - NORTH END OF TUNNEL 2



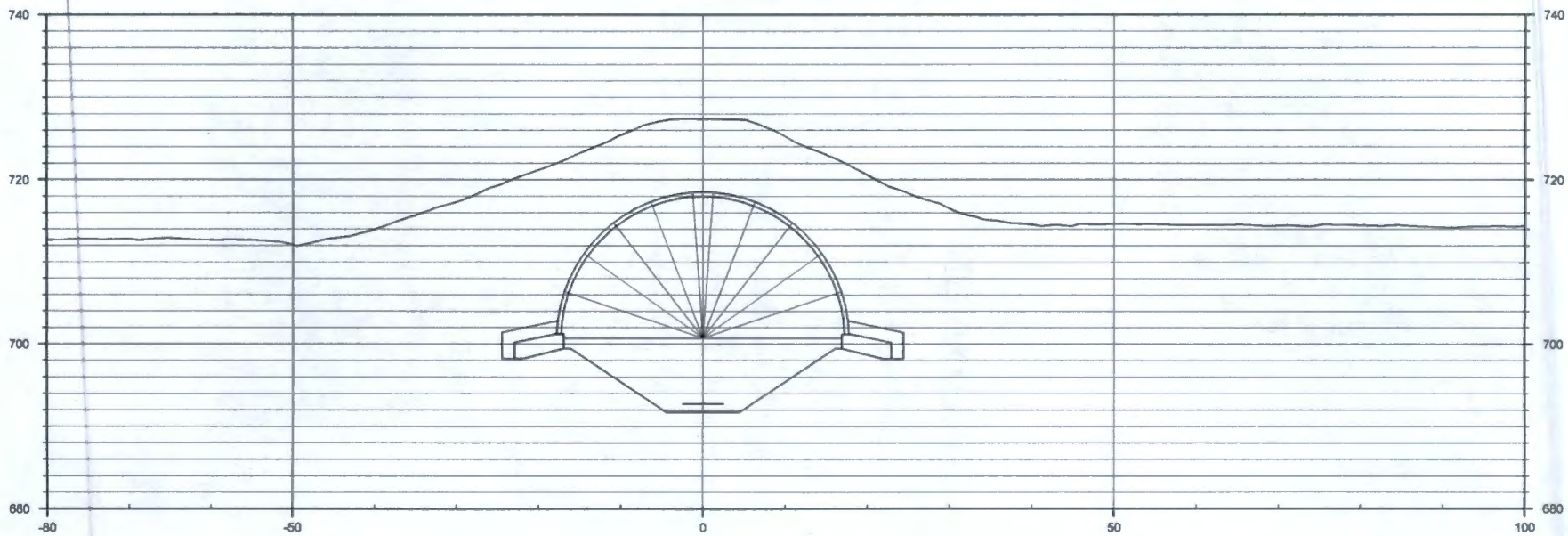
4+00



CROSS SECTIONS
3+32.34 AND 4+00
PUREX TUNNEL 1
ENGINEERING EVALUATION



5+00



6+00

CROSS SECTIONS
5+00 AND 6+00
PUREX TUNNEL 1
ENGINEERING EVALUATION

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LiDAR Digital elevation Model(s)
&
Topographic Contour Maps

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1 Project Area & Synopsis

- This contract consists of obtaining DEM and contour data for the Central Plateau of the Hanford Site. Under a separate contract with Washington Closure Hanford, Aero-Metric completed a LiDAR data acquisition for the entire Hanford site and included most of the Central Plateau. See Exhibit A for project limits. The project area consists of approximately 38,000 acres.
- Aero-Metric obtained Airborne LiDAR data at sufficient density and accuracy to allow for the generation of 0.5m posting DEM grids and 0.3m contours.
- LiDAR survey was supported by on-board Airborne GPS and IMU observations.
- Rogers Surveying was responsible for the ground observations on two base stations during all aerial missions. Aero-Metric was responsible for the computations for the ABGP and IMU measurements.
- Rogers Surveying surveyed and computed the coordinates for approximately 234 vertical points along six profiles and well distributed within the entire Hanford area and representing different terrain types. These points served as true check points to further analyze and correct biases in the LiDAR data.
- See Exhibit B for location of the six selected profiles.

1.1 Coordinate System

- Horizontal system: State Plane NAD83, Washington South Zone, Meters
- Vertical system: NAVD88, Meters

2 LiDAR Survey

2.1 Flight Specifications – See exhibit C for Flight Line Coverage

- Sensor: Optech ALTM.
- Date of data acquisition: April 13, 2008
- Flying Height: 800m / 2634 ft above ground.
- Overlap between flight lines: 50% (100% double coverage)
- System Frequency: 100 KHz
- Scan Frequency: 65 Hz
- Scan Angle: net 24 degrees (12deg on each side)
- Air Speed: 135 kts
- Number of Flight lines: 145
- Geo-Referencing: ABGPS, IMU, and nine ground targets
- Nominal ground resolution: 0.4m
- Mission length: About seven days

2.2 QA/QC Profiles

- See exhibit B for location of the selected QA/QC point profiles.
- See Exhibit D for Point Residual Listing
- About 234 QA/QC points to support analysis and correction to LiDAR data.
- Survey work by Rogers Surveying.

2.3 LiDAR Data Post-Flight / Pre-Processing

- All ABGPS and IMU data, and GPS-obserevations on two base stations are integrated within the computation of the final geo-referencing of each of the flight lines.
- All discrepancies between flight lines are minimized through a number of post-processing algorithms.
- QA/QC points coordinates are used to verify the final accuracy of the derived LiDAR products as described earlier.
- Software used: Optech DASHMAP, Microstation Version 8, TerraSolids TMATCH and TSCAN packages.

2.4 LiDAR Data Editing and Contour Generation

- Post-processed LiDAR data is reduced to 750m by 750m tiles. See exhibit E for Tile Layout and Numbering.
- Classified LiDAR data is visually checked, analyzed, and re-classified if needed. Erroneous points are eliminated.
- After the completion of all editing steps, a LiDAR QC specialist reviewed all data to ensure completeness, conformity to standards, and the accuracy of the data.
- Software used: MicroStation version 8, Terrasolids TSCAN, TMODEL, and TMATCH packages.

3 Final Packaging and Delivery

- The final products are checked against a deliverable list to ensure product completeness. All data used for the project has been archived on permanent archival media for future use if needed by the client.
- The following products are delivered along with this report.
    - Contour data in Microstation Version 8 format.
    - Contour data in Digital Exchange Format (DXF)
    - Contour data in Arc Shapefile format
    - LiDAR data in ASCII .xyz format (All points, First return, Ground, Non-Ground)
    - LiDAR data in .las format
    - DEM Grids in Arc Grid format (All points, First return, Ground, Non-Ground)
    - This LiDAR report including the QA/QC points residuals

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EXHIBIT A – PROJECT AREA

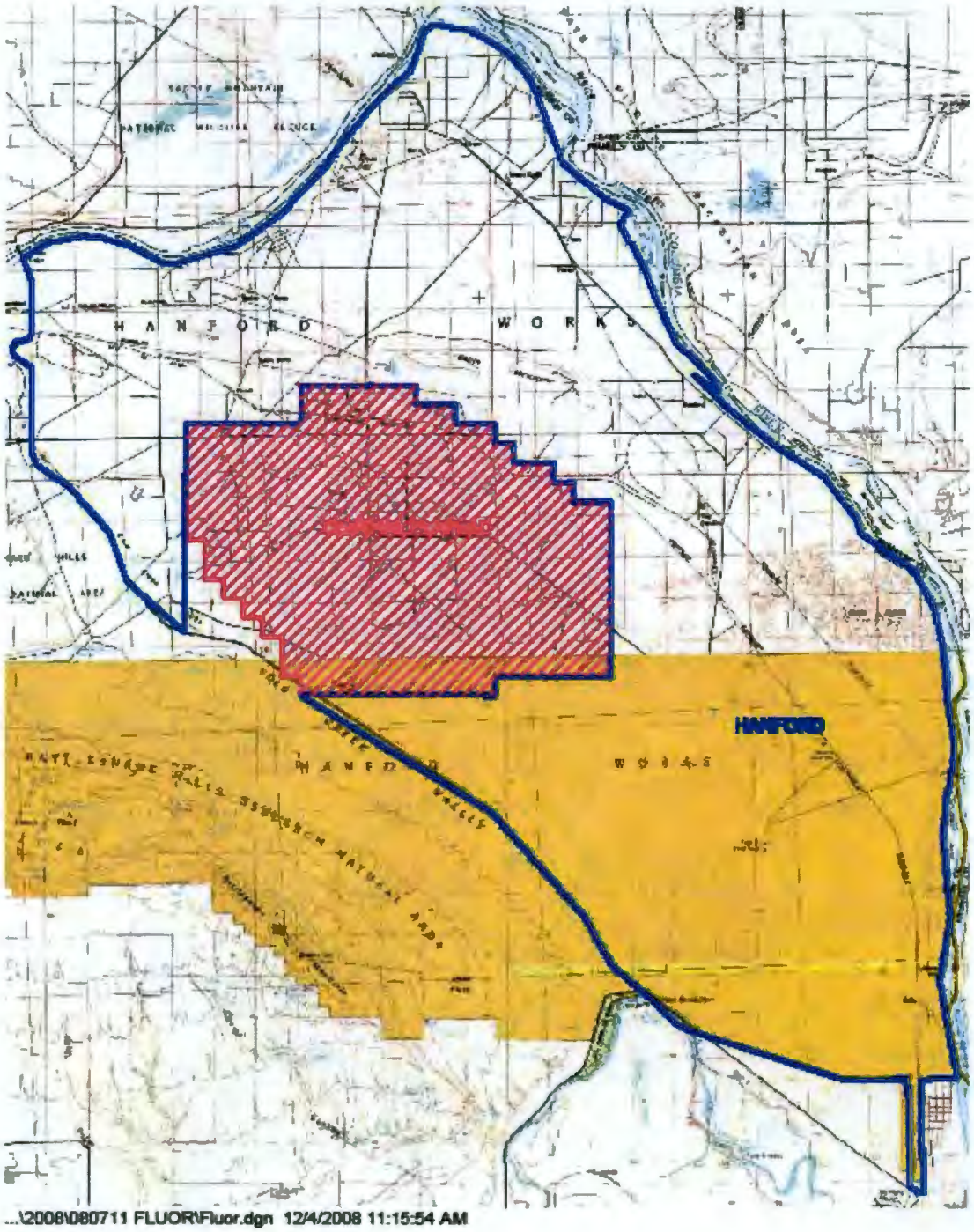

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EXHIBIT B – QA/QC LiDAR PROFILES LOCATION

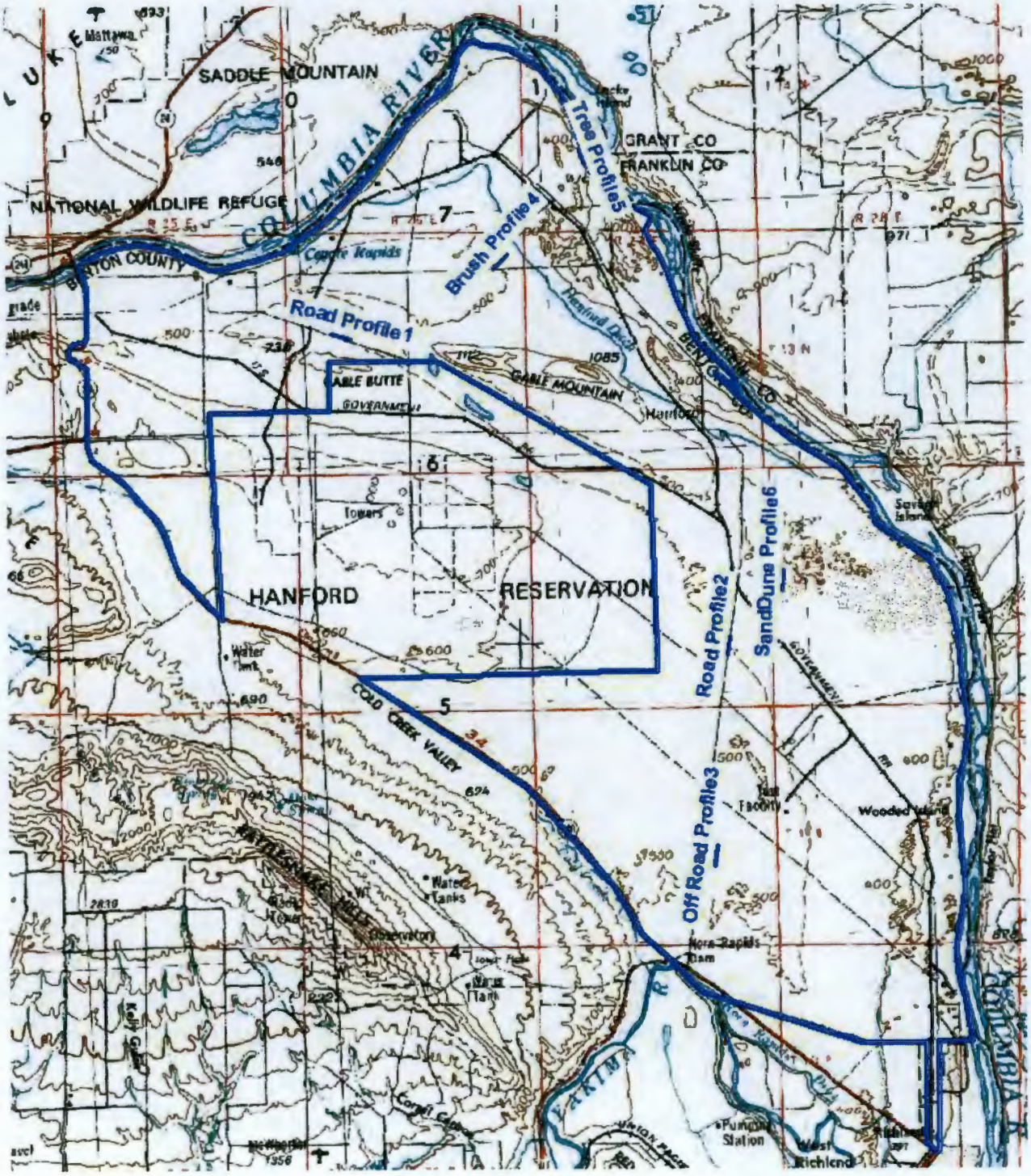

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EXHIBIT C – LiDAR FLIGHT LINE COVERAGE

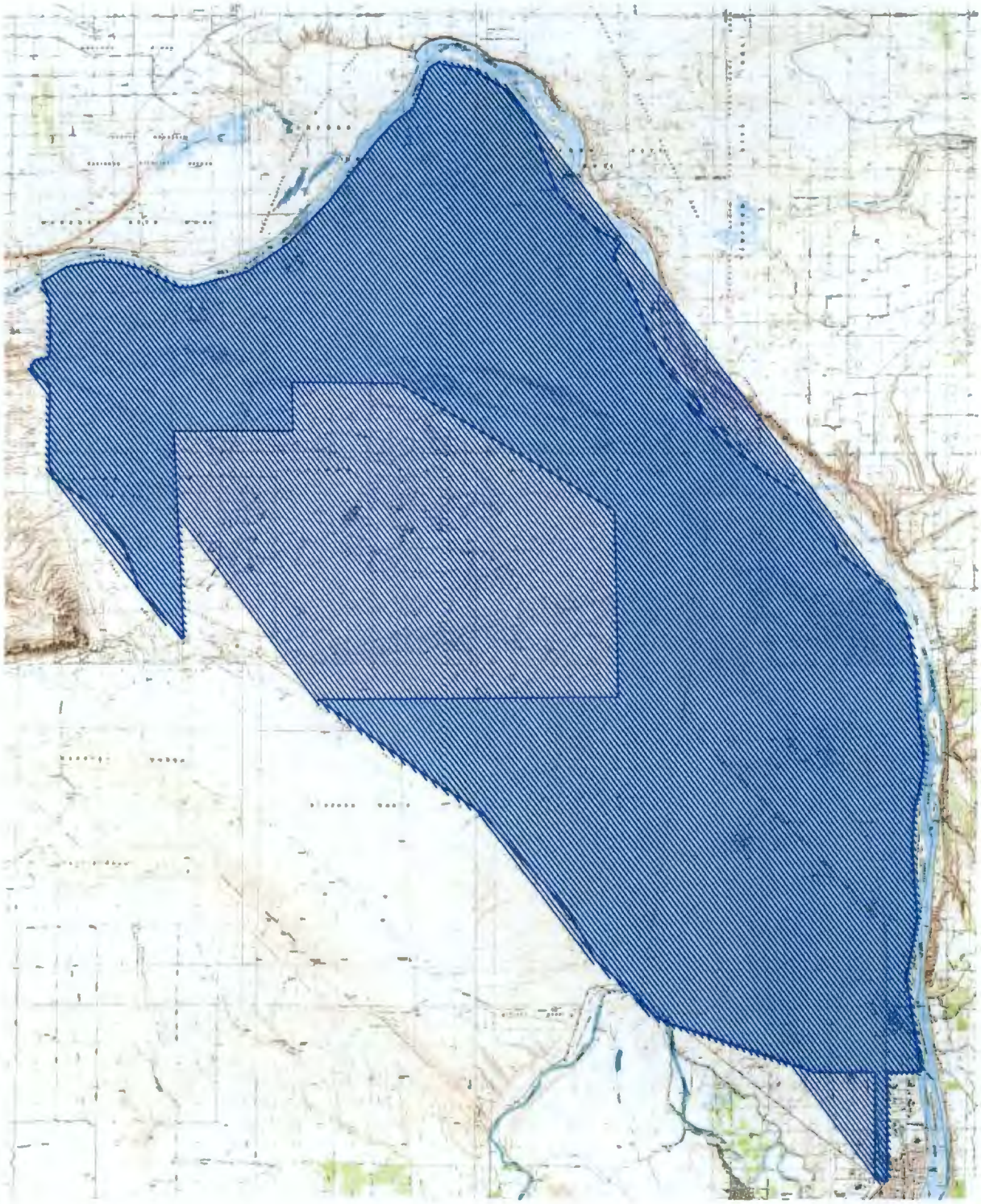

EXHIBIT D – GROUND SURVEYED QA/QC POINTS RESIDUALS (LiDAR minus Survey)

Number	Easting	Northing	Known Z	Laser Z	Dz
1060	587475.238	132228.964	143.798	144.280	+0.482
1249	579340.993	149856.447	118.679	118.970	+0.291
1251	579354.060	149816.955	117.872	118.140	+0.268
1257	579398.173	149685.686	117.363	117.580	+0.217
1250	579348.422	149834.166	118.097	118.270	+0.173
1256	579373.489	149694.794	118.975	119.140	+0.165
1258	579408.160	149654.826	118.016	118.160	+0.144
1263	579450.924	149526.348	117.780	117.920	+0.140
1247	579321.640	149916.351	119.914	120.050	+0.136
1260	579429.732	149589.940	118.158	118.280	+0.122
1259	579419.576	149621.120	118.200	118.310	+0.110
1205	575725.372	145382.413	153.510	153.610	+0.100
1206	575734.716	145392.197	153.144	153.240	+0.096
1235	576160.150	145857.677	140.360	140.450	+0.090
1029	585334.472	130096.641	145.601	145.690	+0.089
1242	579280.772	150039.035	120.312	120.400	+0.088
1084	587531.404	132654.877	148.093	148.180	+0.087
1046	585393.347	130504.843	140.355	140.440	+0.085
1229	576067.969	145757.301	143.657	143.740	+0.083
1074	587504.092	132451.257	144.460	144.540	+0.080
1225	576008.265	145692.311	144.430	144.510	+0.080
1111	584732.979	121081.044	161.161	161.240	+0.079
1106	584722.465	120983.923	162.562	162.640	+0.078
1037	585356.787	130251.641	142.384	142.460	+0.076
1252	579358.353	149803.759	117.784	117.860	+0.076
1063	587481.152	132273.435	144.935	145.010	+0.075
1107	584723.326	120993.677	162.786	162.860	+0.074
1043	585381.737	130425.046	139.898	139.970	+0.072
1104	584718.804	120948.671	160.868	160.940	+0.072
1127	584775.960	121488.399	162.498	162.570	+0.072
1047	585397.229	130530.652	140.739	140.810	+0.071
1090	587543.694	132745.898	144.630	144.700	+0.070
1238	576207.541	145909.804	139.670	139.740	+0.070
1209	575781.473	145443.937	152.011	152.080	+0.069
1239	579254.962	150117.591	120.811	120.880	+0.069
1033	585345.655	130173.495	143.872	143.940	+0.068
1087	587535.486	132683.870	143.213	143.280	+0.067
1241	579270.765	150068.564	120.543	120.610	+0.067
1234	576143.187	145839.927	140.815	140.880	+0.065
1105	584720.514	120966.552	161.768	161.830	+0.062
1034	585348.104	130190.540	143.500	143.560	+0.060
1044	585385.662	130451.978	139.930	139.990	+0.060
1013	585285.770	129762.754	159.731	159.790	+0.059
1113	584737.235	121121.823	160.511	160.570	+0.059
1203	575697.710	145352.397	154.161	154.220	+0.059
1068	587493.500	132364.239	147.042	147.100	+0.058
1119	584755.026	121290.156	162.702	162.760	+0.058
1079	587517.153	132544.162	143.644	143.700	+0.056
1233	576126.535	145821.649	141.154	141.210	+0.056
1048	585400.833	130555.520	141.045	141.100	+0.055
1067	587490.288	132341.425	143.815	143.870	+0.055
1202	575684.795	145338.702	154.425	154.480	+0.055
1031	585339.057	130128.201	144.916	144.970	+0.054
1129	584780.205	121525.340	162.716	162.770	+0.054
1058	587470.086	132193.265	145.097	145.150	+0.053
1056	587468.194	132178.359	147.378	147.430	+0.052
1246	579311.631	149944.598	119.948	120.000	+0.052
1262	579444.609	149546.715	117.958	118.010	+0.052
1200	575649.474	145299.913	155.988	156.040	+0.052
1065	587485.476	132308.395	144.339	144.390	+0.051
1240	579262.006	150095.471	120.599	120.650	+0.051
1032	585343.442	130157.930	144.230	144.280	+0.050
1243	579289.207	150013.874	120.320	120.370	+0.050
1244	579297.440	149987.930	120.431	120.480	+0.049
1040	585369.572	130340.238	140.922	140.970	+0.048
1092	587549.771	132792.058	145.612	145.660	+0.048
1095	587555.680	132838.165	145.482	145.530	+0.048
1039	585365.902	130314.026	141.343	141.390	+0.047
1224	575993.975	145676.430	144.603	144.650	+0.047
1248	579330.740	149888.793	119.423	119.470	+0.047
1054	587466.279	132160.286	146.184	146.230	+0.046
1078	587515.376	132531.208	144.984	145.030	+0.046
1254	579351.470	149761.925	120.035	120.080	+0.045
1053	585417.062	130667.336	141.175	141.220	+0.045
1219	575922.100	145597.732	147.805	147.850	+0.045
1066	587486.884	132318.500	143.376	143.420	+0.044
1024	585322.692	130014.839	147.557	147.600	+0.043
1109	584729.398	121043.603	161.517	161.560	+0.043
1226	576025.399	145710.966	143.857	143.900	+0.043
1101	587570.353	132945.220	146.418	146.460	+0.042
1208	575766.562	145427.564	152.239	152.280	+0.041
1006	585262.488	129602.806	162.470	162.510	+0.040

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Number	Easting	Northing	Known Z	Laser Z	Dz
1035	585352.545	130221.264	142.901	142.940	+0.039
1030	585336.726	130112.466	145.282	145.320	+0.038
1228	576052.502	145740.052	143.252	143.290	+0.038
1110	584730.962	121062.110	161.533	161.570	+0.037
1253	579341.285	149792.516	120.363	120.400	+0.037
1064	587483.223	132290.597	143.904	143.940	+0.036
1094	587552.304	132812.394	145.675	145.710	+0.035
1207	575750.411	145409.761	152.525	152.560	+0.035
1227	576037.938	145724.396	143.285	143.320	+0.035
1216	575856.934	145526.814	149.476	149.510	+0.034
1073	587502.724	132438.206	147.827	147.860	+0.033
1210	575796.525	145460.724	151.277	151.310	+0.033
1237	576190.571	145891.082	139.908	139.940	+0.032
1080	587519.405	132561.432	143.929	143.960	+0.031
1100	587568.481	132936.009	146.499	146.530	+0.031
1245	579303.030	149972.045	120.070	120.100	+0.030
1261	579438.794	149564.664	118.360	118.390	+0.030
1026	585327.239	130047.124	146.662	146.690	+0.028
1028	585331.994	130079.151	145.972	146.000	+0.028
1108	584725.530	121014.277	161.683	161.710	+0.027
1017	585301.952	129873.589	153.884	153.910	+0.026
1204	575711.747	145367.563	153.994	154.020	+0.026
1236	576176.449	145875.905	140.004	140.030	+0.026
1036	585354.638	130236.508	142.604	142.630	+0.026
1052	585414.120	130646.630	141.215	141.240	+0.025
1083	587529.062	132635.560	145.155	145.180	+0.025
1218	575900.312	145574.535	148.485	148.510	+0.025
1220	575937.207	145614.189	147.205	147.230	+0.025
1093	587551.306	132802.381	145.146	145.170	+0.024
1077	587512.095	132506.571	145.116	145.140	+0.024
1214	575834.228	145501.400	149.636	149.660	+0.024
1115	584744.477	121190.555	160.397	160.420	+0.023
1221	575952.192	145630.292	146.527	146.550	+0.023
1015	585293.665	129816.196	156.978	157.000	+0.022
1021	585313.338	129950.528	149.899	149.920	+0.021
1062	587478.885	132256.844	144.919	144.940	+0.021
1004	585257.252	129565.131	162.220	162.240	+0.020
1022	585316.063	129969.702	149.060	149.080	+0.020
1075	587507.324	132472.236	142.870	142.890	+0.020
1096	587559.724	132865.958	145.550	145.570	+0.020
1009	585271.984	129666.647	162.731	162.750	+0.019
1217	575878.912	145550.191	149.131	149.150	+0.019
1027	585329.591	130063.183	146.312	146.330	+0.018

Number	Easting	Northing	Known Z	Laser Z	Dz
1223	575985.550	145667.458	145.123	145.140	+0.017
1011	585278.672	129712.912	161.833	161.850	+0.017
1019	585308.711	129919.113	151.453	151.470	+0.017
1116	584747.972	121223.200	160.274	160.290	+0.016
1038	585361.199	130281.578	141.915	141.930	+0.015
1076	587510.869	132493.952	143.015	143.030	+0.015
1137	584797.178	121687.712	161.266	161.280	+0.014
1222	575968.795	145648.424	146.196	146.210	+0.014
1000	585246.671	129492.270	161.737	161.750	+0.013
1102	587571.431	132954.897	147.827	147.840	+0.013
1213	575831.101	145497.699	149.727	149.740	+0.013
1117	584751.321	121255.658	160.448	160.460	+0.012
1042	585377.796	130397.331	140.069	140.080	+0.011
1005	585259.702	129583.398	162.299	162.310	+0.011
1128	584777.534	121507.256	162.779	162.790	+0.011
1061	587478.126	132249.504	143.640	143.650	+0.010
1141	584805.858	121768.837	160.970	160.980	+0.010
1122	584760.344	121339.695	163.011	163.020	+0.009
1134	584790.705	121626.756	164.801	164.810	+0.009
1255	579363.387	149725.734	119.651	119.660	+0.009
1057	587468.520	132182.242	147.091	147.100	+0.009
1215	575835.194	145502.674	149.841	149.850	+0.009
1230	576084.177	145775.166	143.871	143.880	+0.009
1192	569862.857	142446.807	162.902	162.910	+0.008
1232	576114.464	145808.402	142.592	142.600	+0.008
1098	587566.081	132914.210	147.343	147.350	+0.007
1201	575671.847	145324.480	155.603	155.610	+0.007
1010	585275.209	129689.221	162.454	162.460	+0.006
1097	587564.037	132898.331	146.784	146.790	+0.006
1126	584772.523	121455.126	161.684	161.690	+0.006
1140	584804.218	121751.800	161.454	161.460	+0.006
1050	585407.594	130601.992	141.325	141.330	+0.005
1086	587533.314	132667.314	147.765	147.770	+0.005
1199	569932.756	142401.189	163.135	163.140	+0.005
1088	587536.923	132696.760	143.995	144.000	+0.005
1023	585320.291	129998.692	148.027	148.030	+0.003
1125	584768.839	121420.448	162.887	162.890	+0.003
1124	584766.584	121399.722	164.638	164.640	+0.002
1211	575812.652	145478.706	150.638	150.640	+0.002
1091	587548.132	132778.387	145.349	145.350	+0.001
1041	585373.808	130368.986	140.440	140.440	+0.000
1114	584740.801	121155.878	160.291	160.290	-0.001
1045	585389.520	130478.627	140.093	140.090	-0.003

AERO-METRIC
SEATTLE

Number	Easting	Northing	Known Z	Laser Z	Dz
1089	587540.288	132723.323	142.843	142.840	-0.003
1003	585254.801	129548.103	162.133	162.130	-0.003
1025	585324.996	130030.991	147.073	147.070	-0.003
1071	587498.852	132409.974	143.704	143.700	-0.004
1008	585268.861	129644.743	162.734	162.730	-0.004
1049	585404.251	130579.220	141.234	141.230	-0.004
1007	585265.531	129623.343	162.595	162.590	-0.005
1103	587573.207	132968.568	147.715	147.710	-0.005
1130	584783.234	121555.343	162.065	162.060	-0.005
1131	584784.841	121571.940	163.065	163.060	-0.005
1018	585306.447	129903.713	152.287	152.280	-0.007
1138	584800.327	121720.404	162.597	162.590	-0.007
1002	585251.722	129525.940	161.938	161.930	-0.008
1118	584753.793	121272.855	161.348	161.340	-0.008
1133	584788.533	121609.581	164.980	164.970	-0.010
1051	585410.866	130624.061	141.312	141.300	-0.012
1139	584802.055	121733.653	162.622	162.610	-0.012
1014	585289.664	129788.968	158.363	158.350	-0.013
1177	569628.397	142530.490	163.243	163.230	-0.013
1012	585282.180	129737.468	160.914	160.900	-0.014
1191	569848.162	142452.023	162.914	162.900	-0.014
1069	587494.380	132371.971	147.055	147.040	-0.015
1135	584792.679	121646.923	163.746	163.730	-0.016
1112	584735.214	121104.017	162.159	162.140	-0.019
1070	587495.528	132385.484	142.999	142.980	-0.019
1182	569706.150	142502.735	163.059	163.040	-0.019
1136	584794.917	121668.615	161.131	161.110	-0.021
1020	585311.009	129934.725	150.683	150.660	-0.023
1123	584763.534	121367.574	162.844	162.820	-0.024
1212	575830.018	145496.613	149.854	149.830	-0.024
1231	576097.714	145790.088	142.735	142.710	-0.025
1016	585297.757	129844.389	155.466	155.440	-0.026
1179	569658.787	142519.618	163.147	163.120	-0.027
1121	584758.670	121323.442	163.868	163.840	-0.028
1195	569908.362	142430.497	162.828	162.800	-0.028
1072	587501.372	132428.969	147.999	147.970	-0.029
1082	587527.493	132623.357	145.660	145.630	-0.030
1099	587567.080	132924.839	149.243	149.210	-0.033
1173	569566.925	142552.223	163.494	163.460	-0.034
1193	569878.215	142441.384	162.844	162.810	-0.034
1178	569643.461	142525.105	163.186	163.150	-0.036
1180	569674.172	142514.127	163.112	163.070	-0.042
1132	584786.451	121589.690	164.235	164.190	-0.045

Number	Easting	Northing	Known Z	Laser Z	Dz
1196	569923.688	142425.157	162.775	162.730	-0.045
1001	585248.956	129507.996	161.826	161.780	-0.046
1197	569938.539	142419.792	162.706	162.660	-0.046
1188	569801.879	142468.503	162.889	162.840	-0.049
1120	584756.750	121307.326	163.880	163.830	-0.050
1194	569893.434	142436.026	162.820	162.770	-0.050
1181	569689.956	142508.539	163.101	163.050	-0.051
1168	569487.341	142580.612	163.714	163.660	-0.054
1198	569953.261	142414.648	162.647	162.590	-0.057
1161	569375.906	142620.454	164.080	164.020	-0.060
1163	569406.553	142609.476	163.960	163.900	-0.060
1185	569752.675	142486.048	162.983	162.920	-0.063
1183	569722.146	142497.138	163.018	162.950	-0.068
1186	569770.259	142479.803	162.948	162.880	-0.068
1174	569581.657	142547.040	163.420	163.350	-0.070
1184	569736.533	142491.981	162.964	162.890	-0.074
1164	569425.572	142602.690	163.896	163.820	-0.076
1172	569550.840	142558.164	163.559	163.480	-0.079
1158	569330.482	142636.593	164.052	163.970	-0.082
1190	569831.975	142457.913	162.892	162.810	-0.082
1153	569246.648	142666.541	164.233	164.150	-0.083
1151	569216.008	142677.606	164.225	164.140	-0.085
1150	569200.706	142683.108	164.215	164.130	-0.085
1176	569612.442	142536.096	163.326	163.240	-0.086
1148	569169.730	142694.183	164.237	164.150	-0.087
1165	569440.403	142597.353	163.857	163.770	-0.087
1169	569502.527	142575.177	163.697	163.610	-0.087
1170	569518.665	142569.451	163.660	163.570	-0.090
1081	587523.830	132593.560	146.301	146.210	-0.091
1159	569345.229	142631.411	164.051	163.960	-0.091
1167	569471.549	142586.268	163.791	163.700	-0.091
1171	569534.668	142563.805	163.611	163.520	-0.091
1160	569360.614	142625.943	164.082	163.990	-0.092
1156	569300.130	142647.349	164.125	164.030	-0.095
1146	569138.005	142705.530	164.279	164.180	-0.099
1143	569092.588	142721.835	164.329	164.230	-0.099
1189	569816.997	142463.222	162.890	162.790	-0.100
1154	569262.345	142660.946	164.172	164.070	-0.102
1157	569314.751	142642.178	164.092	163.990	-0.102
1055	587467.086	132172.439	149.014	148.910	-0.104
1162	569391.019	142614.984	164.016	163.910	-0.106
1166	569456.249	142591.751	163.830	163.720	-0.110
1149	569185.447	142688.599	164.214	164.100	-0.114

AERO-METRIC

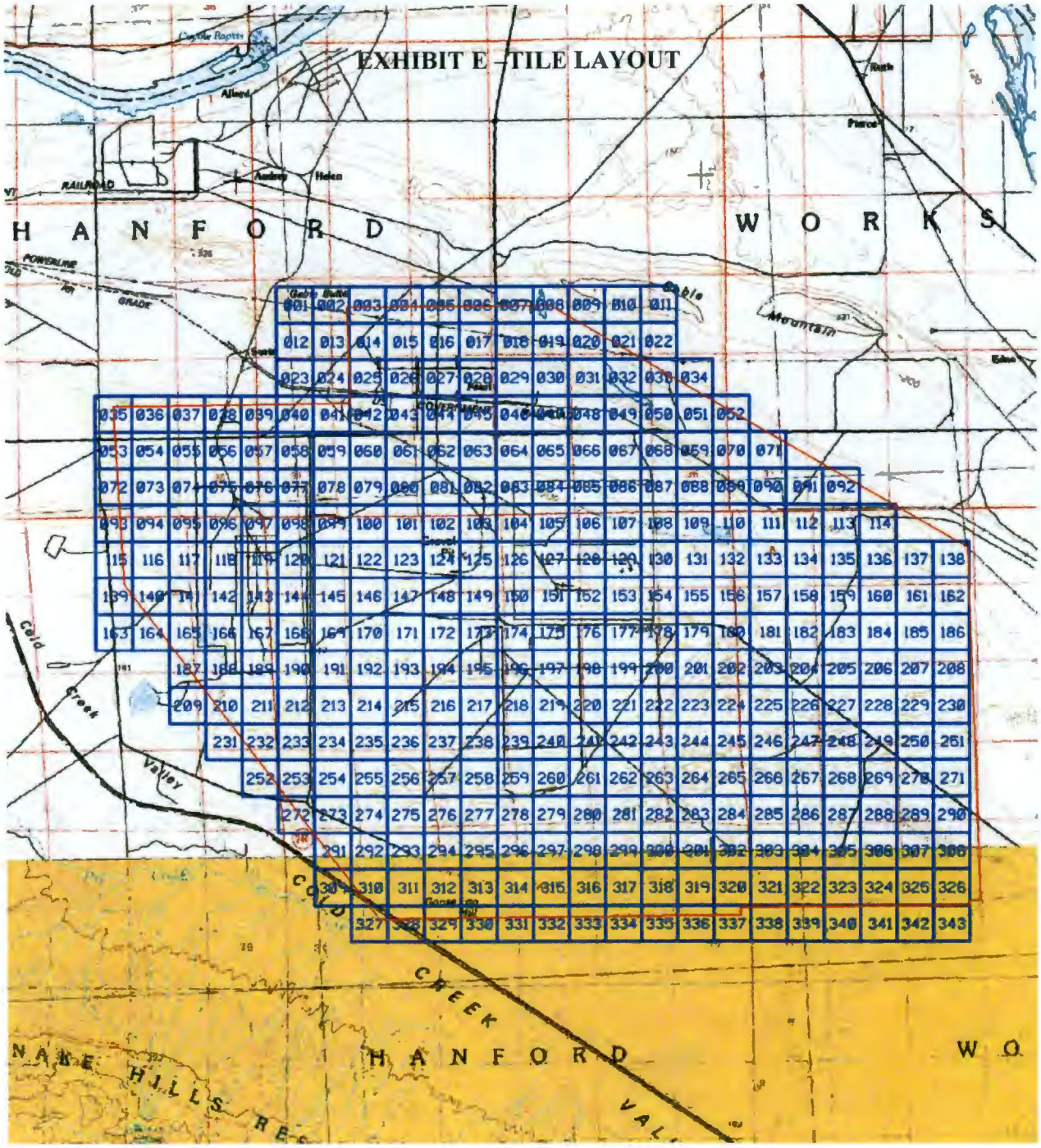
SEATTLE

Number	Easting	Northing	Known Z	Laser Z	Dz
1145	569123.233	142710.848	164.295	164.180	-0.115
1085	587532.544	132661.711	148.386	148.270	-0.116
1175	569596.356	142541.784	163.387	163.270	-0.117
1144	569108.301	142716.259	164.329	164.210	-0.119
1187	569786.206	142474.105	162.944	162.820	-0.124
1147	569153.788	142699.953	164.268	164.140	-0.128
1152	569230.707	142672.242	164.238	164.100	-0.138
1155	569282.224	142653.724	164.129	163.990	-0.139
1059	587473.812	132219.970	148.368	148.180	-0.188

Average dz	+0.011
Minimum dz	-0.188
Maximum dz	+0.482
Average magnitude	0.051
Root mean square	0.072
Std deviation	0.071

AERO-METRIC

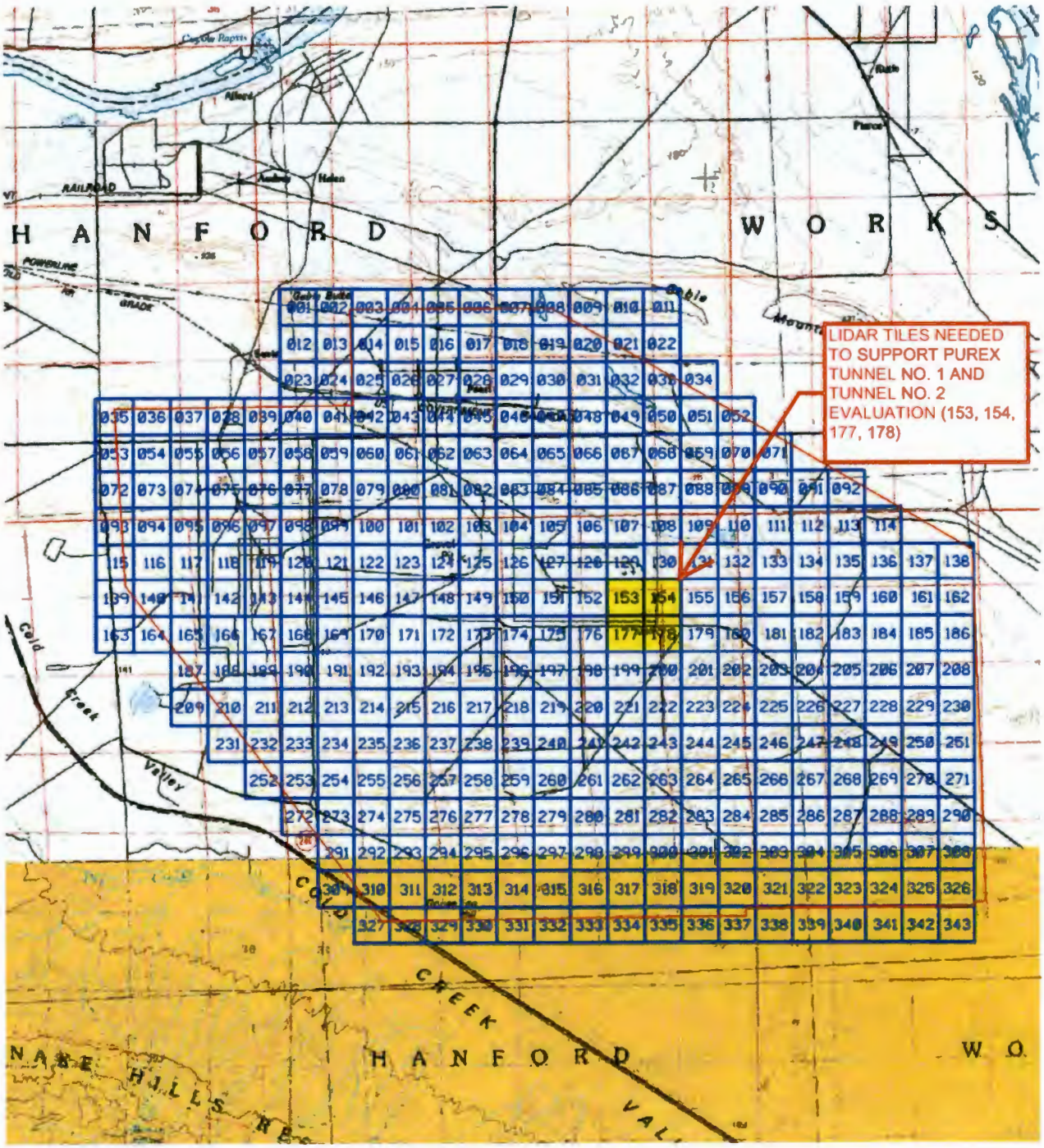
SEATTLE



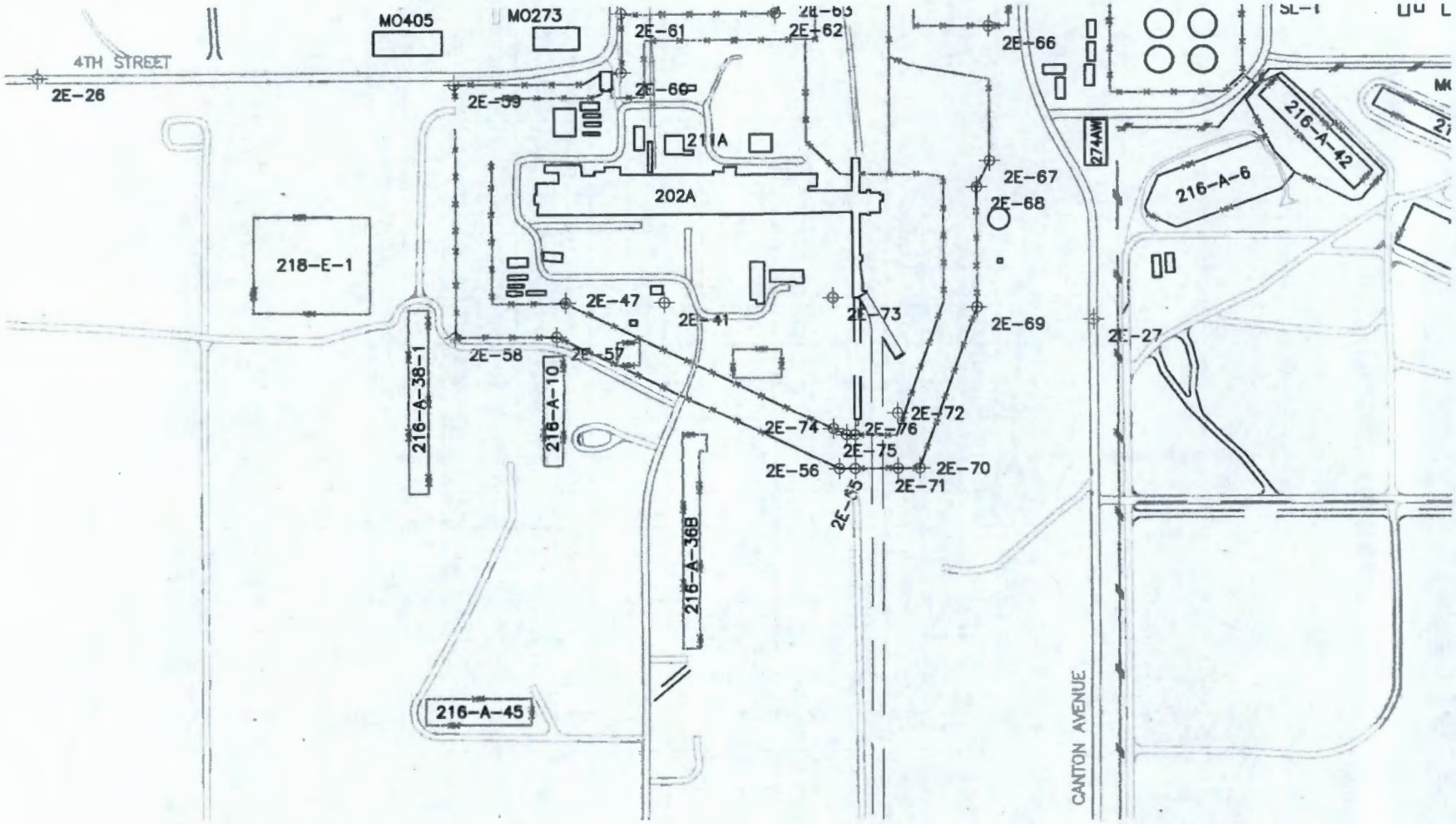
- **Number of Tiles within project area: 343**
- **Tile size: 750m**

AERO-METRIC
SEATTLE

EXHIBIT E –TILE LAYOUT



- **Number of Tiles within project area: 343**
- **Tile size: 750m**


4TH STREET
2E-26
M0405
M0273
2E-61
2E-62
2E-66
2E-59
2E-60
211A
202A
274AW
216-A-42
216-A-6
2E-67
2E-68
218-E-1
2E-47
2E-41
2E-73
2E-69
2E-27
2E-58
2E-57
216-A-38-1
216-A-10
2E-72
2E-74
2E-76
2E-75
2E-56
2E-70
2E-71
2E-55
216-A-36B
216-A-45
CANTON AVENUE

SURVEY MONUMENT DATA

STATION: 2E-41			LOCATION CODE: 200E	
HORIZONTAL			VERTICAL	
DATUM	COORDINATES		DATUM	HEIGHTS
	LATITUDE	LONGITUDE		ELLIPSOIDAL
NAD83(1991)	46°32'54.878454"	119°31'15.333733"	NAD83(1991)	196.737 m
	NORTHING	EASTING		ORTHOMETRIC
WCS83S(1991)	135,539.989 m	575,080.970 m	NAVD88	218.115 m
200E AREA (PUREX CONTROL)	N	W		ORTHOMETRIC
	39,557.00	48,600.00	NGVD29	712.216 usft
				ORTHOMETRIC
			200E (STAMPED)	711.99

DESCRIPTION

BRASS CAP MONUMENT ON SOUTH SIDE PUREX BLDG 202-A, INSIDE PUREX YARD FENCE. SOUTHEAST OF PIPEFITTERS SHOP, SOUTH OF STEAMLINE.

HORIZONTAL CONTROL

DATUM	INST./METHOD	MONUMENTS USED	LENGTH	MISCLOSURE	ADJUST
NAD 83/91	GPS/NETWORK	USACE GPS NET			TRIMNET
WCS83S/91	"	"			"
200E	— HELD STAMPED VALUES FOR PUREX CONTROL —			NOT TIED TO 200E NETWORK	

VERTICAL CONTROL

DATUM	INST./METHOD	MONUMENTS USED	LENGTH	MISC. (mm/√k)	ADJUST
NAVD88	NA-2/ 3-WIRE	USACE VERTICAL NET			TRIMNET
NGVD29	"	"			"

REFERENCE

DATUM	FILE NO.	REQ. NO.	BERNOULLI	OTHER
NAD 83/91	2ESE-087	962-108		
200E (H&V)	HELD STAMPED VALUES FOR PUREX CONTROL			
NAVD 88	2ESE-087, 2EXX-001	971-032	PROJ 1/74 200E 88	L-272
NGVD29	"	"	" 200E 29	"

ch2m	Calculation Title: PUREX Tunnel 1 Engineering Evaluation	Project Number: 693839.BS Project: PUREX Tunnel 1 Client: CHPRC

Attachment C

Earth Pressure Recommendations

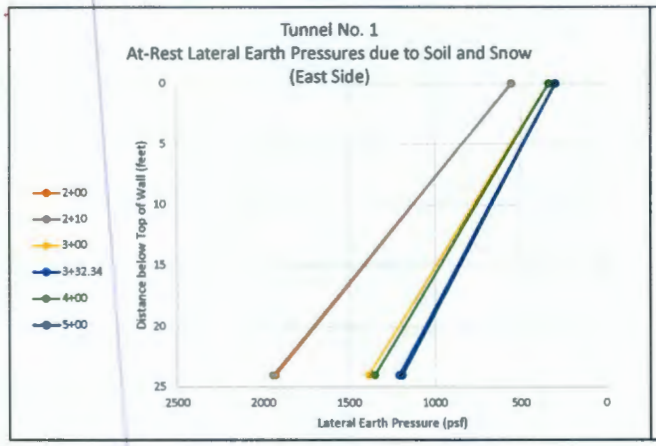
Tunnel No. 1
Lateral Earth Pressures and Roof Loads

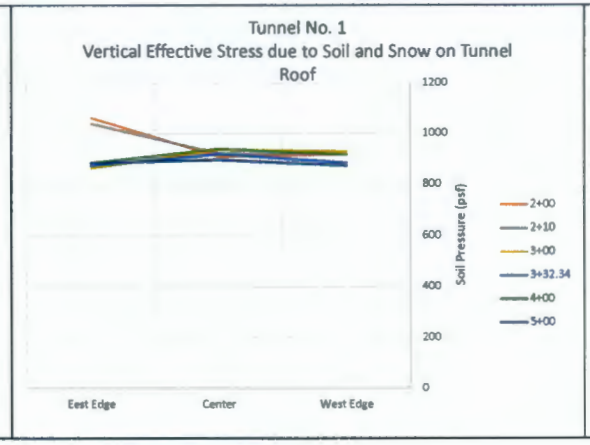
Prepared by:	M. Kacmarcik	6/12/2017
Checked by:	Y. Bougataya	6/13/2017

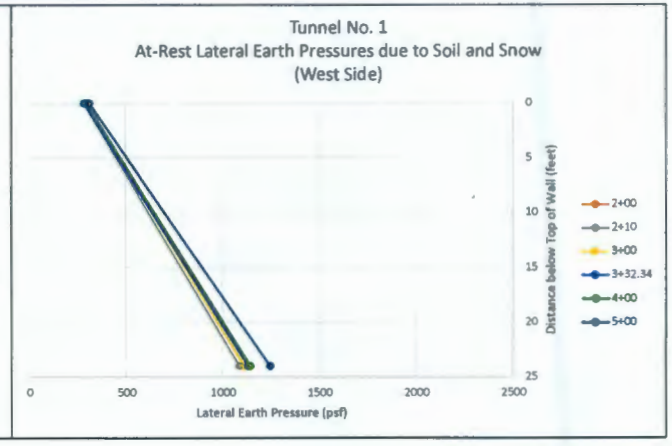
γ_{moist}	110	pcf	*Range is 105 to 115 pcf*
ϕ	30	deg	
c	0	psf	
K0	0.50	--	
SNOW	15	psf	

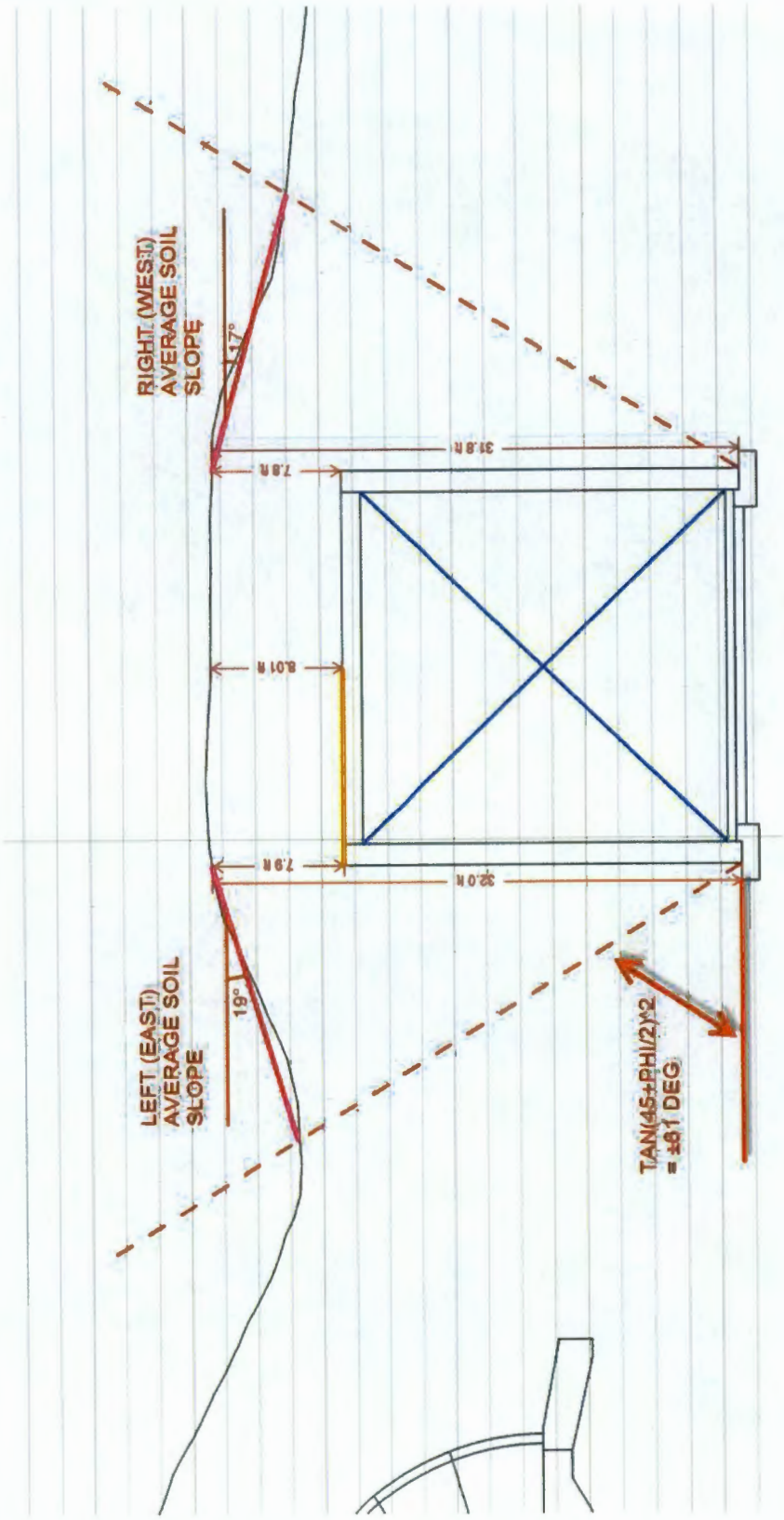
	LEFT (EAST) SIDE								ROOF						RIGHT (WEST) SIDE							
			SOIL + SNOW				SOIL + SNOW		SOIL HEIGHT (FEET)			VERTICAL SOIL + SNOW LOAD (PSF)					SOIL + SNOW				SOIL + SNOW	
STA	Soil Height Over Footing (feet)	Average Soil Slope (deg)	Vertical Effective Stress at Wall Top (psf)	Vertical Effective Stress at Wall Bot (psf)	Inclined At-Rest Coefficient (Koi)	Adjusted Equivalent Fluid Unit Weight (pcf/LF)	At-Rest Lateral Earth Pressure at Wall Top (psf)	At-Rest Lateral Earth Pressure at Wall Bot (psf)	East Edge	Center	West Edge	East Edge	Center	West Edge	Soil Height Over Footing (feet)	Average Soil Slope (deg)	Vertical Effective Stress at Wall Top (psf)	Vertical Effective Stress at Wall Bot (psf)	Inclined At-Rest Coefficient (Koi)	Adjusted Equivalent Fluid Unit Weight (pcf/LF)	At-Rest Lateral Earth Pressure at Wall Top (psf)	At-Rest Lateral Earth Pressure at Wall Bot (psf)
1+45	*33.5*	*0*	*1104*	*3700*	*0.30*	*55*	*552.0*	*1850.0*	9.9	9.8	5.6	1104	1096.3	631	29.6	-25	631	3271	0.29	32	182.2	944.3
2+00	***33.2***	***3***	***1060***	***3667***	***0.53***	***58***	***557.7***	***1929.5***	9.5	8.1	8.2	1060	908.2	917	32.2	-21	917	3557	0.32	35	294.2	1141.1
2+10	*32.9*	*6*	*1088*	*3634*	*0.53*	*59*	*555.2*	*1843.7*	9.3	8.2	8.2	1038	921.4	917	32.2	-23	917	3557	0.30	34	279.3	1083.6
3+00	31.7	-12	862	3502	0.40	44	341.4	1386.9	7.7	8.4	8.3	862	935.7	928	32.2	-22	928	3557	0.31	34	290.2	1112.3
3+32.34	31.7	-18	873	3502	0.35	38	301.6	1209.9	7.8	8.2	7.9	873	919.2	884	31.9	-21	884	3524	0.32	35	283.6	1130.6
4+00	32.0	-13.6	884	3535	0.38	42	338.1	1351.9	7.9	8.4	8.2	884	939	917	32.2	-21	917	3557	0.32	35	294.2	1141.1
5+00	32.0	-19	884	3535	0.34	37	298.1	1192.1	7.9	8.0	7.8	884	896.1	873	31.8	-17	873	3513	0.35	39	308.9	1242.9

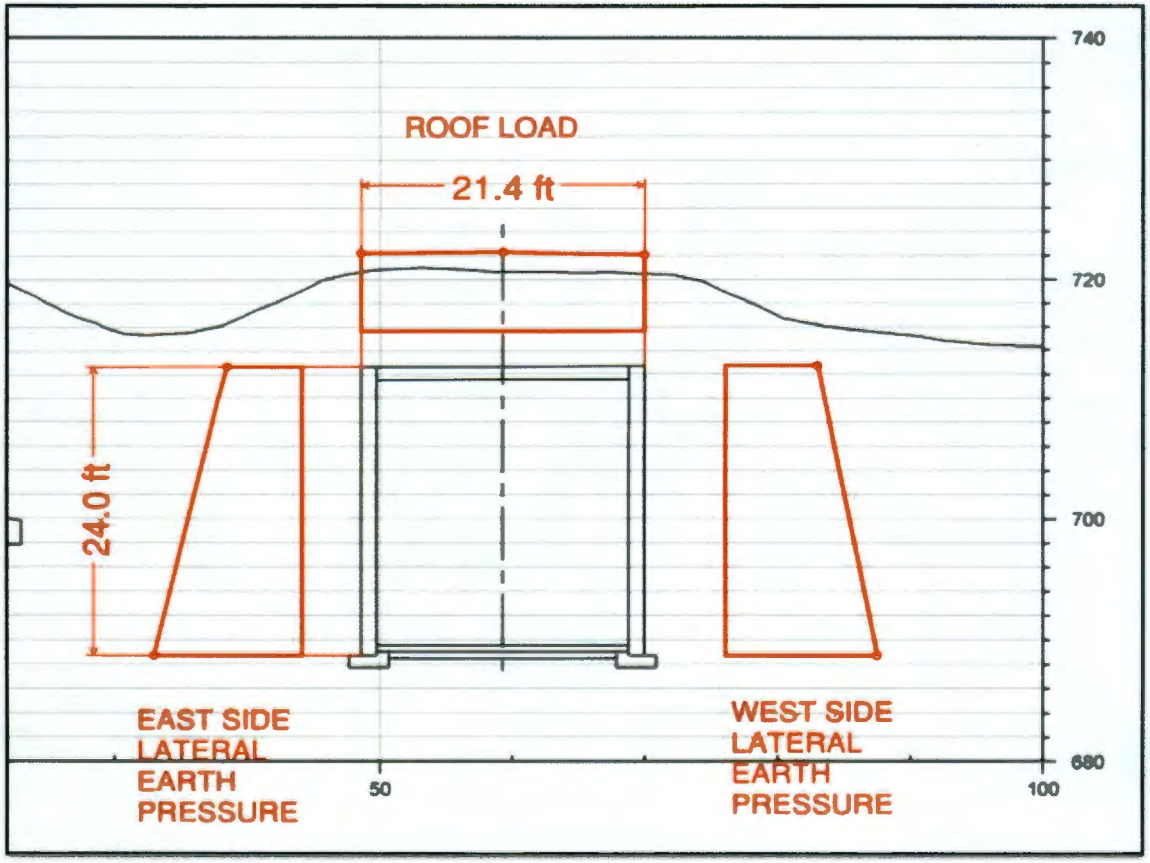
NOTES:
Earth pressure values reported for left (east) side, STA 1+45, 2+00, and 2+10, shown in *red italics*, act against concrete wall. All other earth pressures act against wood timber wall.
Soil pressure profile at STA 1+45 is approximate at edge of LIDAR topography at Tunnel No. 1 Water Filled Door. Not included in summary plots due to uncertain topography.
At-Rest Lateral earth pressure coefficients are adjusted for sloping ground surface at each sections following recommendations of Brooker and Ireland.
STA 2+10 is located at the May 9, 2017 breach of Tunnel No. 1

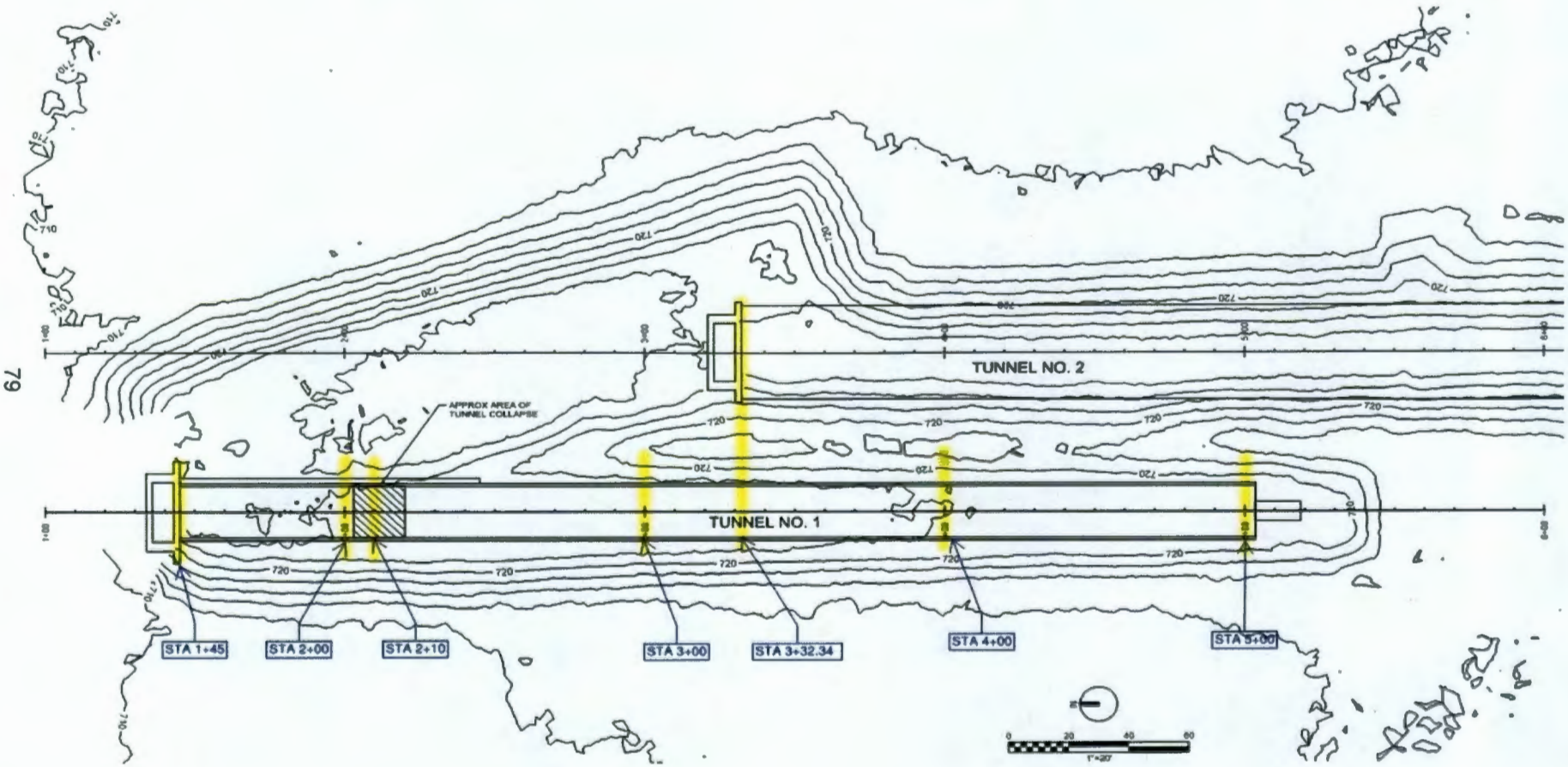







RIGHT (WEST)
AVERAGE SOIL
SLOPE
17°
31.8 ft
7.9 ft
8.01 ft
7.9 ft
32.0 ft
LEFT (EAST)
AVERAGE SOIL
SLOPE
19°
TAN(45+PHI/2)^2
= ±8.1 DEG


ROOF LOAD
21.4 ft
24.0 ft
EAST SIDE
LATERAL
EARTH
PRESSURE
WEST SIDE
LATERAL
EARTH
PRESSURE
740
720
700
680
50
100


TUNNEL NO. 2
TUNNEL NO. 1
APPROX AREA OF TUNNEL COLLAPSE
STA 1+45
STA 2+00
STA 2+10
STA 3+00
STA 3+32.34
STA 4+00
STA 5+00
720
710
0
20
40
60
1"=20'

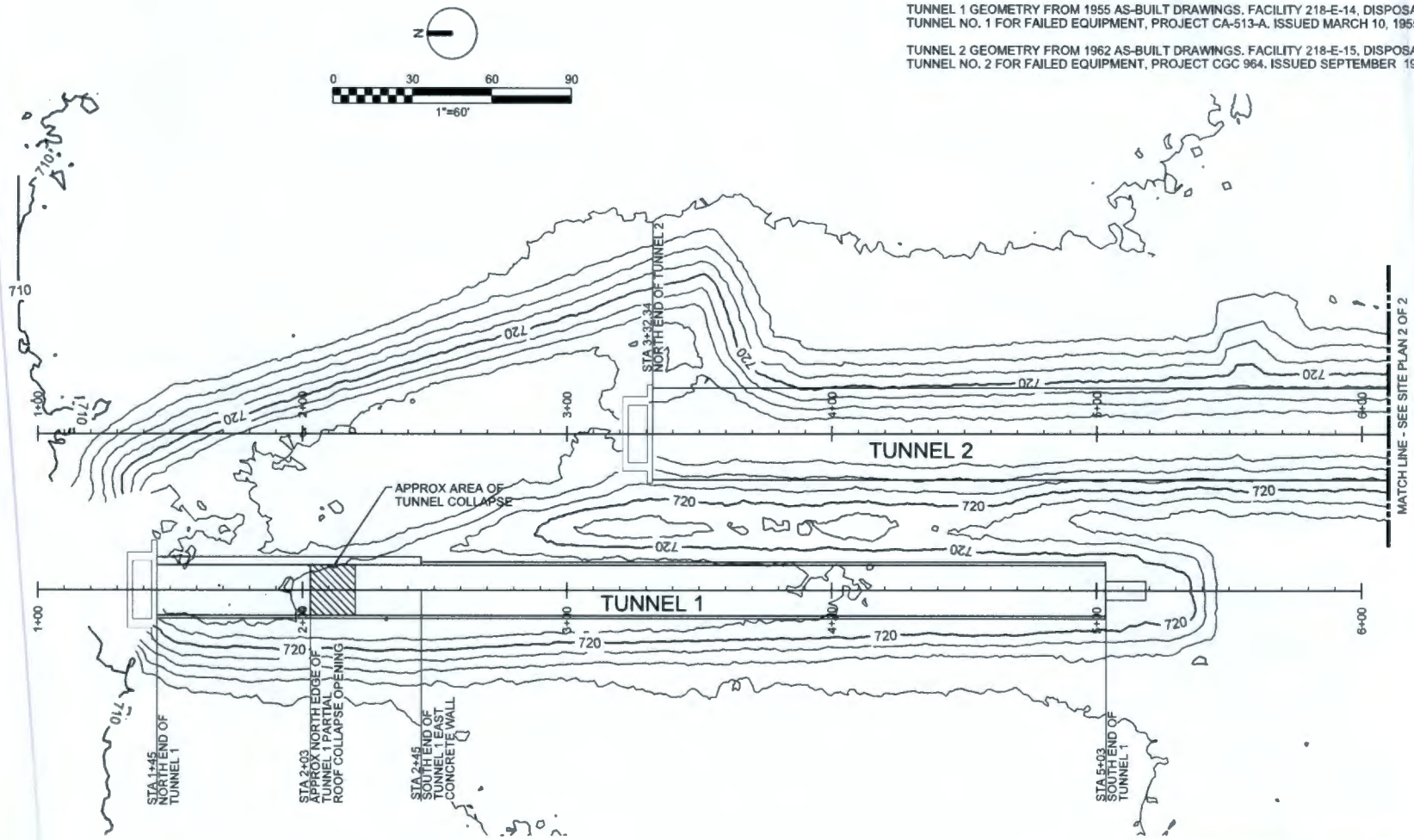
TOPOGRAPHY FROM 2008 *LIDAR CENTRAL PLATEAU REPORT* PREPARED BY AERO-METRIC FOR FLUOR HANFORD. DATA ACQUISITION APRIL 13, 2008. HORIZONTAL DATUM: STATE PLANE NAD83, WASHINGTON SOUTH ZONE, METERS. VERTICAL DATUM: NAVD88, METERS.

HORIZONTAL SYSTEM CONVERTED TO HANFORD SITE DATUM, AREA 200E (PUREX CONTROL), FEET, USING SURVEY MONUMENT COORDINATE DATA FOR MONUMENT 2E-41.

VERTICAL SYSTEM CONVERTED TO HANFORD SITE DATUM, AREA 200E (PUREX CONTROL), FEET, USING SURVEY MONUMENT COORDINATE DATA FOR MONUMENT 2E-41. VERTICAL DATUM SHIFT FROM LIDAR TOPOGRAPHY, -1.100 METERS.

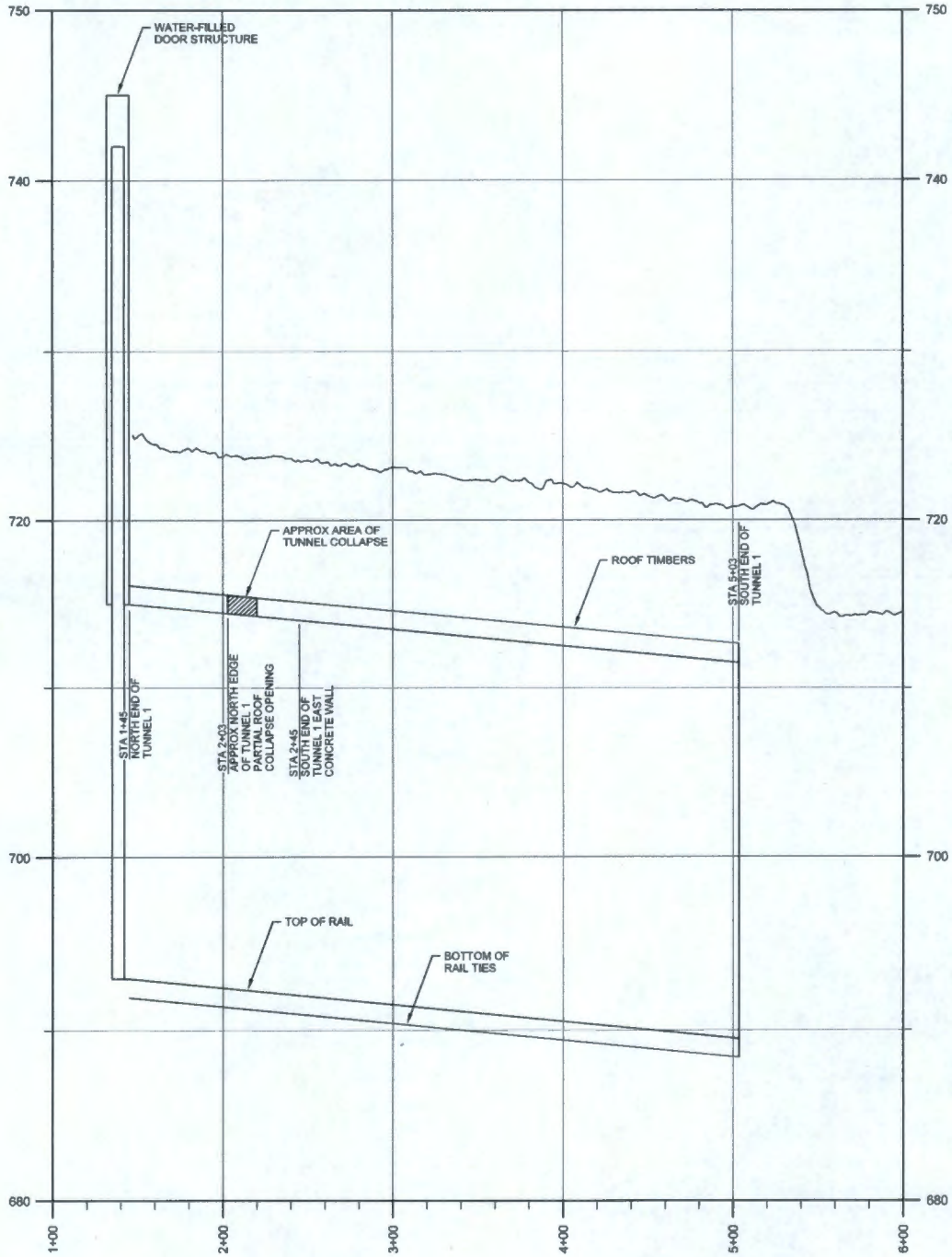
TUNNEL 1 GEOMETRY FROM 1955 AS-BUILT DRAWINGS. FACILITY 218-E-14, DISPOSAL TUNNEL NO. 1 FOR FAILED EQUIPMENT, PROJECT CA-513-A. ISSUED MARCH 10, 1955.

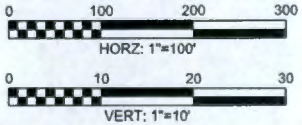
TUNNEL 2 GEOMETRY FROM 1962 AS-BUILT DRAWINGS. FACILITY 218-E-15, DISPOSAL TUNNEL NO. 2 FOR FAILED EQUIPMENT, PROJECT CGC 964. ISSUED SEPTEMBER 1962.



TOPOGRAPHIC SITE PLAN

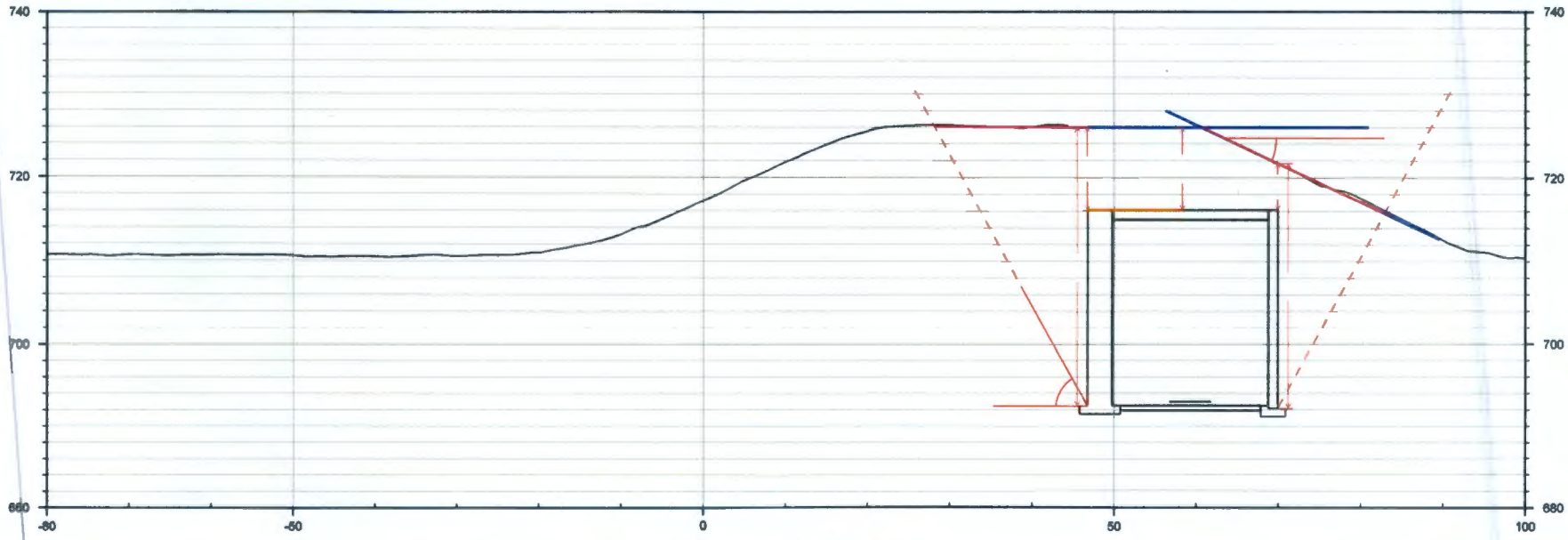
PUREX TUNNEL 1
ENGINEERING EVALUATION



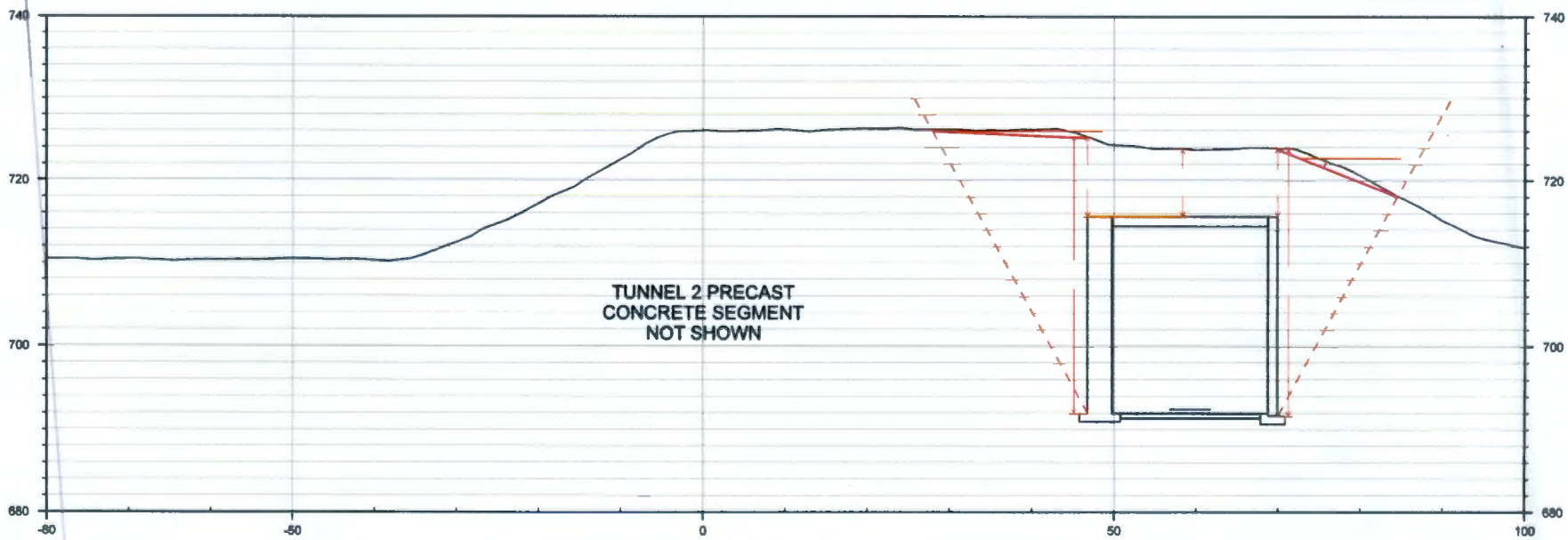


TUNNEL 1 PROFILE

PUREX TUNNEL 1
ENGINEERING EVALUATION

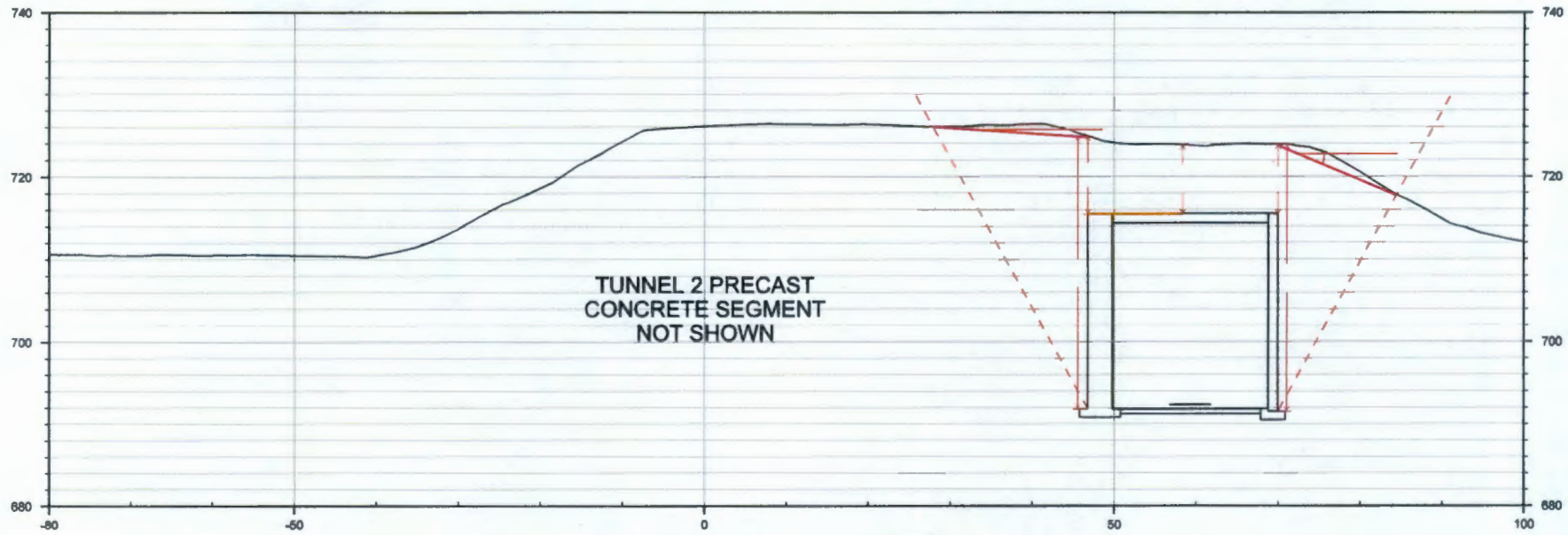


1+45 - NORTH END OF TUNNEL 1

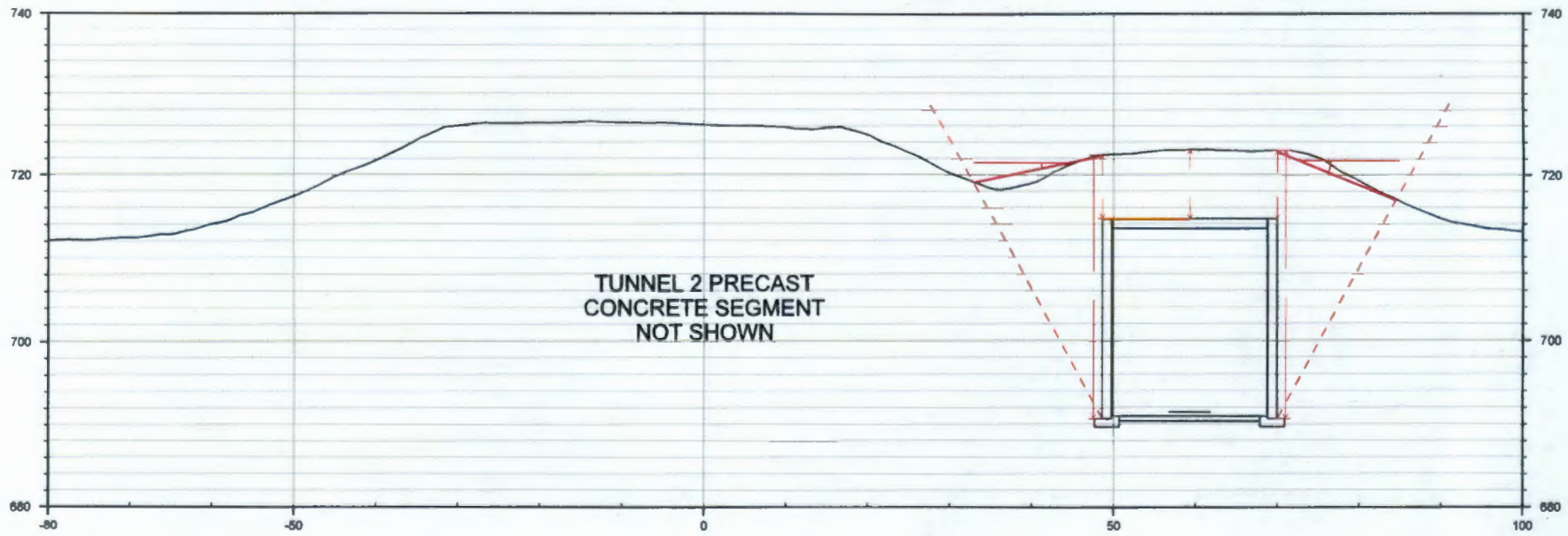


2+00

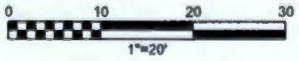
CROSS SECTIONS
1+45 AND 2+00
PUREX TUNNEL 1
ENGINEERING EVALUATION



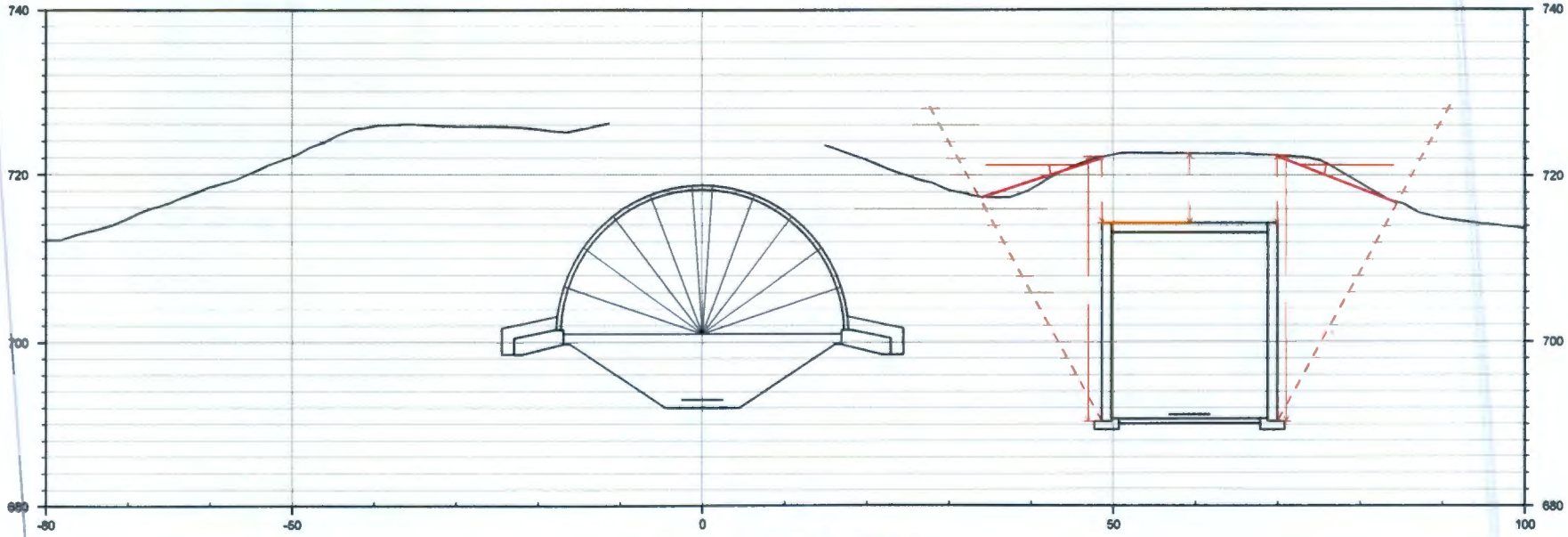
2+10 - TUNNEL 1 PARTIAL ROOF COLLAPSE OPENING



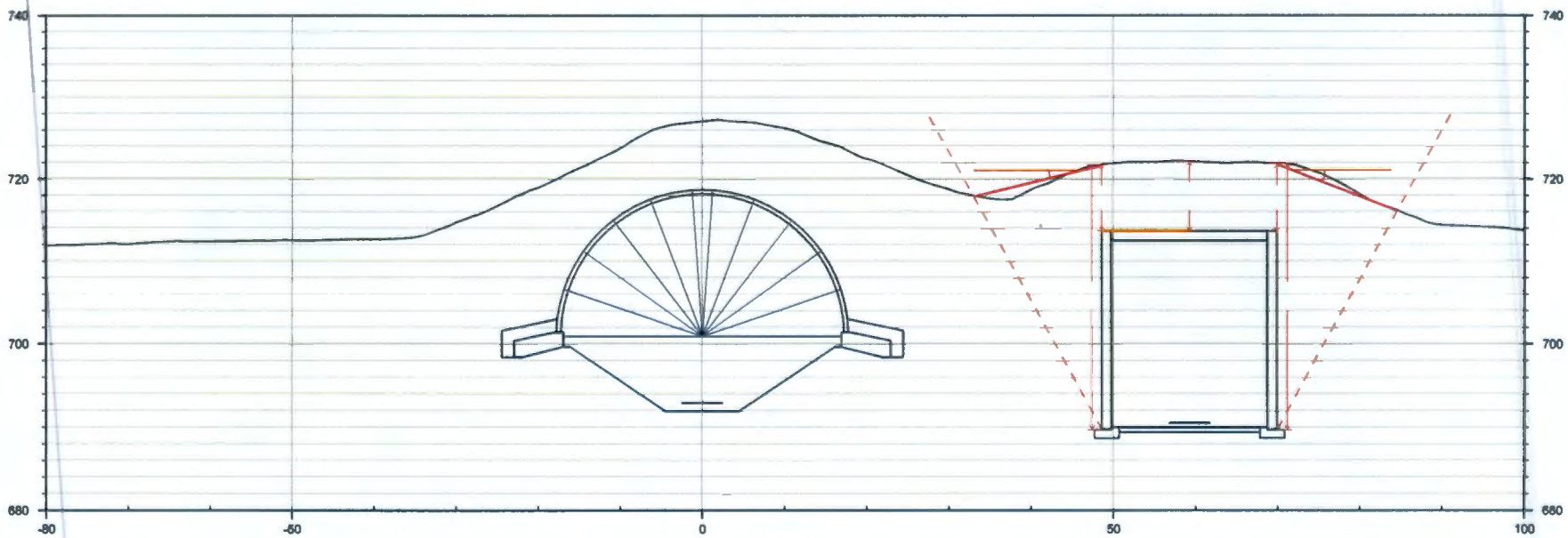
3+00



CROSS SECTIONS
2+10 AND 3+00
PUREX TUNNEL 1
ENGINEERING EVALUATION

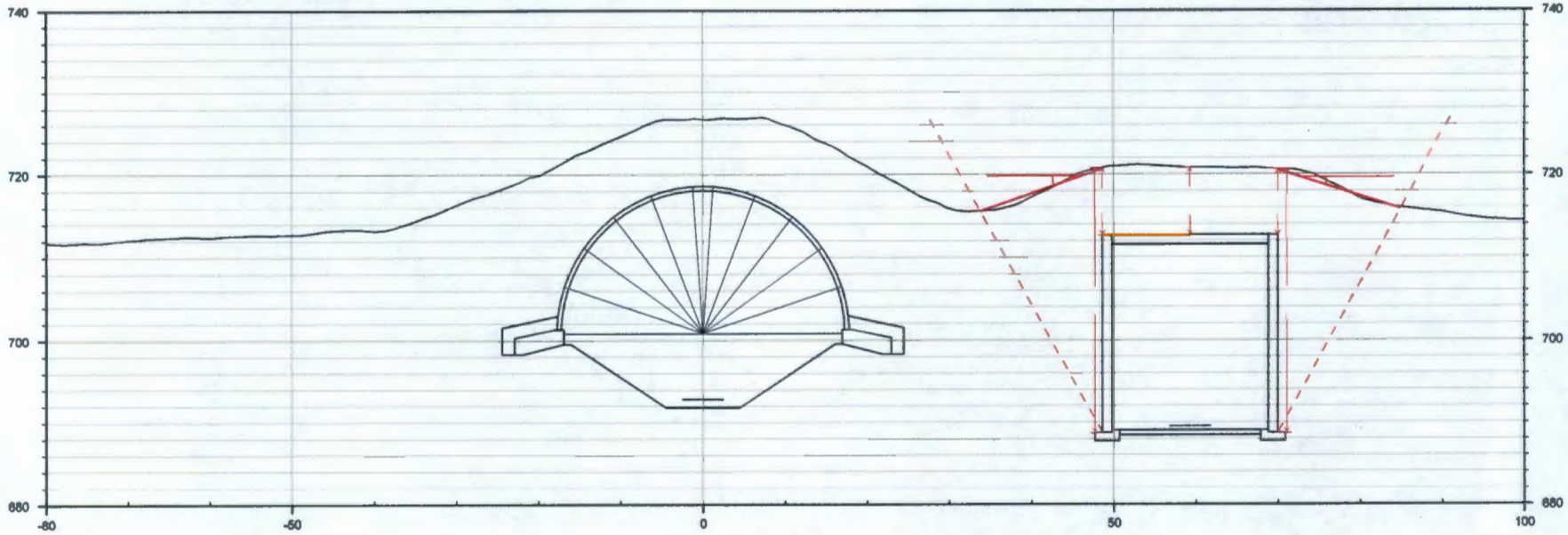


3+32.34 - NORTH END OF TUNNEL 2

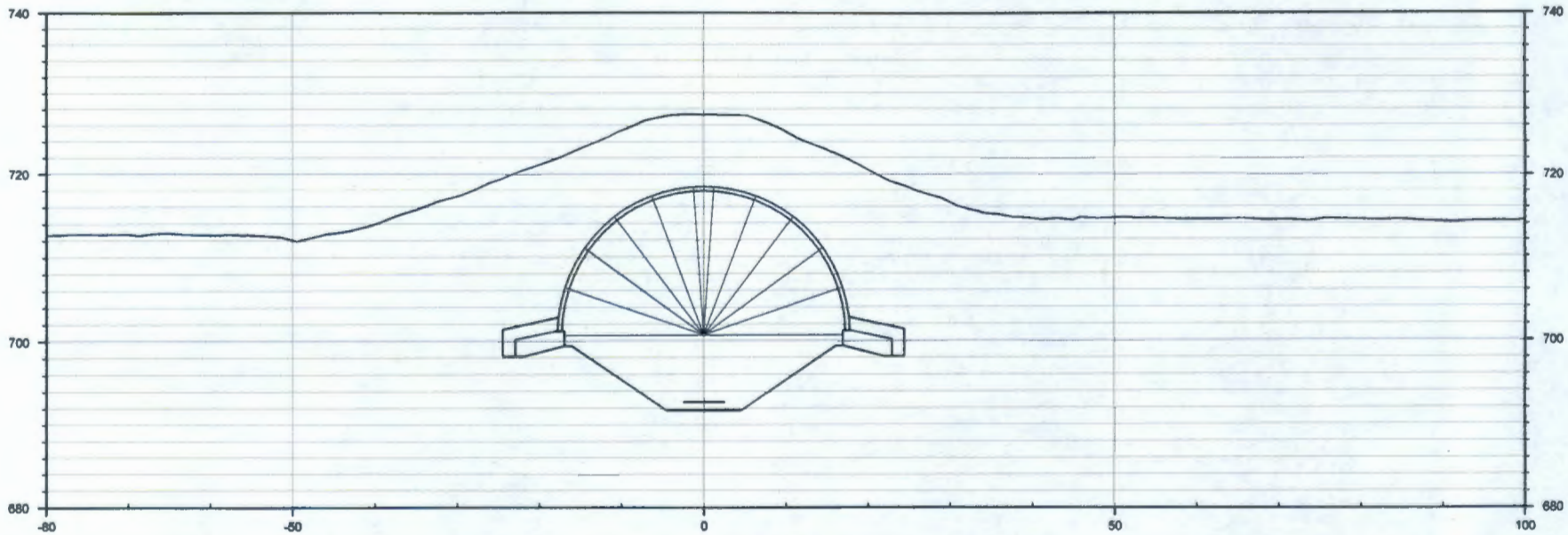


4+00

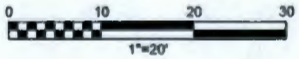
CROSS SECTIONS
3+32.34 AND 4+00
PUREX TUNNEL 1
ENGINEERING EVALUATION



5+00



6+00



CROSS SECTIONS
5+00 AND 6+00
PUREX TUNNEL 1
ENGINEERING EVALUATION

ch2m	Calculation Title: PUREX Tunnel 1 Engineering Evaluation	Project Number: 693839.BS Project: PUREX Tunnel 1 Client: CHPRC

Attachment D

Foundation Bearing Pressure Evaluation

Bearing Capacity Evaluation

ch2m

PROJECT: Hanford PUREX Tunnel 1 Evaluation
PROJECT #: 693839.BS
CREATED BY: Mark Kacmarcik/CVO **DATE:** 6/2/2017 **MODIFIED:** 6/23/2017
REVIEWED BY: Youssef Bougataya/CVO **DATE:** 6/12/2017

Given: Tunnel No. 1 Design Drawings
Tunnel No. 1 Construction Specification
Tunnel No. 1 Historical Photos

Find: Estimated Ultimate Bearing Capacity of Tunnel 1 Strip Footings
Allowable Bearing Capacity of Tunnel 1 Strip Footings

Assumptions: Footing geometry is as shown in 1955 as-built drawings for Tunnel 1
Footing subgrade is relatively undistrubed
Factor of Safety of 3.0 is appropriate for estimation of allowable bearing pressures
others as noted below in calculation.

References:

- Coduto, D. *Foundation Design.* Prentice Hall. New Jersey. 2000.
- McCarthy, D. 2002. Essentials of Soil Mechanics and Foundations. 6th Ed. Prentice Hall.
- Dames & Moore (1984) Report of Soils Investigation: Proposed Process Facility Modification Project, Purex Plant, Hanford Washington. Prepared for U.S. Department of Energy. 10805-136-05. Jan. 4.
- Shannon & Wilson (2015) Geotechnical Engineering Study (Rev 2) West Replacement Footings for 324 Building 'B' Cell Excavation, Hanford Reservation, Washington. Submitted to Kurion, Inc. 22-1-03078-010. Feb. 27.

1.0 Soil Properties

At this time, there is no site specific geotechnical data available. Soil properties are based uon the interpretation of construction photographs, and from a 1984 Dames & Moore report which provides foundation recommendations for a proposed Process Facility Modifications approximately 0.25 miles west of the PUREX storage tunnels, in similar subsurface materials.

Original Tunnel 1 Construction Photograph showing construction of Tunnel 1 footings

Subsurface Properties:

From construction photos, undisturbed earth appears to be dense fluvial/glaciofluvial deposits of the Hanford Formation as noted in Dames & Moore (1984). Excavation walls stand vertical to near vertical in many places, with some ravelling to angle of repose possibly due to drying of moist soil. There is no apparent shoring used. There is neither groundwater nor dewatering infrastructure visible, and groundwater is noted to be deep at the PUREX facility. Detail photos of footing subgrade are not available. It is assumed that the material beneath the footings is similar to the material visible at and above the footings. Properties assumed herein should be checked against available geologic literature and available geotechnical studies in similar materials, if available.

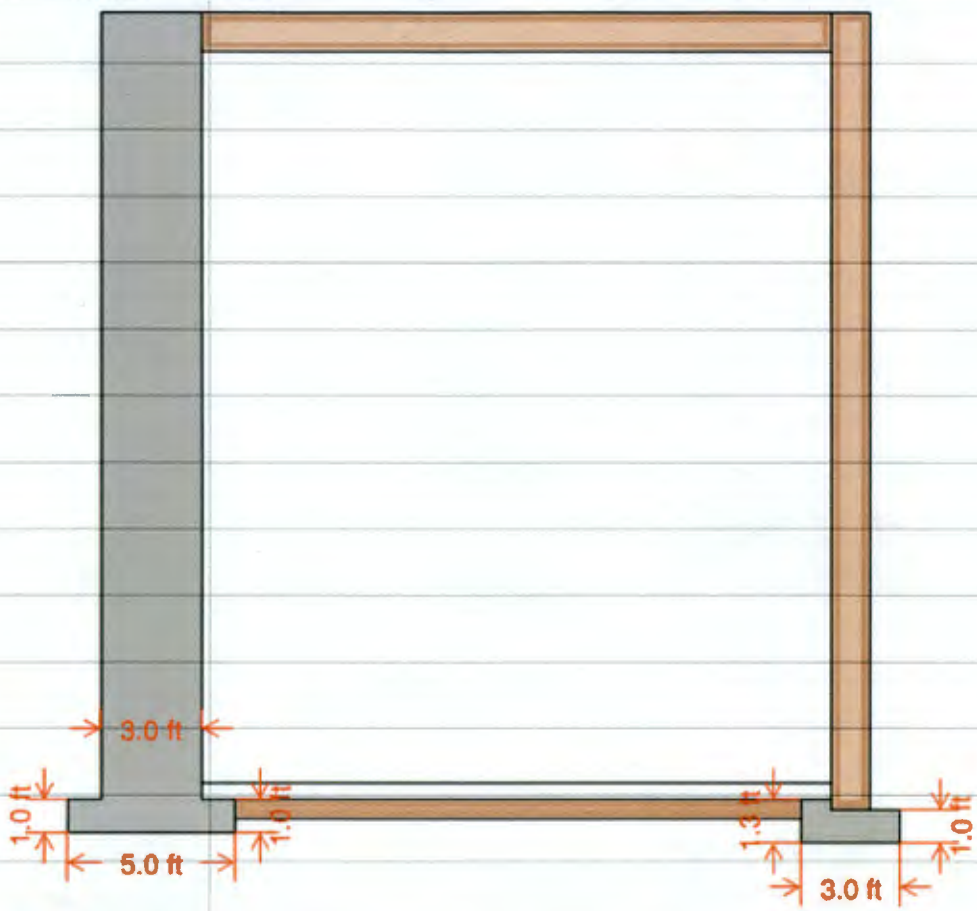
$\phi := 34deg$	Assumed internal friction angle of soils, from 1984 Dames & Moore Report
$c := 0psf$	Assumed effective stress cohesion, from 1984 Dames & Moore Report
$\gamma_{moist} := 125pcf$	Assumed average moist unit weight of in-situ soils, average value reported by Shannon and Wilson (2015).
$\gamma_{water} := 62.4pcf$	Unit weight of water, assumed.

2.0 Subsurface Profile

The subsurface profile is assumed to be uniform within the depths of stress influence beneath the footings. Groundwater is assumed to be significantly deeper than the footings, and total stresses are equal to effective stresses.

3.0 Footing Geometry

The footings for Tunnel 1 consist of two parallel strip footings.
Footings beneath the concrete wall (100 feet long) are 5 feet wide and 1 foot below grade in tunnel.
Footings beneath the timber walls are 3 feet wide and 1.3125 feet below grade in tunnel.



$\alpha := 0deg$	Footing inclination (0 indicates horizontal footings)
$B_{conc} := 5ft$	Width of footing beneath concrete wall
$B_{timber} := 3ft$	Width of footings beneath timber walls
$L_{conc} := 100ft$	Footing length, beneath concrete wall
$L_{timber} := 358ft$	Footing length, beneath timber wall

$\beta := 0deg$ Transverse slope inside tunnel is horizontal

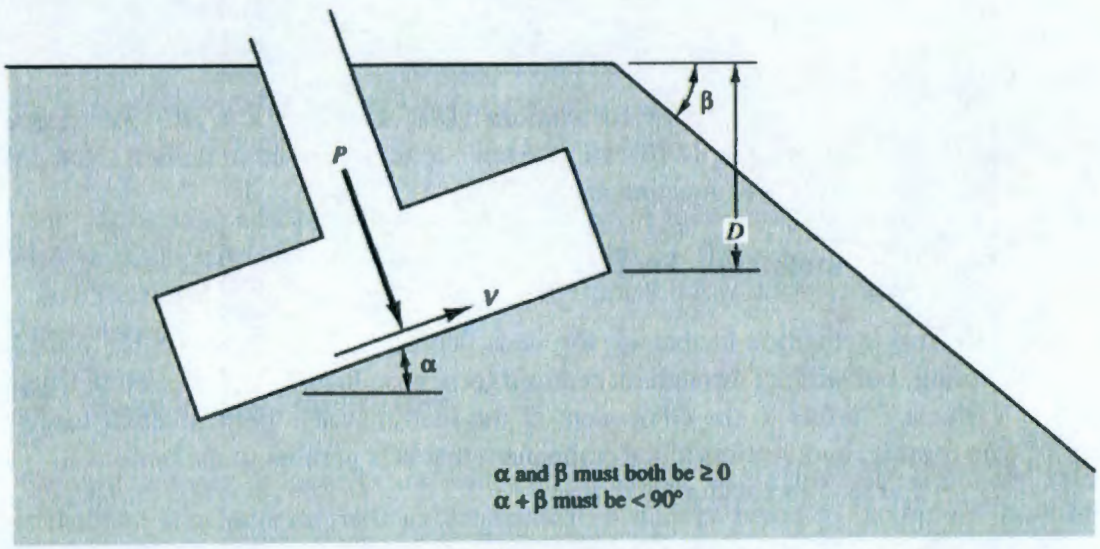
$D_{conc} := 1.0ft$ Footing depth below concrete wall

$D_{timber} := 1ft + 3.75in = 1.313\ ft$ Footing depth below timber walls

$\sigma_{zD.conc} := D_{conc} \cdot \gamma_{moist} = 125 \cdot psf$ In-situ vertical effective stress at depth, D

$\sigma_{zD.timber} := D_{timber} \cdot \gamma_{moist} = 164 \cdot psf$ In-situ vertical effective stress at depth, D

6.2 Estimate Bearing Capacity Using Vesic's Bearing Capacity Formula:



Footing Parameters for Vesic's Bearing Capacity Formula (adapted from Coduto (2001) Figure 6.8).

$$q_{ult} := c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + \sigma_{zD}(D_{ftg}) \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma$$

where: s_c, s_q, s_γ = shape factors
d_c, d_q, d_γ = depth factors
i_c, i_q, i_γ = load inclination factors
b_c, b_q, b_γ = base inclination factors
g_c, g_q, g_γ = ground inclination factors

Bearing Capacity Factors:

$$N_q := e^{\pi \tan(\phi)} \tan\left(45 \cdot deg + \frac{\phi}{2}\right)^2 \qquad N_q = 29.44$$

$$N_c := \begin{cases} 5.14 & \text{if } \phi = 0 \\ \dfrac{N_q - 1}{\tan(\phi)} & \text{otherwise} \end{cases}$$

$N_c = 42.164$

$$N_\gamma := 2\left(N_q + 1\right)\tan(\phi)$$

$N_\gamma = 41.064$

Footing Shape Factors

$$s_c(B, L) := 1.0 + \left(\frac{N_q}{N_c}\right)\left(\frac{B}{L}\right)$$

$s_{c.conc} := s_c\left(B_{conc}, L_{conc}\right) = 1.035$

$s_{c.timber} := s_c\left(B_{timber}, L_{timber}\right) = 1.006$

$$s_q(B, L) := 1.0 + \left(\frac{B}{L}\right)\cdot\tan(\phi)$$

$s_{q.conc} := s_q\left(B_{conc}, L_{conc}\right) = 1.034$

$s_{q.timber} := s_q\left(B_{timber}, L_{timber}\right) = 1.006$

$$s_\gamma(B, L) := 1.0 - 0.4\cdot\left(\frac{B}{L}\right)$$

$s_{\gamma.conc} := s_\gamma\left(B_{conc}, L_{conc}\right) = 0.98$

$s_{\gamma.timber} := s_\gamma\left(B_{timber}, L_{timber}\right) = 0.997$

Depth Factors:

$$k_{conc} := \frac{D_{conc}}{B_{conc}} = 0.2 \qquad k_{timber} := \frac{D_{timber}}{B_{timber}} = 0.437$$

$$d_c(k) := 1 + 0.4\cdot k$$

$d_{c.conc} := d_c\left(k_{conc}\right) = 1.08$

$d_{c.timber} := d_c\left(k_{timber}\right) = 1.175$

$$d_q(k) := 1 + 2\cdot k\cdot\tan(\phi)\cdot(1 - \sin(\phi))^2$$

$d_{q.conc} := d_q\left(k_{conc}\right) = 1.052$

$d_{q.timber} := d_q\left(k_{timber}\right) = 1.115$

$d_\gamma := 1.00$ for all ϕ

$d_{\gamma.conc} := d_\gamma$

$d_{\gamma.timber} := d_\gamma$

Load Inclination Factors:

$Q := 10lbf$ — Total footing load (magnitude is irrelevant, only the ratio of P to V is necessary for evaluation of *m*)

$P := Q$ — Component of load that acts perpendicular to bottom of footing.

$V := 0lbf$ — Component of load that acts parallel to bottom of footing.

$A_{conc} := B_{conc} \cdot L_{conc} = 500\ ft^2$ — Area of footing beneath concrete wall

$A_{timber} := B_{timber} \cdot L_{timber} = 1074\ ft^2$ — Area of footing beneath timber wall

$$m_{conc} := \frac{2 + \frac{B_{conc}}{L_{conc}}}{1 + \frac{B_{conc}}{L_{conc}}} = 1.952$$

For load inclination transverse to axis of tunnel

$$m_{timber} := \frac{2 + \frac{B_{timber}}{L_{timber}}}{1 + \frac{B_{timber}}{L_{timber}}} = 1.992$$

For load inclination transverse to axis of tunnel

$$i_q(A, m) := \left(1 - \frac{V}{P + \frac{A \cdot c}{\tan(\phi)}}\right)^m$$

$i_{q.conc} := i_q(A_{conc}, m_{conc}) = 1$

$i_{q.timber} := i_q(A_{timber}, m_{timber}) = 1$

$$i_\gamma(A, m) := \left(1 - \frac{V}{P + \frac{A \cdot c}{\tan(\phi)}}\right)^{m+1}$$

$i_{\gamma.conc} := i_\gamma(A_{conc}, m_{conc}) = 1$

$i_{\gamma.timber} := i_\gamma(A_{timber}, m_{timber}) = 1$

$$i_c(i_q) := i_q - \left(\frac{1 - i_q}{N_c \cdot \tan(\phi)}\right)$$

$i_{c.conc} := i_c(i_{q.conc}) = 1$

$i_{c.timber} := i_c(i_{q.timber}) = 1$

Ground Inclination Factors:

Vesic's factors for accounting for inclined ground surface

$\beta = 0 \cdot deg$

$g_c := 1 - \frac{\beta}{147deg}$ $\qquad g_c = 1$

$g_q := (1 - \tan(\beta))^2$ $\qquad g_q = 1$

$g_\gamma := g_q$ $\qquad g_\gamma = 1$

Base Inclination Factors:

Vesic's factors for inclined base of footing.

$\alpha = 0 \cdot deg$ Angle of inclination of base of footing.

$b_c := 1 - \frac{\alpha}{147deg}$ $\qquad b_c = 1$

$b_q := \left(1 - \frac{\alpha \cdot \tan(\phi)}{57deg}\right)^2$ $\qquad b_q = 1$

$b_\gamma := b_q$ $\qquad b_\gamma = 1$

Estimate Ultimate and Allowable Bearing Capacity Using Vesic's Extended Bearing Capacity Equation:

$$q_{ult.conc} := c \cdot N_c \cdot s_{c.conc} \cdot d_{c.conc} \cdot i_{c.conc} \cdot b_c \cdot g_c \cdots + \sigma_{zD.conc} \cdot N_q \cdot s_{q.conc} \cdot d_{q.conc} \cdot i_{q.conc} \cdot b_q \cdot g_q \cdots + 0.5 \cdot \gamma_{moist} \cdot B_{conc} \cdot N_\gamma \cdot s_{\gamma.conc} \cdot d_{\gamma.conc} \cdot i_{\gamma.conc} \cdot b_\gamma \cdot g_\gamma$$

$q_{ult.conc} = 16579 \cdot psf$

$$q_{ult.timber} := c \cdot N_c \cdot s_{c.timber} \cdot d_{c.timber} \cdot i_{c.timber} \cdot b_c \cdot g_c \cdots + \sigma_{zD.timber} \cdot N_q \cdot s_{q.timber} \cdot d_{q.timber} \cdot i_{q.timber} \cdot b_q \cdot g_q \cdots + 0.5 \cdot \gamma_{moist} \cdot B_{timber} \cdot N_\gamma \cdot s_{\gamma.timber} \cdot d_{\gamma.timber} \cdot i_{\gamma.timber} \cdot b_\gamma \cdot g_\gamma$$

$q_{ult.timber} = 13088 \cdot psf$

$FS := 3$ Factor of safety for use in estimating allowable bearing pressures. Typical value consistent with standard of practice for strip footings. Higher factors of safety may be justified considering the poorly-defined subsurface conditions and the consequences of failure

$q_{allow.conc} := \frac{q_{ult.conc}}{FS} = 5526 \cdot psf$ Allowable Bearing Capacity for Concrete Wall Footing

$q_{allow.timber} := \frac{q_{ult.timber}}{FS} = 4363 \cdot psf$ Allowable Bearing Capacity for Timber Wall Footing