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11. REMARKS

Structural steel gantry support, reinforced concrete, and gantry rail design by R. Hood, Attachments I, IV, and V. Concrete invert by F. Duan, Attachment II.

Attachment II, Concrete Invert checked by Yiming Sun.

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01	Main body of analysis completely revised to include the design loads of the Preliminary Waste Package Transport and Emplacement Equipment Design, DI: BCA000000-01717-0200-00012 REV 00 and of the Repository Ground Support Analysis for Viability Assessment, DI: BCAA00000-01717-0200-00004 REV 00. Attachment I, invert design for gantry support revised for design loads in the Preliminary Waste Package Transport and Emplacement Equipment Design, DI: BCA000000-01717-0200-00012 REV 00. Attachment II, invert design revised for design loads in the Repository Ground Support Analysis for Viability Assessment, DI: BCAA00000-01717-0200-00004 REV 00 and in the Preliminary Waste Package Transport And Equipment Design, DI: BCA00000-01717-0200-00012 REV 00. Attachment III, no changes. Added Attachment IV, Reinforced Concrete Design. Added Attachment V, Gantry Rail Design. Initial Issue

1. PURPOSE

The purpose of this analysis is to develop the design of the emplacement drift invert for the Underground Facility portion of the Engineered Barrier Segment (EBS) of the repository. The underground facility portion of the EBS consists of emplacement drift openings, emplacement drift backfill (if needed to enhance the EBS) and invert system.

The objective of this analysis is to support the Viability Assessment with an emplacement drift invert design compatible with the ground control system, the subsurface waste handling system, and the waste package support system.

The scope of this analysis covers design of the emplacement drift invert structure that provides support for the ground control system, the subsurface waste handling system, and the waste package support system. This analysis will evaluate concrete and steel materials for the invert structure and will determine the invert configuration, the structural properties, and the strengths of materials proposed for the invert. This analysis will include investigation of loadings from construction operations, ground control, waste package handling for emplacement and retrieval and for off normal conditions, and waste package support (pedestal and pier). This analysis will identify interfaces with ground control, waste handling, and waste package support systéms as shown in Reference 5.30. Developments in the design of the waste package emplacement equipment, power supply system, waste package support system, and in-drift monitoring will be incorporated. The impact of potential application of fill material (which may be added to enhance the EBS) and any future backfilling needed will be addressed.

The design of the emplacement drift openings, emplacement drift backfill, and waste package support pier assembly is not covered by this analysis.

2. QUALITY ASSURANCE

The EBS and the emplacement area of the ground control system are classified as QA-1 and QA-2, according to *Classification of the Preliminary MGDS Repository Design* (TBV-228) (Reference 5.5, page 17), therefore, the emplacement drift invert is considered quality affecting and subject to *Quality Assurance Requirements and Description (QARD)*, (Reference 5.11) requirements.

This design analysis activity has been evaluated in accordance with QAP-2-0, *Conduct of Activities*, and has been determined to be applicable to the requirements of the QARD (Reference 5.11). The outputs of this analysis are subject to QA controls in accordance with NLP-3-18, *Documentation of QA Controls on Drawings, Specifications, Design Analyses, and Technical Documents*.

The existing/unconfirmed input data used in this analysis are preliminary and unconfirmed and, therefore, the outputs require confirmation. Because of the preliminary nature of this analysis, the formal tracking system described in NLP-3-15, *To Be Verified (TBV) and To Be Determined (TBD) Monitoring System*, is not applicable. The conclusions from this design analysis cannot be used as input to documents supporting procurement, fabrication, or construction without further confirmation.

3. METHOD

The emplacement drift invert will be designed by hand calculations and computer analyses, considering rock, thermal, and seismic loads. Installation and operating loads and loads from potential backfilling materials are also considered in the invert design.

4. DESIGN INPUTS

4.1 DESIGN PARAMETERS

4.1.1 Category 1 rock mass mechanical properties used in this analysis are as follows:

Young's modulus: 7.76 GPa (Reference 5.21, Table 4, page 48)
Poisson's ratio: 0.21 (Reference 5.27, Table 7-6, page 7-13)
Cohesion: 1.5 MPa (Reference 5.21, Table 5, page 49)
Friction angle: 43° (Reference 5.21, Table 6, page 50)
Tensile Strength: 1.32 MPa for the Tsw2 unit (Reference 5.21, Table 8, page 52)

These rock mass mechanical properties are based on full peripheral mapping data obtained during the ESF construction. Compared to similar properties based on scanline mapping data, rock mass properties are generally lower, and therefore, are considered to be conservative for this analysis.

4.1.2 The remaining applicable design parameters used in this analysis are included in Section 4.3 as assumptions.

4.2 CRITERIA

The following design criteria, applicable to this analysis, were developed in response to requirements in the *Repository Design Requirements Document* (RDRD) (Reference 5.1), and to related requirements in the *Engineered Barrier Design Requirements Document* (EBDRD) (Reference 5.6).

4.2.1 The Repository Segment shall be designed so that facilities (inverts) are easily maintained. Maintainability considerations shall include the use of durable materials. These criteria are addressed (Section 7.2) by the selected use of concrete or steel components for the invert segment, which enhances durabilit and allows for ease of maintenance and/or replacement with cast-in-place concrete if required (EBDRD 3.2.5.2.8.A.1 and RDRD 3.2.5.2.8.A.1).

4.2.2 The Repository Segment shall be designed for a maintainable service life of at least 100 years (RDRD 3.2.5.4.A) which is exceeded by Assumption 4.3.2 which assumes a service life of at least 150 years for the EBS (CDA EBDRD 3.2.5.4). Both of these requirements equal or exceed Key 016 of Section 4.3.1. This criterion is addressed (Section 7.2) by the selected design of the invert segment using concrete or steel, which allows access for maintenance and/or replacement for the service life of the structure.

4.2.3 Geologic Repository Operations Area Systems, Structures, and Components important to safety shall be designed to accommodate natural phenomena such as earthquakes. This criterion is in the analysis (Section 7.3) following the methodology of Reference 5.2. The concrete invert is designed to withstand loads shown in Section 4.3.1, Key 064 (EBDRD 3.2.6.1.A and B and RDRD 3.2.6.1.A).

4.2.4 The design of structures shall include the effects of stresses and movements resulting from variations in temperature, including the effect of emplaced waste packages. This criterion is addressed (Sections 7.1.5 and 7.3) in the analysis and the concrete invert was designed to withstand these loads (EBDRD 3.2.6.1.D and RDRD 3.2.6.1.D).

4.2.5 The Repository Segment facilities shall be designed to incorporate the use of noncombustible and heat resistant materials. This criterion is addressed (Section 7.2) by specifying concrete or steel materials for the invert segment (EBDRD 3.2.6.2.2 and RDRD 3.2.6.2.2.D).

4.2.6 All designs shall comply with U.S. Nuclear Regulatory Commission direction supplemented by the criteria of DOE Order 6430.1A to the extent there is no conflict. The applicable criteria of DOE Order 6430.1A, i.e., Division 1 (General Requirements) Sections 0109 (Reference Standards and Guides), 0111-1 (General) 0111-2 (Loads), 0111-3 (Structural Systems for Buildings and Other Structures), 0111-99 (Special Facilities); Division 3 (Concrete), Sections 0320 (Concrete Reinforcement), 0340 (Precast Concrete); Division 5 (Metals) Sections 0512 (Buildings and Other Structures), 0532 Metal Fastening; and Division 13 (Special Facilities) Sections 1300-1 (Coverage and Objectives) and 1300-3.2 (Safety Class Items) are addressed throughout this analysis (EBDRD 3.3.1.B and RDRD 3.3.1.A, 3.3.4.B).

4.2.7 The Repository Segment shall accommodate the emplacement concept (TBD) selected during advanced conceptual design. Advanced conceptual design related to the emplacement drift invert has changed from a scenario of fill material to currently one of structural materials, i.e., steel or concrete. In addition, the advanced conceptual design showed emplacement with railcars; however, Key

066, Section 4.3.1 requires gantry emplacement (EBDRD 3.2.3.3.A.8 and RDRD 3.2.3.2.2.A.7) (Section 7.1.5).

4.3 ASSUMPTIONS

All assumptions below require confirmation as the design proceeds.

4.3.1 The following Key Assumptions from the *Controlled Design Assumptions Document* (CDA) (Reference 5.3) relate to the design of the emplacement drift invert segment.

Key 011: Waste packages will be emplaced in-drift in a horizontal mode. (Sections 4.3.5 and 7.1.5)

Key 016: The repository will be designed for a retrievability period of up to 100 years after initiation of emplacement. (Section 4.2.2)

Key 064: The seismic design of repository Systems, Structures and Components important to safety shall be based on the methodology presented in Reference 5.2. (Section 7.3)

For the emplacement drift invert, seismic design parameters in Reference 5.4, are assumed to correspond to Frequency-Category I, Reference 5.2. Based on Reference 5.4, Table 1, page 6, a mean peak horizontal acceleration of 0.27 g and mean peak horizontal velocity of 16 cm/sec are obtained. These values are further assumed to be applicable both to vertical and horizontal ground motions. As a conservative consideration, factors for reduction of ground motion with depth (Reference 5.4, Table 3, page 17) are not used in the analysis. Furthermore, based on Reference 5.2, page 3-21, a seismic wave frequency of 10 Hz is chosen for this analysis.

The seismic waves are numerically represented by the sinusoidal velocity waves (P-wave and S-wave) propagating vertically upwards through the emplacement drift.

The following parameters are at the earth's surface and apply to analysis for vibratory ground motion in both the horizontal and vertical directions:

Peak acceleration: 0.27g (Reference 5.4, Table 1) (Rounded to 0.3 g and used in Section 7.3 and Attachment II)

Peak velocity: 16 cm/sec (Reference 5.4, Table 1) (Section 7.3, Table 1 and Attachment II)

Frequency: 10 Hz (Reference 5.2) (Section 7.3, Table 1 and Attachment II)

Duration: up to 3.0 sec (Reference 5.2) (Section 7.3, Table 1 and Attachment II)

Key 066: Waste packages will be placed center in-drift, on pedestals, using gantry emplacement (Sections 4.3.5, 7.1.5 and 7.1.3).

Key 070: The following diameters are assumed for underground openings: Emplacement Drift (TBD) (Sections 4.3.5 and 7.1.5).

4.3.2 The following Engineered Barrier Design Requirements Document (EBDRD) assumptions from the CDA (Reference 5.3) relate to the design of the emplacement drift invert segment.

CDA EBDRD 3.2.5.4: EBS structures, systems, and components shall be designed for a maintainable preclosure service life of at least 150 years following first emplacement of waste (Section 7.2).

CDA EBDRD 3.7.1.J.2: The waste package mass shall not exceed 83,000 Kg (Sections 4.3.6 and 7.3).

4.3.3 The following Design Concept Subsurface (DCSS) assumptions from Reference 5.3 relate to the design of the emplacement drift invert segment.

DCSS 023: Maximum allowable preclosure rock surface temperature in the emplacement drift will be 200° C (Section 4.3.4).

DCSS 027: Concrete (subject to restrictions on chemical composition of cementitious materials) and steel are allowable preclosure construction material in all openings (Section 7.2).

DCSS 034: A single ground support type will be used in the emplacement drifts. Candidate ground support types under consideration (Sections 4.3.4 and 7.2):

- Precast concrete
- Cast-in-place concrete
- Steel sets.

DCSS 037: Invert material will consist of concrete/crushed tuff material combination. Other material additives may be used as necessary. Note, the concrete/crushed tuff combination has been abandoned due to consideration of full lining systems. Concrete material is being used for the invert (Section 7.2).

4.3.4 Materials for the emplacement drift invert segment will be of reinforced concrete or structural steel and shall have the following design strengths in this analysis (Sections 7.2 and 8.2):

- Concrete: compressive strength 62.1 MPa (f'_c=9000 psi) minimum
- Concrete Reinforcement: yield strength 413.7 MPa (60000 psi) minimum
- Structural steel: yield strengths 248.22 MPa (36,000 psi) and 344.75 Mpa (50,000 psi) depending on applications shown in Attachment I. The tunnel is judged to be a non-corrosive environment because of the generally dry conditions (Reference 5.26, Section 6.3.2.2), therefore corrosion allowance beyond reserve capacity of the steel members is not considered necessary.

Concrete and steel meet the requirements of Section 4.2.5 for the maximum emplacement drift temperature of 200° C (Section 4.3.3, DCSS 023). Design strength of concrete was selected because it represents the lower range of high strength concrete; i.e., high-strength concretes have specified compressive strengths of 6000 psi (41.31 MPa) or greater (ACI 363R-92, Chapter 1). Design strengths of structural steel were selected because they represent the most frequently used grades of steel (ASTM A36 and ASTM A572).

The following properties are assigned to the concrete invert for use in the numerical analysis.

Young's modulus: 27.58 GPa (4×10^6 psi, ACI-318, Section 8.5)
Poisson's ratio: 0.21 (ACI 363R-92, Section 5.4)
Density: 2000 kg/M^3 (Reference 5.29, page 8-4)

4.3.5 Emplacement drift diameter will be 5.5 meters maximum in this analysis. Section 4.3.1, Key 070 does not assume a diameter for the emplacement drift. The value assumed above allows waste packages placed horizontally center in-drift, on pedestals, using gantry emplacement (Section 4.3.1, Key 011 and Key 066). Any change to the emplacement drift diameter will most probably be a small increase which will not have a significant impact on the invert analysis. (Section 7.1.5)

4.3.6 Construction and operating loads shall have the following maximum values in this analysis:

TBM: 285 MT (2796 kN)
TBM transport dolly wheel load: 20 MT (196 kN)
Impact load: 25 percent of TBM dolly wheel load which is reasonable for a one time removal of the TBM in each emplacement drift.
Gantry: 60 MT (588 kN) Derived in Attachment I.
Waste package: 83 MT (814 kN) Rounded to 85 MT (834 kN) in Attachment I and bounded at 90MT (883kN) in Attachment II. (Section 4.3.2, EBDRD 3.7.1.J.2) The waste package was bounded at 883kN for analysis of the concrete invert because the waste package is stored directly on the invert

which results in more impact to the design with any waste package weight change.

Waste package emplacement impact: Load will not exceed the vertical seismic value of 0.3g.

The weight of the tunnel boring machine (TBM) will be 285 MT which is reasonable for a TBM of 5.5 meters diameter. The TBM will be removed from the completed drift over the construction access rails and will be supported by two rail mounted dollies, each with eight wheels. The weight of the TBM will be supported equally between the dollies. The weight of the TBM (285 MT) divided by sixteen wheels results in a wheel load of 17.8 MT which is rounded up to 20 MT (196 KN) for this analysis and provides an upper bounding value. A system of rollers could also be used and could be designed to be within the upper bound of 20 MT per wheel. The 20 MT wheel load will be larger than any construction wheel load. Construction wheel loads for locomotives and muck cars will be the largest construction loads and will not exceed 20 MT per wheel. Spacing of dolly wheels along a single track will not be less than three feet between centerline of wheels. Using two feet diameter wheels, the three feet spacing allows a minimum of one foot between outside perimeter of wheels.

Gantry weight of 60 MT is reasonable for a heavy steel frame and lifting mechanism, including drive motors, necessary to handle waste packages of 85 MT. (Section 7.3 and Attachment I)

4.3.7 The configurations of the emplacement drift invert segment developed in Section 7.1.5 and analyzed in Attachments I and II, are based on preliminary layout of drift opening, gantry emplacement, steel ground support, and precast concrete liner segments. Figure II-4, shown in Attachment II and extracted from Reference 5.8 shows the waste package support layout, specifically the steel support, pier, precast concrete invert segment, and related dimensions to be used as the basis for the precast concrete invert configuration in Figure II-3 of Attachment II. Figure II-3 shows the dimensions of an upper bounded condition for this design analysis.

4.3.8 Initial stresses used in this computer model are estimated based on gravitational stresses generated by the overburden weight. The horizontal to vertical in situ stress ratio of 0.5 was used in the computer model. (Reference 5.22, pages 15 and 19) (Attachment II, page II-67)

4.3.9 A 60 percent initial ground relaxation prior to the installation of invert and lining is assumed and used in this analysis. The rational is based on Reference 5.19, Section 7.12.4.1. (Section 7.3)

4.3.10 The hoop stress level induced in the concrete invert by the heat output from emplaced waste packages will remain in a range not exceeding 15 MPa.

(Reference 5.15, Sections 7.6.2.2.1 and 7.6.2.2.2) (Section 7.3 and Attachment II, Section 4)

4.3.11 A uniform spacing of 28 m for the emplacement drifts is assumed. (Reference 5.9, page 56) (Attachment II, page II-50)

4.3.12 A normal stiffness of 50 GPa and a shear strength of 10 GPa are assumed for simulating the interface between the tunnel rock and concrete invert. The values are based on the summary of joint stiffness values for the tuff. (Reference 5.10, page 19) (Attachment II, page II-53)

4.3.13 A live load of 24 kPa (500 psf) will be used to design the cover plate for the steel invert. This uniform load exceeds the load required for storage facilities (UBC, Table 16-A) by a factor of two and is used to provide durability to the steel invert allowing multiple reuse of the invert. (Attachment I)

4.3.14 Loads on the emplacement drift invert from equipment used during off-normal conditions and other off-normal loads will not exceed the operating loads in Section 4.3.6. Equipment used during off-normal conditions will access the emplacement drift using the gantry rails, or over a layer of crushed rock fill material. Gantry rails can support multiple wheel loads equal to the gantry wheel loading. Crushed rock fill will spread any wheel loads and reduce the loading on the inverts. Off-normal equipment loads can be managed to not exceed operating loads. Some off-normal loads, such as a dropped waste package, may damage the invert and repair or replacement of the invert may be required. (Section 7.3)

4.3.15 Loads on the emplacement drift invert from backfill materials to enhance the EBS will not exceed the loads of the waste package support and loaded gantry. (Sections 7.1.4 and 7.3 and Attachment II)

4.3.16 A soft layer underneath the concrete invert is assumed to have one tenth of the Young's modulus of the rock mass and to have the same Poisson's ratio as the rock mass. This soft layer represents a layer of grout to be injected under the invert after installation to provide uniform support for the invert. (Attachment II)

4.3.17 A bulk density of 2274 kg/m^3 is used for the TSw2 unit. (Reference 5.19, Attachment II, page II-67)

4.4 CODES AND STANDARDS

4.4.1 American Concrete Institute (ACI)

ACI 117R-90	Standard Specification for Tolerances for Concrete Construction and Materials

ACI 211.1-91	Standard Practice for Slecting Proportions for Normal, Heavyweight and Mass Concrete
ACI 301-96	Standard Specification for Structural Concrete
ACI 305R-91	Hot Weather Concreting
ACI 306R-88	Cold Weather Concreting
ACI 318/318R-95	Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95).
ACI 363R-92	State-of-the-Art Report on High-Strength Concrete (Reapproved 1997)

4.4.2 American Institute of Steel Construction (AISC)

AISC MO16-89	AISC Manual of Steel Construction, Allowable Stress Design, Ninth Edition, 1989.

4.4.3 American Railway Engineering and Maintenance-of-Way Association (AREMA)

AREMA-97	AREA Manual for Railway Engineering

4.4.4 Not used.

4.4.5 American Society for Testing and Materials (ASTM)

ASTM A6/A6M-96b	Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling
ASTM A36/A36M-96	Standard Specification for Carbon Structural Steel
ASTM A307-94	Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength
ASTM A325-96a	Standard Specification for Structural Steel Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength
ASTM A510/A510M-96	Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel

ASTM A563-96	Standard Specification for Carbon and Alloy Steel Nuts
ASTM A572/A572M-97	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel
ASTM A615/A615M-96a	Standard Specification for Deformed and Plain Billet Steel Bars for Concrete Reinforcement
ASTM C31/C31M-96	Standard Practice for Making and Curing Concrete Test Specimens in the Field
ASTM C33-97	Standard Specification for Concrete Aggregates
ASTM C39-96	Standard Test Methods for Compressive Strength of Cylindrical Concrete Specimens
ASTM C94-97	Standard Specification for Ready-Mixed Concrete
ASTM C109/C109M-95	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2 Inch or 50 mm Cube Specimens)
ASTM C138-92	Standard Test Method for Unit Weight,Yield and Air Content (Gravimetric) of Concrete
ASTM C143-90a	Standard Test Method for Slump of Hydraulic Cement Concrete
ASTM C150-97	Standard Specification for Portland Cement
ASTM C171-97	Standard Specification for Sheet Materials for Curing Concrete
ASTM C172-97	Standard Practice for Sampling Freashly Mixed Concrete
ASTM C173-94a	Stardard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
ASTM C231-97	Stardard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
ASTM C260-95	Standard Specification for Air-Entraining Admixtures for Concrete

ASTM C309-97	Standard Specificatition for Liquid Membrane-Forming Compounds for Curing Concrete
ASTM C494-92	Standard Specification for Chemical Admixtures for Concrete
ASTM C1064-86 (Reapproved 1993)	Standard Test Methods for Temperature of Freshly Mixed Portland Cement Concrete
ASTM C1107-97	Standard Specification for Package Dry, Hydraulic-Cement Grout (Nonshrink)
ASTM D75-87	Standard Practice for Sampling Aggregates
ASTM F436-93	Standard Specification for Hardened Steel Washers

4.4.6 American Welding Society (AWS)

AWS D1.1-98	Structural Welding Code-Steel, 16th Edition

4.4.7 Concrete Reinforcing Institute (CRSI)

CRSI-DA4-90	Manual of Standard Practice, 1990, 25th Edition

4.4.8 Department of Energy (DOE) Orders

DOE 6430.1A-89	General Design Criteria

4.4.9 International Conference of Building Officials (ICBO)

UBC-97	Uniform Building Code (UBC)

4.4.10 Precast Prestressed Concrete Institute (PCI)

PCI MNL 116-85	Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products, Third Edition

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5.31 CRWMS M&O 1997. *Software Qualification Report for Fast Lagrangian Analysis of Continua (FLAC) Version 3.30.* CSCI Number 30022 V3.3. DI: 30022-2003, REV 00. Las Vegas, Nevada: CRWMS M&O. MOL.19980123.0651.

6. USE OF COMPUTER SOFTWARE

6.1 STAAD-III, Version 22.3a, (Reference 5.13), is the computer software used for the analysis of the steel invert presented in Attachment I. The computer software has been verified and validated, according to QAP-SI-series of CRWMS M&O Computer Software Quality Assurance procedures. STAAD-III is a specialized computer code developed specifically for solving structural analysis problems.

The computer software used in this analysis is appropriate for this application since the STAAD-III program was specifically selected and validated for the purpose of analyzing and designing steel frames and accessories. The program was used within the validated range as described in the verification and validation documentation. The program was obtained from Software Configuration Management in accordance with appropriate procedures.

The computer software is installed on an IBM-compatible PC equipped with a Pentium microprocessor.

6.2 Fast Lagrangian Analysis of Continua (FLAC) Version 3.3 (Reference 5.31, and Reference 5.12), a finite difference code, was used to perform the mechanical analysis of the concrete invert segment as presented in Attachment II. The analysis was performed on a Pentium PC. FLAC is approved for use in design in accordance with M&O Computer Software Quality Assurance procedures. FLAC is appropriate for the applications used in this analysis. FLAC was obtained from Software Configuration Management in accordance with the applicable M&O procedures. FLAC software was used within the range of validation as specified in software qualification documentation.

6.3 Computational support software Mathcad Plus 6.0 was used in Attachments I and IV to perform structural calculations for determining steel and concrete requirements. User defined formulas, inputs and results are shown in attachments. Mathcad represents equations, text and graphics as would be seen in a text book. Mathematical computations are performed internally. Mathcad is appropriate for this application.

The computer software is installed on a Compaq Desk Pro with a Pentium microprocessor.

6.4 Graphic support software MicroStation 95 was used in Attachments IV and V to perform measurement analysis of areas and to determine the centroid of areas and volumes. Results are shown in attachments. MicroStation 95 is appropriate for this application.

The graphic software is installed on a Compaq Desk Pro with a Pentium microprocessor.

7. DESIGN ANALYSIS

7.1 INTRODUCTION

7.1.1 General

This design analysis develops configurations for the emplacement drift invert suitable for use with steel, precast concrete, and cast-in-place concrete ground control systems. The emplacement drift invert is designed to support the subsurface waste handling system gantry and the waste package support system. Loads from construction operations are evaluated and included in the invert design. Loadings from retrieval operations and off-normal conditions are assessed. The impact of fill materials and future backfilling is determined. Interfaces with ground control, waste handling and waste package support systems are identified (Reference 5.30). Impacts from the developments in the design of the waste package emplacement equipment, power supply system, waste package support system, and in-drift monitoring system are identified and incorporated into the design.

7.1.2 Ground Control System

The ground control system design is not part of this analysis. The ground control system may consist of concrete or steel materials or a combination of concrete and steel or may be of rockbolts and shotcrete. This analysis will consider steel sets with and without a steel liner, precast concrete liner segments, and cast-in-place concrete for the ground control. Concrete liners will be not exceed 200 mm thick as analyzed in Reference 5.15, Section 7.6.2.2.2.

7.1.3 Subsurface Waste Handling System

Waste packages will be transported by the subsurface waste emplacement system from the surface to the entrance of the emplacement drift. A rail mounted transporter will carry the waste package to the emplacement drift entrance. The waste package within the transporter will sit on a special railcar designed to hold it in place during transportation. At the emplacement drift entrance the railcar and waste package will be pushed from the transporter onto an unloading dock immediately inside the drift. A rail-mounted gantry (Section 4.3.1 Key 066) will then straddle the waste package and railcar, lift the waste package, and carry it to an assigned location in the emplacement drift. At that location the gantry will lower the waste package onto the waste package support system which is installed on top of the concrete invert system. (Reference 5.14, Section 7.1.1, page 16) The waste package support system will require redesign to be compatible with the steel invert. (Reference 5.8) The ground control system, gantry, waste package support system, and invert systems are shown in Figures 7-1 and 7-2 for illustrative purposes. The power supply for the rail mounted gantry within the emplacement drifts will be provided through a third rail collector system which will be mounted on the drift invert system. (Reference 5.24, Section 8.3, page 62)

7.1.4 Waste Package Support System

The waste package support assembly is of a modular design allowing flexibility in waste package placement within the drifts and component replacement of the support assembly if the support becomes damaged. The waste package support assembly consists of a steel and concrete pier and a steel "V" shaped support that is directly in contact with the waste package. Attachment II, Figure II-38 shows the waste package support layout designed to be placed on the concrete invert. The support pier may be placed or can be designed to be placed directly on the invert of the drift or on top of a concrete invert if installed. The waste package support system keeps the waste package off of the invert and allows drainage along the invert through an opening in the base of the pier. The waste package support system is designed to accommodate the potential application of fill material that would act as a filter bed to allow drainage or any future backfilling that may be needed. (Reference 5.8 and Section 4.3.15)

7.1.5 Invert Configurations

The emplacement drifts are configured for in-drift horizontal emplacement of the waste packages in accordance with assumption in Section 4.3.1 Key 011, and will be placed center in-drift, on pedestals, using gantry emplacement in accordance with the assumption in Section 4.3.1 Key 066. The emplacement drift invert system is considered part of the underground facility portion of the EBS and the invert will form the support structure for the subsurface waste emplacement system and the waste package support system. The invert may also be part of the ground control system in the emplacement drift forming the base support structure for the ground control structural system and be capable of withstanding loads resulting from installation and from thermally induced strains from the

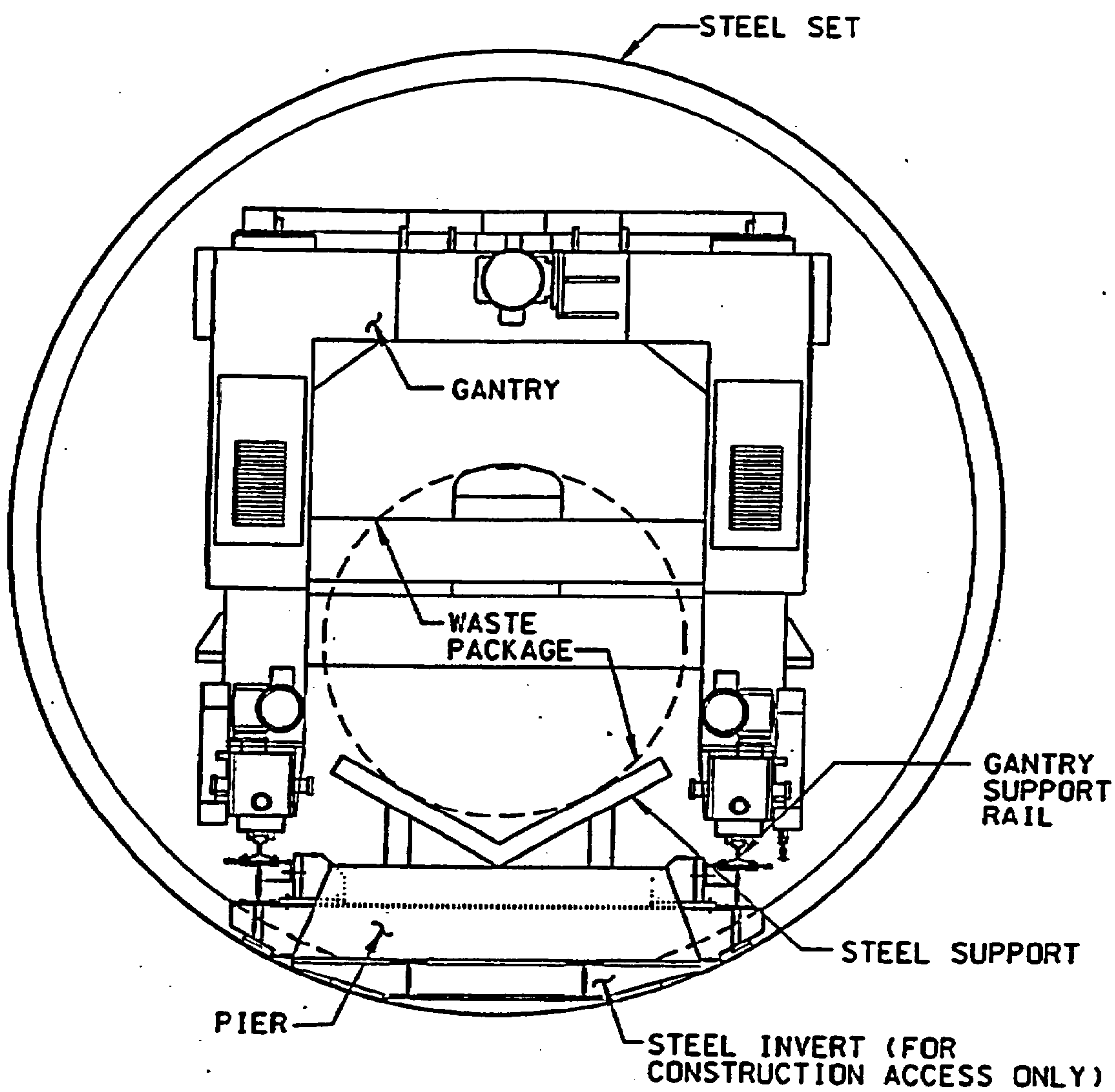


FIGURE 7-1
STEEL INVERT

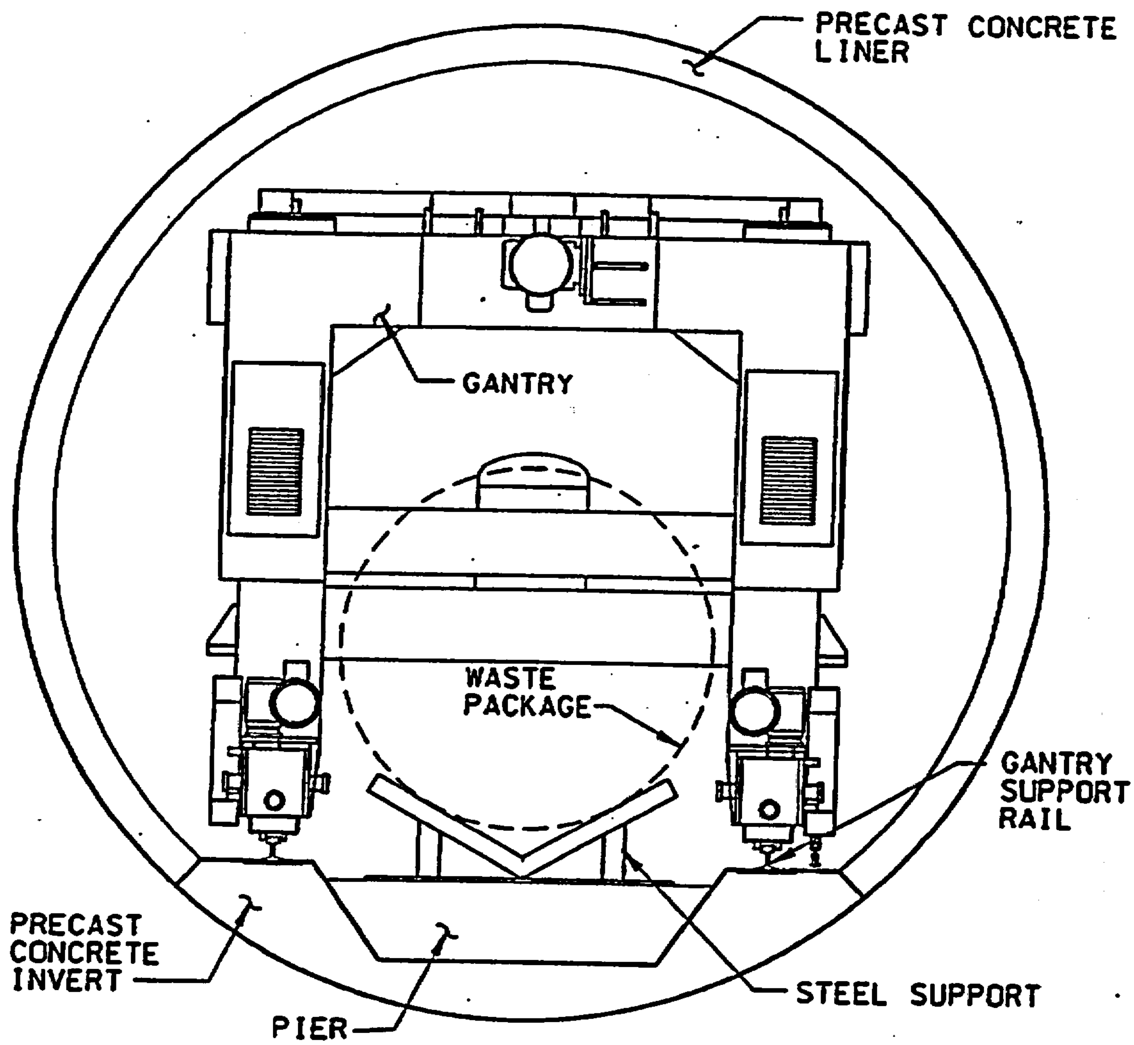
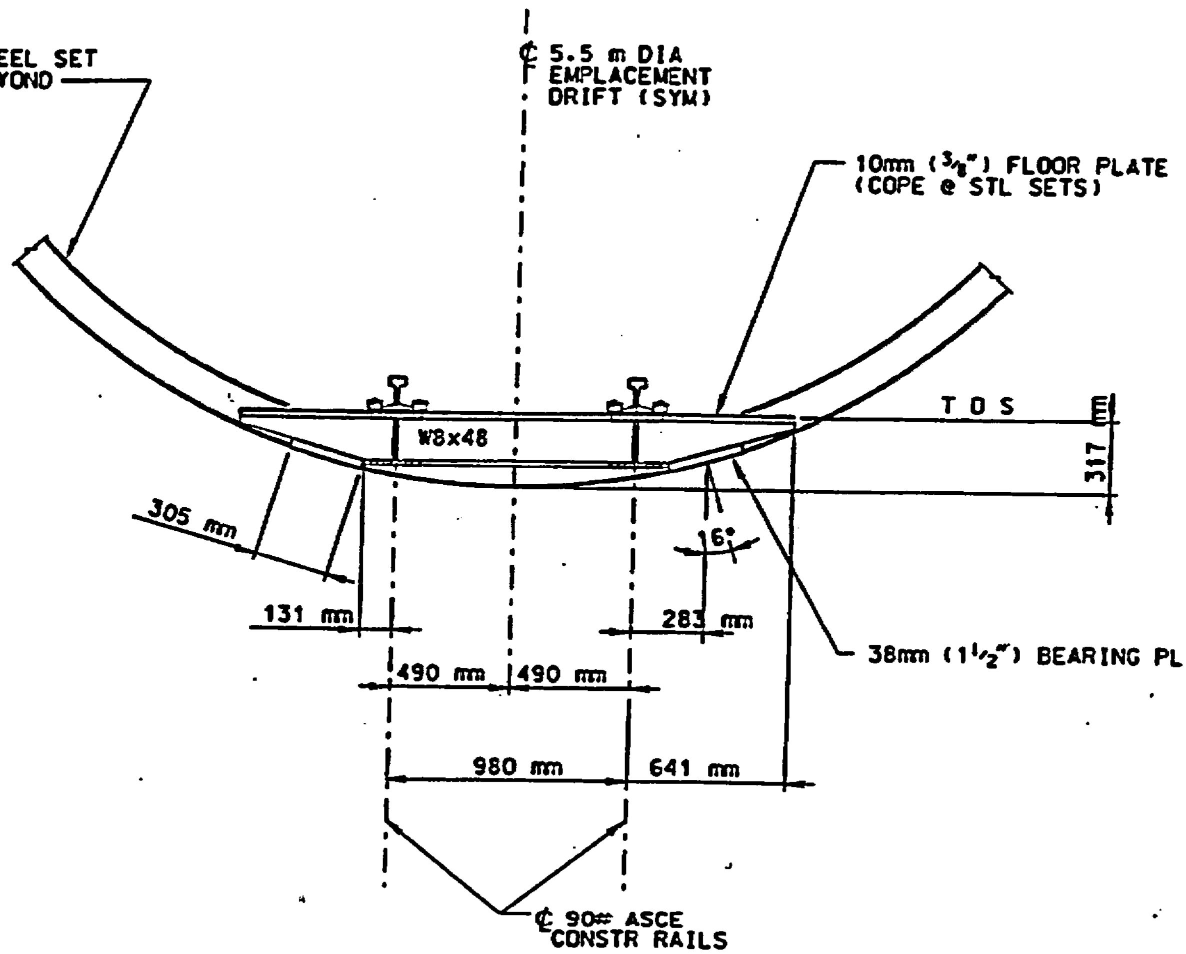


FIGURE 7-2
CONCRETE INVERT



NOTE:

1. STEEL INVERT WILL BE ALL WELDED CONSTRUCTION - AWS D1.1
2. STEEL INVERT TO BE REMOVED PRIOR TO INSTALLING GANTRY RUNWAY BEAM AND RAIL. SEE FIGURE 7-4.
3. STEEL INVERT TO BE REMOVED PRIOR TO CAST-IN-PLACE CONCRETE INVERT. SEE FIGURE 7-5.

FIGURE 7-3
STEEL INVERT SECTION
FOR CONSTRUCTION ACCESS

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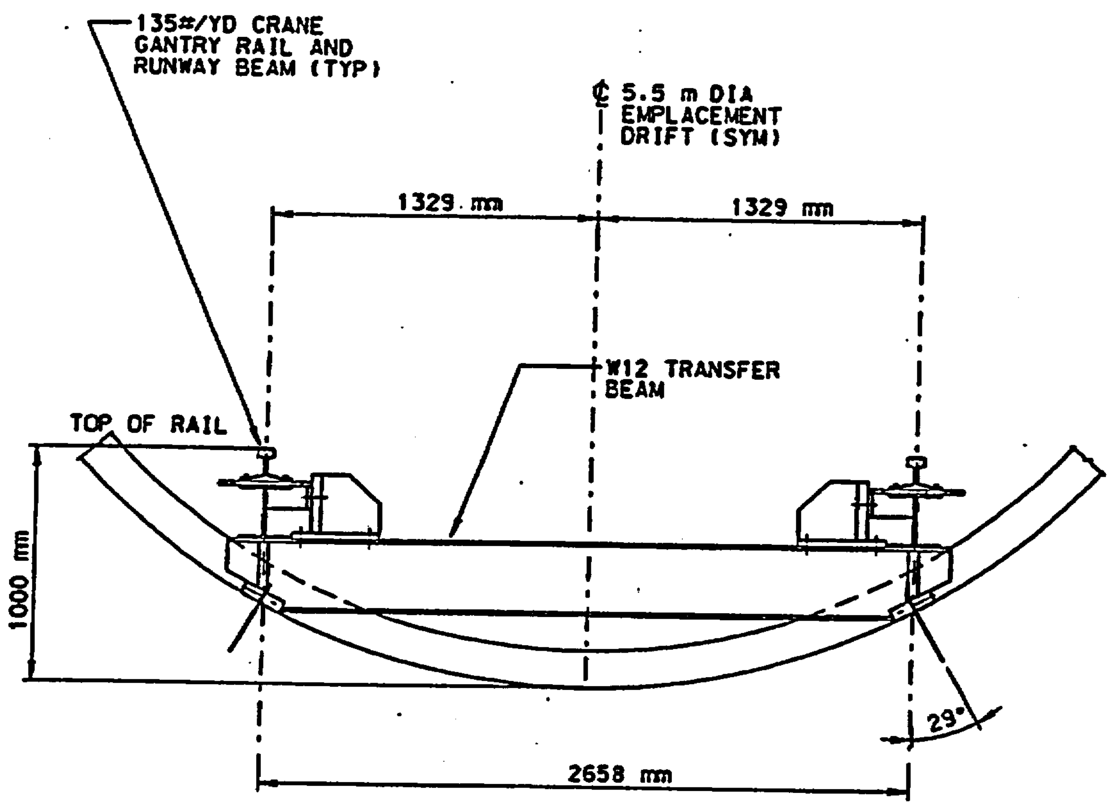

NOTE

1. SEE FIGURE 7-3 FOR CONSTRUCTION ACCESS INVERT.

FIGURE 7-4
GANTRY SUPPORT STEEL
INVERT ELEVATION

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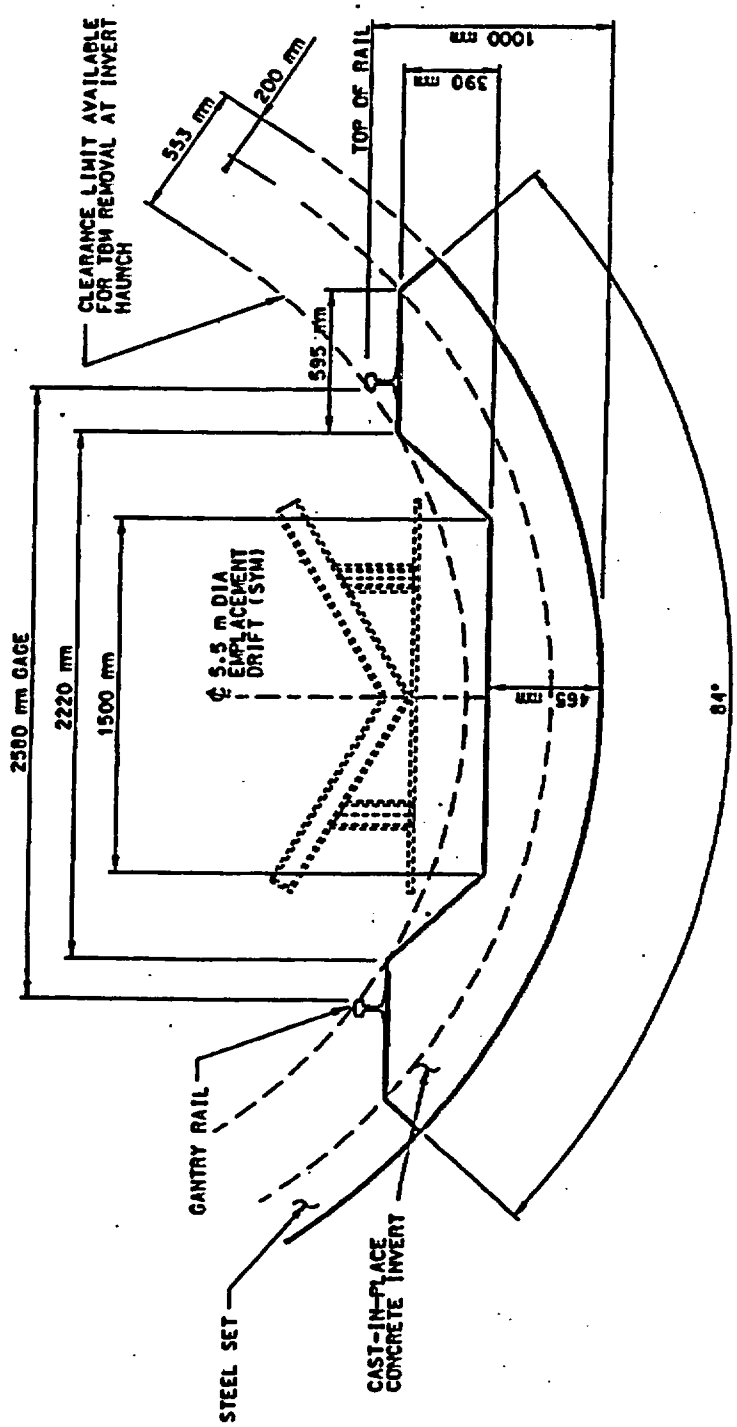

NOTE:
1. SEE FIGURE 7-3 FOR CONSTRUCTION ACCESS INVERT.

FIGURE 7-5
CAST-IN-PLACE CONCRETE INVERT & HAUNCHES FOR REPOSITORY

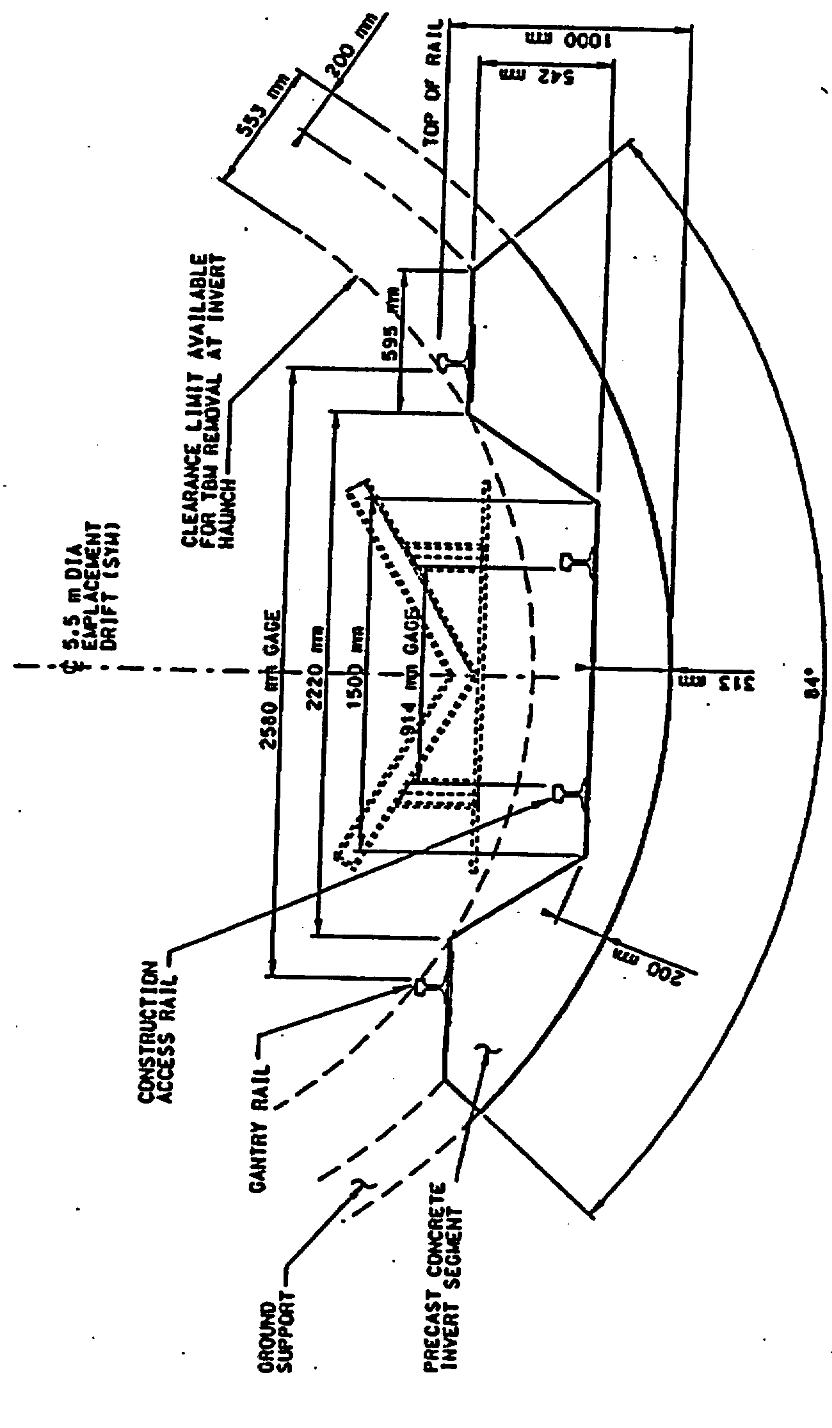

FIGURE 7-6
PRECAST CONCRETE INVERT
FOR CONSTRUCTION & REPOSITORY

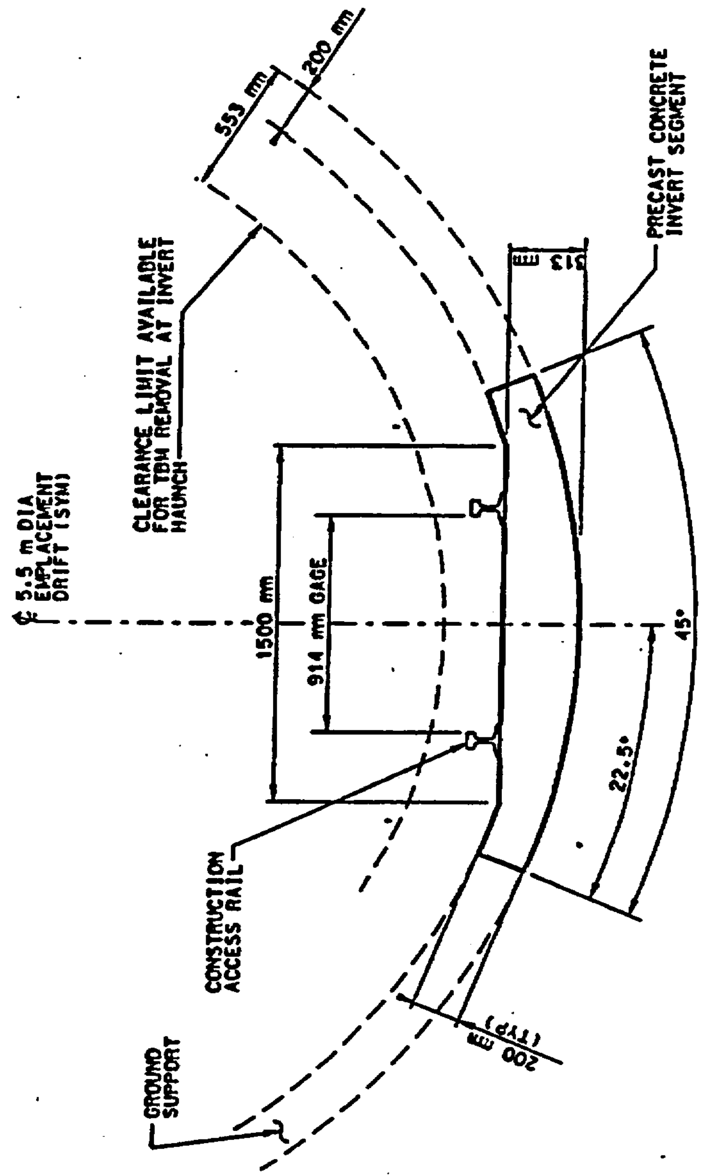

FIGURE 7-7
PRECAST CONCRETE INVERT FOR CONSTRUCTION ACCESS

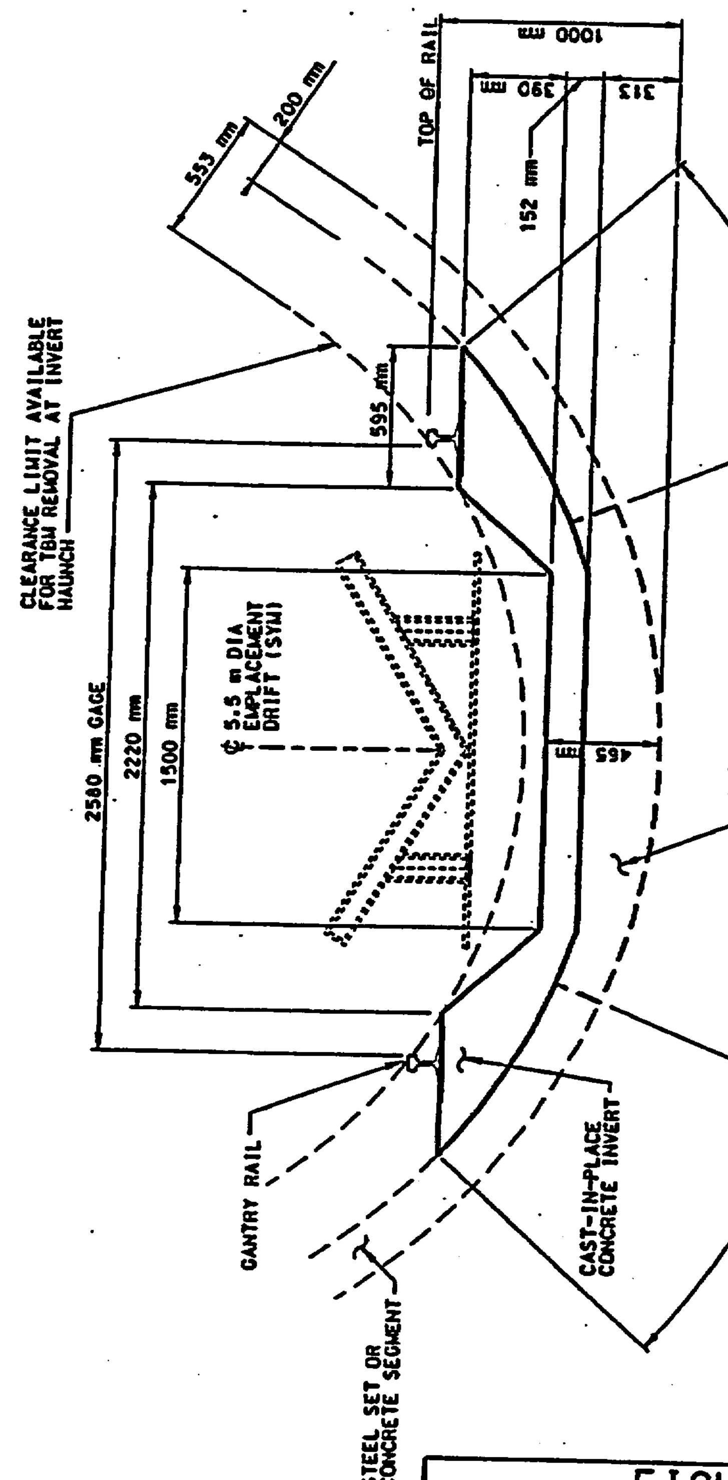

FIGURE 7-8
CAST-IN-PLACE CONCRETE INVERT & HAUNCHES FOR REPOSITORY

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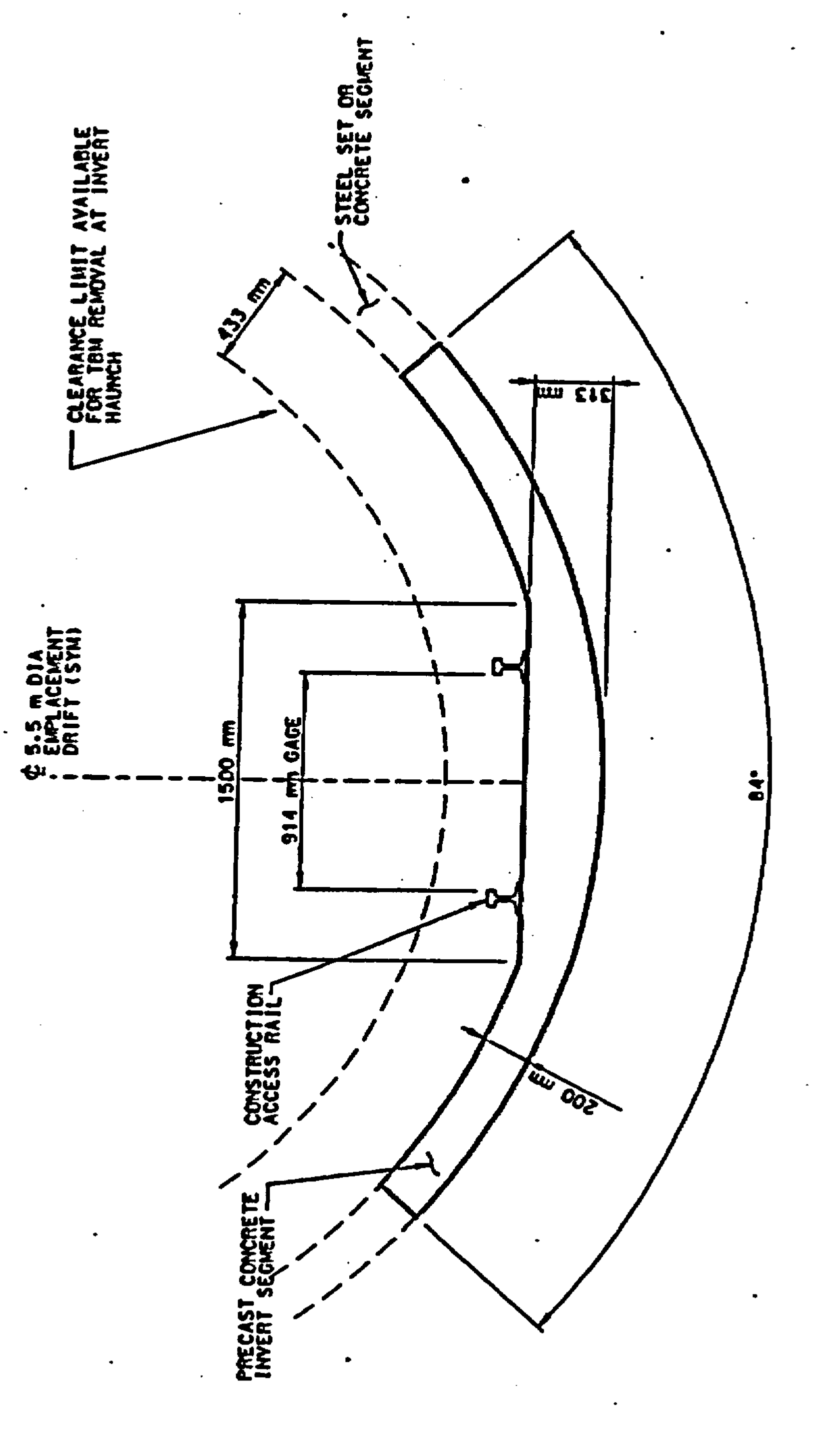


FIGURE 7-9
PARTIAL PRECAST CONCRETE INVERT FOR CONSTR ACCESS

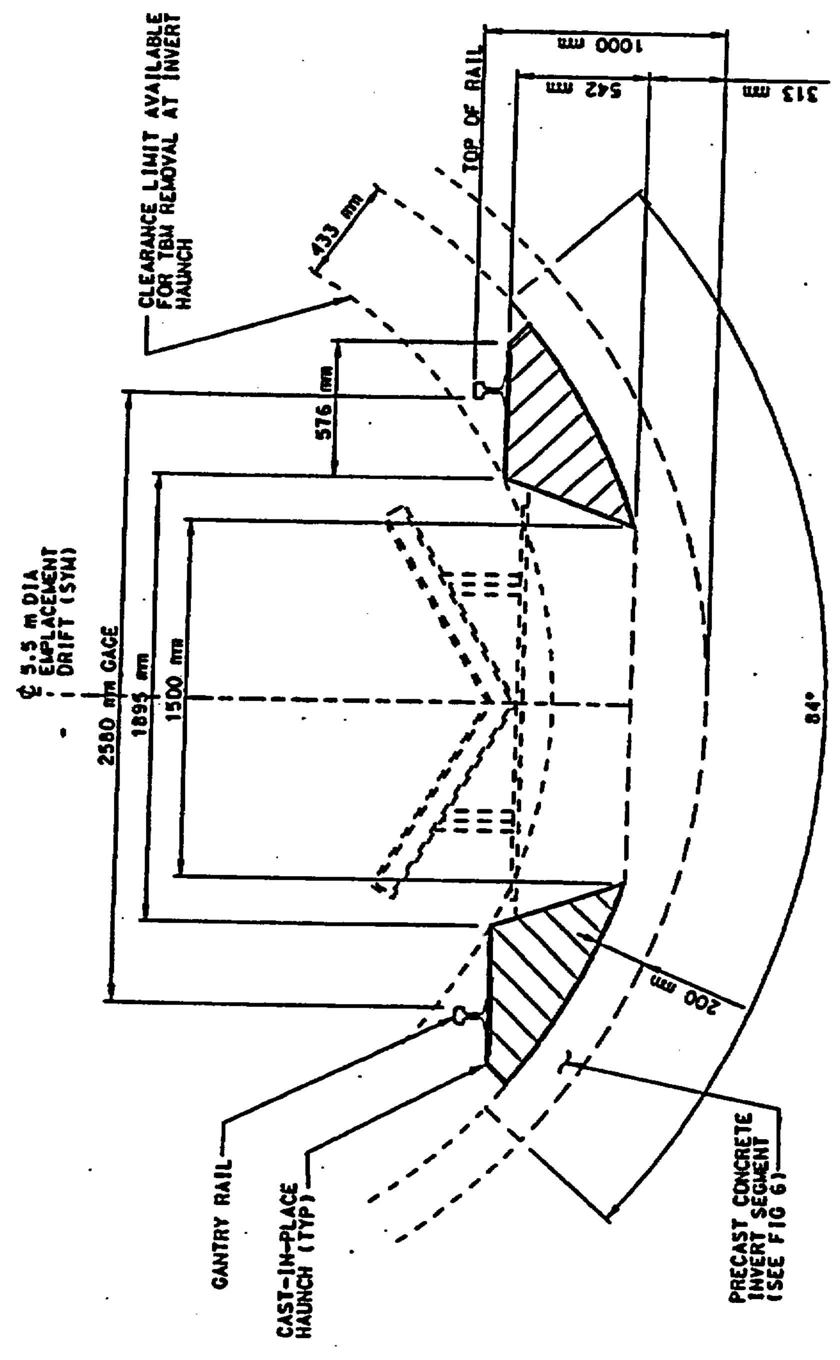


FIGURE 7-10
CAST-IN-PLACE CONCRETE HAUNCHES FOR REPOSITORY

hot emplaced waste packages, Section 4.2.4, creating a preclosure rock surface temperature of 200° C maximum, Section 4.3.3, DCSS 023.

The invert structure must also accommodate a number of other functions. During excavation of the emplacement drift, the invert structure will be installed behind the TBM head and will support the TBM construction rail.

The invert must be robust enough to accommodate loads from the TBM trailing gear and rail traffic for muck removal and materials handling. On completion of emplacement drift excavation, the TBM will be partially disassembled and backed through the drift over the rail on rollers (or dollies) designed to support the heavy TBM.

The configurations of the emplacement drift invert segment shown in Attachments I and II, are based on preliminary evaluation of waste package handling in accordance with the criteria in Section 4.2.7, and assumptions in Section 4.3.1 Key 011 and Key 066. All dimensions shown defining the invert geometries are in accordance with the assumption in Section 4.3.7, and are used here to determine the minimum properties of the invert materials. The precast invert configuration shown in Figure II-37 is used as the basis for the analysis of the concrete invert in Attachment II. Diameter of the emplacement drift opening is TBD in accordance with the assumption in Section 4.3.1, Key 070, but is assumed to be 5.5 meters maximum (Section 4.3.5) for this analysis.

The invert configuration must accommodate the above requirements and must be made of materials suitable for anticipated conditions. Various invert design configurations are described below and analyzed in this analysis.

7.1.5.1 Steel Invert

The steel invert will be a two part system consisting of a construction support system and an emplacement gantry support system. The construction support system will carry construction loads and will be removed for reuse upon completion of the emplacement drift excavation. The construction rail will be carried on special steel supports placed between the steel sets. The gantry support system will serve the subsurface waste handling operations and will be a permanent part of the repository. The gantry rails will be mounted on heavy steel supports placed between the steel sets. Figures 7-3 and 7-4 show the configuration of the steel invert. Both parts of the steel invert will be constructed of steel materials consisting of structural shapes and plates. This invert will be suitable for use with steel sets and a steel liner as a ground control system.

7.1.5.2 Steel Invert and Cast-in-place Concrete Invert

This alternate will be a two part system consisting of a steel invert for the construction support system and a cast-in-place concrete invert for the emplacement gantry support system. The construction support system will carry construction loads and will be removed for reuse upon completion of the emplacement drift excavation. The construction support system can be used to carry the forming and concrete placement equipment for installing the cast-in-place concrete invert. The cast-in-place gantry support system will serve the subsurface waste handling operations and will be a permanent part of the repository. Figure 7-5 shows the configuration of the cast-in-place concrete invert. The removable steel part of the invert will be of the same configuration described in Section 7.1.5.1 and shown in Figure 7-3. The cast-in-place concrete invert will have the advantage of a controlled alignment that will facilitate rail installation and equipment operation. This invert will be suitable for use with steel sets and a steel liner as a ground control system. The cast-in-place invert will enclose the lower part of the steel sets and steel liner.

7.1.5.3 Precast Concrete Invert

The precast concrete invert will be a one part system consisting of a single concrete structure providing both the construction support system and the emplacement gantry support system. The construction support system will consist of an access rail installed in the center portion of the precast concrete invert to carry construction loads. Once emplacement drift construction is completed and the TBM is extracted, the construction access rail will be removed for reuse and the gantry rails will be installed on the raised haunches of the invert. The gantry support system will serve the subsurface waste handling operations and will be a permanent part of the repository. Figure 7-6 shows the configuration of the precast concrete invert. This invert will be suitable for use with steel sets and a steel liner, a concrete segmental liner, or a cast-in-place liner as a ground control system.

7.1.5.4 Precast Concrete Invert With Cast-in-place Concrete Invert And Haunches

This alternate will be a two part system consisting of a precast concrete invert for the construction support system and a cast-in-place concrete invert with haunches for the emplacement gantry support system. The construction support system will consist of an access rail installed in the center portion of the precast concrete invert to carry construction loads. Once emplacement drift construction is completed and the TBM extracted the construction access rail will be removed for reuse. The construction support system can be used to carry the forming and concrete placement equipment for installing the cast-in-place concrete invert. A cast-in-place concrete invert with haunches will then be installed for the gantry support system to serve the subsurface waste handling operations. Both parts of the invert system will be a permanent part of the repository. Figures 7-7 and 7-8 show the configuration of the precast concrete invert and the cast-in-place invert. The cast-in-place part of this alternate will have the advantage of a controlled alignment that will facilitate rail installation and equipment operation. This invert will be suitable for use with steel sets and a steel liner, a concrete segmental liner, or a cast-in-place liner as a ground control system.

7.1.5.5 Partial Precast Concrete Invert with Cast-in-place Concrete Haunches

This alternate will be a two part system consisting of a partial precast concrete invert for the construction support system and cast-in-place concrete haunches added to the precast portion for the emplacement gantry support system. The construction support system will consist of an access rail installed in the center portion of the partial precast concrete invert to carry construction loads. Once emplacement drift construction is completed and the TBM extracted, the construction access rail can be used to carry the forming and concrete placement equipment for installing the cast-in-place concrete haunches. After haunch construction is completed the construction access rail will be removed for reuse. The cast-in-place haunches will provide the gantry support system that will serve the subsurface waste emplacement system. Both parts of the invert system will become a permanent part of the repository. Figures 7-9 and 7-10 show the configuration of the partial precast concrete invert and cast-in-place haunches. The cast-in-place haunches in this alternate will have the advantage of a controlled alignment that will facilitate rail installation and equipment operation. This invert will be suitable for use with steel sets and a steel liner, a concrete segmental liner, or a cast-in-place liner as a ground control system.

7.1.6 IN-DRIFT MONITORING

In-drift monitoring will be achieved with the use of a proposed "remote inspection gantry" that would operate over the emplacement drift invert gantry rails. The monitoring would include vision systems, thermal instruments, radiological instruments and air and gas instruments. (Reference 5.25, Section 7.6.5, page 55)

7.2 INVERT MATERIALS

Two types of materials are considered for the emplacement drift invert system in this analysis, inverts constructed from either concrete or steel. Two types of invert materials are considered because of potential performance assessment concerns stemming from extensive use of concrete in the emplacement drifts. If concrete becomes unacceptable, the alternative steel material will be used. Invert material consisting of a combination of concrete and crushed tuff, Section 4.3.3 DCSS 037, was considered and abandoned when full lining systems were selected.

The invert segment is usually part of the ground support acting in ring compression and forming the foundation for any ground support structure. Assumption 4.3.3 DCSS 034 addresses three types of ground support; i.e., precast concrete, cast-in-place concrete, and steel sets.

Steel and concrete (both precast and cast-in-place) materials are proposed for use for the emplacement drift invert in accordance with assumptions in Section 4.3.3 DCSS 027. Steel and concrete materials satisfy the criteria in Sections 4.2.1, 4.2.2 and 4.2.5. Steel and concrete are durable materials that can be maintained as necessary for a service life of 150 years. Concrete, cast-in-place or as precast invert segments or steel inverts can be repaired as necessary or replaced with like materials. Steel and concrete materials are noncombustible and heat resistant. Structural steel yield strength is 248.22 MPa minimum and concrete compressive strength is 62.1 MPa, Section 4.3.4, for this analysis.

Section 7.1.5 describes invert design configurations with steel, steel and cast-in-place concrete, precast concrete, precast and cast-in-place concrete and precast concrete with cast-in-place concrete haunches. Invert configurations will be suitable for use with steel sets and a steel liner, a concrete segmental liner or a cast-in-place liner as ground support.

Structural steel ground support will consist of steel sets made of wide flange shapes. The steel ground support may be a continuous steel ring or a partial steel ring supported on a precast concrete invert. Where the partial steel ring is supported on a precast concrete invert, the invert will be designed as described below.

Where a continuous steel ring (steel set) and a steel liner is used for ground support, a two part steel invert, Section 7.1.5.1 and Figure 7-3, will be placed between the steel sets and will support the construction access rail and be designed with clearances and load capacity to allow removal of the TBM. After the emplacement drift is completed and the TBM removed, the construction access rail and steel invert will be removed for use in constructing another emplacement drift. The gantry support rail will then be installed on a steel support beam independent of the steel sets.

As an alternative to attaching the gantry rail to a steel support beam, a cast-in-place concrete invert, Section 7.1.5.2 and Figure 7-5, can be used for gantry rail support. Initially a steel invert, Figure 7-3, would be installed to support the construction access rail and for TBM removal. After the drift is completed and the TBM extracted, the access rail and steel invert will be removed for reuse. A cast-in-place concrete invert, Figure 7-5, will then be installed to carry the gantry support rail.

Where steel sets with a steel liner, precast concrete segmental liner, or cast-in-place concrete liner are used for ground support, precast concrete inverts or a combination of precast invert segments and cast-in-place inverts, Sections 7.1.5.3, 7.1.5.4 and 7.1.5.5, will be used. A cast-in-place concrete invert and/or liner may be placed as a second stage ground support following installation of rockbolts or steel sets. A cast-in-place liner would most likely include a cast-in-place invert over precast inverts installed for construction access, Figures 7-7 and 7-9.

A precast concrete invert suitable for construction and repository, Section 7.1.5.3 and Figure 7-6, can be used with the ground control support described above. The precast invert would support both the construction access rail and the gantry rail. The construction access rail will be removed prior to installing the gantry rail.

A combination precast and cast-in-place concrete invert, Section 7.1.5.4 and Figures 7-7 and 7-8, are also suitable for use with the ground control support described above. Figure 7-7 shows a precast invert segment that will be compatible with the ground support and will carry the construction access rail. Once the emplacement drift construction is complete a cast-in-place concrete invert, Figure 7-8, is installed directly over the precast invert. The cast-in-place portion will carry the gantry support rail.

Another combination of precast and cast-in-place concrete is shown in Figures 7-9 and 7-10. A partial precast concrete invert, suitable for use with the ground control described above, is installed to support the construction access rail, Figure 7-9. Once the emplacement drift construction is completed cast-in-place concrete haunches, Figure 7-10, are installed to carry the gantry support rail.

7.3 LOADING CONDITIONS

As part of the EBS, the emplacement drift invert provides support for the following loading conditions resulting from construction and waste emplacement operations:

- Ground control structures
- Construction access rail
- TBM removal by rail
- Waste package handling during emplacement and retrieval
- Waste package handling during recovery from off-normal conditions. (Off-normal equipment loads will not exceed operating loads.) (Section 4.3.14)
- Emplacement drift backfill to enhance the EBS. (Loads will not exceed the loads of the waste package and gantry.) (Section 4.3.15)

The emplacement drift invert segment is subjected to dead (rock) loads, seismic loads, thermal loads, installation loads, construction loads, and operating loads. An allowance for thermally induced stress is shown in Section 4.3.10. Seismic loads are shown in Section 4.3.1 Key 064. Construction and operating loads are shown in Section 4.3.6 and are bounded by the following related values in this analysis:

TBM: 285 MT (2795 kN)
TBM transport dolly wheel load: 20 MT (196 kN)
Gantry: 60 MT (588 kN)
Waste package: 85 MT (834 kN) Bounded at 90 MT (884 kN) for concrete inverts.
Waste package emplacement impact: Load will not exceed the vertical seismic value of 0.3g.

Steel Invert Loads

Loads on the steel invert for construction and gantry support are:

Construction Support

- Live load on cover plate: 24 kPa (500 psf) Exceeds UBC-97, Table 16-A, Storage, by a factor of two
- TBM transport dolly wheel load: 20 MT (196 kN) Section 4.3.6
- Impact load : 25 percent of TBM dolly wheel load Section 4.3.6

Gantry Support

- Gantry load: 60 MT (588 kN) Section 4.3.6
- Waste package load: 85 MT (834 kN) Section 4.3.6
- Seismic load: 0.27g, used as a percentage of weight, is included in Reference 5.18, file GANTRY-H, used in Attachment I. Section 4.3.1
- Thermal load: Not applied. Expansion joint in steel members can be readily added in final design.

Seismic load is not applied to the construction support invert because there is a low probability that the site seismic event will occur at the time the TBM is being removed. As it is proposed that the construction support invert be removed and reused, no waste package loading for the invert is considered. In addition, thermal loading is not considered because the invert will be removed and reused. If the steel invert remains in place an allowance for expansion under thermal load needs to be evaluated.

Combinations of dead load, live load, and seismic load specifically related to the analysis of the invert are shown in Attachment I.

The steel invert is analyzed in Attachment I in accordance with the criteria and assumptions of this analysis and adheres to the dimensions shown in Figure II-37, Attachment II for the construction and gantry rails. The steel invert is modeled with the loading conditions above and the results are shown in Section 8.

<u>Concrete Invert Loads</u>

For the two dimensional FLAC models, the invert segments and lining segments are numerically installed after a 60 percent elastic ground relaxation, (Reference 5.19, Section 7.12.4.1) due to excavation, has taken place. The balance of ground relaxation will load the invert and lining.

Seismic and thermal loads incorporate the criteria requirements of Sections 4.2.3 and 4.2.4 and the assumption of Section 4.3.3, DCSS 023. Reference 5.4 provides the seismic acceleration and velocity values in Section 4.3.1 for analyzing the seismic impacts of emplacement drift equipment on the invert.

Installation loads include forces from handling of the precast invert and forces from the expansion of the precast concrete ground support wall and crown segments. Handling and placing of the precast invert will be by lifting lugs installed in the invert segment at two locations. (Attachment IV) Forces from expanding the precast concrete ground support segments and installing key wedges, by inspection, will not exceed the heat induced hoop stress shown in Table 1.

The cast-in-place concrete invert Figure 7-5 and the cast-in-place haunches Figures 7-8 and 7-10 are not loaded in the same manner as the precast invert loadings shown in Table 1, i.e., rock load, TBM load, and heat induced load are not applicable. The precast invert is subjected to all the loads shown in Table 1. The cast-in-place concrete invert will be analyzed based on evaluation of the applicable stresses developed in this analysis for the precast invert and will not be modeled separately.

Table 1 identifies the loads a precast concrete invert will be subjected to during preclosure of the repository. The concrete invert segment is analyzed as continuous along the emplacement drift alignment.

Table 1. Loads of Concern with A Precast Concrete Invert Segment

Type	Description	Magnitude	Orientation and Distribution	Sources
I	In situ rock loadthat acts on the liner and invert after installation. This load is caused by elastic ground relaxation of the in situ stresses.	Invert is installed after 60 % elastic ground relaxation	Both horizontally and vertically	Reference 5.19, Section 7.12.4.1
II	TBM transportation load. This is a moving load that is concentrated at wheel contact points.	196 kN per contact	Vertically on the construction rail on the invert	Section 4.3.6
III	Gantry load. This load is a moving load, concentrated at wheel or roller contact points. Listed is the total weight of gantry.	588 kN	Vertically on both shoulders of the invert	Section 4.3.6
IV	Individual waste package weight	883 kN	Vertically on the inverts.	Section 4.3.6
V	Load range of heat-induced concrete liner's hoop stress (force) that will transfer to the invert in form of axial thrust acting on the liner/invert connection joints.	15 MPa at 200 mm thickness	Hoop direction.	Section 4.3.10

Type	Description	Magnitude	Orientation and Distribution	Sources
VI	Potential earthquake induced dynamic load on the invert.	Acceleration 0.3g Frequency: 10 Hz Peak ground velocity: 16 cm/s Duration 3 sec.	Both P- and S- waves propagate vertically towards the ground surface	Section 4.3.1

Loading Combinations for the Concrete Invert

Scenario 1: Pre-emplacement rock load (I) + TBM Transportation Load (II)

Scenario 2: Pre-emplacement rock load (I) + Gantry Weight (III) with Waste Package Weight (IV)

Scenario 3: Pre-emplacement rock load (I) +Heat-Induced load (V) + Emplaced Waste Package Weight (IV)

Scenario 4: Pre-emplacement rock load (I) + Heat-Induced load (V) + Emplaced Waste Package Weight (IV) + Seismic Load (VI)

Scenario 5: Pre-emplacement rock load (I) + Gantry Weight (III) with Waste Package Weight (IV) + Emplaced Waste Package Weight (IV)

The precast concrete invert configuration is analyzed in Attachment II in accordance with the criteria and assumptions of this analysis. The concrete invert is modeled with the loading conditions above and the results are shown in Section 8.

8. CONCLUSIONS

This analysis is based on existing, unconfirmed input data and the use of any data from this analysis as input to documents supporting construction, fabrication, or procurement is required to be controlled as TBV in accordance with NLP-3-15.

Conclusions and recommendations are presented and shown in the analyses and figures included in attachments. Attachments are summarized below.

8.1 The structural steel invert is analyzed in Attachment I. Figures I-1 through I-7 of Attachment I, show the plan, elevation, sections, and details developed by this analysis for the steel invert and components. The steel invert consists of two major parts:

A. A removable/reusable steel section placed between the steel sets supporting construction access rails, Figures I-1 through I-4. The steel invert for construction access was designed in Attachment I using ASTM-A36 steel materials and consists of the following major components:

- Floor Plate: 3/8 inch thick with 23.9 kPa (500 psf) live load.
- Framing: W8 x 48 with 20 MT wheel load.
- Construction Rail Size: 438 N/m (90 pounds/yard) Attachment I

B. A gantry runway beam and rail anchored to a structural steel support, Figures I-5 through I-7. The gantry runway and steel support were designed in Attachment I using ASTM-A572 steel materials and consists of the following major components:

- Gantry Runway Beam: W8 x 67 with 406.5 kN (91.4 kips) per wheel
- Gantry Rail Size: 657 N/m (135 pounds/yard) Attachment V

The steel invert, as analyzed, is satisfactory for the TBM loads shown in Section 4.3.6.

8.2 The precast and cast-in-place concrete invert is analyzed in Attachment II for the loads shown in Table 1 and concrete stresses were determined. Figure II-3 of Attachment II shows a section through the invert analyzed. Figure II-2 shows an invert configuration based on using a 600 mm diameter wheel which lowers the top of the haunches and demonstrates the feasibility of using a larger wheel diameter. This configuration is bounded by the invert section shown in Figure II-3 and was not analyzed. Using concrete stresses determined in Attachment II, the concrete invert is analyzed in Attachment IV to determine steel reinforcement and concrete compressive strength required. A concrete compressive strength of 62.1 MPa (Section 4.3.4) is suitable. Steel reinforcement is shown in Figure IV-1. Figures IV-2, IV-3, and IV-4 show a plan and details.

The construction rail size is 438 N/m (90 pounds/yard), Attachment I and the gantry rail size is 657 N/m (135 pounds/yard), Attachment V, the same size as used for the steel invert, for this analysis. The precast concrete invert, as analyzed, is satisfactory for the operating loads in Section 4.3.6.

8.3 The configuration of casting-in-place an invert with haunches, Figure 7-8, over an installed precast invert and adjacent ground support, Figure 7-7, involves the additional cost of anchoring the cast-in-place section to the installed precast invert. Anchoring the cast-in-place section is necessary for stability against movement of the cast-in-place section during loading. Transfer of ground support loads to the cast-in-place invert through any anchoring system would most likely over stress the cast-in-place section. No further analysis of this alternative is considered.

8.4 Casting the concrete haunches in-place (Figures 7-9 and 7-10, Attachment II, Part 4, and Figure II-1) on a precast invert appeared more costly than a precast invert that included the haunches because the haunches must be attached to the precast invert with a system of dowels including threaded sleeves installed in the precast invert and steel reinforcement attached to the dowels that are inserted into the sleeves.

8.5 Design of the steel invert supports construction and TBM loads and allows removal and reuse of the invert part placed between the steel sets. The gantry rail structural steel support is bolted to the tunnel floor and conforms to waste package placement centered horizontally in-drift, on pedestals, using gantry emplacement (Section 4.3.1, Key 011 and Key 066).

Design of the precast concrete invert conforms to waste package placement centered horizontally in-drift, on pedestals, using gantry emplacement (Section 4.3.1, Key 011 and Key 066).

8.6 The steel invert, designed to support construction access, is only usable with structural steel ground support and is removed and reused in another drift. The gantry runway beam and rail are installed for gantry support and remain in place for emplacement drift waste emplacement operations.

8.7 Attachment IV analyzes the steel reinforcement required for the precast concrete invert based on a concrete compressive strength of 62.1 MPa. Steel reinforcement required for the precast invert is shown in Figure IV-1 and is compatible with the concrete compressive strength.

9. ATTACHMENTS

ATTACHMENTS	DESCRIPTION
I	Structural Steel Invert
II	Concrete Invert
III	Miscellaneous Reference Data
IV	Reinforced Concrete Design
V	Gantry Rail Design

ATTACHMENT I
STRUCTURAL STEEL INVERT

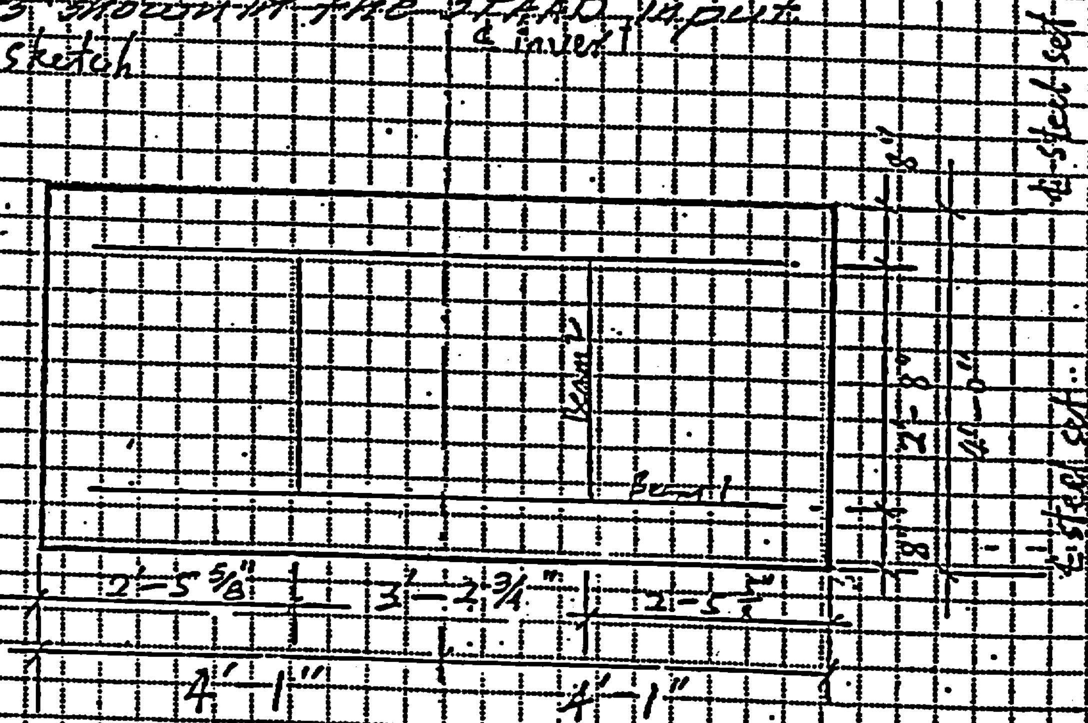
DOE policy requires the subsurface design be performed using metric units. Much source information (e.g., vendor data/steel member sizes) used for design, however, is available only in English units. Because of this, calculations are generally performed in English units. The results are converted to metric units in the main body of the analysis, followed by the corresponding English values in parenthesis.

Floor plate for invert cover for W10 Beam

thickness of plate 3/8" (ASTM-A36)

Load #500/SF (Section 4.3.13)

For computer modeling the plate is supported by the invert beams 1 & 2 at points shown in the STAAD input
See sketch



plan

Max stress is 4.2 ksi < 27.0 ksi = 0.75 Fy OK (AISC p. 5-48)

Max deflection is 0.175 in < L/240 (UBC Table 16-D) OK

$\frac{38\,3/4"}{240} = 0.16"$

See p I-3 to I-9 of I-54 for STAAD III run

Use 3/8" plate

```
****************************************************
*                                                  *
*           S T A A D - III                        *
*           Revision 22.3a                         *
*           Proprietary Program of                 *
*           Research Engineers, Inc.               *
*           Date=    MAR 18, 1998                  *
*           Time=     9:16:11                      *
*                                                  *
*      USER ID: Research Engineers, Inc.           *
****************************************************
```

```
 1. STAAD SPACE COVER PLATE FOR INVERT
 2. UNIT POUND INCH
 3. JOINT COORDINATES
 4. * FILENAME: COVER-PL.STD
 5. * PLATE SIZE 4.1' X 4'(BETWEEN STEELSETS)
 6. * SUPPORTED BY BEAM, CANTILEVERED 5" AT EDGE
 7. * LIVE LOAD 500 LB PER SQUARE FOOT
 8. * MAX ALLOW DEFLECTION L/240
 9. 1  -49 0 0 ;2   -44 0 0 ;3   -19 0 0 ;4 0 0 0
10. 5   19 0 0 ;6    44 0 0 ;7    49 0 0
11. 11 -49 0  8;12 -44 0 8 ;13 -19 0 8 ;14 0 0 8
12. 15  19 0  8;16  44 0 8 ;17  49 0 8
13. 21 -49 0 24;22 -44 0 24;23 -19 0 24;24 0 0 24
14. 25  19 0 24;26  44 0 24;27  49 0 24
15. 31 -49 0 40;32 -44 0 40;33 -19 0 40;34 0 0 40
16. 35  19 0 40;36  44 0 40;37  49 0 40
17. 41 -49 0 48;42 -44 0 48;43 -19 0 48;44 0 0 48
18. 45  19 0 48;46  44 0 48;47  49 0 48
19. ELEMENT INCIDENCE
20. 1 1 2 12 11;2 2 3 13 12;3 3 4 14 13
21. 4 4 5 15 14;5 5 6 16 15;6 6 7 17 16
22. 11 11 12 22 21;12 12 13 23 22;13 13 14 24 23
23. 14 14 15 25 24;15 15 16 26 25;16 16 17 27 26
24. 21 21 22 32 31;22 22 23 33 32;23 23 24 34 33
25. 24 24 25 35 34;25 25 26 36 35;26 26 27 37 36
26. 31 31 32 42 41;32 32 33 43 42;33 33 34 44 43
27. 34 34 35 45 44;35 35 36 46 45;36 36 37 47 46
28. ELEMENT PROPERTY
29. 1 TO 6 11 TO 16 21 TO 26 31 TO 36 THICKNESS 0.375
30. CONSTANT
31. E STEEL ALL
32. DENSITY STEEL ALL
33. SUPPORT
34. 12 TO 16 23 25 32 TO 36  PINNED
35. LOADING 1
36. SELFWEIGHT Y -1.0
37. ELEMENT LOAD
38. 1 TO 6 11 TO 16 21 TO 26 31 TO 36 PRESSURE GY -3.5
39. PERFORM ANALYSIS
```

COVER PLATE FOR INVERT

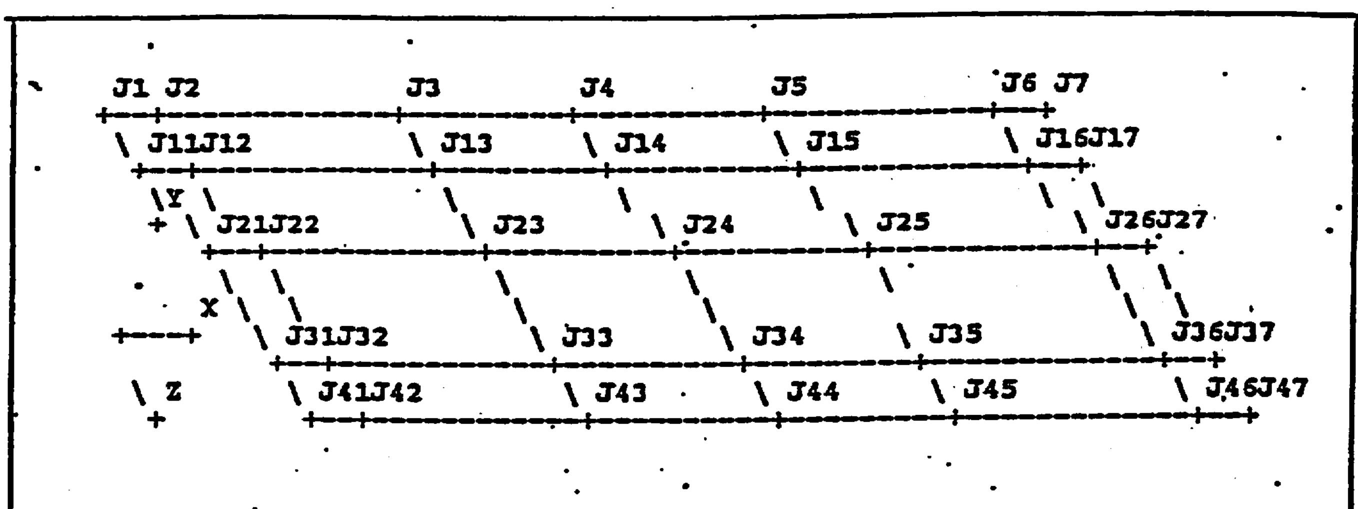
User ID: Research Engineers, Inc.
-- PAGE NO. 2

P R O B L E M   S T A T I S T I C S

```
NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS =     35/    24/    12
ORIGINAL/FINAL BAND-WIDTH =     8/     8
TOTAL PRIMARY LOAD CASES  =     1, TOTAL DEGREES OF FREEDOM =    210
SIZE OF STIFFNESS MATRIX =   11340 DOUBLE PREC. WORDS
REQRD/AVAIL. DISK SPACE =   12.19/ 1181.4 MB,   EXMEM = 1962.9 MB

++ Processing Element Stiffness Matrix.          9:16:11
++ Processing Global Stiffness Matrix.           9:16:12
++ Processing Triangular Factorization.          9:16:12
++ Calculating Joint Displacements.              9:16:12
++ Calculating Member Forces.                    9:16:12

 40. PAREMETER
 41. DMAX 0.133
 42. CODE AISC
 43. PRINT SUPPORT REACTION
```



COVER PLATE FOR INVERT

SUPPORT REACTIONS -UNIT POUN INCH STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
12	1	.00	1550.74	.00	.00	.00	.00
13	1	.00	1286.67	.00	.00	.00	.00
14	1	.00	1589.62	.00	.00	.00	.00
15	1	.00	1286.67	.00	.00	.00	.00
16	1	.00	1550.74	.00	.00	.00	.00
23	1	.00	1217.16	.00	.00	.00	.00
25	1	.00	1217.16	.00	.00	.00	.00
32	1	.00	1550.74	.00	.00	.00	.00
33	1	.00	1286.67	.00	.00	.00	.00
34	1	.00	1589.62	.00	.00	.00	.00
35	1	.00	1286.67	.00	.00	.00	.00
36	1	.00	1550.74	.00	.00	.00	.00

************** END OF LATEST ANALYSIS RESULT **************

44. PRINT ELEMENT STRESS

User ID: Research Engineers, Inc.

```
COVER PLATE FOR INVERT                                   -- PAGE NO.    4

ELEMENT FORCES     FORCE,LENGTH UNITS= POUN INCH
--------------
FORCE OR STRESS = FORCE/WIDTH/THICK, MOMENT = FORCE-LENGTH/WIDTH

ELEMENT  LOAD        QX          QY          MX          MY             MXY
                    VONT        VONB         FX          FY             FXY

   1      1        5.89      -73.09      -38.72      -93.05           18.12
                3704.95     3704.95         .00         .00             .00
  TOP :  SMAX= -1417.76  SMIN= -4204.47  TMAX=  1393.35  ANGLE=  16.9
  BOTT:  SMAX=  4204.47  SMIN=  1417.76  TMAX=  1393.35  ANGLE=  16.9

   2      1       37.36      -29.43      -33.49      -53.02           20.15
                2478.56     2478.56         .00         .00             .00
  TOP :  SMAX=  -890.26  SMIN= -2800.73  TMAX=   955.23  ANGLE=  32.1
  BOTT:  SMAX=  2800.73  SMIN=   890.26  TMAX=   955.23  ANGLE=  32.1

   3      1      -27.19      -41.25        -.59      -54.55            5.60
                2351.66     2351.66         .00         .00             .00
  TOP :  SMAX=     -.59  SMIN= -2351.95  TMAX=  1175.68  ANGLE=   5.9
  BOTT:  SMAX=  2351.95  SMIN=      .59  TMAX=  1175.68  ANGLE=   5.9

   4      1       27.19      -41.25        -.59      -54.55           -5.60
                2351.66     2351.66         .00         .00             .00
  TOP :  SMAX=     -.59  SMIN= -2351.95  TMAX=  1175.68  ANGLE=  -5.9
  BOTT:  SMAX=  2351.95  SMIN=      .59  TMAX=  1175.68  ANGLE=  -5.9

   5      1      -37.36      -29.43      -33.49      -53.02          -20.15
                2478.56     2478.56         .00         .00             .00
  TOP :  SMAX=  -890.26  SMIN= -2800.73  TMAX=   955.23  ANGLE= -32.1
  BOTT:  SMAX=  2800.73  SMIN=   890.26  TMAX=   955.23  ANGLE= -32.1

   6      1       -5.89      -73.09      -38.72      -93.05          -18.12
                3704.95     3704.95         .00         .00             .00
  TOP :  SMAX= -1417.76  SMIN= -4204.46  TMAX=  1393.35  ANGLE= -16.9
  BOTT:  SMAX=  4204.46  SMIN=  1417.76  TMAX=  1393.35  ANGLE= -16.9

  11      1      -39.01      182.19      -14.45       64.34           23.08
                3538.17     3538.17         .00         .00             .00
  TOP :  SMAX=  3012.60  SMIN=  -883.65  TMAX=  1948.13  ANGLE= -15.2
  BOTT:  SMAX=   883.65  SMIN= -3012.60  TMAX=  1948.13  ANGLE= -15.2

  12      1      -12.65       18.02      -22.61       21.35           16.73
                2041.56     2041.56         .00         .00             .00
  TOP :  SMAX=  1151.67  SMIN= -1205.51  TMAX=  1178.59  ANGLE= -18.6
  BOTT:  SMAX=  1205.51  SMIN= -1151.67  TMAX=  1178.59  ANGLE= -18.6

  13      1       18.16       41.33        5.46       -3.67            2.20
                 376.51      376.51         .00         .00             .00
  TOP :  SMAX=   254.35 'SMIN=  -178.18  TMAX=   216.26  ANGLE=  12.9
  BOTT:  SMAX=   178.18 'SMIN=  -254.35  TMAX=   216.26  ANGLE=  12.9

  14      1      -18.16       41.33        5.46       -3.67           -2.20
                 376.51      376.51         .00         .00             .00
  TOP :  SMAX=   254.35  SMIN=  -178.18  TMAX=   216.26  ANGLE= -12.9
  BOTT:  SMAX=   178.18  SMIN=  -254.35  TMAX=   216.26  ANGLE= -12.9
```

COVER PLATE FOR INVERT

ELEMENT FORCES FORCE,LENGTH UNITS= POUN INCH

FORCE OR STRESS = FORCE/WIDTH/THICK, MOMENT = FORCE-LENGTH/WIDTH

ELEMENT	LOAD	QX VONT	QY VONB	MX FX	MY FY	MXY FXY
15	1	12.65	18.02	-22.61	21.35	-16.73
		2041.56	2041.56	.00	.00	.00
TOP :	SMAX=	1151.67	SMIN= -1205.51	TMAX=	1178.59	ANGLE= 18.6
BOTT:	SMAX=	1205.51	SMIN= -1151.67	TMAX=	1178.59	ANGLE= 18.6
16	1	39.01	182.19	-14.45	64.34	-23.08
		3538.17	3538.17	.00	.00	.00
TOP :	SMAX=	3012.60	SMIN= -883.65	TMAX=	1948.12	ANGLE= 15.2
BOTT:	SMAX=	883.65	SMIN= -3012.60	TMAX=	1948.12	ANGLE= 15.2
21	1	-39.01	-182.19	-14.45	64.34	-23.08
		3538.17	3538.17	.00	.00	.00
TOP :	SMAX=	3012.60	SMIN= -883.65	TMAX=	1948.13	ANGLE= 15.2
BOTT:	SMAX=	883.65	SMIN= -3012.60	TMAX=	1948.13	ANGLE= 15.2
22	1	-12.65	-18.02	-22.61	21.35	-16.73
		2041.56	2041.56	.00	.00	.00
TOP :	SMAX=	1151.67	SMIN= -1205.51	TMAX=	1178.59	ANGLE= 18.6
BOTT:	SMAX=	1205.51	SMIN= -1151.67	TMAX=	1178.59	ANGLE= 18.6
23	1	18.16	-41.33	5.46	-3.67	-2.20
		376.51	376.51	.00	.00	.00
TOP :	SMAX=	254.35	SMIN= -178.18	TMAX=	216.26	ANGLE= -12.9
BOTT:	SMAX=	178.18	SMIN= -254.35	TMAX=	216.26	ANGLE= -12.9
24	1	-18.16	-41.33	5.46	-3.67	2.20
		376.51	376.51	.00	.00	.00
TOP :	SMAX=	254.35	SMIN= -178.18	TMAX=	216.26	ANGLE= 12.9
BOTT:	SMAX=	178.18	SMIN= -254.35	TMAX=	216.26	ANGLE= 12.9
25	1	12.65	-18.02	-22.61	21.35	16.73
		2041.56	2041.56	.00	.00	.00
TOP :	SMAX=	1151.67	SMIN= -1205.51	TMAX=	1178.59	ANGLE= -18.6
BOTT:	SMAX=	1205.51	SMIN= -1151.67	TMAX=	1178.59	ANGLE= -18.6
26	1	39.01	-182.19	-14.45	64.34	23.08
		3538.17	3538.17	.00	.00	.00
TOP :	SMAX=	3012.59	SMIN= -883.65	TMAX=	1948.12	ANGLE= -15.2
BOTT:	SMAX=	883.65	SMIN= -3012.59	TMAX=	1948.12	ANGLE= -15.2
31	1	5.89	73.09	-38.72	-93.05	-18.12
		3704.95	3704.95	.00	.00	.00
TOP :	SMAX=	-1417.76	SMIN= -4204.47	TMAX=	1393.35	ANGLE= -16.9
BOTT:	SMAX=	4204.47	SMIN= 1417.76	TMAX=	1393.35	ANGLE= -16.9
32	1	37.36	29.43	-33.49	-53.02	-20.15
		2478.56	2478.56	.00	.00	.00
TOP :	SMAX=	-890.26	SMIN= -2800.73	TMAX=	955.23	ANGLE= -32.1
BOTT:	SMAX=	2800.73	SMIN= 890.26	TMAX=	955.23	ANGLE= -32.1

```
   COVER PLATE FOR INVERT                                          User ID: Research Engineers, Inc.
                                                                   -- PAGE NO.    6

  ELEMENT FORCES    FORCE,LENGTH UNITS= POUN INCH
  --------------
  FORCE OR STRESS = FORCE/WIDTH/THICK, MOMENT = FORCE-LENGTH/WIDTH
 ELEMENT  LOAD          QX           QY          MX          MY           MXY
                       VONT         VONB         FX          FY           FXY

   33      1        -27.19        41.25        -.59      -54.55         -5.60
                   2351.66      2351.66         .00         .00           .00
      TOP :  SMAX=       -.59  SMIN= -2351.95  TMAX=  1175.68  ANGLE=   -5.9
      BOTT:  SMAX=    2351.95  SMIN=      .59  TMAX=  1175.68  ANGLE=   -5.9

   34      1         27.19        41.25        -.59      -54.55          5.60
                   2351.66      2351.66         .00         .00           .00
      TOP :  SMAX=       -.59  SMIN= -2351.95  TMAX=  1175.68  ANGLE=    5.9
      BOTT:  SMAX=    2351.95  SMIN=      .59  TMAX=  1175.68  ANGLE=    5.9

   35      1        -37.36        29.43      -33.49      -53.02         20.15
                   2478.36      2478.56         .00         .00           .00
      TOP :  SMAX=    -890.26  SMIN= -2800.73  TMAX=   955.23  ANGLE=   32.1
      BOTT:  SMAX=    2800.73  SMIN=   890.26  TMAX=   955.23  ANGLE=   32.1

   36      1         -5.89        73.09      -38.72      -93.05         18.12
                   3704.95      3704.95         .00         .00           .00
      TOP :  SMAX=   -1417.76  SMIN= -4204.46  TMAX=  1393.35  ANGLE=   16.9
      BOTT:  SMAX=    4204.46  SMIN=  1417.76  TMAX=  1393.35  ANGLE=   16.9

 ********************END OF ELEMENT FORCES********************

  45. PRINT JOINT DISPLACEMENT
```

JOINT DISPLACEMENT (INCH RADIANS) STRUCTURE TYPE = SPACE

JOINT	LOAD	X-TRANS	Y-TRANS	Z-TRANS	X-ROTAN	Y-ROTAN	Z-ROTAN
1	1	.00000	.03038	.00000	.00239	.00000	.00066
2	1	.00000	.03934	.00000	.00421	.00000	.00167
3	1	.00000	-.01466	.00000	-.00303	.00000	-.00235
4	1	.00000	-.02913	.00000	-.00506	.00000	.00000
5	1	.00000	-.01466	.00000	-.00303	.00000	.00235
6	1	.00000	.03934	.00000	.00421	.00000	-.00167
7	1	.00000	.03038	.00000	.00239	.00000	-.00066
11	1	.00000	-.02093	.00000	.00977	.00000	.00460
12	1	.00000	.00000	.00000	.00730	.00000	.00238
13	1	.00000	.00000	.00000	-.00052	.00000	-.00154
14	1	.00000	.00000	.00000	-.00075	.00000	.00000
15	1	.00000	.00000	.00000	-.00052	.00000	.00154
16	1	.00000	.00000	.00000	.00730	.00000	-.00238
17	1	.00000	-.02093	.00000	.00977	.00000	-.00460
21	1	.00000	-.12524	.00000	.00000	.00000	.00687
22	1	.00000	-.09963	.00000	.00000	.00000	.00669
23	1	.00000	.00000	.00000	.00000	.00000	-.00036
24	1	.00000	-.05656	.00000	.00000	.00000	.00000
25	1	.00000	.00000	.00000	.00000	.00000	.00036
26	1	.00000	-.09963	.00000	.00000	.00000	-.00669
27	1	.00000	-.12524	.00000	.00000	.00000	-.00687
31	1	.00000	-.02093	.00000	-.00977	.00000	.00460
32	1	.00000	.00000	.00000	-.00730	.00000	.00238
33	1	.00000	.00000	.00000	.00052	.00000	-.00154
34	1	.00000	.00000	.00000	.00075	.00000	.00000
35	1	.00000	.00000	.00000	.00052	.00000	.00154
36	1	.00000	.00000	.00000	-.00730	.00000	-.00238
37	1	.00000	-.02093	.00000	-.00977	.00000	-.00460
41	1	.00000	.03038	.00000	-.00239	.00000	.00066
42	1	.00000	.03934	.00000	-.00421	.00000	.00167
43	1	.00000	-.01466	.00000	.00303	.00000	-.00235
44	1	.00000	-.02913	.00000	.00506	.00000	.00000
45	1	.00000	-.01466	.00000	.00303	.00000	.00235
46	1	.00000	.03934	.00000	-.00421	.00000	-.00167
47	1	.00000	.03038	.00000	-.00239	.00000	-.00066

************** END OF LATEST ANALYSIS RESULT **************

46. FINISH

*************** END OF STAAD-III ***************

**** DATE= MAR 18,1998 TIME= 9:16:12 ****

Invert Beams for Transport Dolly Wheel Load

LIVE LOAD WHEEL = 20 MT /PER WHEEL

Do Not CALC. SEISMIC LOAD AT SUPPORT BEAMS.

ADD 25% LIVE LOAD FOR IMPACT.
Use American Society Civil Engineers (ASCE) #90 YARD RAIL (See STAAD analysis p. I-31 of I-54)

WHEEL SPACE = 3' (Section 4.3.6)

USE W8X48 FOR SUPPORT BEAMS (A36M-A36)

BECAUSE LIMIT OF HEIGHT.
ADD 3/8" SHEAR PLATE AT BEAM CONNECTION
CALL "W8X4E"

ADD 2 x 3/8" SHEAR PLATE AT SUPPORT ENDS

Max DEFLECTION = $\frac{L}{1000}$ (LIVE LOAD)
(Ref. 5.7, p 83) $= 0.00329 < \frac{38\frac{3}{4}''}{1000} = 0.03875$

SEE p. I-11 to I-19 of I-54 FOR STAAD III RUN

W8X48 is satisfactory for support beams.
Interaction Ratio is < 1.0

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*****************************************************
*                                                   *
*            S T A A D - III                        *
*            Revision 22.3a                         *
*            Proprietary Program of                 *
*            Research Engineers, Inc.               *
*            Date=    MAR 18, 1998                  *
*            Time=    10:27:57                      *
*                                                   *
*      USER ID: Research Engineers, Inc.            *
*****************************************************
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```
STAAD SPACE W8 BEAM SUPPORT WHEELER OF TBM
* FILENAME: INV-WHEL
UNIT KIP FEET
* 25% IMPACT LOAD
* MAX ALLOWABLE DEFLECTION = L/1000
* LOAD OF WHEELER = 20 MT/PER WHEELER = 44.1K
* WHEEL SPACE = 3'
* ADD 2" X 3/8" SHEAR PLATE AT TAPERED SUPPORT ENDS
* ADD 3/8" SHEAR PLATE AT BEAM CONNECTIONS
* USE ASCE #90 RAIL
* SEISMIC ACCELERATION = 0
JOINT COORDINATES
1 -2.54 0 0;2 -2.04 0 0;3 -1.6 0 0;4 -1.2 0 0
5 0 0 0;6 1.2 0 0;7 1.6 0 0;8 2.04 0 0;9 2.54 0 0
11 -2.54 0 2.67;12 -2.04 0 2.67;13 -1.6 0 2.67;14 -1.2 0 2.67
15 0 0 2.67;16 1.2 0 2.67;17 1.6 0 2.67;18 2.04 0 2.67;19 2.54 0 2.67
21 -1.6 0 0.5;22 -1.6 0 2.17
31 1.6 0 0.5;32 1.6 0 2.17
MEMBER INCIDENCES
1 1 2;2 2 3;3 3 4;4 4 5;5 5 6;6 6 7;7 7 8;8 8 9
11 11 12;12 12 13;13 13 14;14 14 15;15 15 16
16 16 17;17 17 18;18 18 19
21 3 21;22 21 22;23 22 13
31 7 31;32 31 32;33 32 17
UNIT INCH
SUPPORT
1 9 11 19 PINNED
CONSTANT
E STEEL ALL
DENSITY STEEL ALL
START USER TABLE
TABLE 1
WIDE FLANGE
W8X45
*ADD 3/8" PLATE TO INCREASE SHEAR CAPACITY
16.39 8.5 0.77 8.11 0.685 184 60.9 1.96 0 0
END
MEMBER PROPERTIES
4 5 14 15 22 32 TABLE ST W8X48
1 8 11 18 TAPERED 8.5 1.15 6.53 8.11 0.685
2 3 6 7 12 13 16 17 21 23 31 33 UPT 1 W8X45
```

```
* FILENAME: INV-WHEL
42. UNIT KIP FEET
43. LOADING 1
44. SELFWEIGHT Y -1.0
45. MEMBER LOAD
46. 21 TO 23  31 TO 33  UNI GY -0.03
47. LOADING 2 CASE1
48. MEMBER LOAD
49. 22 32 CON GY -44.1 0.83
50. LOADING 3 CASE2
51. MEMBER LOAD
52. 21 31 CON GY -44.1 0.1
53. JOINT LOAD
54. 13 17 FY -33.1
55. LOADING 4 CASE3
56. JOINT LOAD
57. 13 FY -33.1
58. 17 FY -33.1
59. 3  FY -44.1
60. 7  FY -44.1
61. LOAD COMBINATION 11 STATIC LOADING CASE1
62. 1 1.0 2 1.0
63. LOAD COMBINATION 12 STATIC LOADING CASE2
64. 1 1.0 3 1.0
65. LOAD COMBINATION 13 IMPACT LOADING CASE1
66. 1 1.0 2 1.25
67. LOAD COMBINATION 14 IMPACT LOADING CASE2
68. 1 1.0 3 1.25
69. LOAD COMBINATION 15 IMPACT LOADING CASE3
70. 1 1.0 4 1.25
71. PERFORM ANALYSIS

      P R O B L E M   S T A T I S T I C S
      -----------------------------------

   NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS =     22/    22/     4
   ORIGINAL/FINAL BAND-WIDTH =    16/     4
   TOTAL PRIMARY LOAD CASES  =     4, TOTAL DEGREES OF FREEDOM =    120
   SIZE OF STIFFNESS MATRIX =    3600 DOUBLE PREC. WORDS
   REQRD/AVAIL. DISK SPACE =  12.86/ 1180.9 MB,  EXMEM = 1962.9 MB

++ Processing Element Stiffness Matrix.                 10:27:58
++ Processing Global Stiffness Matrix.                  10:27:58
++ Processing Triangular Factorization.                 10:27:58
++ Calculating Joint Displacements.                     10:27:58
++ Calculating Member Forces.                           10:27:58

  72. UNIT INCH
  73. PARAMETERS
  74. CODE AISC
  75. PROFILE W8 ALL
  76. PRINT SUPPORT REACTION
```

User ID: Research Engineers, Inc.
W8 BEAM SUPPORT WHEELER OF TBM -- PAGE NO. 3

* FILENAME: INV-WHEL

SUPPORT REACTIONS -UNIT KIP INCH STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	.00	.24	.00	.00	.00	.00
	2	.00	22.13	.00	.00	.00	.00
	3	.00	42.45	.00	.00	.00	.00
	4	.00	44.10	.00	.00	.00	.00
	11	.00	22.38	.00	.00	.00	.00
	12	.00	42.69	.00	.00	.00	.00
	13	.00	27.91	.00	.00	.00	.00
	14	.00	53.30	.00	.00	.00	.00
	15	.00	55.37	.00	.00	.00	.00
9	1	.00	.24	.00	.00	.00	.00
	2	.00	22.13	.00	.00	.00	.00
	3	.00	42.45	.00	.00	.00	.00
	4	.00	44.10	.00	.00	.00	.00
	11	.00	22.38	.00	.00	.00	.00
	12	.00	42.69	.00	.00	.00	.00
	13	.00	27.91	.00	.00	.00	.00
	14	.00	53.30	.00	.00	.00	.00
	15	.00	55.37	.00	.00	.00	.00
11	1	.00	.24	.00	.00	.00	.00
	2	.00	21.97	.00	.00	.00	.00
	3	.00	34.75	.00	.00	.00	.00
	4	.00	33.10	.00	.00	.00	.00
	11	.00	22.21	.00	.00	.00	.00
	12	.00	34.99	.00	.00	.00	.00
	13	.00	27.70	.00	.00	.00	.00
	14	.00	43.68	.00	.00	.00	.00
	15	.00	41.62	.00	.00	.00	.00
19	1	.00	.24	.00	.00	.00	.00
	2	.00	21.97	.00	.00	.00	.00
	3	.00	34.75	.00	.00	.00	.00
	4	.00	33.10	.00	.00	.00	.00
	11	.00	22.21	.00	.00	.00	.00
	12	.00	34.99	.00	.00	.00	.00
	13	.00	27.70	.00	.00	.00	.00
	14	.00	43.68	.00	.00	.00	.00
	15	.00	41.62	.00	.00	.00	.00

************** END OF LATEST ANALYSIS RESULT **************

77. PRINT SECTION MAX DISPLACEMENT

W8 BEAM SUPPORT WHEELER OF TBM

* FILENAME: INV-WHEL

MAX MEMBER SECTION DISPLACEMENTS

UNIT= INCH FOR FPS AND CM FOR METRIC/SI SYSTEM

MEMBER	MAX DISP	LOCATION	LOAD	L/DISPL
1	.00020	3.50	15	29369
2	.00031	2.64	15	16906
3	.00034	2.40	15	14253
4	.00303	7.20	15	4747
5	.00303	7.20	15	4747
6	.00034	2.40	15	14253
7	.00031	2.64	15	16907
8	.00018	2.50	15	33821
11	.00016	3.50	14	37227
12	.00025	2.64	14	21431
13	.00027	2.40	14	18041
14	.00240	7.20	14	6008
15	.00240	7.20	14	6008
16	.00027	2.40	14	18041
17	.00025	2.64	14	21432
18	.00014	2.50	14	42870
21	.00007	3.50	13	0
22	.00329	10.02	13	6085
23	.00007	2.50	13	0
31	.00007	3.50	13	0
32	.00329	10.02	13	6085
33	.00007	2.50	13	0

************ END OF SECT DISPL RESULTS ***********

78. PRINT MAXFORCE ENV NSE 4

W8 BEAM SUPPORT WHEELER OF TBM

* FILENAME: INV-WHEL

MEMBER FORCE ENVELOPE

ALL UNITS ARE KIP INCH

MAX AND MIN FORCE VALUES AMONGST ALL SECTION LOCATIONS

MEMB		FY/ FZ	DIST DIST	LD LD	MZ/ MY	DIST DIST	LD LD	FX	DIST	LD
1	MAX	55.37	.00	15	.00	.00	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	.21	6.00	1	-332.12	6.00	15			
		.00	6.00	15	.00	6.00	15	.00	6.00	15
2	MAX	55.34	.00	15	-1.37	.00	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	.19	5.28	1	-624.24	5.28	15			
		.00	5.28	15	.00	5.28	15	.00	5.28	15
3	MAX	.08	.00	14	-2.42	.00	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	.00	4.80	2	-624.08	4.80	15			
		.00	4.80	15	.00	4.80	15	.00	4.80	15
4	MAX	.06	.00	1	-2.75	.00	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	.00	14.40	14	-624.49	14.40	15			
		.00	14.40	15	.00	14.40	15	.00	14.40	15
5	MAX	.00	.00	1	-2.75	14.40	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-.06	14.40	15	-624.49	.00	15			
		.00	14.40	15	.00	14.40	15	.00	14.40	15
6	MAX	.00	.00	3	-2.42	4.80	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-.08	4.80	15	-624.08	.00	15			
		.00	4.80	15	.00	4.80	15	.00	4.80	15
7	MAX	-.19	.00	1	-1.37	5.28	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-55.34	5.28	15	-624.24	.00	15			
		.00	5.28	15	.00	5.28	15	.00	5.28	15
8	MAX	-.21	.00	1	.00	6.00	13			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-55.37	6.00	15	-332.12	.00	15			
		.00	6.00	15	.00	6.00	15	.00	6.00	15
11	MAX	43.68	.00	14	.00	.00	13			
		.00	.00	1	.00	.00	1	.00	.00	1

• FILENAME: INV-WHEL

	MIN	.21	6.00	1	-262.01	6.00	14			
		.00	6.00	15	.00	6.00	15	.00	6.00	15
12	MAX	43.65	.00	14	-1.37	.00	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	.19	5.28	1	-492.43	5.28	14			
		.00	5.28	15	.00	5.28	15	.00	5.28	15
13	MAX	.08	.00	15	-2.42	.00	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	.00	4.80	2	-493.10	4.80	14			
		.00	4.80	15	.00	4.80	15	.00	4.80	15
14	MAX	.06	.00	1	-2.75	.00	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	.00	14.40	13	-493.51	14.40	14			
		.00	14.40	15	.00	14.40	15	.00	14.40	15
15	MAX	.00	.00	15	-2.75	14.40	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-.06	14.40	14	-493.51	.00	14			
		.00	14.40	15	.00	14.40	15	.00	14.40	15
16	MAX	.00	.00	4	-2.42	4.80	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-.08	4.80	13	-493.10	.00	14			
		.00	4.80	15	.00	4.80	15	.00	4.80	15
17	MAX	-.19	.00	1	-1.37	5.28	1			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-43.65	5.28	14	-492.43	.00	14			
		.00	5.28	15	.00	5.28	15	.00	5.28	15
18	MAX	-.21	.00	1	.00	6.00	15			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-43.68	6.00	14	-262.01	.00	14			
		.00	6.00	15	.00	6.00	15	.00	6.00	15
21	MAX	53.17	.00	14	.00	.00	14			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-2.00	6.00	14	-166.51	6.00	13			
		.00	6.00	15	.00	6.00	13	.00	6.00	15
22	MAX	27.73	.00	13	.00	20.04	4			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-27.52	20.04	13	-440.74	10.02	13			
		.00	20.04	15	.00	20.04	15	.00	20.04	15
23	MAX	.00	.00	4	.00	.00	4			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-27.57	6.00	13	-165.27	.00	13			
		.00	6.00	15	.00	6.00	15	.00	6.00	15
31	MAX	53.17	.00	14	.00	4.50	4			
		.00	.00	1	.00	.00	1	.00	.00	1

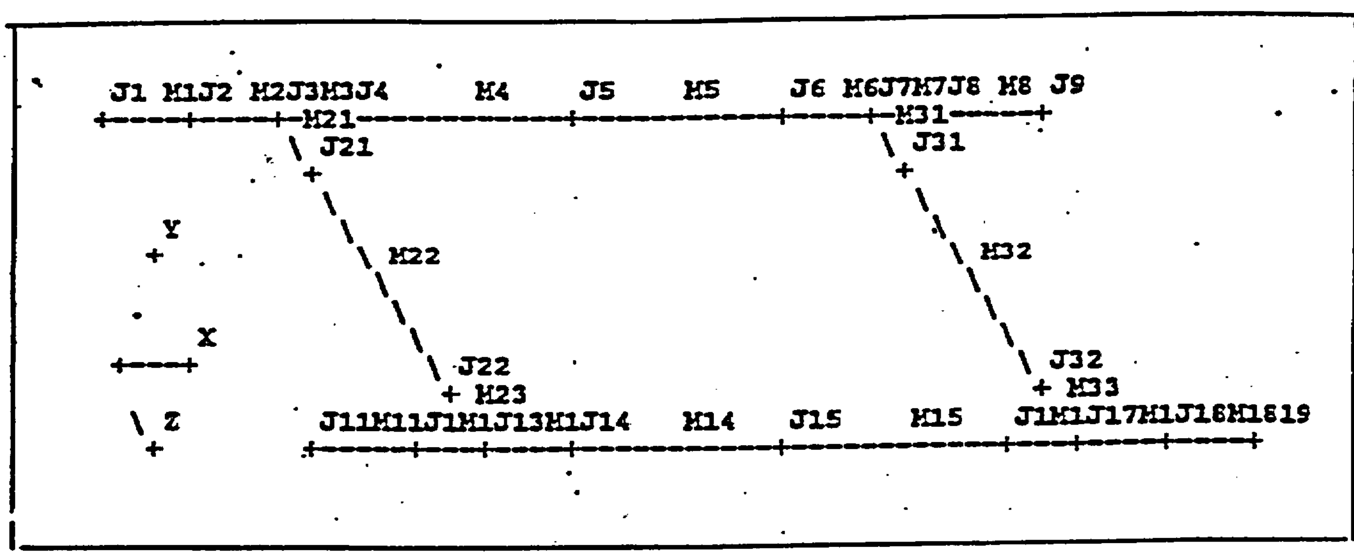
WS BEAM SUPPORT WHEELER OF TBM

* FILENAME: INV-WHEL

	MIN	-2.00	6.00	14	-166.51	6.00	13				
		.00	6.00	15	.00	6.00	15	.00	6.00	15	
32	MAX	27.73	.00	13	.00	.00	4				
		.00	.00	1	.00	.00	1	.00	.00	1	
	MIN	-27.52	20.04	13	-440.74	10.02	13				
		.00	20.04	15	.00	20.04	15	.00	20.04	15	
33	MAX	.00	.00	4	.00	4.50	4				
		.00	.00	1	.00	.00	1	.00	.00	1	
	MIN	-27.57	6.00	13	-165.28	.00	13				
		.00	6.00	15	.00	6.00	15	.00	6.00	15	

********** END OF FORCE ENVELOPE FROM INTERNAL STORAGE **********

79. CHECK CODE



W8 BEAM SUPPORT WHEELER OF TBM

User ID: Research Engineers, Inc.
-- PAGE NO. 8

* FILENAME: INV-WHEL

STAAD-III CODE CHECKING - (AISC)
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE NOTED)

MEMBER		TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1		TAP ERED	PASS	SHEAR -Y	.512	15
			.00	.00	-332.12	6.00
2	ST	W8X45	PASS	AISC- H1-3	.607	15
			.00	.00	-624.24	5.28
3	ST	W8X45	PASS	AISC- H1-3	.607	15
			.00	.00	-624.08	4.80
4	ST	W8X 48	PASS	AISC- H1-3	.607	15
			.00	.00	-624.49	14.40
5	ST	W8X 48	PASS	AISC- H1-3	.607	15
			.00 C	.00	-624.49	.00
6	ST	W8X45	PASS	AISC- H1-3	.607	15
			.00 C	.00	-624.08	.00
7	ST	W8X45	PASS	AISC- H1-3	.607	15
			.00 C	.00	-624.24	.00
8		TAP ERED	PASS	SHEAR -Y	.512	15
			.00	.00	.00	6.00
11		TAP ERED	PASS	SHEAR -Y	.404	14
			.00	.00	-262.01	6.00
12	ST	W8X45	PASS	AISC- H1-3	.479	14
			.00	.00	-492.43	5.28
13	ST	W8X45	PASS	AISC- H1-3	.479	14
			.00	.00	-493.10	4.80
14	ST	W8X 48	PASS	AISC- H1-3	.480	14
			.00	.00	-493.51	14.40
15	ST	W8X 48	PASS	AISC- H1-3	.480	14
			.00 C	.00	-493.51	.00
16	ST	W8X45	PASS	AISC- H1-3	.479	14
			.00 C	.00	-493.10	.00
17	ST	W8X45	PASS	AISC- H1-3	.479	14
			.00 C	.00	-492.43	.00
18		TAP ERED	PASS	SHEAR -Y	.404	14
			.00	.00	.00	6.00
21	ST	W8X45	PASS	SHEAR -Y	.225	14
			.00 C	.00	.00	.00
22	ST	W8X 48	PASS	SHEAR -Y	.566	13
			.00 C	.00	-166.51	.00
23	ST	W8X45	PASS	AISC- H1-3	.161	13
			.00 C	.00	-165.27	.00
31	ST	W8X45	PASS	SHEAR -Y	.225	14
			.00 C	.00	.00	.00
32	ST	W8X 48	PASS	SHEAR -Y	.566	13
			.00 C	.00	-166.51	.00

W8 BEAM SUPPORT WHEELER OF TBM

* FILENAME: INV-WHEL

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
33	ST W8X45	PASS	AISC- H1-3	.161	13
		.00 C	.00	-165.28	.00

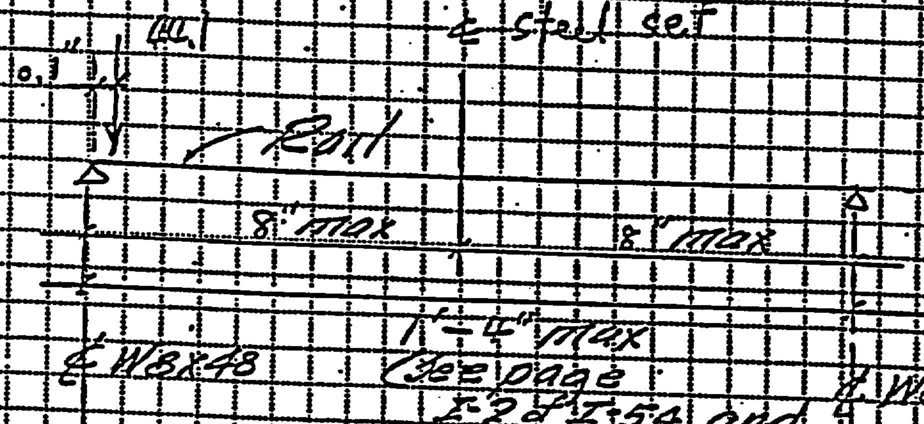
80. FINISH

*************** END OF STAAD-III ***************

**** DATE= MAR 18,1998 TIME= 10:27:58 ****

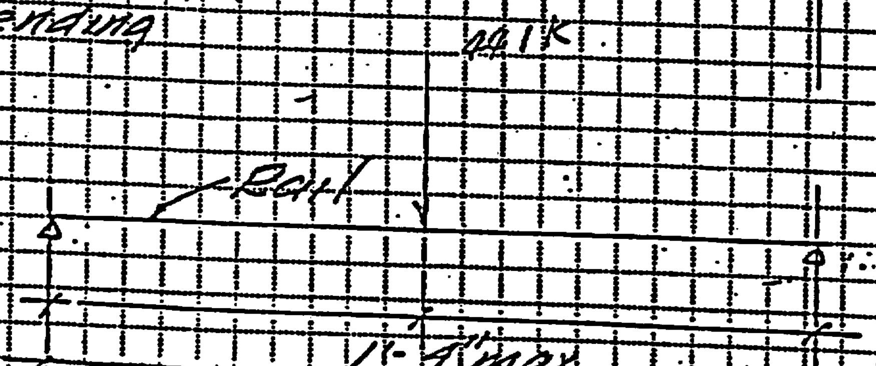
ASCE #90/yard rail: Analyze rail for TBM wheel loads and 25% Impact load, conservative for a one time operation. (Section 4.3.6)

Load 1 TBM Wheel = 44.1 K (= 20 MT)
at edge for shear



1'-4" max (See page I-2 of I-54 and Figures I-1 to I-7 pages I-48 to I-54)

Load 2
at center for bending



Maximum Deflection = L/1000
= 0.00582" < 16"/1000 = 0.016

ASCE #90/yard rail is satisfactory for wheel support with 1'4" span. Interaction ratio < 1.0.
See p I-21 to I-26 of I-54 STAAD III RUN

```
***************************************************
*                                                 *
*              S T A A D - III                    *
*              Revision 22.3a                     *
*              Proprietary Progra  of             *
*              Research Engineers, Inc.           *
*              Date=    APR  6, 1998              *
*              Ti e=    15:15:34                  *
*                                                 *
*      USER ID: Research Engineers, Inc.          *
***************************************************
```

```
STAAD SPACE RAIL AT INVERT FOR TBM
* FILENAME: RAIL.STD
UNIT KIP INCH
* TBM WEIGHT = 20 MT/PER ROLLER
* 25  IMPACT LOAD
* MAX DEFLECTION = L/1000
JOINT COORDINATES
1 0 0 0;2 16 0 0
MEMBER INCIDENCES
1 1 2
SUPPORT
1 2  PINNED
CONSTANT
E STEEL ALL
DENSITY STEEL ALL
START  USER TABLE
TABLE 1
ISECTION
ASCE-90
5.37 0.56 5.37 2.62 1.30 5.37 0.31 0   0 0
END
MEMBER PROPERTY
1 UPT 1 ASCE-90
LOADING 1
SELFWEIGHT Y -1.0
LOADING 2 LOAD AT EDGE FOR SHEAR
MEMBER LOAD
1 CON GY -44.1 0.1
LOADING 3 LOAD AT CENTER FOR MOMENT
MEMBER  LOAD
1 CON GY -44.1 8
LOAD COMBINATION 11 STATIC LOADING CASE1
1 1.0 2 1.0
LOAD COMBINATION 12 STATIC LOADING CASE2
1 1.0 3 1.0
LOAD COMBINATION 13 IMPACT LOADING CASE1
1 1.0 2 1.25
LOAD COMBINATION 14 IMPACT LOADING CASE2
1 1.0 3 1.25
PERFORM ANALYSIS
```

```
                                                              User ID: Research Engineers, Inc.
 RAIL AT INVERT FOR TBM                                       -- PAGE NO.     2

 * FILENAME: RAIL.STD

        P R O B L E M   S T A T I S T I C S
        -----------------------------------

   NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS =      2/      1/      2
   ORIGINAL/FINAL BAND-WIDTH =      1/      1
   TOTAL PRIMARY LOAD CASES  =      3, TOTAL DEGREES OF FREEDOM =       6
   SIZE OF STIFFNESS MATRIX =       36 DOUBLE PREC. WORDS
   REQRD/AVAIL. DISK SPACE =   12.00/ 1183.3 MB,   EXMEM = 1969.2 MB

++ Processing Element Stiffness Matrix.                        15:15:35
++ Processing Global Stiffness Matrix.                         15:15:35
++ Processing Triangular Factorization.                        15:15:35

 ***WARNING - IMPROPER LOAD WILL CAUSE INSTABILITY AT JOINT      2
   DIRECTION = MX  PROBABLE CAUSE MODELING PROBLEM       .000E+00
++ Calculating Joint Displacements.                            15:15:35
++ Calculating Member  Forces.                                 15:15:35

   41. SECTION  0.5  ALL
   42. PARAMETERS
   43. CODE AISC
   44. UNL 1.0 ALL
   45. FYL 40.0 ALL
   46. PRINT SECTION MAX DISPLACEMENT
```

RAIL AT INVERT FOR TBM

* FILENAME: RAIL.STD

MAX MEMBER SECTION DISPLACEMENTS

UNIT= INCH FOR FPS AND CM FOR METRIC/SI SYSTEM

MEMBER	MAX DISP	LOCATION	LOAD	L/DISPL
1	.00582	8.00	14	2746

************ END OF SECT DISPL RESULTS ***********

47. PRINT MAXFORCE ENV NS 12

User ID: Research Engineers, Inc.
-- PAGE NO. 4

RAIL AT INVERT FOR TBM

* FILENAME: RAIL.STD

MEMBER FORCE ENVELOPE

ALL UNITS ARE KIP INCH

MAX AND MIN FORCE VALUES AMONGST ALL SECTION LOCATIONS

MEMB		FY/ FZ	DIST DIST	LD LD	MZ/ MY	DIST DIST	LD LD	FX	DIST	LD
1	MAX	54.80	.00	13	.00	16.00	14			
		.00	.00	1	.00	.00	1	.00	.00	1
	MIN	-27.58	16.00	14	-220.56	8.00	14			
		.00	16.00	14	.00	16.00	14	.00	16.00	14

********** END OF FORCE ENVELOPE FROM INTERNAL STORAGE **********

48. PRINT SUPPORT REACTION

RAIL AT INVERT FOR TBM

* FILENAME: RAIL.STD

SUPPORT REACTIONS -UNIT KIP INCH STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	.00	.02	.00	.00	.00	.00
	2	.00	43.82	.00	.00	.00	.00
	3	.00	22.05	.00	.00	.00	.00
	11	.00	43.84	.00	.00	.00	.00
	12	.00	22.07	.00	.00	.00	.00
	13	.00	54.80	.00	.00	.00	.00
	14	.00	27.58	.00	.00	.00	.00
2	1	.00	.02	.00	.00	.00	.00
	2	.00	.28	.00	.00	.00	.00
	3	.00	22.05	.00	.00	.00	.00
	11	.00	.29	.00	.00	.00	.00
	12	.00	22.07	.00	.00	.00	.00
	13	.00	.36	.00	.00	.00	.00
	14	.00	27.58	.00	.00	.00	.00

************** END OF LATEST ANALYSIS RESULT **************

49. CHECK CODE

User ID: Research Engineers, Inc.

RAIL AT INVERT FOR TBM -- PAGE NO. 6

* FILENAME: RAIL.STD

STAAD-III CODE CHECKING - (AISC)
****

ALL UNITS ARE - KIP INCH (UNLESS OTHERWISE NOTED)

MEMBER	TABLE	RESULT/ FX	CRITICAL COND/ MY	RATIO/ MZ	LOADING/ LOCATION
1	ST ASCE-90	PASS	AISC- H1-3	.963	14
		.00 C	.00	-220.56	8.00

50. FINISH

*************** END OF STAAD-III ***************

**** DATE= APR 6,1998 TIME= 15:15:35 ****

Design Figures:

See design figures for construction access invert and gantry structural support framing, pages I-48 thru I-54. Construction access invert framing is unchanged from Rev. 00. Gantry structural support framing has been modified with revised structural calculation included.

Calculation Purpose:

Calculate new Gantry Support Reactions due to increase Gantry Dead load and Waste Package Load. Gantry reactions will be used to design gantry runway support structure.

Data:

The STAAD-III computer analysis, Ref. 5.18, file GANTRY-H. Maximum Gantry Dead Load and Waste Package Load used:

Gantry Dead Load = 45 MT
Waste Package Load = 69 MT

Design Procedure:

Determine maximum gantry support reaction to crane runway beam by scaling up Ref. 5.18, file GANTRY-H, support reactions. The new Gantry dead weight is estimated by proportioning the new and original Waste Package Loads. The maximum gantry support reaction will then be calculated by proportioning the new and original gantry dead load + Waste Package Load. The new Waste Package Load is:

Waste Package Load = 85 MT (Section 4.3.6)

Calculate new Gantry Dead Load (GDL):

GDL = Original Gantry Dead Load x New Waste Package Load / Original Waste Package Load

$$\text{GDL} = 45 \cdot \frac{85}{69} = 55.435$$

Use Gantry Dead Load = 60 MT (Section 4.3.6)

Calculate scale factor to be applied to original GANTRY-H support reactions:

$$\text{SRSF} = \frac{85 + 60}{(69 + 45)} = 1.272$$

kip = 1000·lbf Mathcad unit definition

Maximum support reactions occur for Load Case 102 at joint 118 from Ref. 5.18, file GANTRY-H:

$Fx := 38.74 \cdot kip$

$Fy := 145.74 \cdot kip$

$Fz := 34.14 \cdot kip$

Scale up GANTRY-H, support reactions at joint 118, Load Case102:

$Fx_1 := 1.272 \cdot Fx$ $\quad Fy_1 := 1.272 \cdot Fy$ $\quad Fz_1 := 1.272 \cdot Fz$

$Fx_1 = 49.277 \cdot kip$ $\quad Fy_1 = 185.381 \cdot kip$ $\quad Fz_1 = 43.426 \cdot kip$

Determine Maximum wheel load forces for runway beam design, 2 wheels per column reaction:

Maximum wheel load X- direction $MWL_x := \frac{Fx_1}{2}$, $MWL_x = 24.639 \cdot kip$

Maximum wheel load Y- direction $MWL_y := \frac{Fy_1}{2}$, $MWL_y = 92.691 \cdot kip$

Maximum wheel load Z - direction $MWL_z := \frac{Fz_1}{2}$, $MWL_z = 21.713 \cdot kip$

Runway beam design parameters:

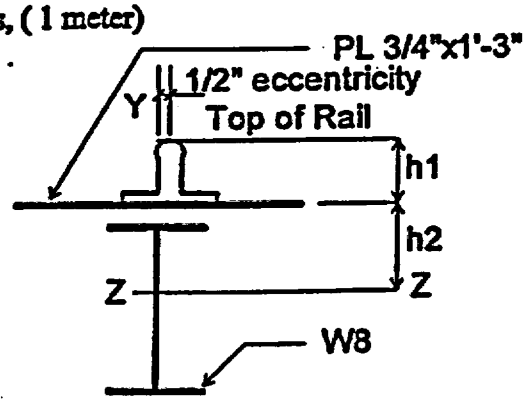
Wheel spacing, Ref. 5.14, Figure 7.4.2 = 39.4 inches, (1 meter)

Trial Sections: W8 Beam and Plate 3/4"x 1'-3" section.

Try h1 + h2 = 8.853 inches

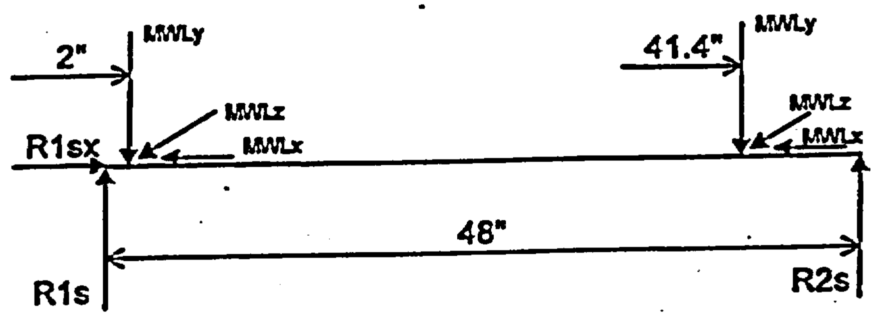
$h1 := 5.375 \cdot in$

$h2 := 3.478 \cdot in$



Calculate runway beam maximum shear, moment and reactions

Maximum Beam Shear : Position wheel load 2 inches from centerline of support for calculation of maximum shear, neglect beam weight :



$R1s_y := MWL_y \cdot \left[\left(\frac{46}{48}\right) + \left(\frac{6.6}{48}\right)\right]$, $R1s_y = 101.573 \cdot kip$

$R2s_y := MWL_y \cdot \left[\left(\frac{2}{48}\right) + \left(\frac{41.4}{48}\right)\right]$, $R2s_y = 83.808 \cdot kip$

$R1s_z := MWL_z \cdot \left[\left(\frac{46}{48}\right) + \left(\frac{6.6}{48}\right)\right]$, $R1s_z = 23.794 \cdot kip$

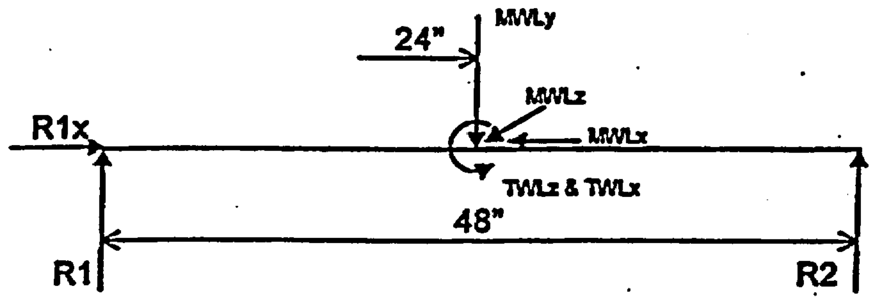
$R2s_z := MWL_z \cdot \left[\left(\frac{2}{48}\right) + \left(\frac{41.4}{48}\right)\right]$, $R2s_z = 19.632 \cdot kip$

$R1s_x := MWL_x \cdot 2$, $R1s_x = 49.277 \cdot kip$

Note: Analysis uses simple span condition for calculation of shears and moments. Runway beam is actually continuous with Simple support occurring at beam splices, 40'-0" on center. Analysis is conservative for flexure, however beam design is controlled by shear.

Determine maximum flexural moments about the X and Y axis, neglect beam weight:

Maximum eccentricity for vertical wheel load $e := 0.5 \cdot in$



$MWL_x = 24.639 \cdot kip$, $MWL_y = 92.691 \cdot kip$, $MWL_z = 21.713 \cdot kip$

$TWL_z := (h1 + h2) \cdot MWL_x$, $TWL_z = 18.177 \cdot ft \cdot kip$, $TWL_x := MWL_y \cdot e$

Sum moments about R2 about Z axis:

$R1_y := \dfrac{MWL_y \cdot 2 \cdot ft + TWL_z}{4 \cdot ft}$, $R1_y = 50.89 \cdot kip$

Sum moments about R1 about Z axis:

$R2_y := \dfrac{MWL_y \cdot 2 \cdot ft - TWL_z}{4 \cdot ft}$, $R2_y = 41.801 \cdot kip$

Maximum midspan moment about Z axis:

$TZ_{max} := R1_y \cdot 24 \cdot in$, $TZ_{max} = 1221.351 \cdot in \cdot kip$

Sum moments about R2 about Y axis:

$R1_z := \left(\dfrac{MWL_z}{2}\right)$, $R1_z = 10.857 \cdot kip$

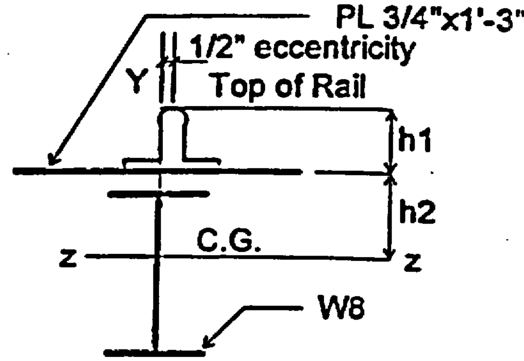
Sum moments about R1 about Z axis:

$R2_z := \dfrac{MWL_z}{2}$, $R2_z = 10.857 \cdot kip$

Maximum midspan moment about Y axis:

$TY_{max} := R1_z \cdot 24 \cdot in$, $TY_{max} = 260.556 \cdot in \cdot kip$

Runway beam Design Data:



Maximum Y- Axis Beam Shear: $R1s_y = 101.573 \cdot kip$

Maximum Z- Axis Beam Shear: $R1s_z = 23.794 \cdot kip$

Maximum Moment about the Z-Axis: $TZ_{max} = 101.779 \cdot ft \cdot kip$

Maximum Moment about the Y-Axis: $TY_{max} = 260.556 \cdot in \cdot kip$

Axial Load from 2 wheels $P_a := 2 \cdot MWL_x$ $P_a = 49.277 \cdot kip$

Plate Section Properties: Try PL 3/4"x 1'-3" $F_{yc} := 50 \cdot \frac{kip}{in^2}$

$A_c := 11.25 \cdot in^2$ $d_c := 15 \cdot in$ $t_{wc} := 0.75 \cdot in$ $x_c := .375 \cdot in$ $I_{xc} := 210 \cdot in^4$ $I_{yc} := .52 \cdot in^4$

Wide Flange Section Properties: Try W8x67 $F_{yw} := 50 \cdot \frac{kip}{in^2}$

$A := 19.7 \cdot in^2$ $d := 9.0 \cdot in$ $t_w := .57 \cdot in$ $I_x := 272 \cdot in^4$ $t_f := .935 \cdot in$ $b_f := 8.28 \cdot in$

$I_{wy} := 88.6 \cdot in^4$

Calculate nuetral axis location of composite section:

$$Y := \frac{A_c \cdot (d + t_{wc} - x_c) + A \cdot \frac{d}{2}}{A_c + A} \qquad Y = 6.272 \cdot in$$

Calculate Strong Axis moment of inertia:

$$I_z := I_x + I_{yc} + A_c \cdot (d + t_{wc} - x_c - Y)^2 + A \cdot \left(Y - \frac{d}{2}\right)^2 \qquad I_z = 442.7 \cdot in^4$$

Calculate section modulus for top (St) and (Sb) flanges:

$$S_t := \frac{I_z}{(d + t_{wc} - Y)} \qquad S_t = 127.286 \cdot in^3$$

$$S_b := \frac{I_z}{Y} \qquad S_b = 70.583 \cdot in^3$$

Calculate weak axis moment of inertia Iwy for W8 and cap plate:

$$I_{yy} := I_{wy} + I_{xc} \qquad I_{yy} = 298.6 \cdot in^4$$

Calculate moment of inertia of top flange and cap plate about the weak axis of W section (Iy):

$I_{yt} := I_{xc} + \frac{b_f^3}{12} \cdot t_f$ $\qquad I_{yt} = 254.23 \cdot in^4$

Section modulus of top flange and cap plate:

$S_{yt} := \frac{I_{yt}}{\left(\frac{d_c}{2}\right)}$ $\qquad S_{yt} = 33.897 \cdot in^3$

Runway Beam Design, Ref. 5.7

Check Y-axis beam shear: $f_{vy} := \frac{R1s_y}{t_w \cdot d}$ $\qquad f_{vy} = 19.8 \cdot \frac{kip}{in^2}$ $\qquad$ < Fvy = 0.4*Fy = 20 ksi AISC, Eq. F4-1, page 5-49 (Shear Ok)

Check Z-axis beam shear: $f_{vz} := \frac{R1s_z}{t_{wc} \cdot d_c}$ $\qquad f_{vz} = 2.115 \cdot \frac{kip}{in^2}$ $\qquad$ < Fvy = 0.4*Fy = 20 ksi AISC, Eq. F4-1, page 5-49 (Shear Ok)

Calculate additional moment about Y-axis due to 1/2" eccentricity. Use flexural analogy, Ref 5.20, page 410 :

Beam Span $\quad L := 48 \cdot in$

$TY1 := \frac{TWL_x}{(d + t_{wc})} \cdot \frac{L}{4}$ $\qquad TY1 = 57.04 \cdot in \cdot kip$

Calculate localized top flange stress due to bending under wheel load:

Use 135 lb crane rail, see Attachment V, page V-10, 90lb rail was used to this point. Difference is negligible to the preceeding calculations.

Maximum wheel load: $\quad MWL_y = 92.691 \cdot kip$

Moment of inertia of rail section: $\quad I_{rail} := 50.8 \cdot in^4$ $\quad$ AISC, Table page 1-113 $\qquad t_{fl} := t_f + t_{wc}$

Moment of inertia of top flange: $\quad I_{tf} := \left(\frac{t_{fl}^3}{12}\right) \cdot b_f$ $\qquad I_{tf} = 3.301 \cdot in^4$

Using bf in computing Itf is conservative.

Calculate localized top flange stress due to bending under wheel load: Continued-

$t_1 := 2 \cdot t_f + 0.75 \cdot in$ $d := 9.75 \cdot in$ Top and Bottom Flange and top plate thicknesses

$$f_{bwl} := \frac{MWL_y \cdot t_{fl}}{8 \cdot (I_{rail} + I_{tf})} \cdot \left[2 \cdot (I_{rail} + I_{tf}) \cdot \frac{d - 2 \cdot t_1}{t_w}\right]^{\frac{1}{4}}$$ Ref. 5.7, page 86.

$f_{bwl} = 1.952 \cdot \frac{kip}{in^2}$ (Top flange bending stress due to localized wheel load)

Check top flange stress (axial compression and Bending): $Fy := 50 \cdot \frac{kip}{in^2}$ $F_{bz} := 33 \cdot \frac{kip}{in^2}$ $F_{by} := 33 \cdot \frac{kip}{in^2}$

$r_y := \left(\frac{I_{yy}}{A_c + A}\right)^{0.5}$ $r_y = 3.106 \cdot in$ $L_u := 48 \cdot in$ (r_y controls)

$r_z := \left(\frac{I_z}{A_c + A}\right)^{0.5}$ $r_z = 3.782 \cdot in$ $L_u := 48 \cdot in$

Calculate allowable compressive stress, controlled by plate yield stress $F_y = 50$ ksi

$\frac{L_u}{r_y} = 15.453$ $F_a := 28.71 \cdot \frac{kip}{in^2}$ AISC, Table C-50, page 3-17, for lu/ry = 16

Axial stress from 2 wheels: $f_a := \frac{P_a}{A_c + A}$ $f_a = 1.592 \cdot \frac{kip}{in^2}$ $\frac{f_a}{F_a} = 0.055$ < 0.15 use AISC EQ. H1-3

$f_{bzt} := \frac{TZ_{max}}{S_t} + f_{bwl}$ $f_{bzt} = 11.547 \cdot \frac{kip}{in^2}$

$f_{byt} := \frac{TY_{max} + TY1}{S_{yt}}$ $f_{byt} = 9.369 \cdot \frac{kip}{in^2}$

AISC Equation H1-3

$C_1 := \frac{f_a}{F_a} + \frac{f_{bzt}}{F_{bz}} + \frac{f_{byt}}{F_{by}}$ $C_1 = 0.689$ < 1.0 Ok

Check stress in bottom flange: $S_{bz} := 70.583 \cdot in^3$

$f_{bzb} := \frac{TZ_{max}}{S_{bz}}$ $f_{bzb} = 17.304 \cdot \frac{kip}{in^2}$ <Fbz, ok

Check deflection Y-axis: $E := 29000 \cdot \frac{kip}{in^2}$ $L := 48 \cdot in$

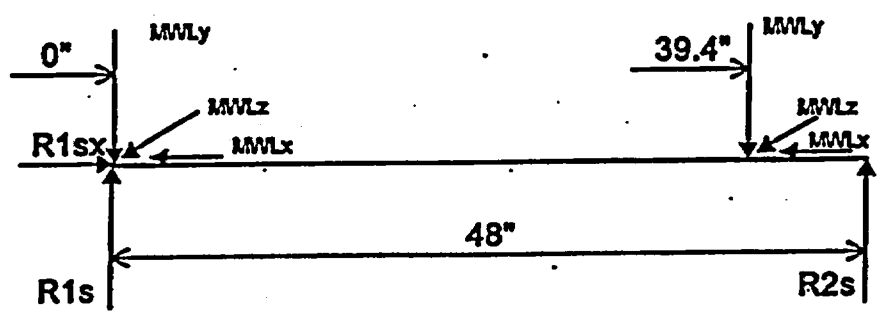
$\Delta_y := \frac{MWL_y \cdot L^3}{48 \cdot E \cdot I_z}$ $\Delta_y = 0.017 \cdot in$ <L/1000 Ok.

Check deflection Z-axis: $E := 29000 \cdot \frac{kip}{in^2}$ $L := 48 \cdot in$

$\Delta_z := \frac{MWL_z \cdot L^3}{48 \cdot E \cdot I_{yt}}$ $\Delta_z = 6.785 \cdot 10^{-3} \cdot in$ <L/400 Ok.

Use W8x67 Fy = 50 ksi and
Plate 15"x 3/4" Fy= 50 ksi

Calculate Maximum Support Reaction to transfer beam:

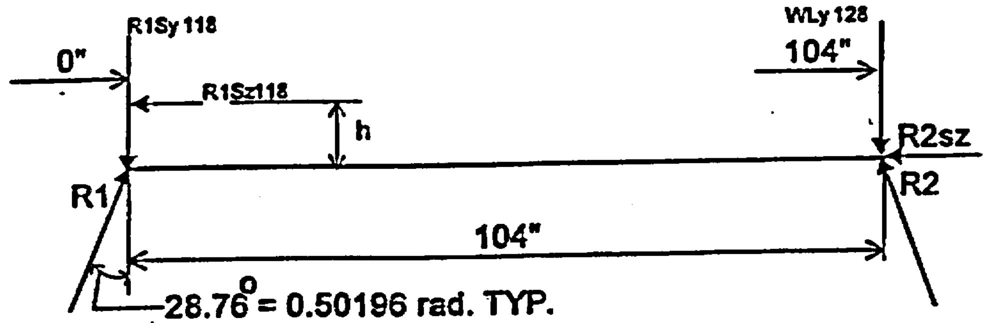


$R1s_y := MWL_y \cdot \left[\left(\frac{48}{48} \right) + \left(\frac{8.6}{48} \right) \right]$, $R1s_y = 109.298 \cdot kip$

$R1s_z := MWL_z \cdot \left[\left(\frac{48}{48} \right) + \left(\frac{8.6}{48} \right) \right]$, $R1s_z = 25.603 \cdot kip$

$R1s_x := MWL_x \cdot 2$, $R1s_x = 49.277 \cdot kip$

Design Transfer Beam: Try W12 with sloped base plate on bottom of beam; Base plate slope to match tunnel wall slope. Analysis assumes frictionless supports due to the required friction angle for a base plate sloped at 28.76 deg. The required friction angle u = f / N = sin 28.76 / cos 28.76 = 0.58 >> allowable coefficient of static friction = 0.08 for steel supported on concrete with a factor of safety = 5 (Support of Temporary loads), Ref. 5.16, page 5-6.



Gantry wheel reaction WLy128 from Ref. 5.18, File Gantry-H, load case 102, joint 128. This reaction is opposite maximum reaction at joint 118.

WLy128 = (gantry column reaction/2 wheels)*(48/48 + 8.6/48)

Gantry column reaction $=R_{g128} := 1.272 \cdot 27.67 \cdot kip$ $\qquad WL_{y128} := \frac{R_{g128}}{2} \cdot \left(\frac{48}{48} + \frac{8.6}{48}\right)$

$WL_{y128} = 20.751 \cdot kip$ $\qquad R1s_{y118} := R1s_y$ $\qquad R1s_{z118} := R1s_z$

$h := 21.7 \cdot in$ $\qquad L := 104 \cdot in$

Calculate Beam Reactions: Note: Horizontal load R1Sz118 location taken from top of PL 3/4"x1'-3" to bottom of W12, $h := 21.7 \cdot in$.

Sum Moments about R2 = 0:

$$R1 := \frac{R1s_{y118} \cdot L + R1s_{z118} \cdot h}{L \cdot \cos(.50196)} \qquad R1 = 130.772 \cdot kip$$

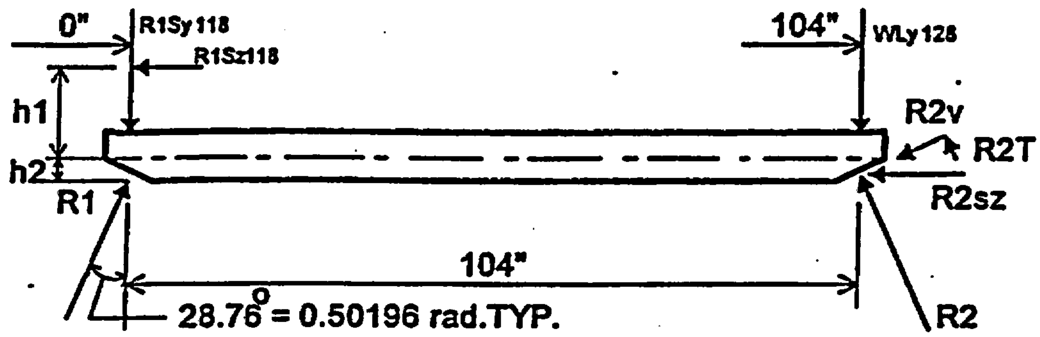
Sum Moments about R1 = 0:

$$R2 := \frac{WL_{y128} \cdot L - R1s_{z118} \cdot h}{L \cdot \cos(.50196)} \qquad R2 = 17.577 \cdot kip$$

Sum Forces in the Z-axis = 0

$$R2sz := R1 \cdot \sin(.50196) - R2 \cdot \sin(.50196) - R1s_{z118} \qquad R2sz = 28.86 \cdot kip$$

Transfer Beam Design continued:



Calculate Axial load in transfer beam:

$P_{axial} := R1 \cdot \sin(.50196) - R1s_{z118}$ $\qquad$ $P_{axial} = 37.317 \cdot kip$

Calculate Maximum Mz moment at centerline of beam and support R1:

$h1 := 15.73 \cdot in$ $\qquad$ $h2 := \frac{11.94}{2} \cdot in$ $\qquad$ For W12x40, see figure I-7

$Mz_{max} := R1 \cdot \sin(.50196) \cdot h2 + R1s_{z118} \cdot h1$ $\qquad$ $Mz_{max} = 778.373 \cdot in \cdot kip$

Calculate Maximum My moment at centerline of beam using 3 inch eccentricity:

$e := 3 \cdot in$ $\qquad$ (See bracket sketch page I-40)

$My_{max} := R1s_{z118} \cdot e$ $\qquad$ $My_{max} = 76.81 \cdot in \cdot kip$

Design beam - Use beam with 8 inch flange to facilitate connection of W8 to W12: Try W12x40,

$F_y := 50 \cdot \frac{kip}{in^2}$

$A := 11.8 \cdot in^2$ $\quad$ $d := 11.94 \cdot in$ $\quad$ $t_w := .295 \cdot in$ $\quad$ $b_f := 8.005 \cdot in$ $\quad$ $t_f := .515 \cdot in$ $\quad$ $k := 1.25 \cdot in$

$I_x := 310 \cdot in^4$ $\quad$ $S_x := 51.9 \cdot in^3$ $\quad$ $I_y := 44.1 \cdot in^4$ $\quad$ $S_y := 11.0 \cdot in^3$ $\quad$ $r_x := 5.13 \cdot in$ $\quad$ $r_y := 1.93 \cdot in$

Calculate allowable stresses using AISC:

$l_u := 104 \cdot in$ $\qquad$ Unbraced length for Z & Y axis

$F_{bz} := 0.60 \cdot F_y$ $\qquad$ $F_{bz} = 30 \cdot \frac{kip}{in^2}$ $\qquad$ Equation (F1-5), Page 5-46

$F_{by} := 0.75 \cdot F_y$ $\qquad$ $F_{by} = 37.5 \cdot \frac{kip}{in^2}$ $\qquad$ Equation (F2-1), Page 5-48

$\frac{l_u}{r_y} = 53.886$ $\qquad$ $F_a := 23.72 \cdot \frac{kip}{in^2}$ $\qquad$ Table C-50, Page 3-17

Calculate beam stresses:

$f_{bz} := \frac{Mz_{max}}{S_x}$ $f_{by} := \frac{My_{max}}{S_y}$ $f_a := \frac{P_{axial}}{A}$

$f_{bz} = 14.998 \cdot \frac{kip}{in^2}$ $f_{by} = 6.983 \cdot \frac{kip}{in^2}$ $f_a = 3.162 \cdot \frac{kip}{in^2}$

Axial compression and bending coefficients $E = 29000 \cdot \frac{kip}{in^2}$

$C_{mx} := 1.0$ $C_{my} := 1.0$ $k := 1.0$

$$F_{ex} := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(\frac{k \cdot l_u}{r_x}\right)^2}$$ $F_{ex} = 363.345 \cdot \frac{kip}{in^2}$

Euler Stress, AISC, page 5-54
Fex = F'ex

$$F_{ey} := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot \left(\frac{k \cdot l_u}{r_y}\right)^2}$$ $F_{ey} = 51.428 \cdot \frac{kip}{in^2}$

Euler Stress, AISC, page 5-54
Fey = Fey

$$C_{H1} := \frac{f_a}{F_a} + \frac{C_{mx} \cdot f_{bz}}{\left(1 - \frac{f_a}{F_{ex}}\right) \cdot F_{bz}} + \frac{C_{my} \cdot f_{by}}{\left(1 - \frac{f_a}{F_{ey}}\right) \cdot F_{by}}$$

$C_{H1} = 0.836$ < 1.0 ok Equation h1-1, page 5-54

$$C_{H2} := \frac{f_a}{0.6 \cdot F_y} + \frac{f_{bz}}{F_{bz}} + \frac{f_{by}}{F_{by}}$$

$C_{H2} = 0.792$ < 1.0 ok Equation H1-2, page 5-54

$$C_{H3} := \frac{f_a}{F_a} + \frac{f_{bz}}{F_{bz}} + \frac{f_{by}}{F_{by}}$$

$C_{H3} = 0.819$ < 1.0 ok Equation H1-3, page 5-54

Use W12x40, Fy =50ksi. Minimum 8" flange width required for W8 to transfer beam connection

Design connection of W8 to W12 transfer beam:

Find W8x67 length of bearing (N) based on Local Web Yielding, AISC, chapter K, page 5-81, Equation (K1-3):

$k := 1.438 \cdot in \quad t_w := .57 \cdot in \quad R := R1s_y \quad F_{yield} := 50 \cdot \frac{kip}{in^2} \quad R = 109.298 \cdot kip$

$$N := \frac{R - .66 \cdot F_{yield} \cdot t_w \cdot 2.5 \cdot k}{.66 \cdot F_{yield} \cdot t_w} \qquad N = 2.216 \cdot in$$

Find W8 length of bearing (N) based on Web Crippling, AISC, chapter K, page 5-81, Equation (K1-5):

$t_w = 0.57 \cdot in \quad t_f := .935 \cdot in \quad d := 9 \cdot in \quad F_y := 50 \cdot \frac{kip}{in^2} \quad$ For: $N = 2.216 \cdot in$

$$R_a := \left[34 \cdot t_w^2 \cdot \left[1 + 3 \cdot \left(\frac{N}{d}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5} \right] \cdot \sqrt{\frac{F_y}{\left(\frac{kip}{in^2}\right)} \cdot \frac{t_f}{t_w}} \cdot \frac{kip}{in^2} \right] \qquad R_a = 135.21 \cdot kip \qquad > 109.298 \text{ kips, ok}$$

$$d_c := \frac{4100 \cdot t_w^3 \cdot 7.07106 \cdot \frac{kip}{in^2}}{R} \qquad d_c = 49.123 \cdot in$$

> dc W8x67 ok, AISC, Eq. K1-8

Conclusion: Bearing length controlled by Local Web Yielding. Provide minimum 2.22 inches of bearing.

Check maximum wheel load permitted by Sidesway Web buckling, AISC, equation or K1-7, pg. 5-82.

$t_w = 0.57 \cdot in$		W8 beam web
$l := 48 \cdot in$		Beam unbraced length
$b_{fw8} := 8.28 \cdot in$		Beam flange width
$d_c := d - 2 \cdot k$	$d_c = 6.124 \cdot in$	Web depth clear of fillets
$h := d - 2 \cdot t_f$	$h = 7.13 \cdot in$	Web depth clear of flanges.

$$c := \frac{\left(\frac{d_c}{t_w}\right)}{\left(\frac{l}{b_{fw8}}\right)} \qquad c = 1.853 \qquad > 1.7$$

Use AISC, equation K1-7, since loaded flange is not restrained against rotation.

$$R := \frac{6800 \cdot kip \cdot t_w^3}{h} \cdot \left(0.4 \cdot c^3\right) \cdot \frac{1}{in^2} \qquad R = 449.73 \cdot kip \qquad > R = 109.298 \text{ kips ok}$$

Design W12 bearing stiffener plates:

Calculate minimum Bearing stiffener thickness with material yield stress = 50 ksi

Minimum stiffener thickness:	ts> 1.5 x minimum weld thickness	AISC, page 4-42
	> tw(beam) x Fy(Beam) / Fy(stiffener)	AISC, page 4-42
	> b / t ratio	Table B5.1, page 5-36
	>Axial stress	AISC,Eq E2-1, page 5-42

ts based on: (minimum weld thickness) $a := \frac{5}{16} \cdot in$ Minimum fillet weld thickness based on material thickness greater than 3/4". AISC, page 5-67, Table J2.4

$t_{s1} := 1.5 \cdot a$ $\quad t_{s1} = 0.469 \cdot in$

ts based on : tw (beam) x Fy(beam) / Fy(stiffiner)

$t_{wbm} := 0.57 \cdot in$ $\quad F_{ybm} := 50 \cdot \frac{kip}{in^2}$ $\quad F_{yst} := 50 \cdot \frac{kip}{in^2}$

$t_{s2} := t_{wbm} \cdot \frac{F_{ybm}}{F_{yst}}$ $\quad t_{s2} = 0.57 \cdot in$

ts based on : $b := 4 \cdot in$ $\quad t_{s3} := \frac{b}{\left(\frac{95}{\sqrt{50}}\right)}$ $\quad t_{s3} = 0.298 \cdot in$ Table B5.1, page 5-36

ts based on axial stress: try $t_s := 1.625 \cdot in$ $\quad h := 10.91 \cdot in$ (l = d- 2bf) $\quad k := 0.75$ AISC, K 1.8, pg 5-82, 0.75h

$b := 3 \cdot in$ Use effective stiffener width concentric below beam bearing

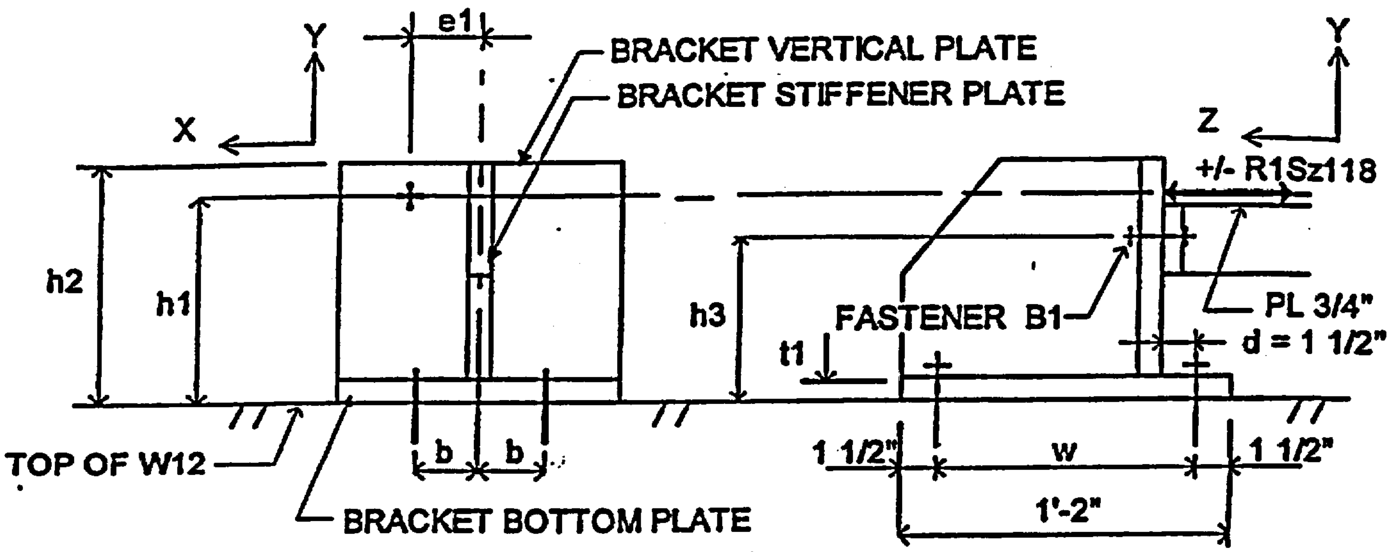
$A := b \cdot t_s$ $\quad I := \frac{b \cdot t_s^3}{12}$ $\quad r := \sqrt{\frac{I}{A}}$ $\quad r = 0.469 \cdot in$ $\quad c := \frac{k \cdot h}{r}$

$c = 17.443$ $\quad F_a := 28.51 \cdot \frac{kip}{in^2}$ AISC, Table C-50, page 3-17, using kh/r=18

$Pall := F_a \cdot b \cdot t_s$ $\quad Pall = 138.986 \cdot kip$ $\quad >$ $\quad R1 = 130.772 \cdot kip$ ok, greater than maximum end reaction

Conclusion: Use 2 - 1 5/8" x 4" stiffener plates full depth of W12 beam, Fy = 50 ksi

Design bracket at beam splice location with total lateral load imparted to one side of bracket with eccentricity equal to e1, see bracket sketch.



Design Bracket Connection: Use 50 ksi yield stress steel $F_y := 50\cdot\frac{kip}{in^2}$

$R1s_{z118} = 25.603\cdot kip$	Maximum shear in the Z direction	$b := 2.75\cdot in$	(half gage for bolt holes W12)
$e1_y := 3\cdot in$	Eccentricity about Y axis	$w := 11\cdot in$	
$h1_z := 9.7\cdot in$	Eccentricity about X axis	$h2 := 11\cdot in$	

Determine number and size of Fasteners connecting bracket to W12 top flange: Try 4 - 7/8" diameter bolts.

$V := R1s_{z118}$ $\quad M_z := R1s_{z118}\cdot h1_z$ $\quad M_y := R1s_{z118}\cdot e1_y$

$V = 25.603\cdot kip$ $\quad M_z = 248.352\cdot in\cdot kip$ $\quad M_y = 76.81\cdot in\cdot kip$

$A := .6013\cdot in^2$ $\quad I_z := A\cdot\left(\frac{w}{2}\right)^2\cdot 4$ $\quad I_y := A\cdot\left[\sqrt{\left(\frac{w}{2}\right)^2 + b^2}\right]^2\cdot 4$

$I_z = 72.757\cdot in^4$ $\quad I_y = 90.947\cdot in^4$

$f_t := \frac{M_z\cdot\frac{w}{2}}{I_z}$ $\quad f_t = 18.774\cdot\frac{kip}{in^2}$ $\quad$ Bolt tensile stess

$f_z := \frac{V}{4\cdot A}$ $\quad f_z = 10.645\cdot\frac{kip}{in^2}$ $\quad$ Direct shear stress along Z axis

Determine number and size of Fasteners: Try 4 - 7/8" diameter bolts, Continued:

$f_{sz} := \frac{M_y \cdot b}{I_y}$ $f_{sz} = 2.323 \cdot \frac{kip}{in^2}$ Rotationl Shear along Z axis

$f_{sx} := \frac{M_y \cdot \frac{w}{2}}{I_y}$ $f_{sx} = 4.645 \cdot \frac{kip}{in^2}$ Rotational shear along X axis

$f_{vt} := \sqrt{(f_z + f_{sz})^2 + f_{sx}^2}$ $f_{vt} = 13.774 \cdot \frac{kip}{in^2}$ Total shear stress per bolt

Find allowableTensile stress:

$F_t := \sqrt{\left(44 \cdot \frac{kip}{in^2}\right)^2 - 4.39 \cdot f_{vt}^2}$ $F_t = 33.213 \cdot \frac{kip}{in^2}$ $> f_t = 18.774 \cdot \frac{kip}{in^2}$ ok AISC, Table J3.3, page 5-74 for A325N bolts

Conclusion use 4 - 7/8" diameter, A325N bolts

Determine plate thickness (t1) for bracket bottom plate connected to top flange of beam: $F_y = 50 \cdot \frac{kip}{in^2}$

$T := A \cdot f_t$ $T = 11.289 \cdot kip$ Maximum bolt tension

$d := 1.5 \cdot in$ Centerline of bolt to face of vertical plate

$b_x := 2 \cdot d$ $b_x = 3 \cdot in$ Effective width of plate resisting flexure, bolt gage/2 > bx/2 , no overlap

$m := \frac{T \cdot d}{b_x}$ $m = 5.644 \cdot \frac{in \cdot kip}{in}$ Moment per inch of plate

$F_b := .75 \cdot F_y$ Allowable bending stress AISC, Equation (F-2.1), page 5-48

$t1 := \sqrt{\frac{6 \cdot m}{F_b}}$ $t1 = 0.95 \cdot in$ Required plate thickness

Use: 1" plate thickness $t1 := 1.0 \cdot in$

Conclusion: Bracket Bottom Plate 1" x 12" x 1'-2" (see above sketch for location)

Design bracket stiffener plate, Ref. 5.17, page 5.3-1

Try: $t2 := .625 \cdot in$ — Thickness of stiffener plate

$L_h := h2 - t1$ $L_h = 10 \cdot in$ — Length of stiffener plate

$As := t2 \cdot L_h$ $As = 6.25 \cdot in^2$ — Area of stiffener

$S := \frac{t2 \cdot L_h^2}{6}$ $S = 10.417 \cdot in^3$ — Section modulus of stiffener

$a := h1_z - \left(\frac{L_h}{2} + t1\right)$ $a = 3.7 \cdot in$ — Eccentricity of load from centroid of plate

$f_c := \left(\frac{R1s_{z118}}{As}\right) + \frac{R1s_{z118} \cdot a}{S}$ — Maximum compressive stress

$f_c = 13.191 \cdot \frac{kip}{in^2}$ $< Fb = 0.6 * Fy$ — AISC, Equation (F1-5)

Check b/t ratio using AISC, Table B5.1, page 5-36, $b/t = \frac{95}{\sqrt{F_y}}$

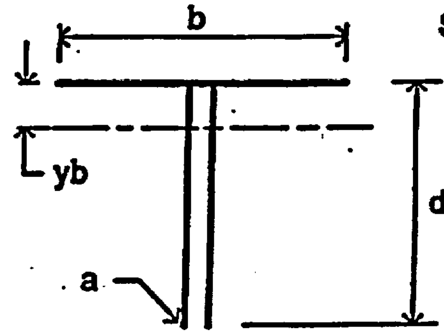
$t := \frac{L_h}{95} \cdot 7.07$ $t = 0.744 \cdot in$ — Check weld size to see if plate thickness controlled by welding requirements.

Determine weld size:

Section properties of weld group

$b := 12 \cdot in$ $d := 10.25 \cdot in$

Section Modulus about point a, see sketch



$S_{wa} := \left[\frac{d^2 \cdot (2 \cdot b + d)}{3 \cdot (b + d)}\right] \cdot 1 \cdot in$ $S_{wa} = 53.908 \cdot in^3$

Polar moment of inertia

$J_w := \left[\frac{\left(b^3 + 8 \cdot d^3\right)}{12} - \frac{d^4}{(b + 2 \cdot d)}\right] \cdot 1 \cdot in$ $J_w = 522.292 \cdot in^4$ $y_b := \frac{d^2}{b + 2 \cdot d}$ $y_b = 3.233 \cdot in$

$M_{zpl} := R1s_{z118} \cdot L_h$ $M_{zpl} = 256.033 \cdot in \cdot kip$ — Moment at top of base plate about Z-axis

$M_y = 76.81 \cdot in \cdot kip$ — Moment about Y-axis, see page 40 for My calculation.

$F_v := R1s_{z118}$ $F_v = 25.603 \cdot kip$ — shear along Z-axis

Calculate stress at point "a"

$f_v := \frac{F_v}{2 \cdot d \cdot 1 \cdot in}$ $f_v = 1.249 \cdot \frac{kip}{in^2}$ Shear stress Z- axis

$f_t := \frac{M_{zpl}}{S_{wa}}$ $f_t = 4.749 \cdot \frac{kip}{in^2}$ Compressive stress

$f_{ry} := \frac{M_y \cdot (d - y_b)}{J_w}$ $f_{ry} = 1.032 \cdot \frac{kip}{in^2}$ Twisting shear stress

$R_w := \sqrt{f_v^2 + f_t^2 + f_{ry}^2}$ $R_w = 5.018 \cdot \frac{kip}{in^2}$ Resultant stress for a 1" effective weld throat

Determine weld size: Use E70 fillet weld, AISC, J2.2a

$F_u := 70 \cdot \frac{kip}{in^2}$ Tensile strength of E70 electrode

$a := \frac{R_w}{0.707 \cdot 0.3 \cdot F_u} \cdot in$ $a = 0.338 \cdot in$

Use 3/8" fillet welds

Check stiffener plate thickness requirement for 2-0.33" fillet welds, Ref 5.20, page 213:

$t_{min} := \frac{2 \cdot a \cdot .707 \cdot .3 \cdot F_u}{0.4 \cdot F_y}$ $t_{min} = 0.502 \cdot in$ Required stiffener plate thickness

Conclusion Bracket Stiffener Plate: Use PL 3/4" x 9 3/8" x 0'-10", b/t ratio controls.

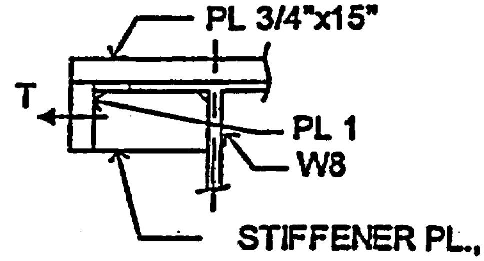
Design fastener B1 (see above sketch) connecting W8 top plate to bracket, using A325N bolts

$h3 := h1_z - 2 \cdot in$ $h3 = 7.7 \cdot in$ Centerline Fastener above top of W12

$T := R1s_{z118} \cdot \frac{h1_z}{h3}$ $T = 32.253 \cdot kip$ Maximun bolt tension for connection to bracket at beam splice location.

Fastener B1: Use 1" diameter A325N bolts, T allowable = 34.6 kip AISC, Table 1-A, page 4-3

Design stiffener plates to reinforce plate PL 1 (see sketch below): Try 3/8" stiffener plates each side of bolt, note 1" bolt spaced equally between stiffener plates:



$t_{st} := .375 \cdot in$

$T = 32.253 \cdot kip$ Tension bolt

$P_t := \frac{T}{2}$ $P_t = 16.127 \cdot kip$ Tension per stiffener plate

$F_t := 0.6 \cdot F_y$ $F_t = 30 \cdot \frac{kip}{in^2}$ Allowable tension stress in plate, AISC, D1, page 5-40

Find required width of plate, " b":

$b := \frac{P_t}{t_{st} \cdot F_t}$ $b = 1.433 \cdot in$ Required width of plate

Plate PL1: Use PL 2" x 4" x 1'-0"
Stiffener Plates: Use b = 3.0 inches, 3 - PL 3/8" x 3" x 0'-5 3/16" Equally spaced along plate PL1

Determine expansion requirements for runway beam.Use runway beam length = 40'-0", fixed for X- axis translation middle, Data:

$L := 240 \cdot in$ Length of beam free to move

$t_b := 200$ Tunnel tempature after closure, degree celcius, (Section 4.3.4, DCSS 023)

$t_{bm} := 50$ Use 50 deg. fabrication temperature, degree fahrenheit

$t_{delta} := (1.8 \cdot t_b + 32) - t_{bm}$ $t_{delta} = 342$ Structural steel temperature change, degree fahrenheit

$c := .00065$ Coefficient of expansion, AISC, page 6-6 for mild steel

$l_{delta} := \frac{c \cdot L \cdot t_{delta}}{100}$ $l_{delta} = 0.534 \cdot in$ Elongation of runway beam, AISC, page 6-6

Center 1" diameter bolt 3 inches from centerline of bracket, see "e1y" on bracket sketch.

Design Bracket Vertical Plate (see bracket sketch above):

$e := e1_y - 0.375 \cdot in$ $e = 2.625 \cdot in$ Distance between centerline of 1" diameter bolt and face of plate.

$T = 32.253 \cdot kip$ bolt tension

$b := 2 \cdot e$ Effective width of plate resisting flexure Spreads at 45 degrees.

$m := T \cdot \frac{e}{b}$ $m = 16.127 \cdot in \cdot \frac{kip}{in}$ Bending moment per inch of plate

$F_b := 0.75 \cdot F_y$ Allowable bending stress in plate, AISC, Equation (F2-1), page 5-48

Bracket Vertical Plate design: Continued

$t_{req} := \sqrt{\frac{6 \cdot m}{F_b}}$ $t_{req} = 1.606 \cdot in$ Required vertical plate thickness.

Bracket Vertical Plate : Use PL 1.5/8" x 10" x 1'-0" , Fy = 50ksi

Design W12 base plate and anchorage to rock: Use 50 ksi yield strength material $F_y := 50 \cdot \frac{kip}{in^2}$

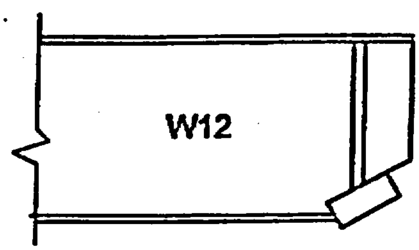
$R1 = 130.772 \cdot kip$ Maximum resultant beam end reaction due to vertical loads.

Try:

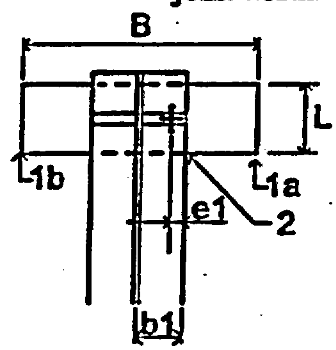
$B := 18 \cdot in$ Baseplate width

$L := 6 \cdot in$ Base plate length

$e_1 := 1.625 \cdot in$ Centroid of load at runway beam splice, see Fig I-5, page I-52, for expansion joint width.



BEAM ELEVATION



BASE PLATE PLAN

Determine bearing pressure under baseplate:

$A := B \cdot L$ $A = 108 \cdot in^2$ Bearing Area

$I := \frac{L \cdot B^3}{12}$ $I = 2.916 \cdot 10^3 \cdot in^4$ Strong Axis moment of inertia

$b_1 := 4 \cdot in$ 1/2 beam flange width

Find bearing pressure at points 1 & 2:

$P_{1a} := \frac{R1}{A} + \frac{R1 \cdot (4 \cdot in - e_1) \cdot \frac{B}{2}}{I}$ $P_{1a} = 2.169 \cdot \frac{kip}{in^2}$ Bearing pressure along edge of base plate located at point 1a

$P_{1b} := \frac{R1}{A} - \frac{R1 \cdot (4 \cdot in - e_1) \cdot \frac{B}{2}}{I}$ $P_{1b} = 0.252 \cdot \frac{kip}{in^2}$ Bearing pressure along edge of base plate located at point 1b

Base plate design: Continued

$p_2 := \frac{R1}{A} + \frac{R1 \cdot (4 \cdot in - e_1) \cdot b_1}{I}$ $p_2 = 1.637 \cdot \frac{kip}{in^2}$ Bearing pressure at exterior edge of stiffener located at point 2

Sum moments at exterior edge of stiffener located at point 2:

$x := \frac{B}{2} - b_1$ $x = 5 \cdot in$ Length of plate extending past edge of stiffener

$m_p := \frac{p_2 \cdot x^2}{2} + \frac{(p_{1a} - p_2) \cdot x^2}{3}$ $m_p = 24.899 \cdot in \cdot \frac{kip}{in}$ Maximum moment per inch of base plate length

$t_{pl} := \sqrt{\frac{6 \cdot m_p}{0.75 \cdot F_y}}$ $t_{pl} = 1.996 \cdot in$ Required base plate thickness

W12 Base Plate: Use 2" thick base plate: Base PL 2" x 7" x 1'-6". Note 6" wide base plate used in calculation, Use 7" wide Base Plate to allow welding base plate to beam bottom flange.

Design anchor bolts: Use A307 threaded bar stock for Anchor bolt design

$F_u := 60 \cdot \frac{kip}{in^2}$ Tensile strength of anchor bolt, AISC, Table 1C, page 4-4.

See page I-35, for horizontal shear force R2sz and free body diagram page I- 36 for anchor bolt forces R2v and R2T.

$R2sz = 28.86 \cdot kip$ Horizontal shear force

$R2v := R2sz \cdot \cos(.50196)$ $R2v = 25.3 \cdot kip$ Shear force along bottom of base plate.

Try using two anchor bolts to restrain load. Shear forces per anchor bolt are:

$V := \frac{R2v}{2}$ $V = 12.65 \cdot kip$ Shear force per anchor bolt

Try using 2 - 1 3/8" diameter A.B.:

$d := 1.375 \cdot in$ Diameter of anchor bolt

$A := \frac{\pi \cdot d^2}{4}$ $A = 1.485 \cdot in^2$ Area of anchor bolt

Anchor bolt design: Continued

$f_v := \frac{V}{A}$	$f_v = 8.519 \cdot \frac{kip}{in^2}$	Anchor bolt shear stress
$F_v := .17 \cdot F_u$	$F_v = 10.2 \cdot \frac{kip}{in^2}$	AISC, Table I-D threads included in shear plane

Conclusion: Use 1 3/8" diameter A307 anchor bolt

Design Summary:

Runway Beam:	W8 x 67
Runway Beam Top Plate:	PL 3/4" x 15" x 39'-10 1/2"
Runway Beam Plate PL1:	PL 2" x 4" x 1'-0"
with Stiffeners	3- PL 3/8" x 3" x 5 5/16
Bolting for runway Beam Bottom Flange connection: to Transfer beam top flange	4- 7/8" diameter, A325

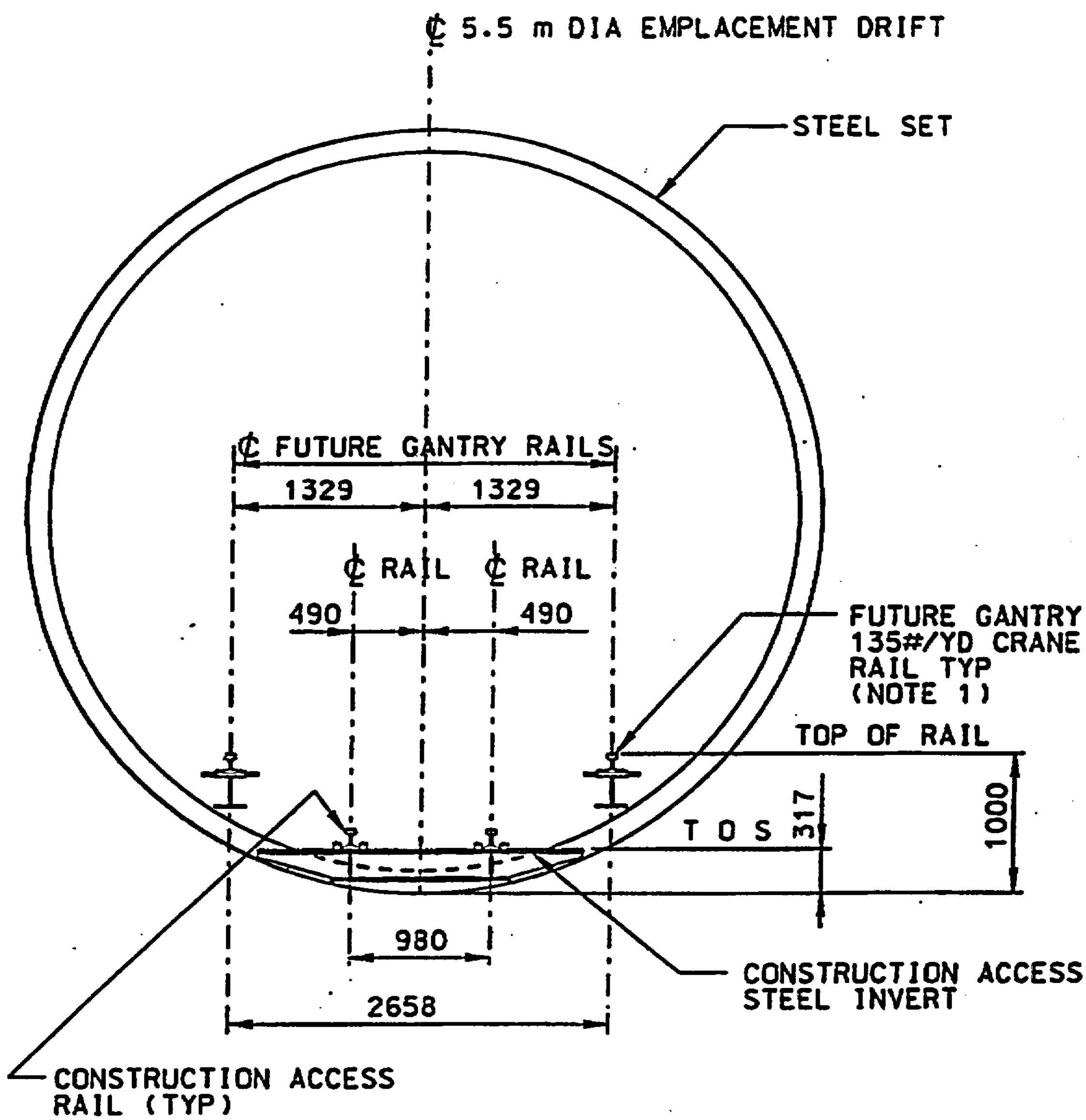
Runway Beam Connection to Bracket Assembly:

Plate PL1:	PL 2" x 4" x 1'-0"
Stiffener plates:	3-PL 3/8" x 3" x 5 3/16"

Transfer Beam:	W12 x 40
Transfer Beam Bearing Stiffeners:	2-PL 1 5/8" x 4" x full depth of W12
Transfer Beam Bearing Plate:	PL 2" x 7" x 1'-6"
Transfer Beam Anchor bolts:	2- 1 3/8" diameter, A307

Bracket Assembly:

Bracket Bottom plate:	PL 1" x 12" x 1'- 2"
Bracket Stiffener Plate:	PL 3/4" x 9 3/8" x 0'-10"
Bracket Vertical Plate:	PL 1 5/8" x 10" x 1'-0"
Bracket Bolting to W8:	2-1" diameter, A325



NOTES:

1: GANTRY RAILS AND RUNWAY SUPPORT STRUCTURE TO BE INSTALLED AFTER REMOVAL OF CONSTRUCTION ACCESS STEEL INVERT.

2. ALL STEEL FABRICATION AND ERECTION TOLERANCES SHALL BE IN ACCORDANCE WITH AISC M016.

ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE NOTED

FIGURE I-1
CONSTRUCTION ACCESS STEEL INVERT ELEVATION

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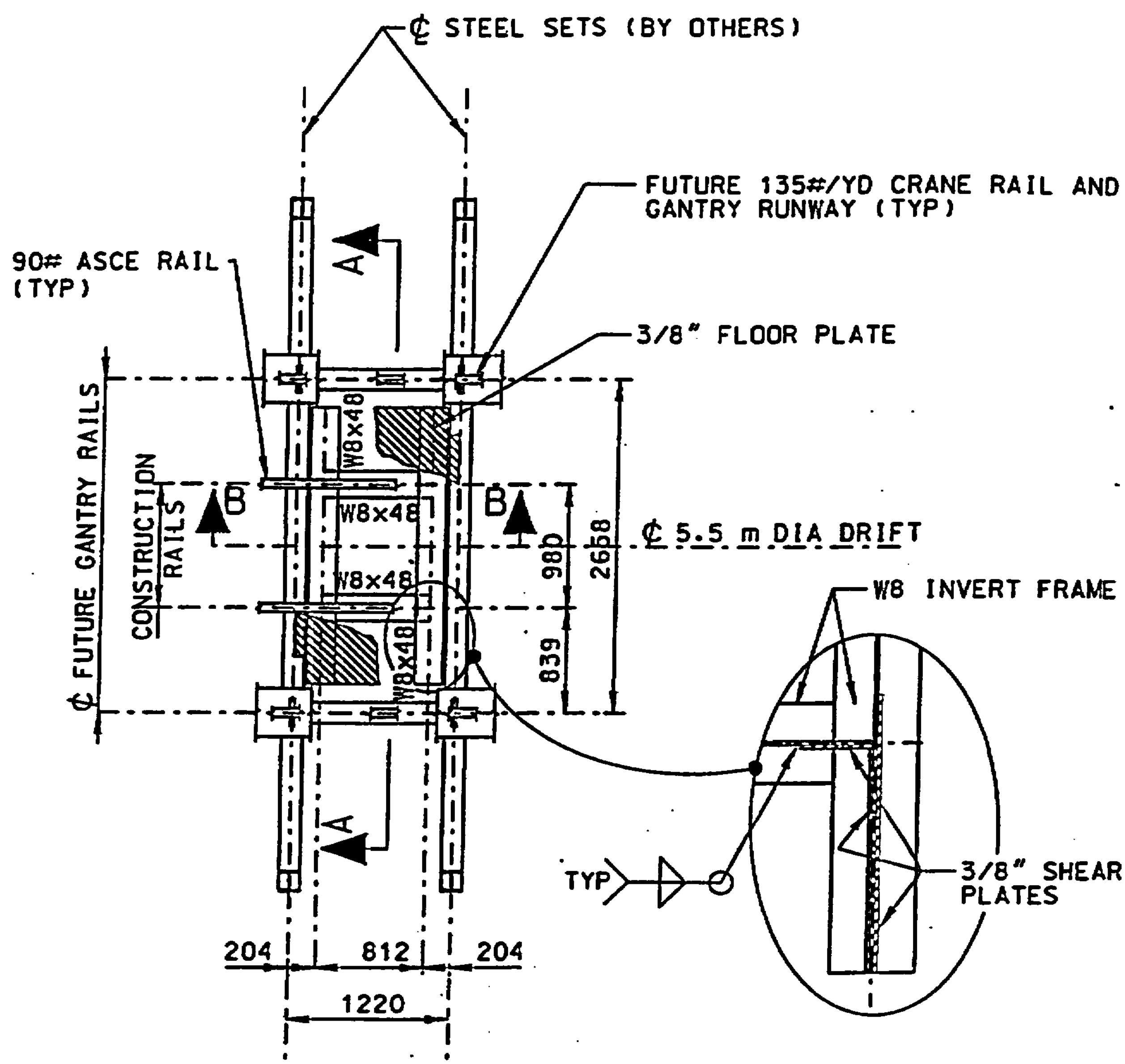

NOTES:

1. STRUCTURAL STEEL MATERIALS FOR THE CONSTRUCTION ACCESS STEEL INVERT SHALL BE IN ACCORDANCE WITH ASTM A6 AND ASTM A36.

2. STEEL INVERT WILL BE ALL WELDED CONSTRUCTION PER AWS D1.1.

ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE NOTED

FIGURE I-2
CONSTRUCTION ACCESS STEEL INVERT PLAN

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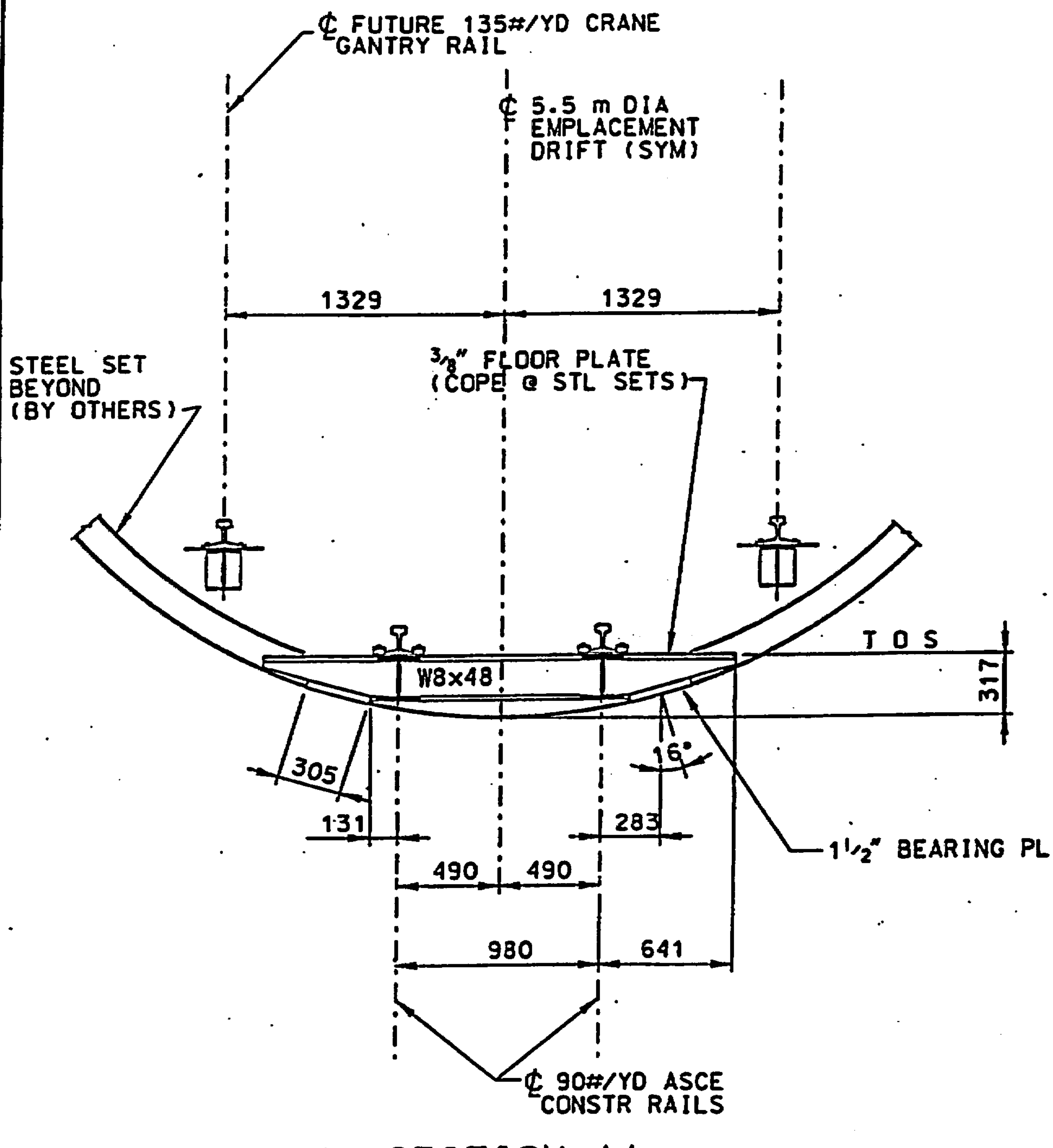

SECTION AA

ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE NOTED

FIGURE I-3
CONSTRUCTION ACCESS
STEEL INVERT SECTION

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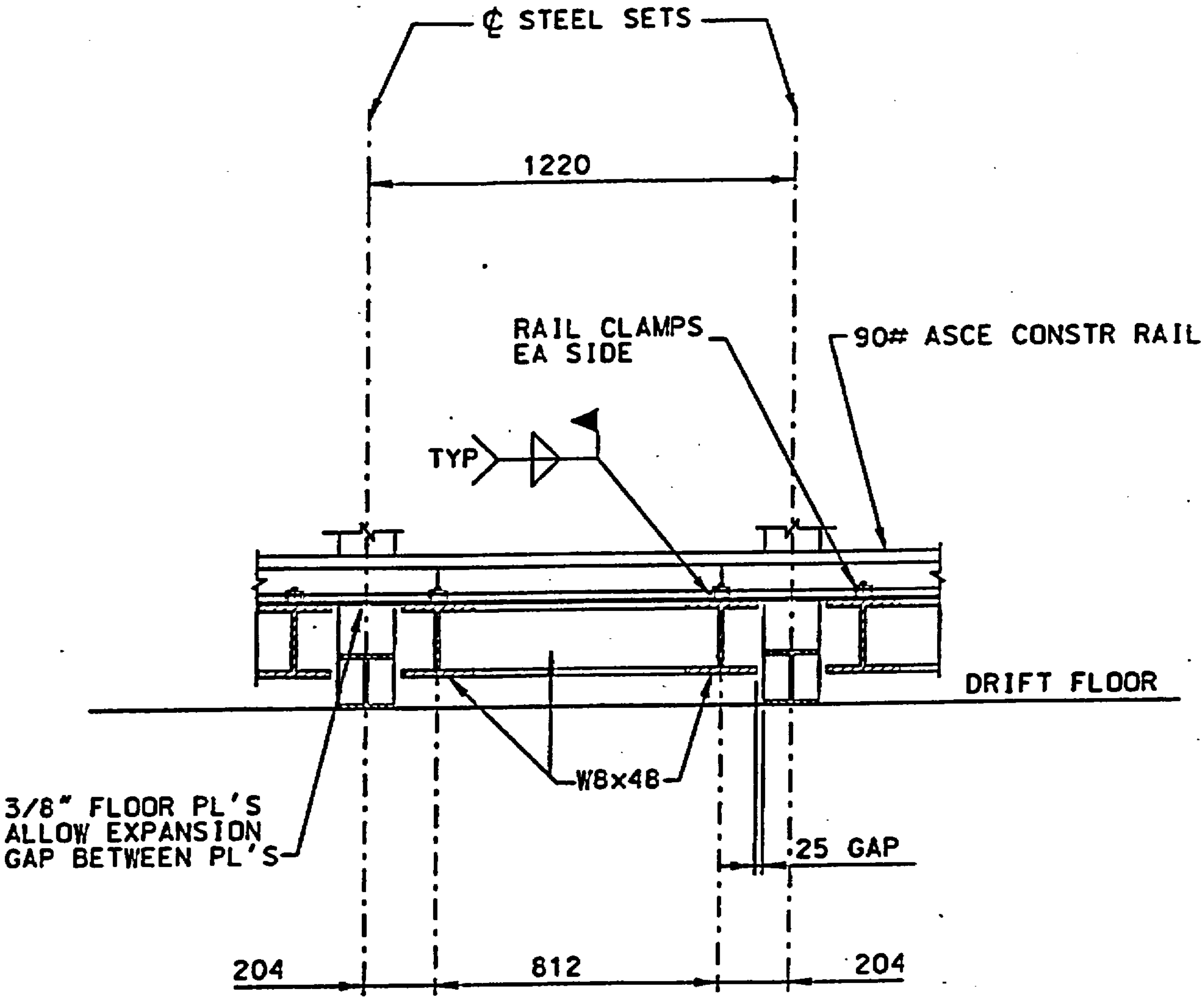

SECTION BB

ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE NOTED

FIGURE I-4
CONSTRUCTION ACCESS STEEL
INVERT SECTION

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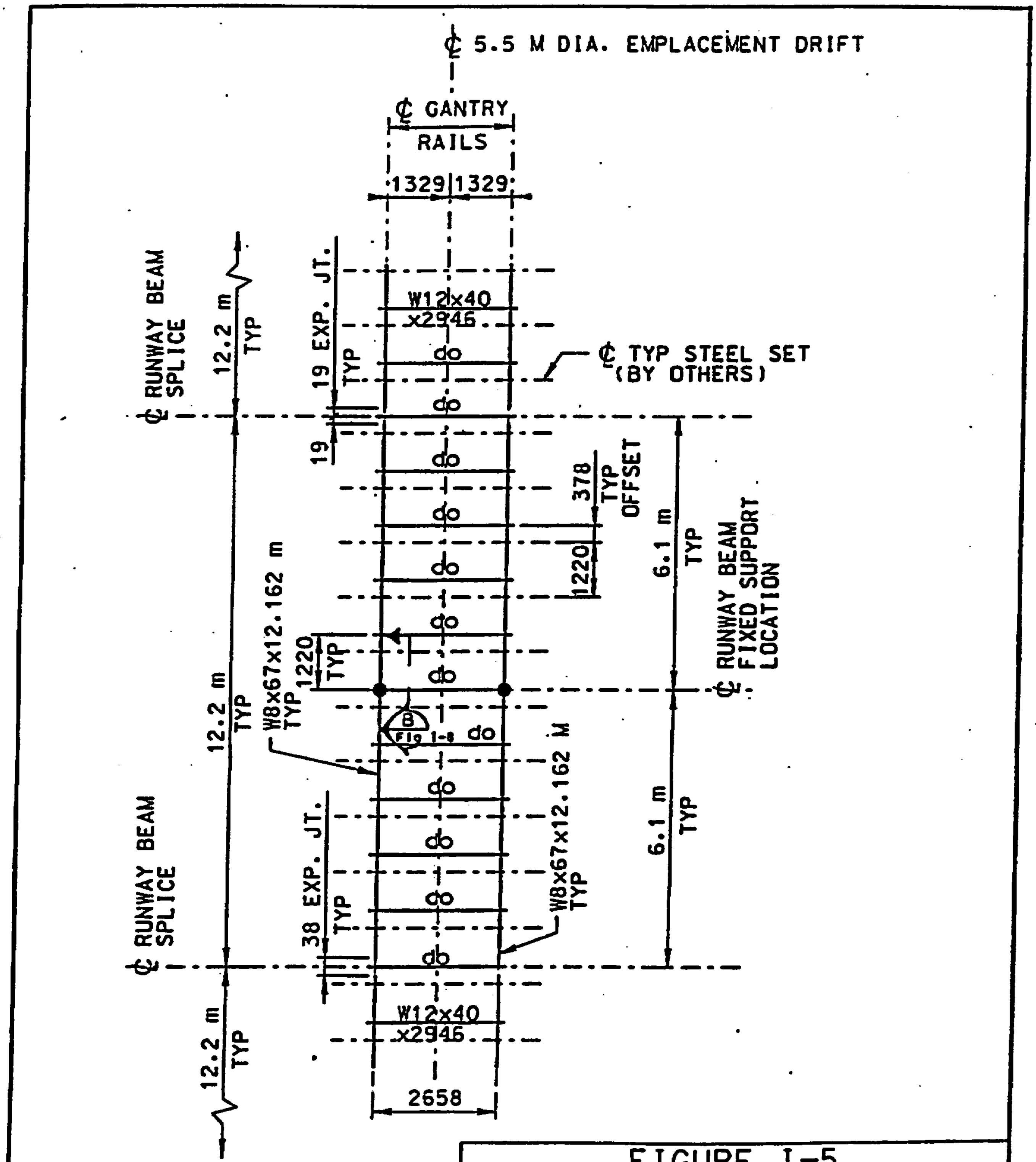
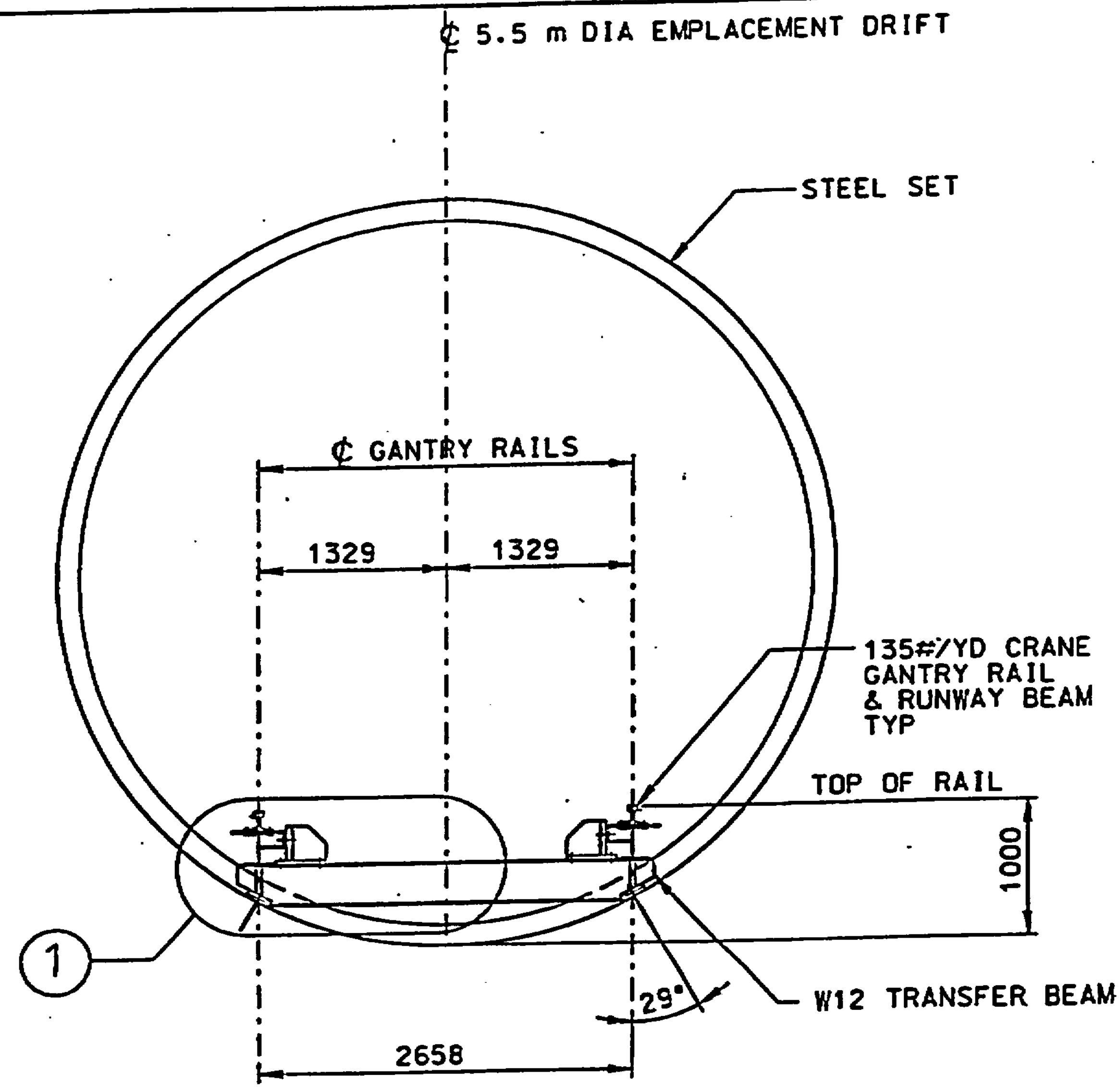
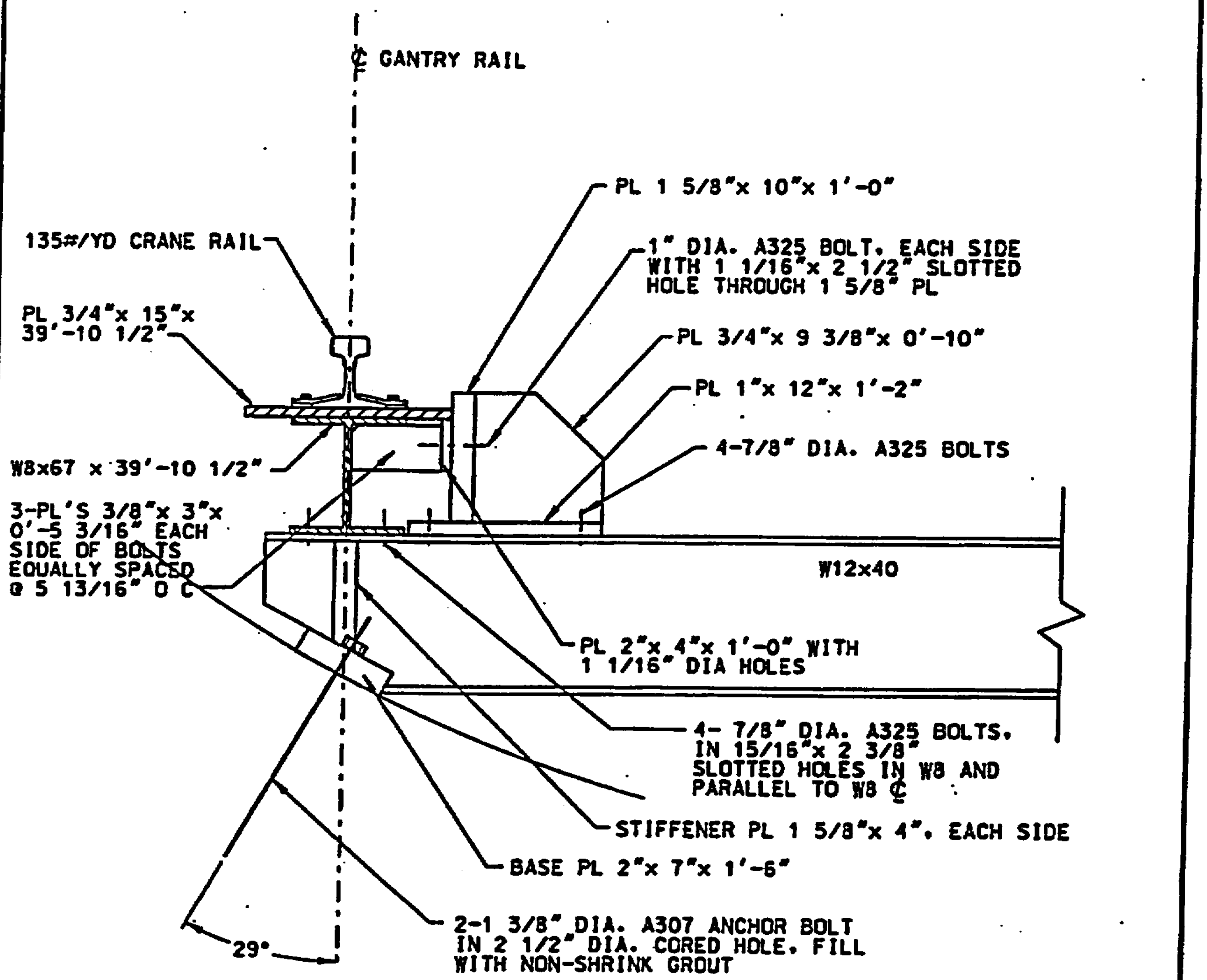


FIGURE I-5
GANTRY RUNWAY STEEL
INVERT FRAMING PLAN



ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE NOTED

FIGURE I-6
GANTRY SUPPORT STEEL
INVERT ELEVATION



NOTES:

1. STRUCTURAL STEEL MATERIALS FOR THE GANTRY RUNWAY SHALL BE IN ACCORDANCE WITH ASTM A6 AND ASTM A572.
2. CONNECTION MATERIAL SHALL BE AS FOLLOWS:
   BOLTS - ASTM A325 MINIMUM.
   NUTS - ASTM A563 MINIMUM.
   WASHERS - ASTM F436 MINIMUM.
   ANCHOR BOLTS - ASTM A307 MINIMUM.
3. INSTALLATION PROCEDURE FOR A325 BOLTS IN SLOTTED HOLES:
   a. SHIM CONNECTIONS AS NEEDED TO PROVIDE FULL CONTACT BETWEEN FAYING SURFACES.
   b. TORQUE BOLTS TO 3 ft-lbs AND DOUBLE NUT.

FIGURE I-7
GANTRY RUNWAY STEEL
INVERT DETAIL

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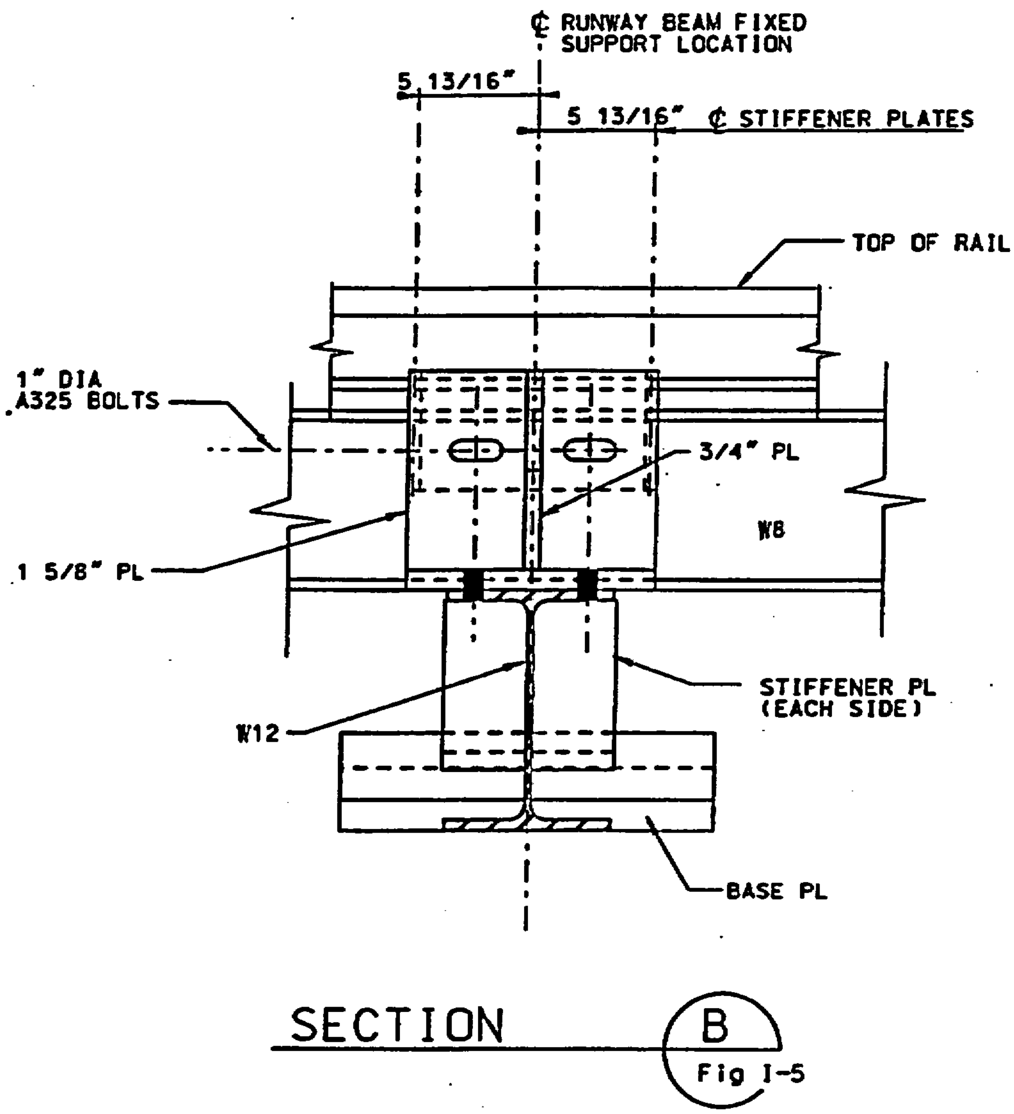

FIGURE I-8
GANTRY SUPPORT STEEL
FIXED SUPPORT DETAIL

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ATTACHMENT II

CONCRETE INVERT

1. INTRODUCTION

Concrete invert segments for emplacement drifts vary in configurations and consist mainly of two key components: a flat pit for supporting the waste package support assembly and two haunches for supporting the gantry rails. Figure II-1 shows a precast invert with a cast-in-place haunch on each side to support the gantry crane rail. The advantage of the cast-in-place haunch is that optimum line and grade of the crane rail can be achieved. The disadvantage of the cast-in-place haunch is the cost and efforts of providing anchorage of the haunch to the precast concrete invert. Anchorage can be achieved with a system of dowels and threaded dowel sleeves which allow the attachment of a rebar cage to reinforce the haunch concrete. An alternative would be to precast the invert with the haunch reinforcement exposed which would be a difficult safety issue to resolve. Casting the haunch in-place appears more costly and time consuming than precasting the haunch with the invert. The cast-in-place haunch will not be analyzed beyond the above conclusion.

Figures II-2 and II-3 show, respectively, the schematic of the precast concrete invert and haunch for two different configurations that will lead to a different haunch height (measured from top of the rail to bottom of the invert) and invert segment width. As represented in Figures II-2 and II-3, a change in the diameter of the gantry wheel from 600 mm to 300 mm could accommodate this change in haunch height. Other invert configurations are possible and can be evaluated as design progressess. The configuration in Figure II-3 is the basis used for the models in this analysis and was developed from the waste package support layout shown in Figure II-4. The configuration of the precast invert can be modified to accommodate variations of the waste package support system.

2. NUMERICAL APPROACH

A two-dimensional finite difference code, FLAC, was used to analyze the stress development in an emplacement drift concrete invert under variable load conditions expected during preclosure. The analyzed concrete invert was in both precast and cast-in-place form. A typical cross-section of emplacement drifts was chosen for this analysis. Numerical simulation began with excavation of emplacement drifts, then was followed by installing the concrete invert and lining, and ended with the detailed calculation of stresses in the invert under each loading condition. In light of the fact that the invert is continuous and that rock strata plunge little along the drift alignment, use of a two-dimensional model instead of a three-dimensional one was considered to be adequate and effective.

Except for the potential seismic load, all other loads discussed in Sections 4 and 7 of the main text of this analysis were considered as static loads. In carrying out numerical simulation using FLAC models under seismic loads, the seismic loading was expressed in terms of a combination of sinusoidal compressional and shear waves with their amplitude, frequency and duration equal to specified values. In addition, the earthquake-induced ground acceleration of 0.27g was rounded off to 0.3g (Section 4.3.1).

3. MODELS AND BOUNDARY CONDITIONS

3.1 BEAM ELEMENT (BE) REPRESENTATION

Figure II-5 illustrates a FLAC model that numerically represents a concrete invert in the middle drift by beam elements. Beam elements are two-dimensional elements with three degrees of freedom (x-translation, y-translation, and rotation) at each end node, and are used to represent a structural member in which bending resistance and limited bending moments are important. The entire invert is subdivided into 14 beam segments which are either numerically bonded at both ends to the underlying rock to simulate the cast-in-place concrete invert scenario or linked to the underlying rock through a continuous interface in order to simulate a precast concrete invert scenario. The model contains five drifts at a uniform spacing of 28 m (Assumption 4.3.11). From the middle drift, the model extends 70 meters (approximately 13 diameters) horizontally to model boundaries and 50 m (approximately 9 diameters) vertically to outer model boundaries. These distances are considered to be sufficiently far away from the drift not only to diminish the boundary effect on numerical results under the static loading condition but also to minimize wave reflections and achieve nearly free-field conditions at the sides of the model. In addition, mutual influence due to excavation of adjacent drifts are taken into account by mining the five drifts in sequence.

During the simulation of seismic loading, viscous boundary conditions are used at the base and top of the FLAC model to prevent the outward propagating waves from reflecting back into the model at those boundaries. The two vertical lateral boundaries are set to be free field conditions. The seismic loads are imposed on the model after equilibrium has been reached under the in situ stress field. Therefore, the initial velocity for each grid point prior to the application of dynamic loads is zero. The sinusoidal velocity waves (P-wave and S-wave) are applied at the bottom boundary and propagate vertically upwards, i.e., at an incidence angle of zero degrees. The shear wave causes horizontal ground vibration (shaking) and is a leading cause of structural damage while the P wave oscillates the ground in compression and tension. As emplacement drifts are experiencing the vibratory motion, the installed concrete inverts and linings will also respond to the seismic loading accordingly.

As was mentioned earlier, the contact between rock and precast concrete invert is numerically represented as an interface that acts just like joints in the rock. Normal and shear stiffness properties as well as friction coefficient for the interface are input parameters. Both overcut caused by TBM vibration and cutter wear will result in the drift diameter varying slightly along the drift axis. Consequently, a precast concrete invert segment will unavoidably have a rather non-uniform contact with excavated rock surface. Furthermore, the contact will be irregular from invert segment to invert segment along the drift axis.

For the interfacial contact between rock and precast concrete, a reasonable assumption is that the shear stiffness of the contact will be very low, and that unknown gap distribution will cause a quite variable normal stiffness distribution, with very low normal stiffness likely during the ground relaxation after installation of the lining. The similarity in elastic moduli between concrete and rock mass leads to a reasonable assumption that there is the similarity between rock joints and concrete/rock interface. Thus, the low bounds of stiffness values for jointed rock specimens are used for simulating the contact between rock and concrete lining. A value of 50 GPa/m for the normal stiffness and a value of 10 GPa/m for the shear stiffness were chosen to represent the interfacial contact between rock and precast concrete lining (Assumption 4.3.12). These values correspond approximately to the lower bounds of the joint stiffness values for the TSw2 unit. A friction angle of 35 degrees was also assumed for the interface between rock and concrete.

It is further pointed out that the BE representation of a concrete invert approximates the invert configuration by a curved beam with uniform thickness. No geometric account is given to such invert details as haunches for hosting gantry rails or flat invert pit for hosting waste package support piers. Therefore, the modeled beam section area is kept to a minimum and the corresponding numerical results are conservative.

3.2 PLANE STRAIN ELEMENT (PSE) REPRESENTATION

Figure II-6 shows a FLAC model that numerically represents the concrete invert by plane strain elements. Plane strain elements are two-dimensional elements with two degrees of freedom at each nodal point. Bending is not represented. Stress components are given at the centroid of each element. These stress components include normal stress in the thrust direction, normal stress in the radial direction, and shear stress. Principal stresses can be calculated based on the stress components in the Cartesian coordinate system. This model is able to represent the geometry of a concrete invert in greater detail than the BE model. For example, the waste package cross-section is represented by a 2 m diameter circle centered at 0.715 m below the drift center (Figure II-4). Accordingly, the primary objective of this model is to help show how stresses are distributed in a concrete invert under a variety of loading conditions.

Drift excavation-induced stress in the invert is realized by allowing a partial ground relaxation (60%) prior to the placement of the concrete invert and lining. The balance of the ground relaxation will load the invert and lining. Other subsequent loads such as gantry and waste package weights on the invert are represented by concentrated nodal forces. Thermally-induced stress in the invert is not directly simulated. Numerical results from the emplacement drift ground support report (Reference 5.15) are used to calculate the total loads in the lining. A quasi-static approach to the seismic loading was adopted for this PSE model. A seismic ground acceleration of 0.3g leads to a force equal to the mass times this acceleration. For the present analysis, an additional static load equal to 0.3W (where W is the weight of single waste package) is applied to the mass center of a waste package both vertically and horizontally. The extra load represents the quasi-static effect of earthquake-induced peak ground acceleration on the invert.

In this PSE model, a thin and soft layer is inserted between concrete and rock elements along the contact. This layer, being softer than both rock and concrete, accounts for 1) gaps and irregularities along the contact, 2) damage or weakening to the exposed rock due to excavation, 3) weak bonding between concrete and rock, and 4) contact grout. The layer should be ductile enough to allow for lateral (circumferential) movement of the concrete invert segment and, in the meantime, be stiff enough to prevent unrealistic excessive shear stress and displacement from developing underneath the concrete invert. The elastic modullus values for this layer are given in Section 4.3.16. With the incorporation of this thin and soft contact layer in the PSE model, both precast and cast-in-place concrete inverts are modeled in the same way. Numerical results apply to both precast and cast-in-place concrete invert scenarios.

4. NUMERICAL RESULTS AND DISCUSSION

All FLAC models constructed are capable of approximating such loading conditions for the invert as excavation, TBM transportation load, waste package weight, gantry weight, and seismic loading. However, the thermal load generated by the heating from emplaced waste packages cannot be represented accurately due to the code limitation. To evaluate the invert stress caused by the long-term thermal loading, numerical results are cited from an emplacement drift ground support report (Reference 5.15) which uses a different numerical code to analyze the stress development in a full lining where the concrete invert forms one segment in the lining. In principle, as a part of this circular liner, the invert segment will experience the same thrust as lining segments do. According to Reference 5.15, Section 7.6.2.2.2, the maximum lining stress caused by elastic ground relaxation and heating combined is 19 MPa for the category-1 rock mass condition and 14 MPa for the category-5 rock mass condition. Furthermore, the minimum compressive lining stress caused by the ground relaxation alone is 4.6 MPa for the category-1 rock mass and 1.4 MPa for the category-5 rock mass. Therefore, the maximum portion of the compressive lining stress caused by the heat alone is 14.4 MPa (= 19 - 4.6) for the category-1 rock mass and 12.6 MPa (=14 - 1.4) for the category-5 rock mass. For this invert analysis, a maximum thermally-induced lining stress of 15 MPa was adopted in computing the total stresses in the concrete invert.

4.1 RESULTS FROM BEAM ELEMENT MODELS

Figure II-7 illustrates the implementation of different loading conditions in the beam element invert model while Figure II-8 shows the loading implementation in the plain strain invert model. TBM transportation load is a moving load and simulated as concentrated loads at wheel contact points. Waste package weights pass through a pedestal and pier to the invert and are simulated as a distributed load. Gantry weight is simulated as a concentrated load in the BE invert mode and as a line load in the PSE invert model. Seismic waves are simulated as body waves that cause dynamic strains in media they propagate through. Loads due to excavation, gantry weight, and waste package weights are static while the seismic loading is dynamic.

4.1.1 Precast Concrete Invert

Figures II-9 through II-14 show the development of thrust in a precast concrete invert under each individual phase of loading. The corresponding bending moment distribution along the invert is shown in Figures II-15 through II-20. Table II-1 summarizes the maximum thrust and bending moment for each loading scenario. Thermal loading condition was not simulated.

Numerical results tabulated in Table II-1 indicate that external loads such as gantry and waste package weights on the invert tend to decrease the thrust but to increase the bending moment in the invert. By recognizing the fact that for a thick wall cylinder with uniform pressure on inside and outside surfaces an increase in the inside pressure will decrease the compressive hoop stress in the cylinder, a decrease in thrust is explained by roughly viewing the invert and lining as a pressurized cylinder. An increase in bending moment can be explained by viewing the invert as a simply-supported beam in which the bending moment increases as the external loads increase. Table II-1 also reveals that the seismic loading causes a momentary increase in thrust from 116.8 to 514.7 kN per linear meter of the invert. Also, the bending moment is increased from 63.0 to 78.1 kN-m per linear meter of the invert. As a result, the maximum momentary increase in the compressive hoop stress caused by a vibratory ground motion of 16 cm/s in the concrete invert is shown to be 4.3 MPa, a small fraction of the concrete strength.

The maximum compressive stress shown in Table II-1 is calculated based on the maximum thrust, the maximum bending moment, and the narrowest sectional area per linear meter of the invert. Let T stand for the maximum thrust, M for the maximum bending moment, h for the thickness at the narrowest section (i.e., 200 mm), A (= 1xh) for the sectional area, and I (=1*h3/12) for the moment of inertia, then,

Maximum compressive stress = [(M/I)(h/2)] + T/A

Maximum tensile stress = [(M/I)(h/2)] - T/A (only if (M/I)(h/2) > T/A).

Table II-1. Maximum Thrust and Bending Moment for a Precast Concrete Invert (with an interface between concrete and rock)

Loading Scenario	Thrust (kN)/m	Bending Moment (kN-m)/m	Max. Compressive Stress (MPa)	Max. Tensile Stress (MPa)
After Excavation	130.6	61.3	9.8	8.5
During TBM Transportation	113.5	66.0	10.5	9.3
During Emplacement: 1 WP + Gantry	108.8	94.6	14.7	13.6
During WP's Sitting on Invert	116.8	63.0	10.0	8.9
During Retrieval: 2 WPs + Gantry	108.5	103.5	16.1	15.0
During Earthquake: WP's in Place	514.7	78.1	14.3	9.1

It must be pointed out that the bending moment values shown in Table II-1 are quite conservative because of the piecewise discontinuous contact numerically represented between rock and lining. The lining and invert segments are represented by a sided polygon while the drift perimeter is represented by a sided polygon. The contact between these two polygons is piecewise continuous.

4.1.2 Cast-In-Place Concrete Invert

Figures II-21 through II-26 show the development of thrust in a cast-in-place concrete invert under each individual phase of loading. The corresponding bending moment distribution along the invert is shown in Figures II-27 to II-32. Table II-2 summarizes the maximum thrust and bending moment development.

Although maximum compressive stresses are close to those tabulated in Table II-1, bending moments are substantially less than those in Table II-1. In light of the fact that a numerically perfect bond between concrete and underlying rock not only ensures a full and uniform contact but also acts as a continuously-supported beam, it is plausible that the bending moment in a cast-in-place concrete invert will be less than that for a precast concrete invert. With contact grouting for the precast concrete invert, the thrust and bending moment will be close to those for a cast-in-place concrete invert. It can be seen from Table II-2 that a vibratory ground motion of 16 cm/s causes a momentary increase of 5.1 MPa in the compressive hoop stress in the concrete invert. The results is consistent with that predicted by the model with an interface present between the rock and concrete. The maximum compressive and tensile stresses shown in Table II-2 are calculated in the same way as discussed in Section 4.1.1 for the precast concrete invert.

Table II-2. Maximum Thrust and Bending Moment for a Cast-In-Place Concrete Invert (with a perfect bond between concrete and rock)

Loading Scenario	Thrust (kN)/m	Bending Moment (kN-m)/m	Max. Compressive Stress (MPa)	Max. Tensile Stress (MPa)
After Excavation	1410.0	14.8	9.3	0
During TBM Transportation	1371.0	16.7	9.4	0
During Emplacement: 1 WP + Gantry	1373.0	11.2	8.5	0
During WP's Sitting on Invert	1368.0	16.3	9.3	0
During Retrieval: 2 WPs + Gantry	1330.0	12.7	8.6	0
During Earthquake: WP's in Place	2312.0	18.7	14.4	0

4.2 RESULTS FROM PLANE STRAIN ELEMENT MODEL

With the plain strain element model for the invert, no distinction is made between precast and cast-in-place concrete inverts. Stress distribution in the concrete invert under different loading scenarios is shown in Figures II-33 through II-38. The stresses shown in each figure are the principal stresses. The sign convention for principal stresses is positive in tension and negative in compression. Stress distribution plots indicate that the major principal stress (σ_1) is in the circumferential direction while the minor principal stress (σ_3) is in the radial direction. From Figures II-33 to II-38, it can be seen that highest stresses occur at the connection areas between the lining and invert. The circumferential stress in the center portion of the invert is uniform. Table II-3 further summarizes the maximum stress development in the concrete invert.

Table II-3. Principal Stresses in a Concrete Invert (based on the PSE element model)

Loading Scenario	Maximum Major Principal Stress (MPa)	Maximum Minor Principal Stress (MPa)
After Excavation	9.8	<1
During TBM Transportation	9.4	<1
During Emplacement: 1 WP + Gantry	9.6	<1
During WP's Sitting on Invert	9.6	<1
During Retrieval: 2 WPs + Gantry	9.4	<1
During Earthquake: WP's in Place	9.7	<1

Compressive stresses in a concrete invert based on the PSE model are in reasonable agreement with those based on BE models. However, the PSE model indicates that there are no tensile stresses developed in the invert, as the model ignores the bending. The PSE model predicts that the stress across the thickness of a concrete invert is below 1 MPa under each loading condition, as shown in Figures II-33 through II-38.

5. SUMMARY AND CONCLUSION

Mechanical response of an emplacement drift concrete invert to the loading combinations from the in situ stress, construction, operation activities, thermal load, and potential seismic events has been analyzed using two-dimensional numerical models. These models have made conservative approximation to the three-dimensional loading conditions. It is also noted that two-dimensional FLAC models used for this analysis do not account for the seismic effect on the rock mass in the longitudinal direction, i.e., along the drift axis. Shear waves vibrating parallel to the drift axis are anticipated to have much less effect on the drift than that vibrating perpendicular to the drift axis, as the former are confined while the latter are nearly unconfined near the drift wall. For the purpose of examining the behavior of a concrete invert in an emplacement drift, a two-dimensional analysis with the longitudinal direction treated as plain strain condition (confinement) is considered to be adequate.

The following summary is based on numerical results from BE and PSE models:

- For a precast concrete invert in a 5.5 m emplacement drift, there is a considerable hoop stress developed in the invert due to excavation, depending on the timing of invert and lining installation with respect to the TBM advance. Gaps and nonuniform contact between concrete and rock have a strong effect on the development of the bending moment in the invert. On the other hand, a momentary increase of 4.3 MPa (Table II-1) for the hoop stress in the concrete invert is predicted during a vibratory ground motion of 16 cm/s caused by a potential seismic event. Excluding the thermally-induced stress, the numerical model predicts that the highest hoop stress in a precast concrete invert without contact grout is about 16 MPa in compression and 15 MPa in tension (Table II-1) under the in situ and seismic loads combined. With an estimated amount of 15 MPa caused alone by the waste package heat, the hoop stress in a precast concrete invert adds up to 31 MPa under the in situ, thermal and seismic loads combined, calling for the concrete with a minimum 31 MPa (about 4,500 psi) allowable strength. Also, tensile stress development in the invert calls for steel reinforcement.

- For a cast-in-place concrete invert in a 5.5 m emplacement drift, external loads such as TBM, gantry and waste package weights have no significant effects on the stresses in the invert. In this respect, the weight of any backfill materials, if used, over the emplaced waste packages is anticipated to have little effect on the invert stresses. A momentary increase of 5.1 MPa (Table II-2) for the hoop stress in the concrete invert is predicted during a vibratory ground motion of 16 cm/s caused by a potential seismic event. Excluding the thermally-induced stress, the load caused by ground relaxation due to drift excavation is the primary consideration. Under the in situ and seismic loads combined, the maximum hoop stress in the concrete invert is shown to be 14.4 MPa (Table II-2). With an estimated amount of 15 MPa caused alone by the waste package heat, the hoop stress in a precast concrete invert adds up to 29.4 MPa under the in situ, thermal and seismic loads combined, calling for the concrete with a minimum 30 MPa (4,350 psi) allowable strength.

- The normal stress component in the direction of thickness of the concrete invert, either precast or cast-in-place, is predicted to be almost negligible (Figures II-33 to II-38). It must be pointed out that a considerable amount of conservatism was made in the modeling, e.g., lowest rock mass category used, and loads such as TBM, gantry and waste package weights represented as line distributed or concentrated loads in a two-dimensional numerical model. With proper contact grout, the stress development in a precast concrete invert will be similar to a cast-in-place concrete invert, i.e., the bending moment and subsequent tensile stress will be reduced. Then, the invert is primarily in compression.

- Reinforcement such as steel bars spaced in each direction at each face will be required in the precast invert for bending stresses from construction handling loads. As the bending moment caused in a precast invert by construction handling and emplacement operation loads is not significant, reinforcement can be conveniently done.

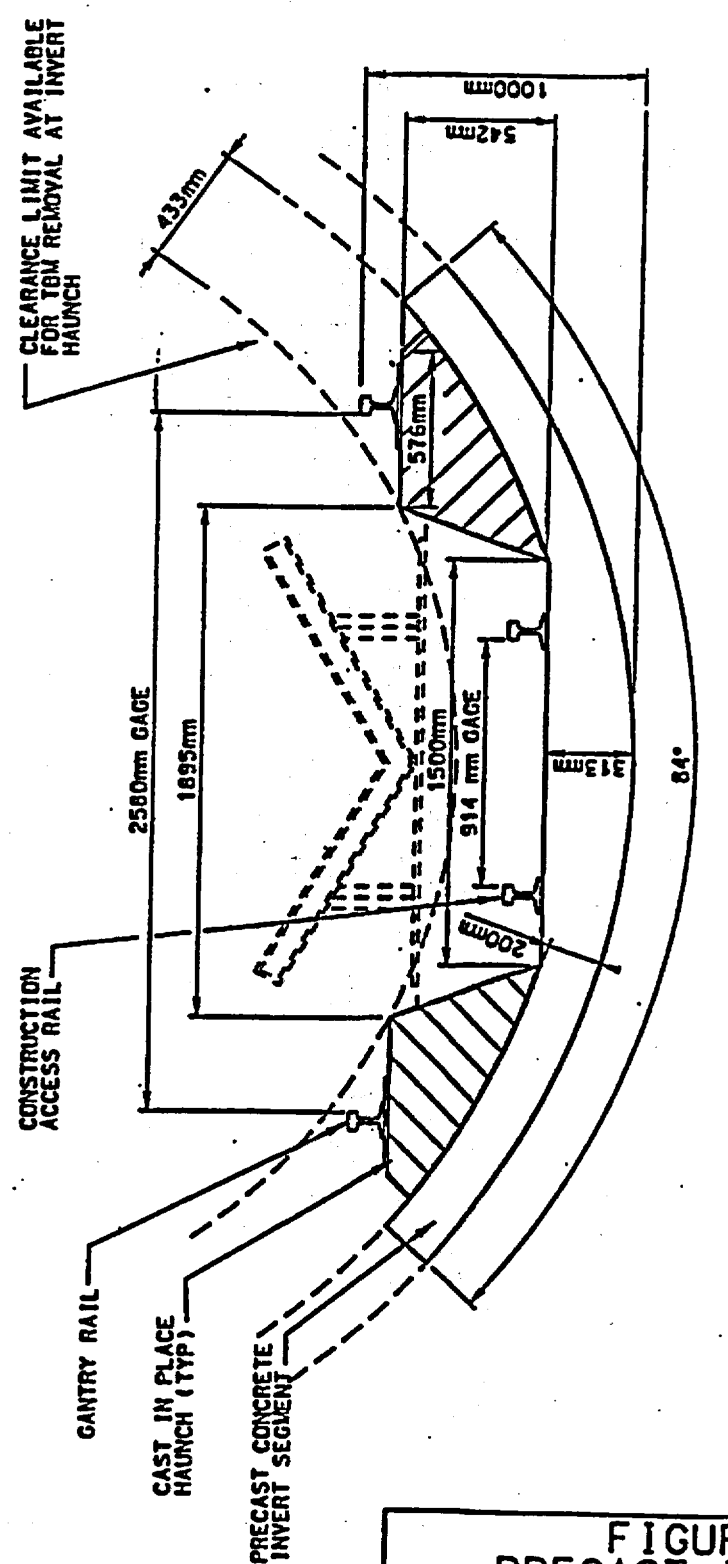

FIGURE II-I
PRECAST INVERT WITH
CAST-IN-PLACE HAUNCH

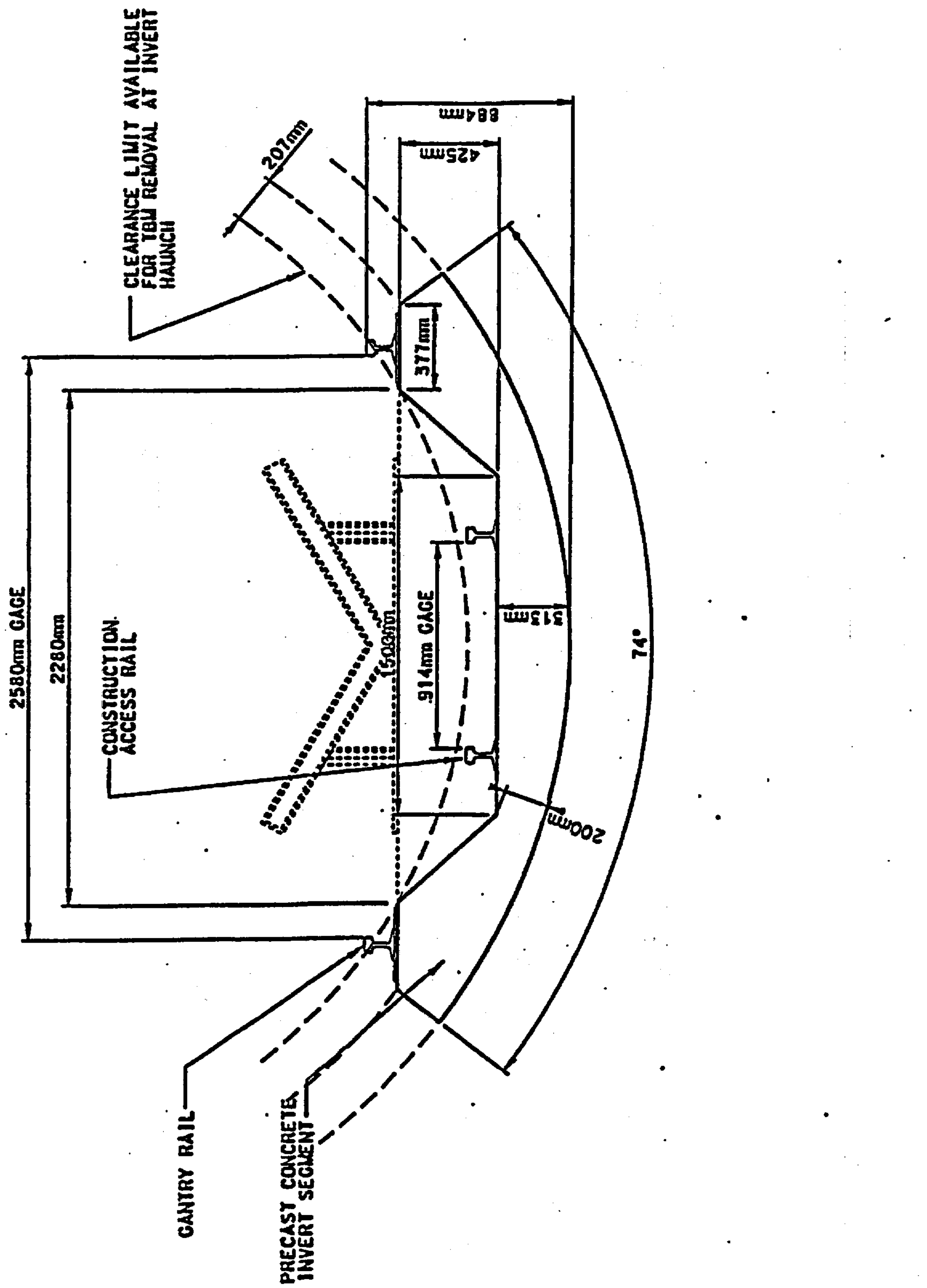

FIGURE II-2
PRECAST INVERT AND HAUNCH
600MM GANTRY WHEEL

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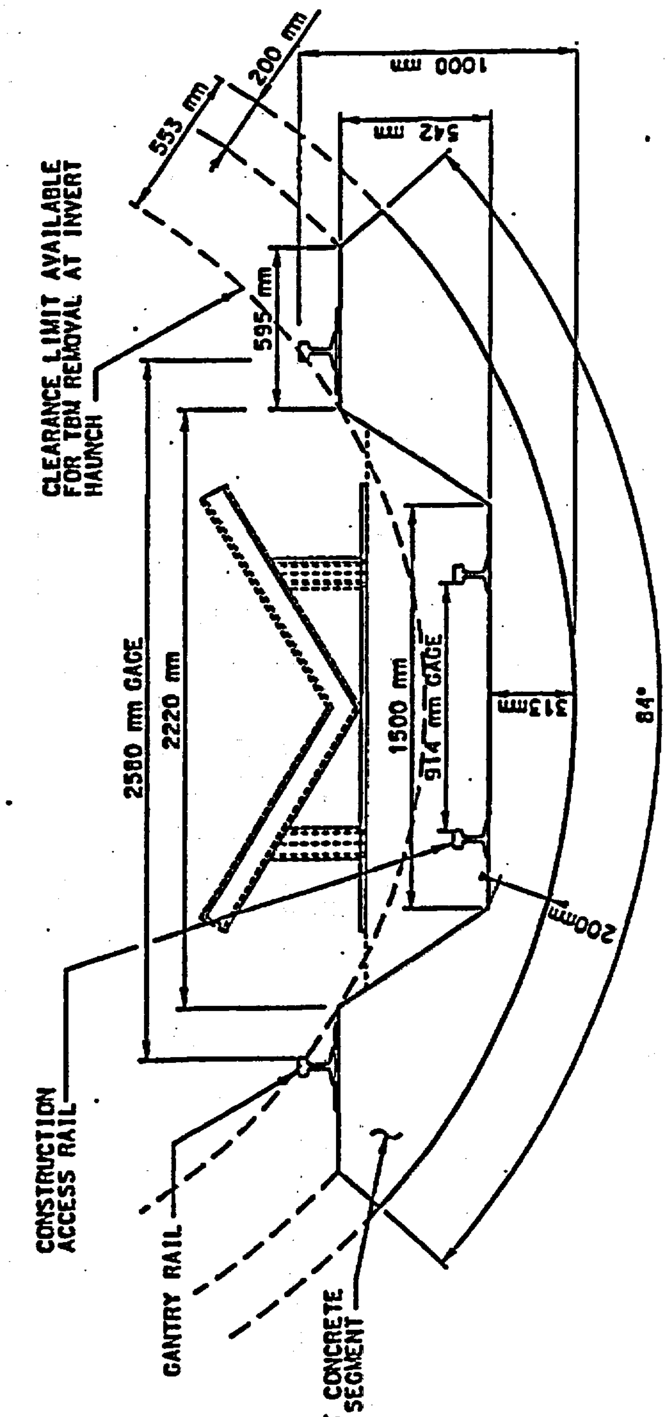

FIGURE II-3
PRECAST INVERT AND HAUNCH
300MM GANTRY WHEEL

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Table 1

WP TYPE	SIZE (diameter x length)	DISTANCE	WP LOADED MASS (kg)
21 PWR UCF	1649.7 x 5335	1933	50,423 est.
44 BWR UCF	1603.9 x 5335	1906	46,424 est.
12 PWR UCF	1297.7 x 5335	1629	32,236 est.
4 PACK DHLW	1706.0 x 3790 est.	1911	30,511 est.
5 PACK DHLW	1970.0 x 3790 est.	2018	35,692 est.

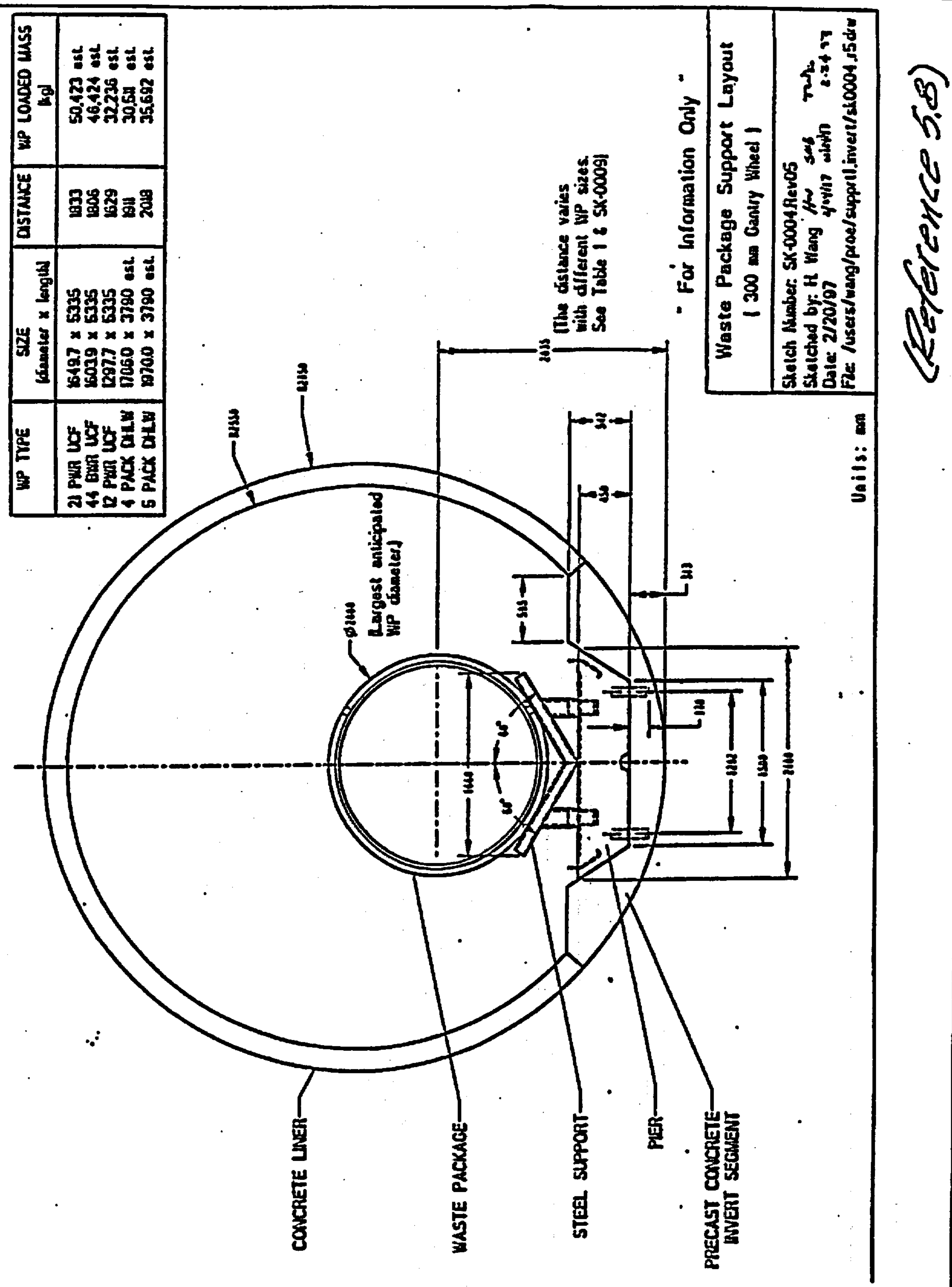


FIGURE II-4
WASTE PACKAGE SUPPORT LAYOUT

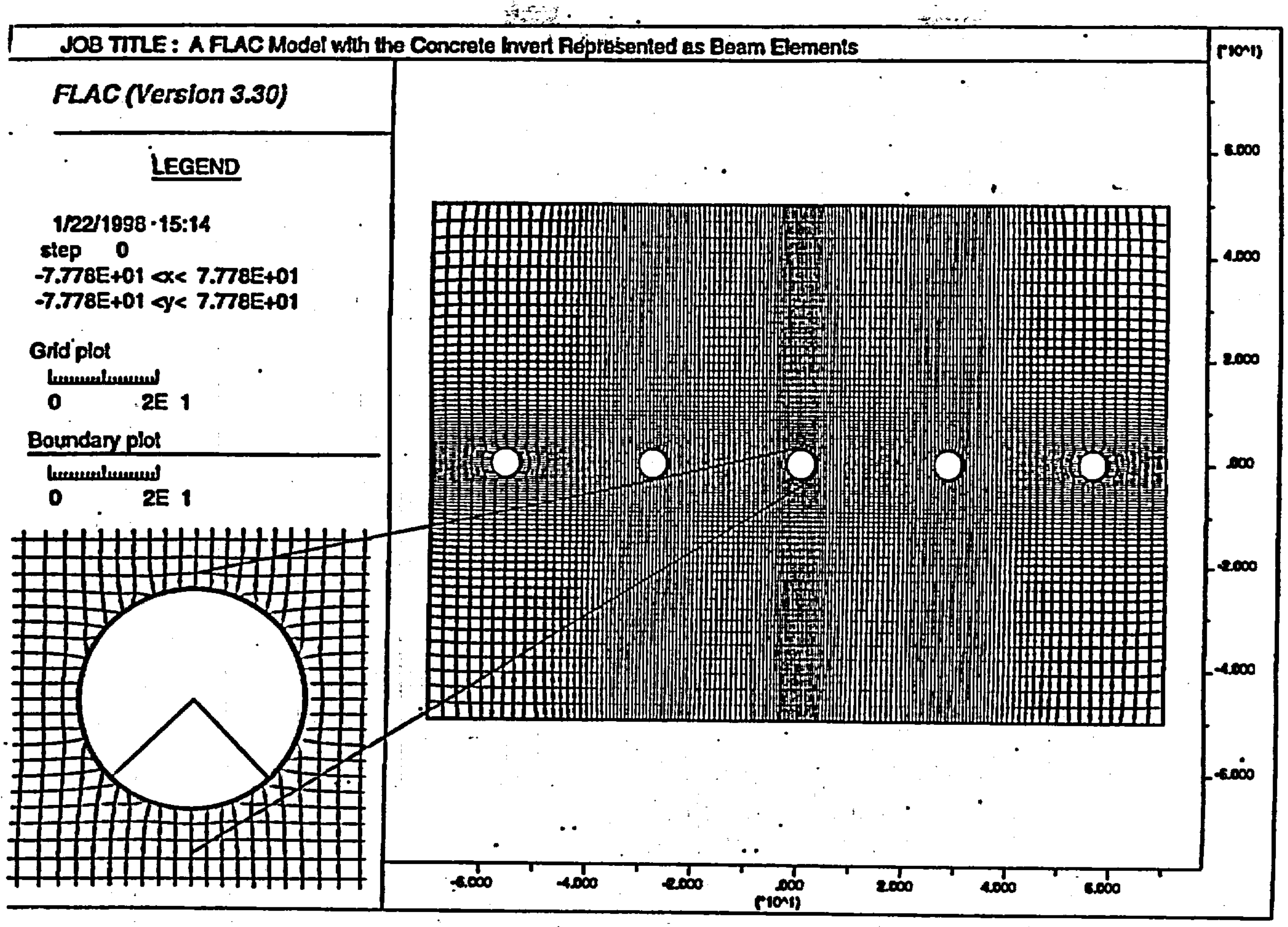


Figure II-5. A FLAC Model with the Concrete Invert Represented as Beam Elements (BE). Dimensions Are in Meters.

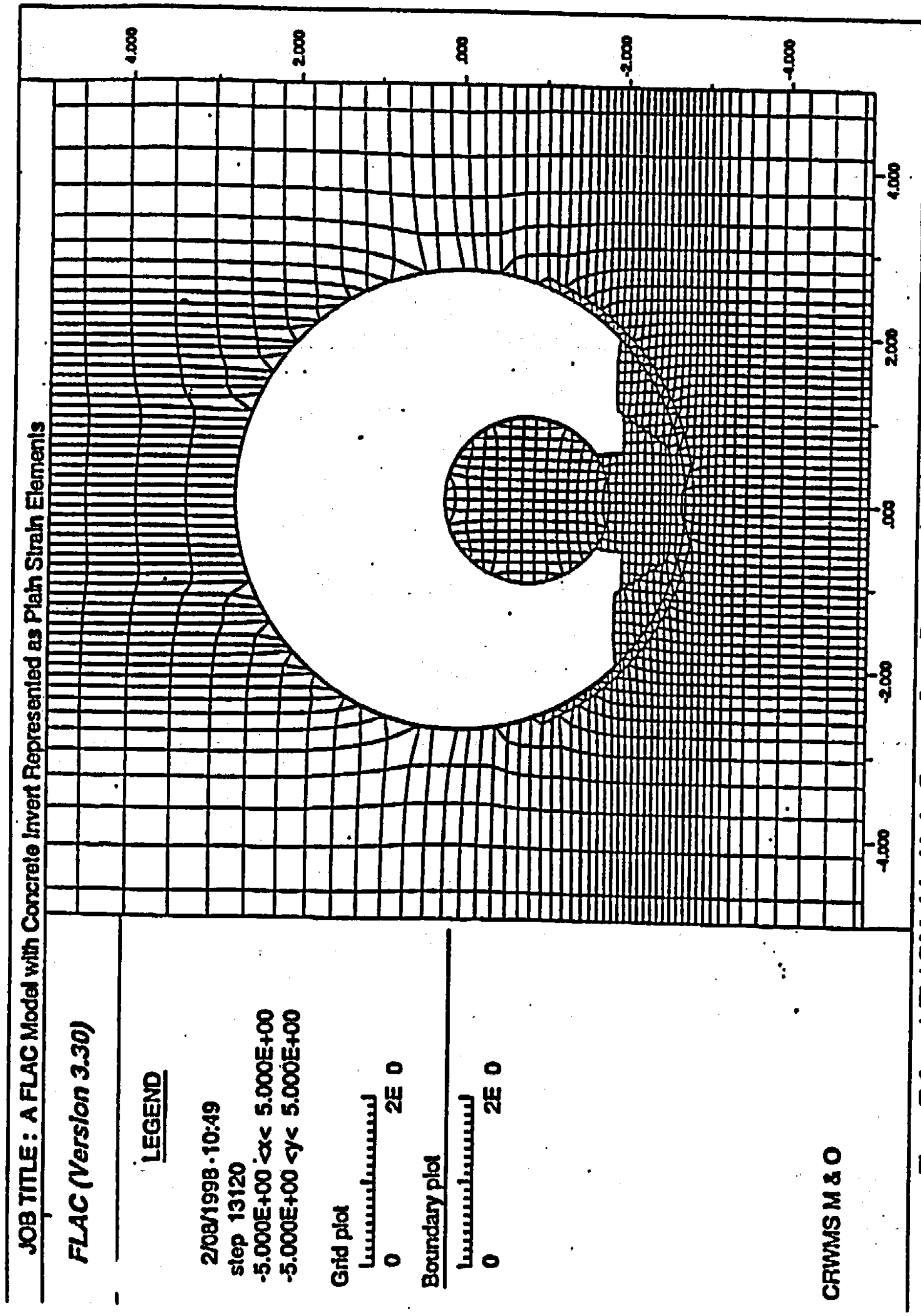


Figure II-6. A FLAC Model with the Concrete Invert Represented as Plane Strain Elements (PSE). Dimensions are in Meters.

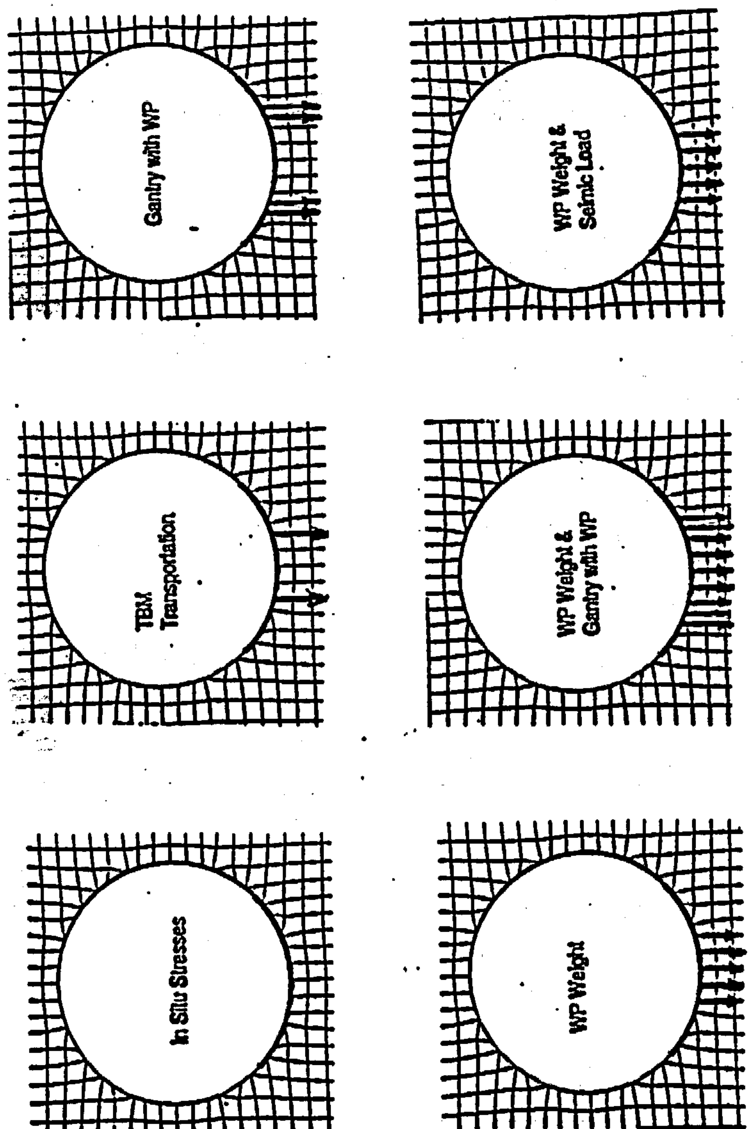


Figure II-7. BE Model: Representing the Loading Conditions.

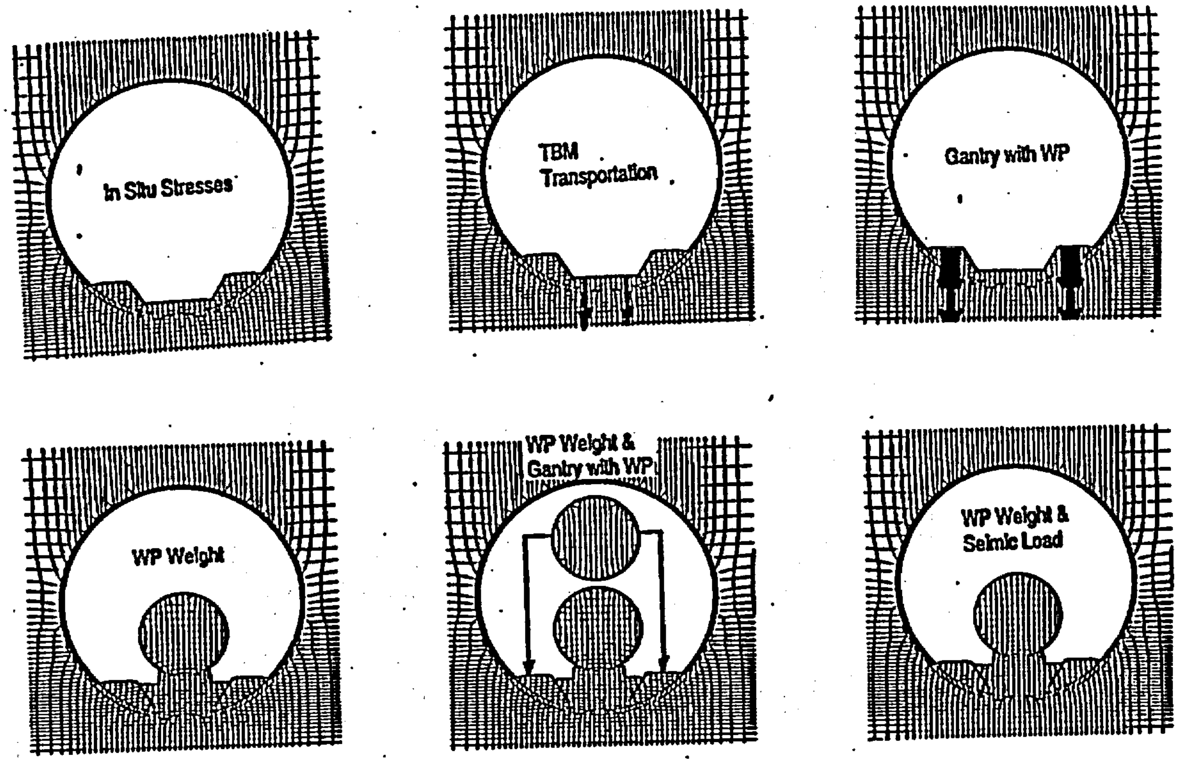


Figure II-8. PSE Model: Representing the Loading Conditions.

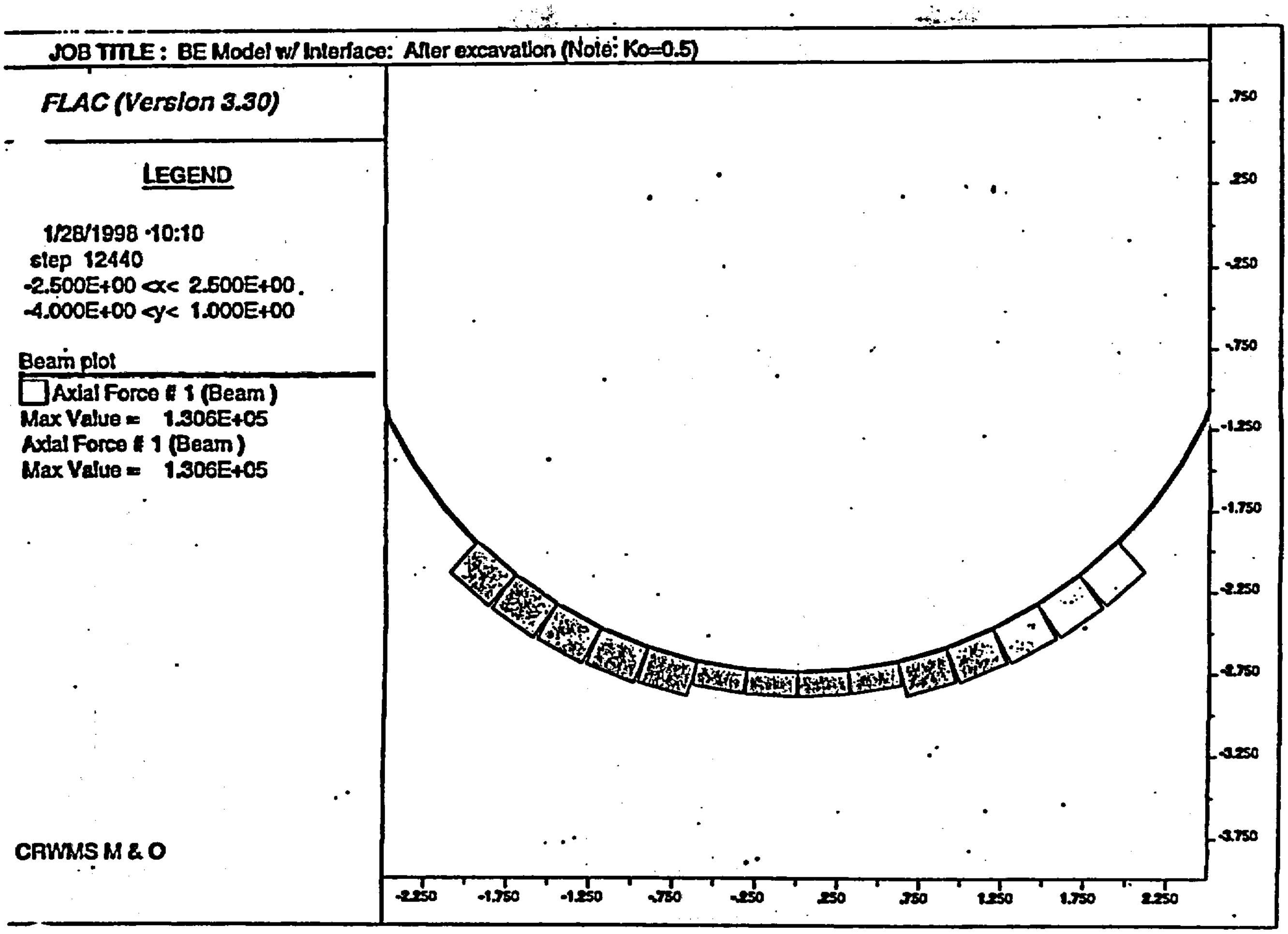


Figure II-9. BE Model with Interface: Axial Force Distribution in the Invert under the In Situ Load. Forces Are in Newtons/m and Dimensions Are in Meters.

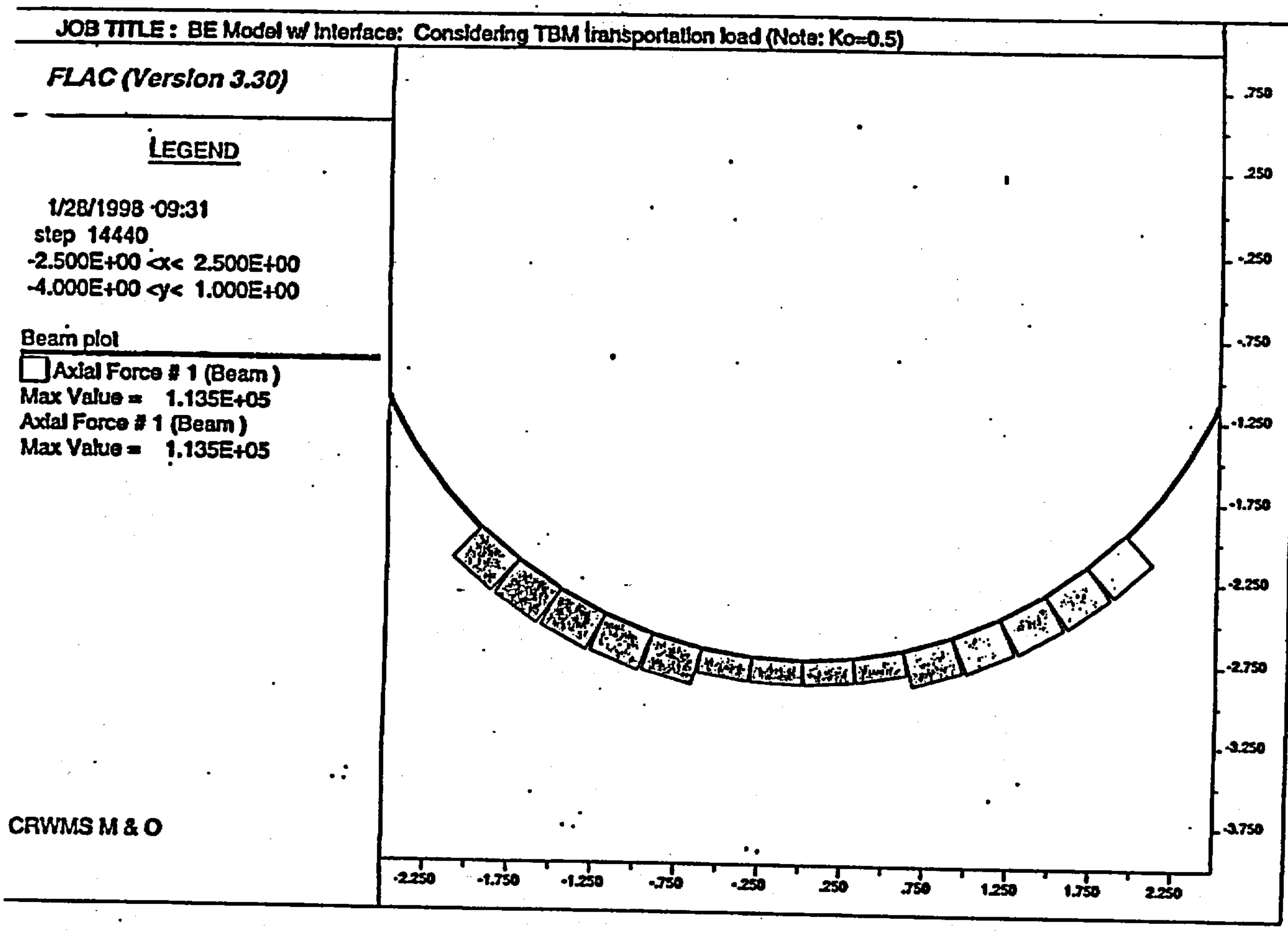


Figure II-10. BE Model with Interface: Axial Force Distribution in the Invert under the TBM Transportation Load. Forces Are in Newtons/m and Dimensions Are in Meters.

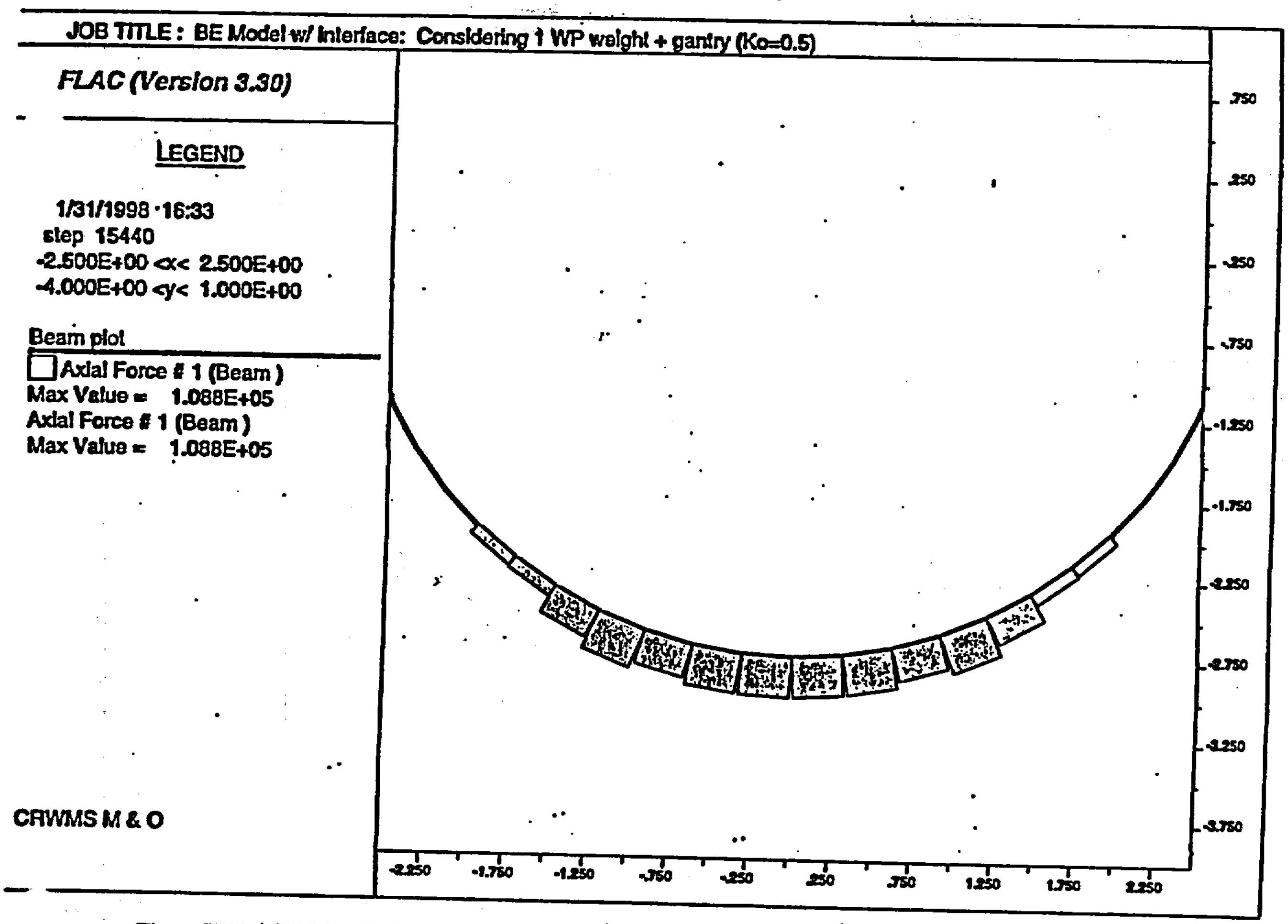


Figure II-11. BE Model with Interface: Axial Force Distribution in the Invert under the Loaded Gantry Weight. Forces Are in Newtons/m and Dimensions Are in Meters.

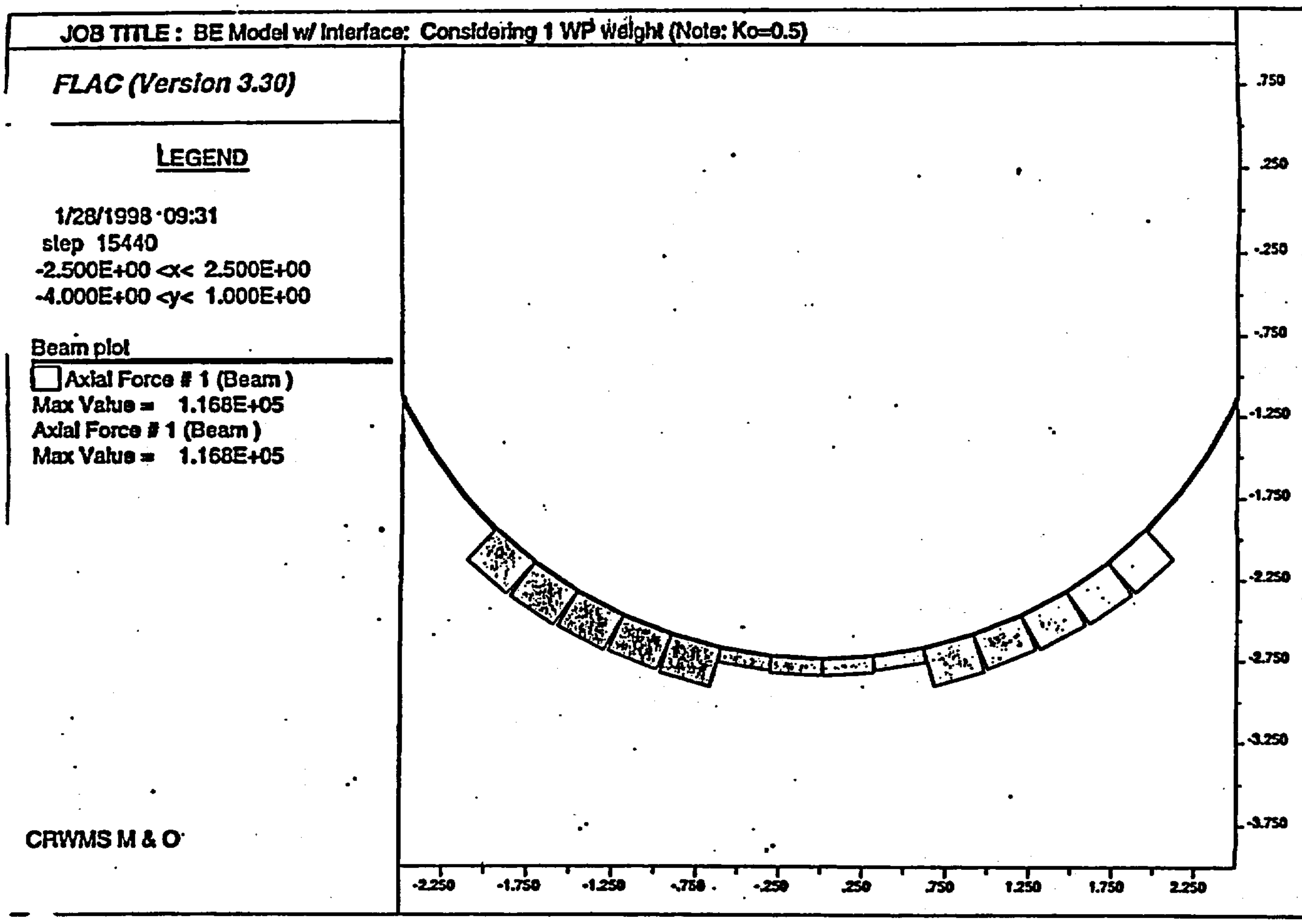


Figure II-12. BB Model with Interface: Axial Force Distribution in the Invert under the Emplaced Waste Package Weight. Forces Are in Newtons/m and Dimensions Are in Meters.

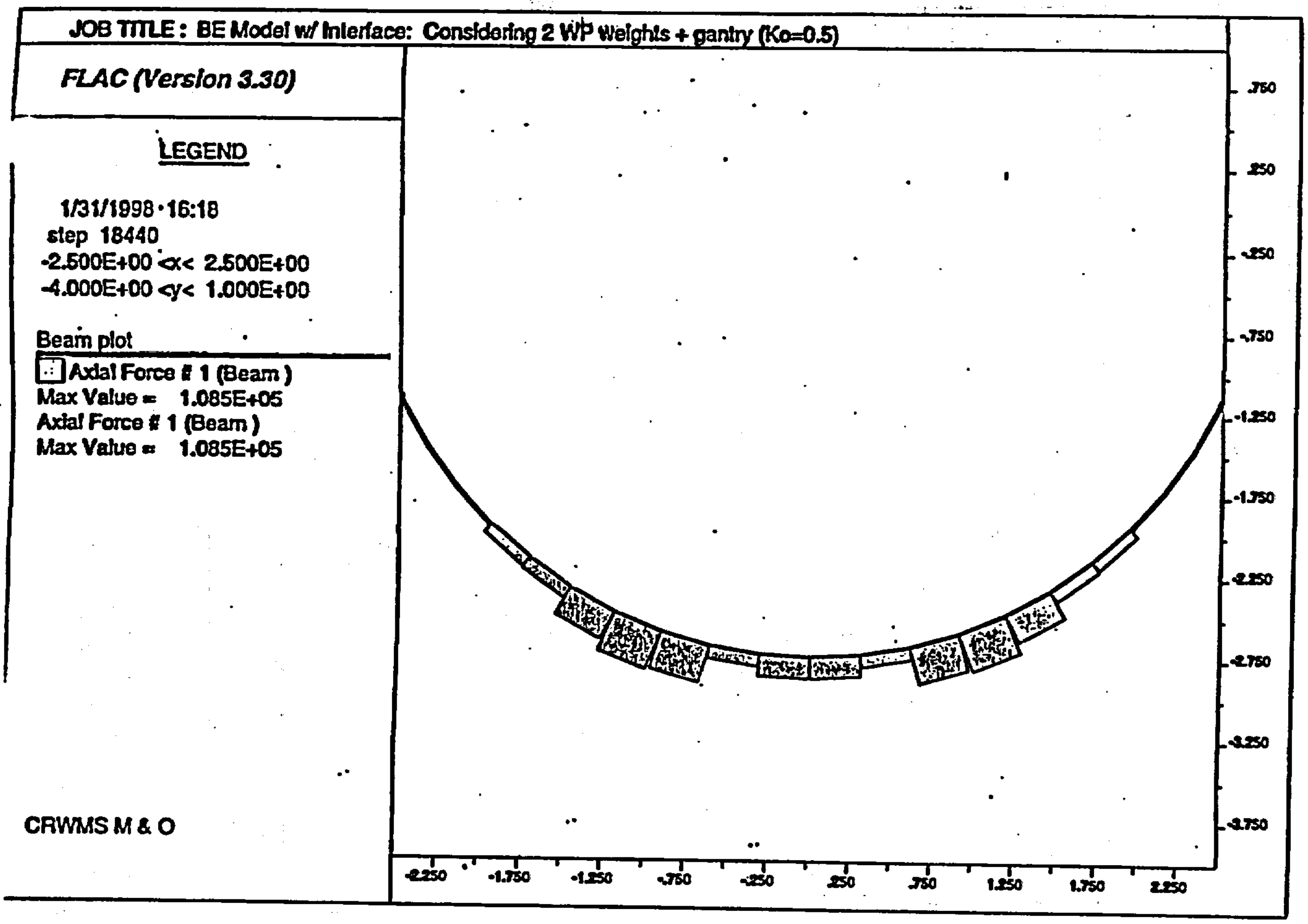


Figure II-13 BE Model with Interface: Axial Force Distribution in the Invert under the Emplaced Waste Package Weight plus the Loaded Gantry Weight. Forces Are in Newtons/m and Dimensions Are in Meters.

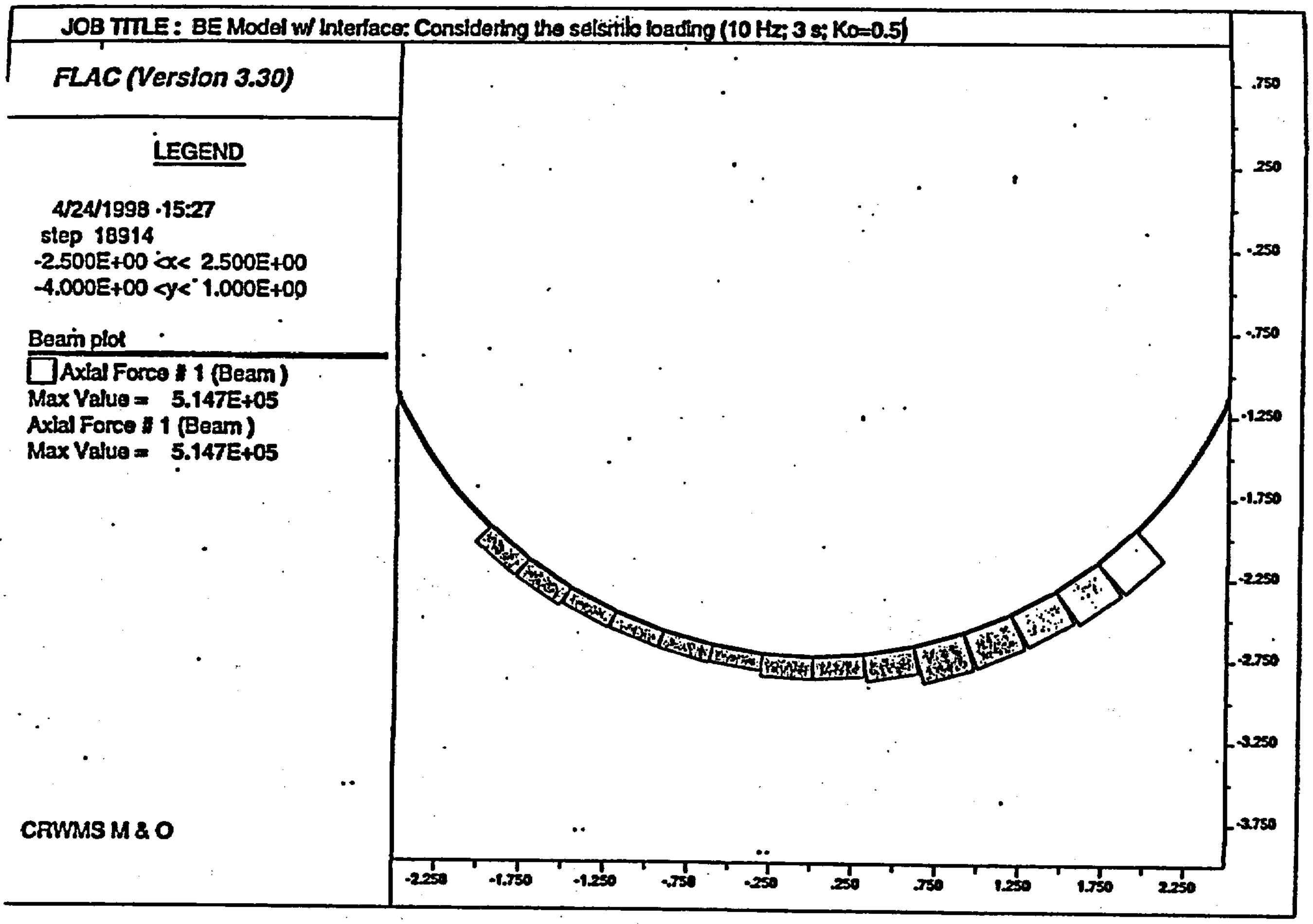


Figure II-14. BE Model with Interface: Axial Force Distribution in the Invert under the Emplaced Waste Package Weight plus the Seismic Load. Forces Are in Newtons/m and Dimensions Are in Meters.

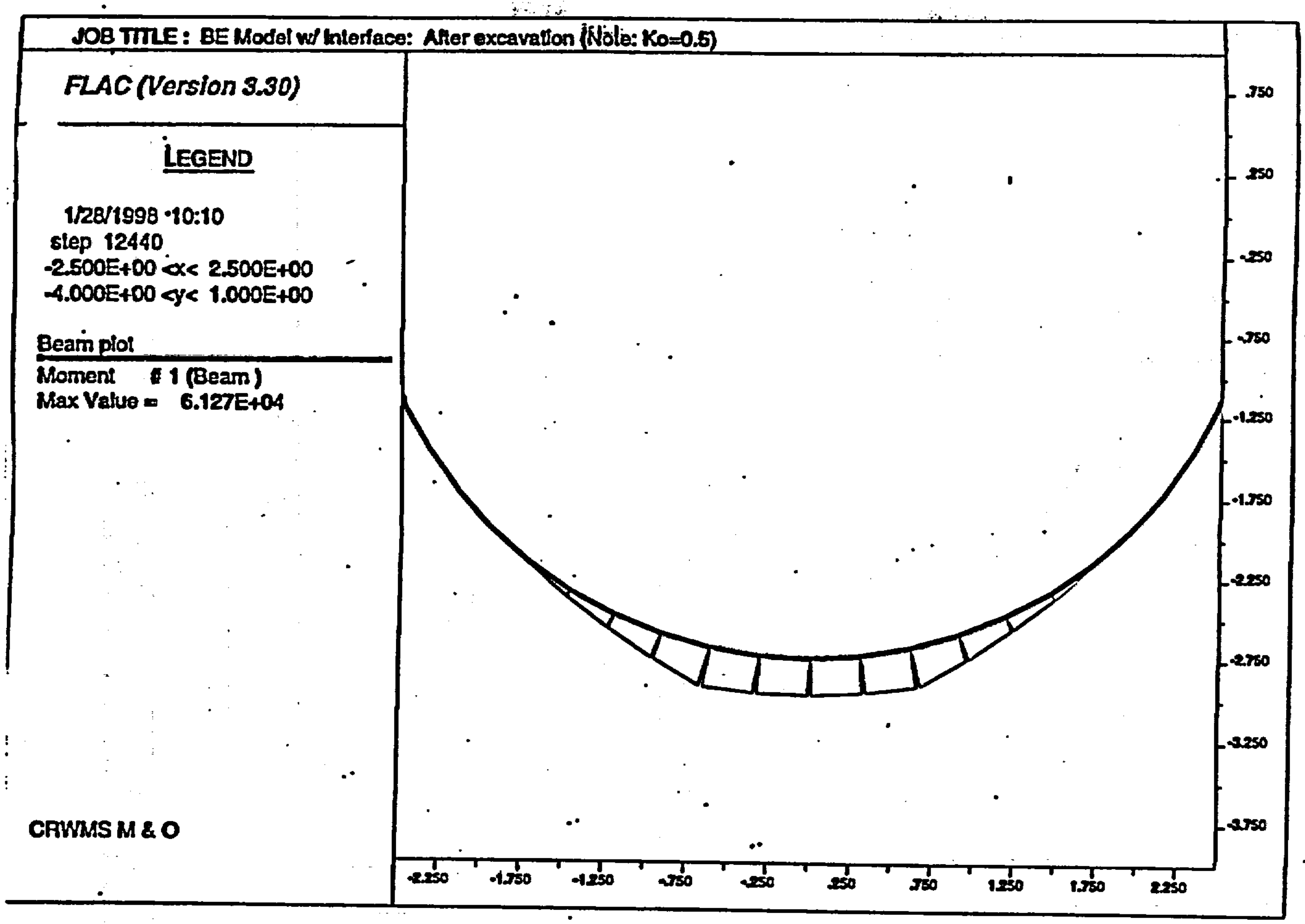


Figure II-15. BE Model with Interface: Bending Moment Distribution in the Invert under the In Situ Load. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

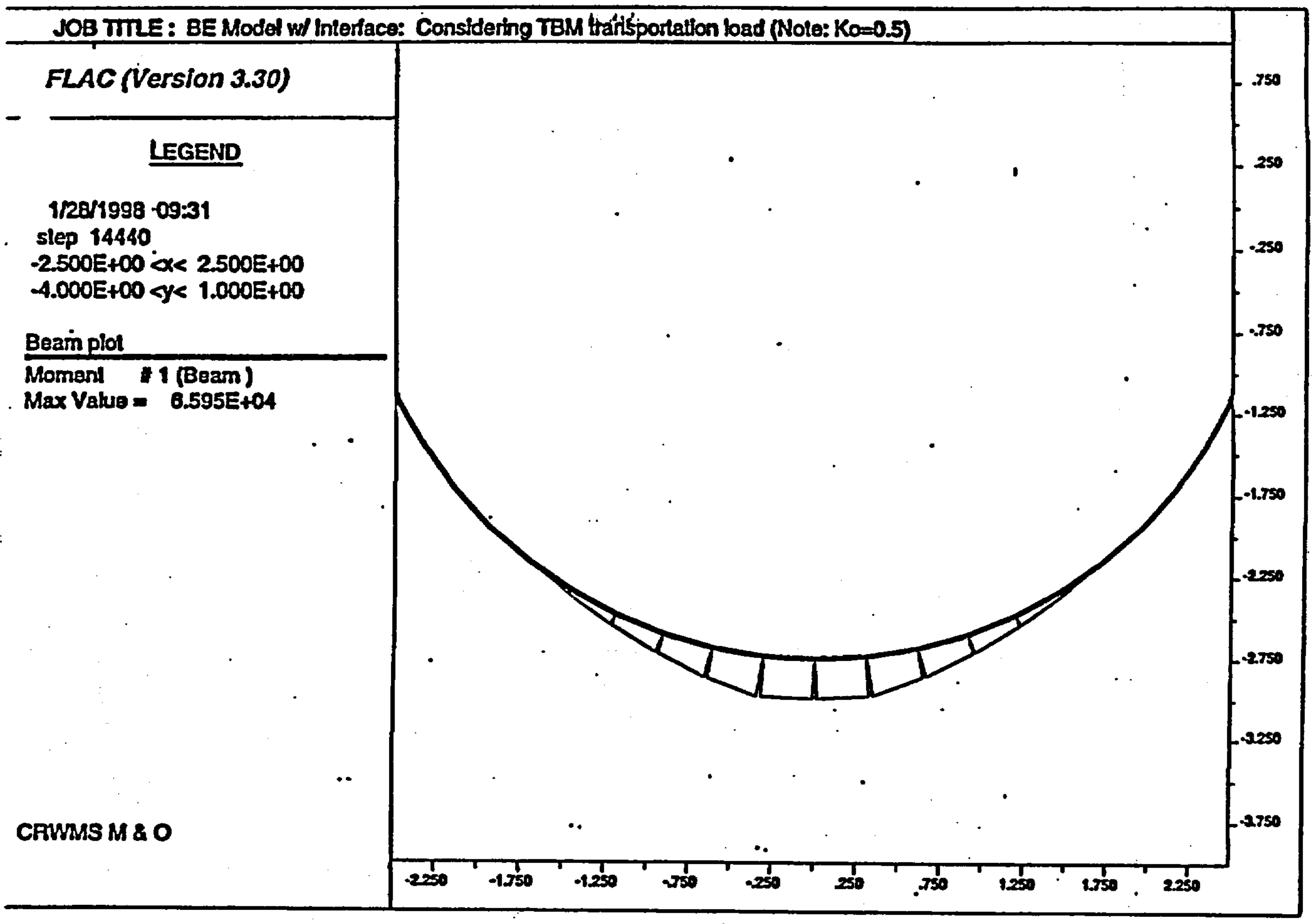


Figure II-16. BE Model with Interface: Bending Moment Distribution in the Invert under the TBM Transportation Load. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

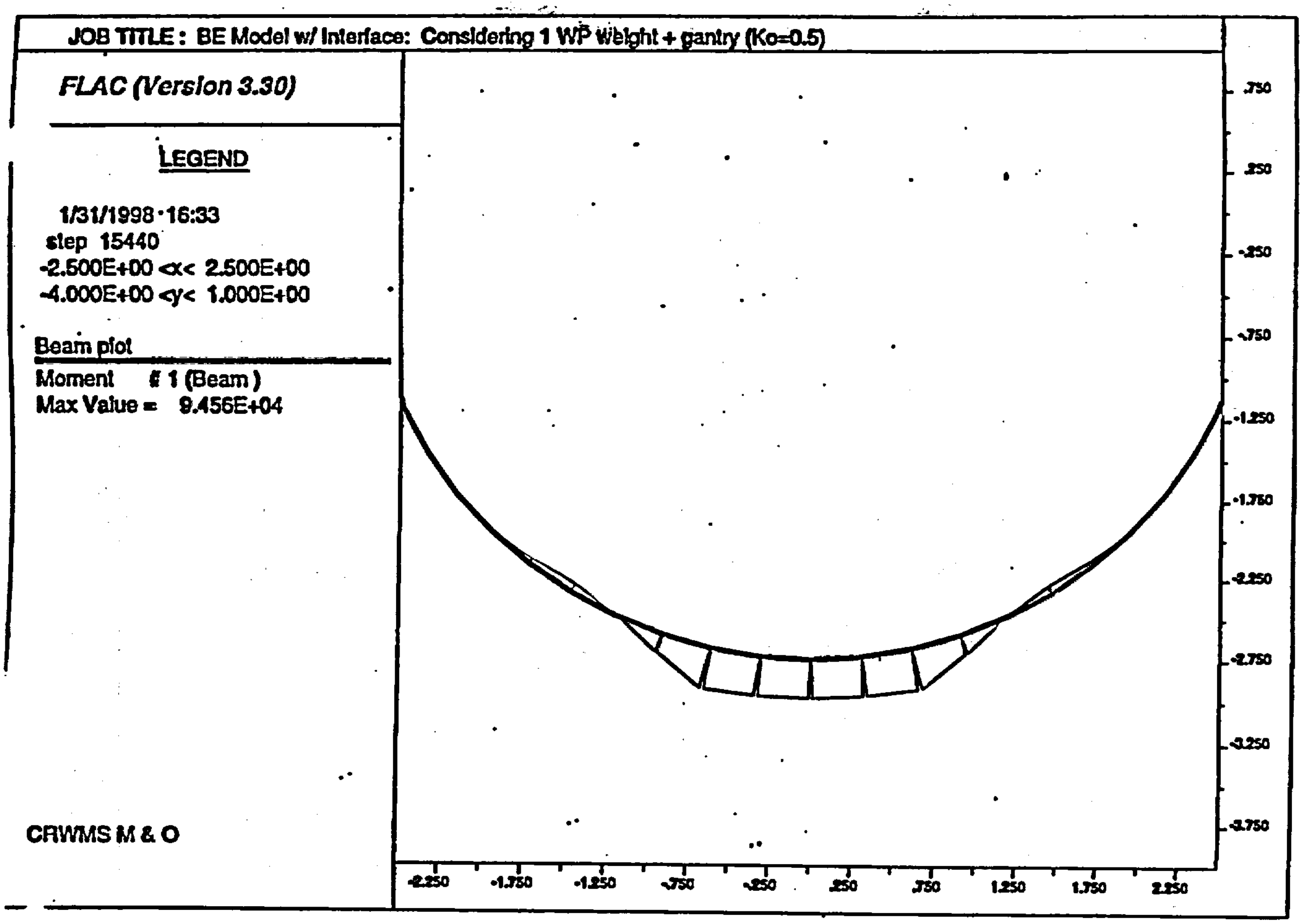


Figure II-17. BE Model with Interface: Bending Moment Distribution in the Invert under the Loaded Gantry Weight. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

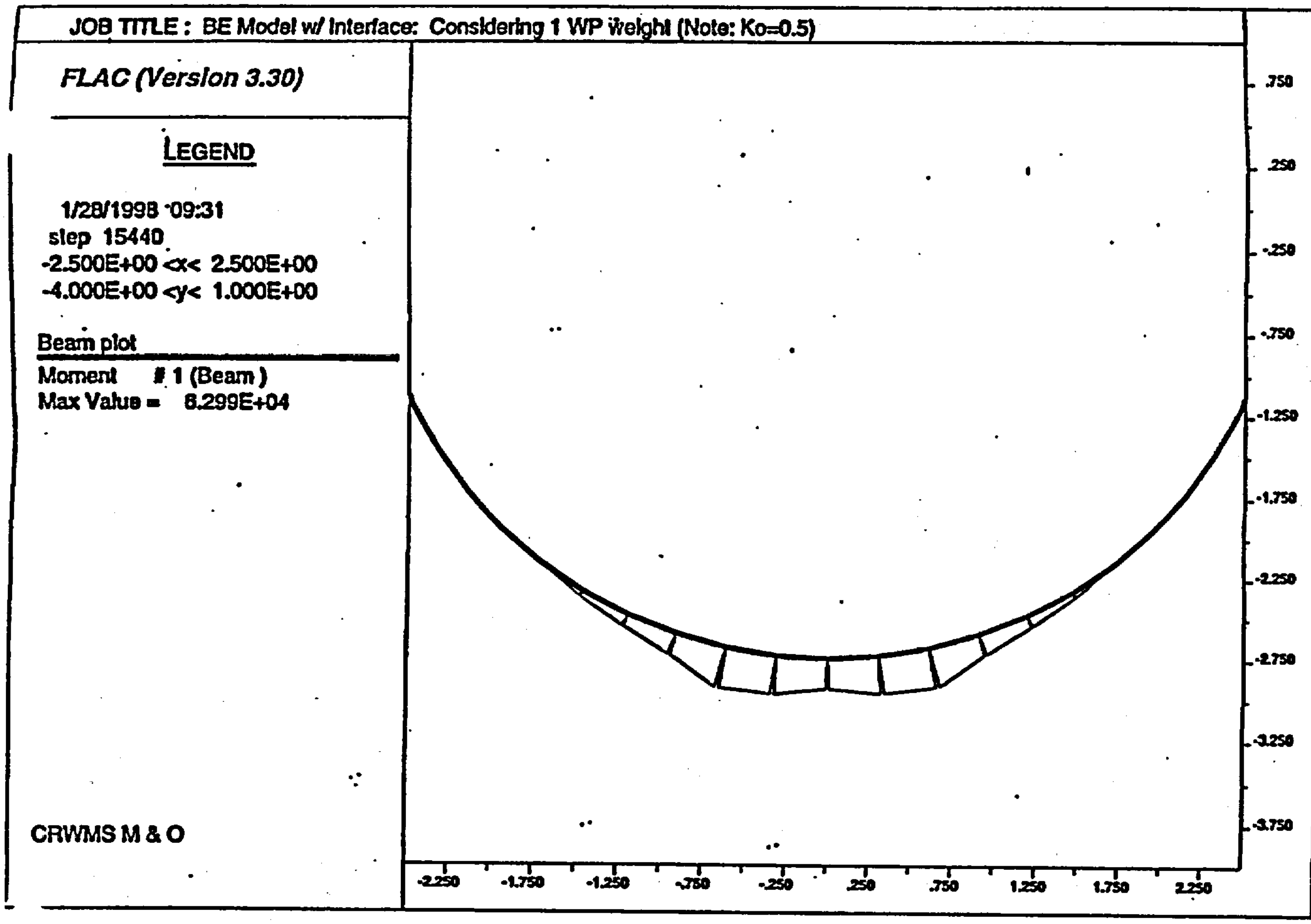


Figure II-18. BE Model with Interface: Bending Moment Distribution in the Invert under the Emplaced Waste Package Weight. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

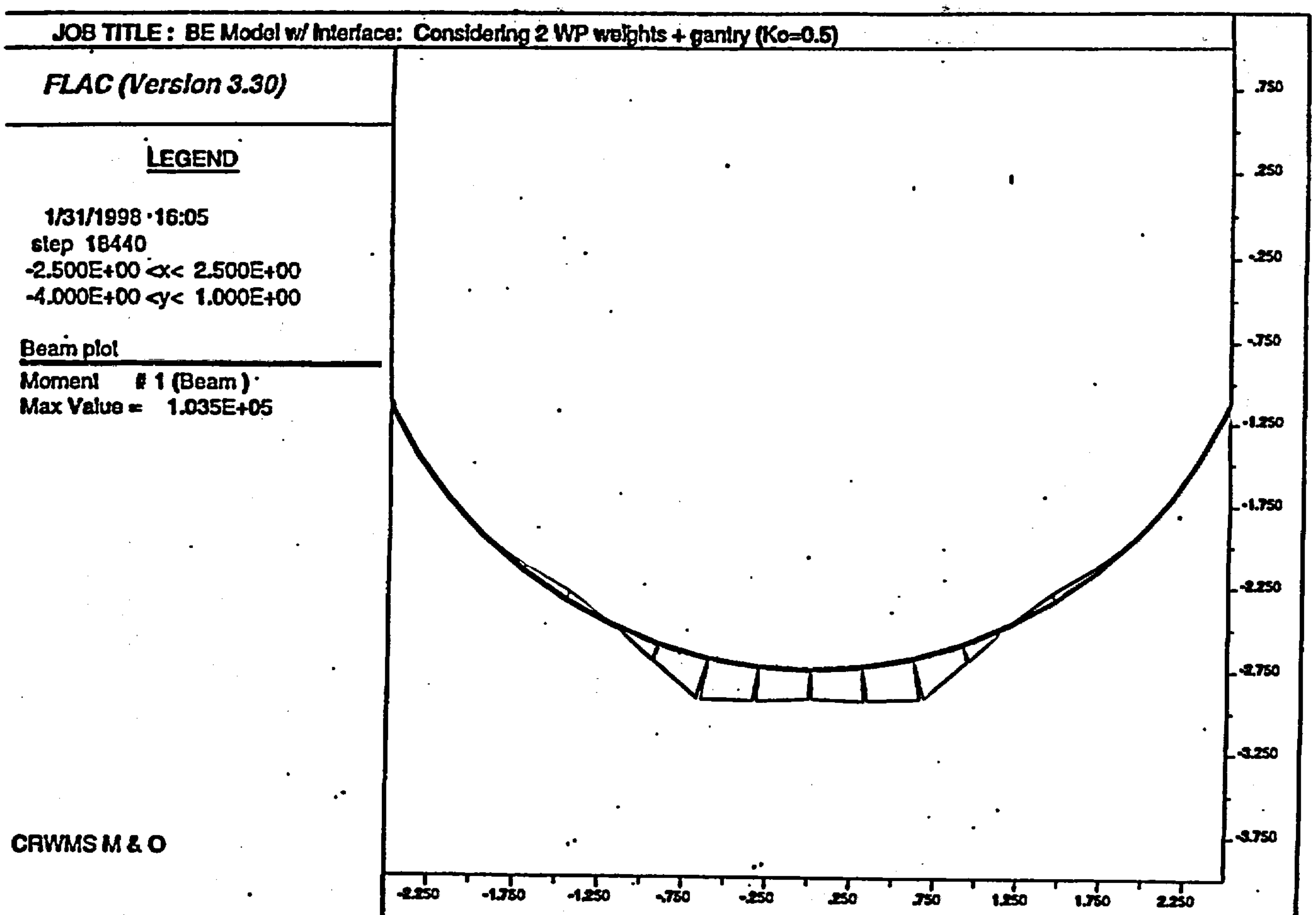


Figure II-19. BE Model with Interface: Bending Moment Distribution in the Invert under the Emplaced Waste Package Weight plus the Loaded Gantry Weight. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

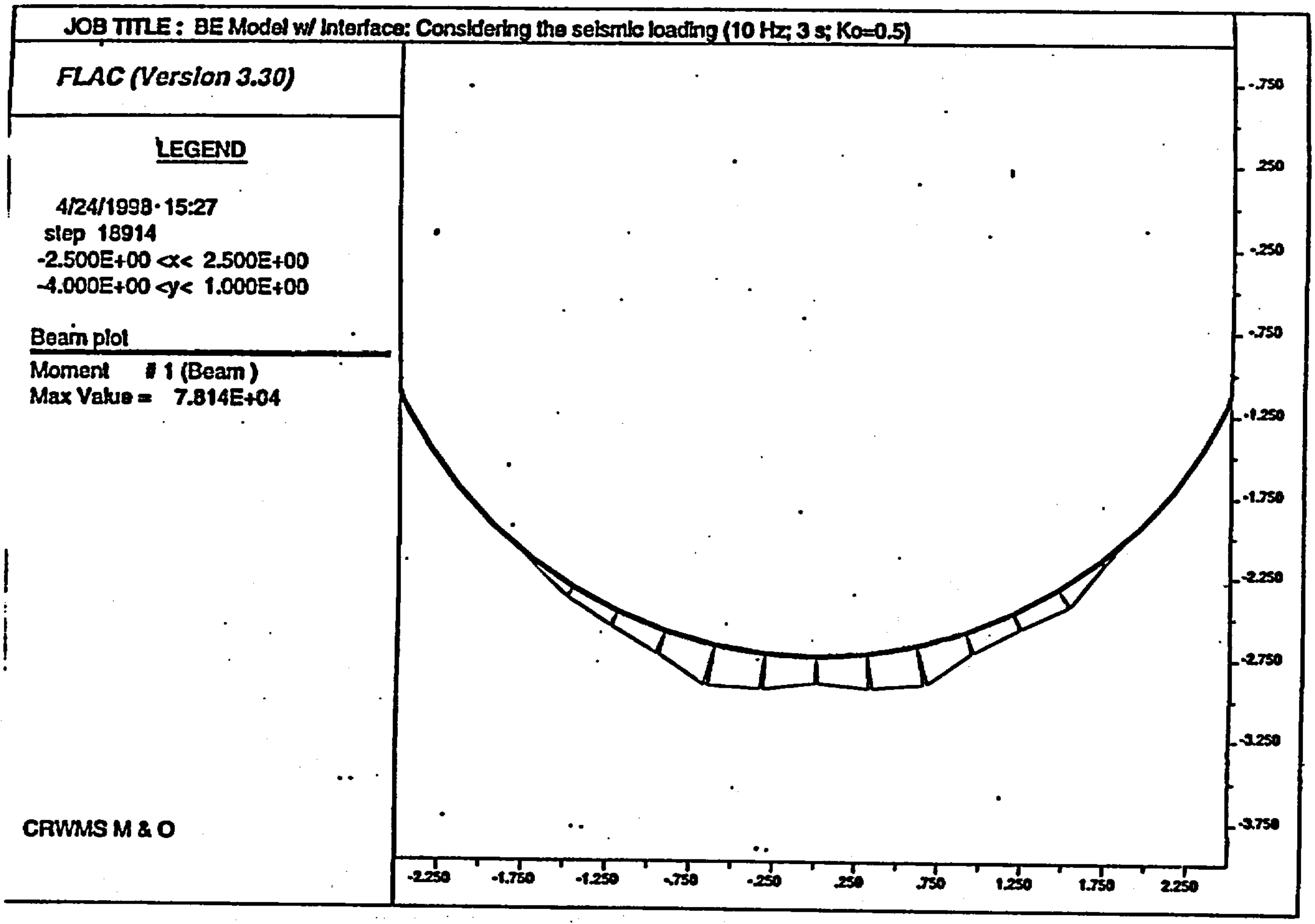


Figure II-20. BE Model with Interface: Bending Moment Distribution in the Invert under the Emplaced Waste Package Weight plus the Seismic Load. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

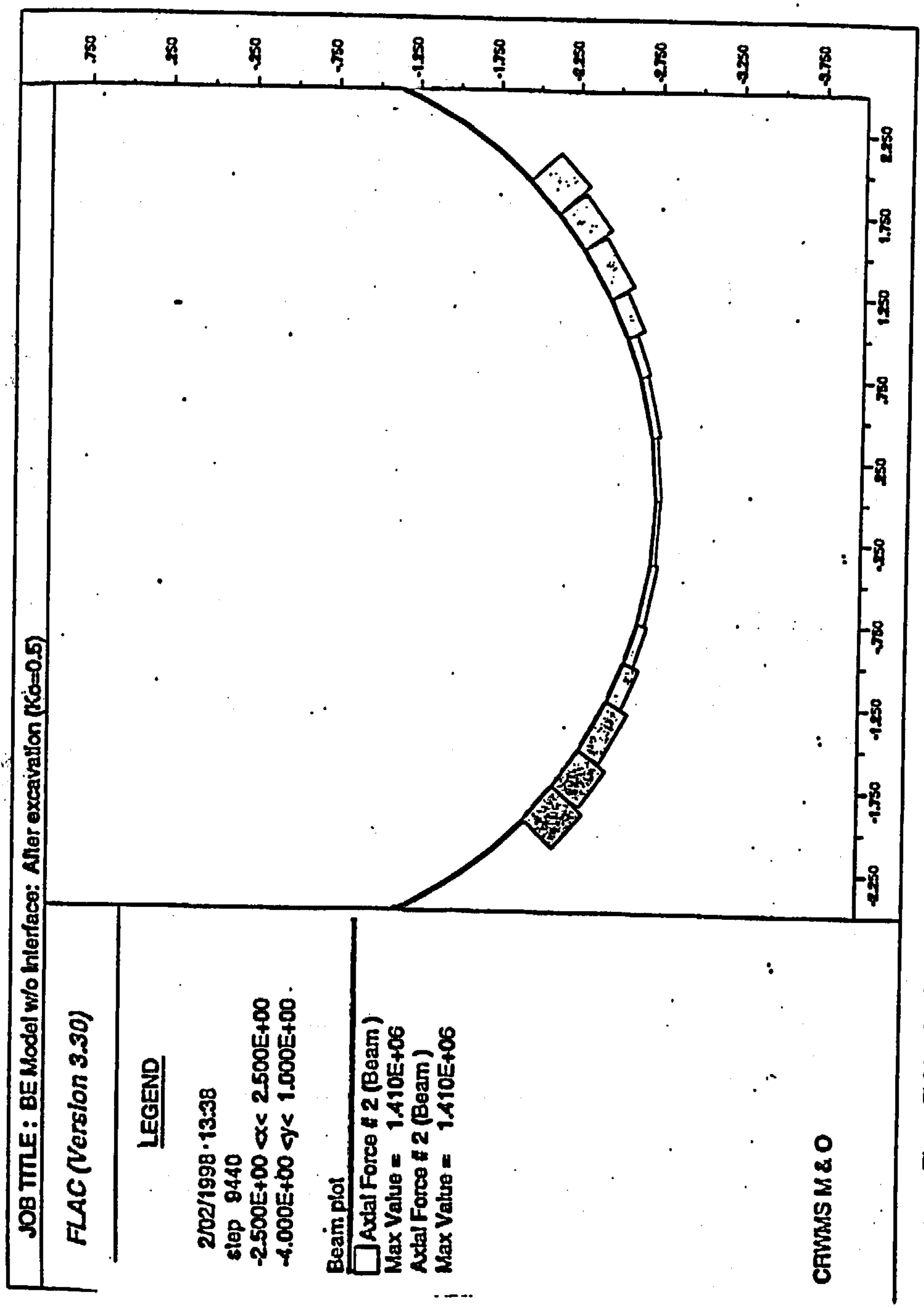


Figure II-21. BE Model without Interface: Axial Force Distribution in the Invert under the In Situ Load. Forces Are in Newtons/m and Dimensions Are in Meters.

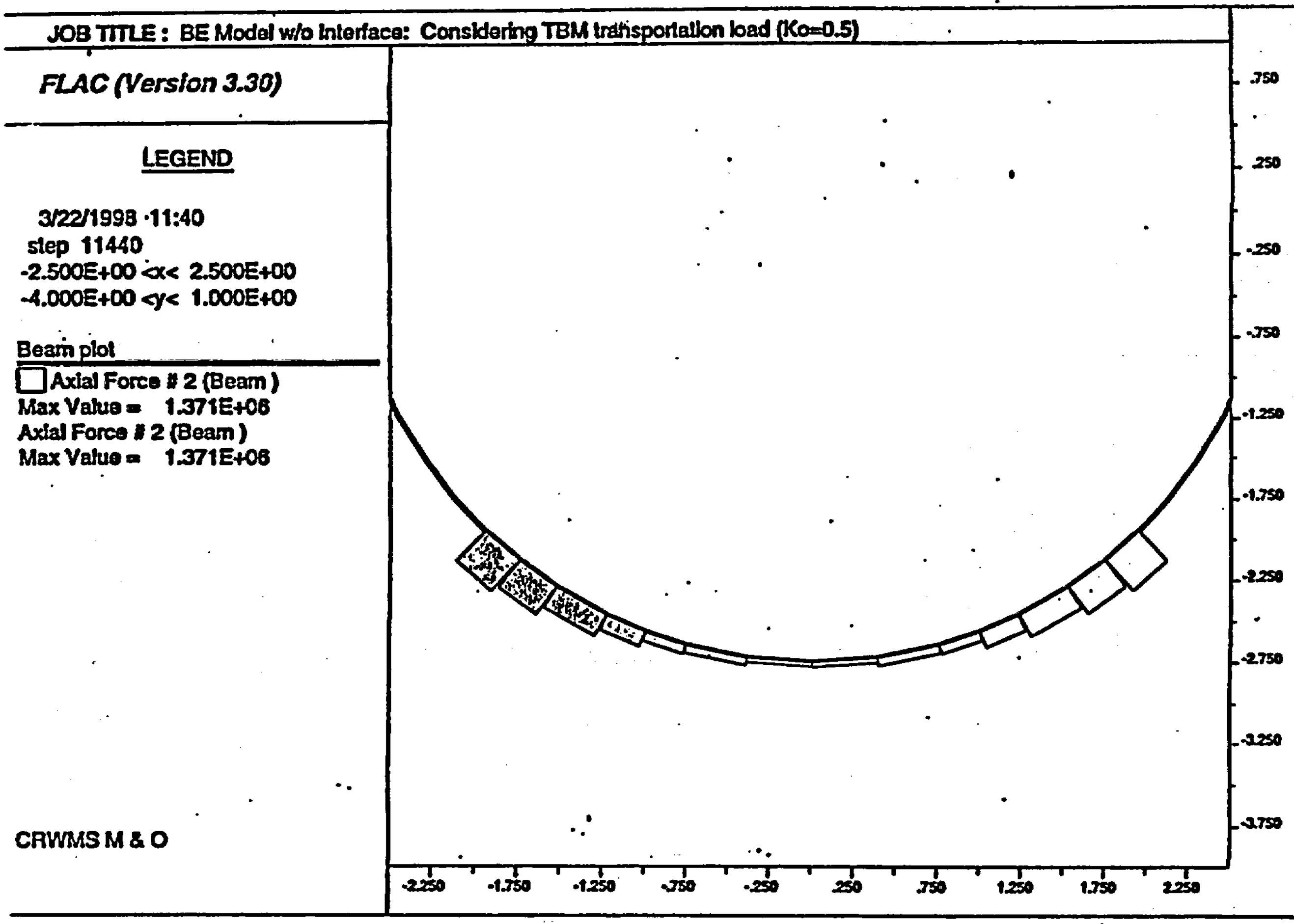


Figure II-22. BB Model without Interface: Axial Force Distribution in the Invert under the TBM Transportation Load. Forces Are in Newtons/m and Dimensions Are in Meters.

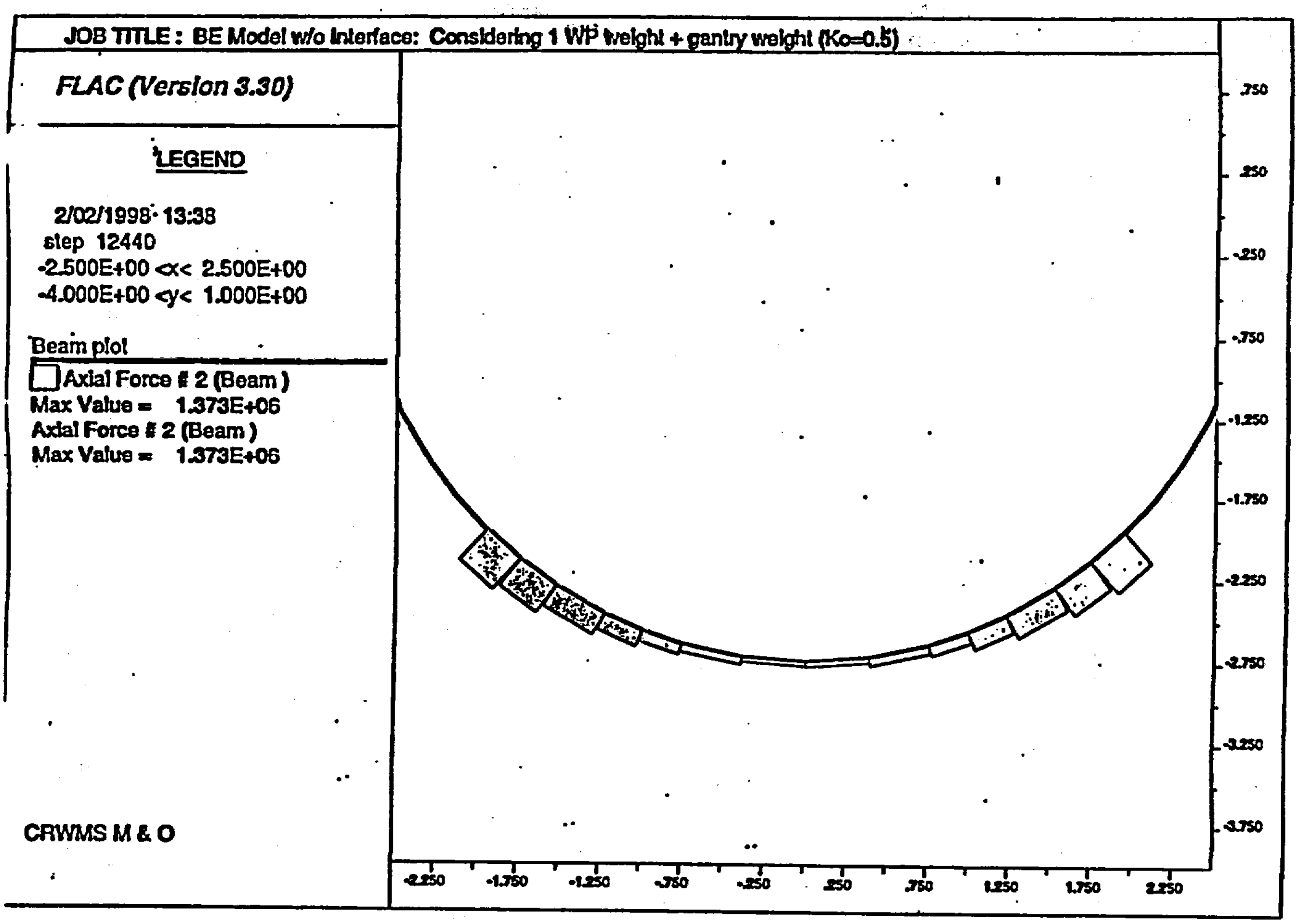


Figure II-23. BE Model without Interface: Axial Force Distribution in the Invert under the Loaded Gantry Weight. Forces Are in Newtons/m and Dimensions Are in Meters.

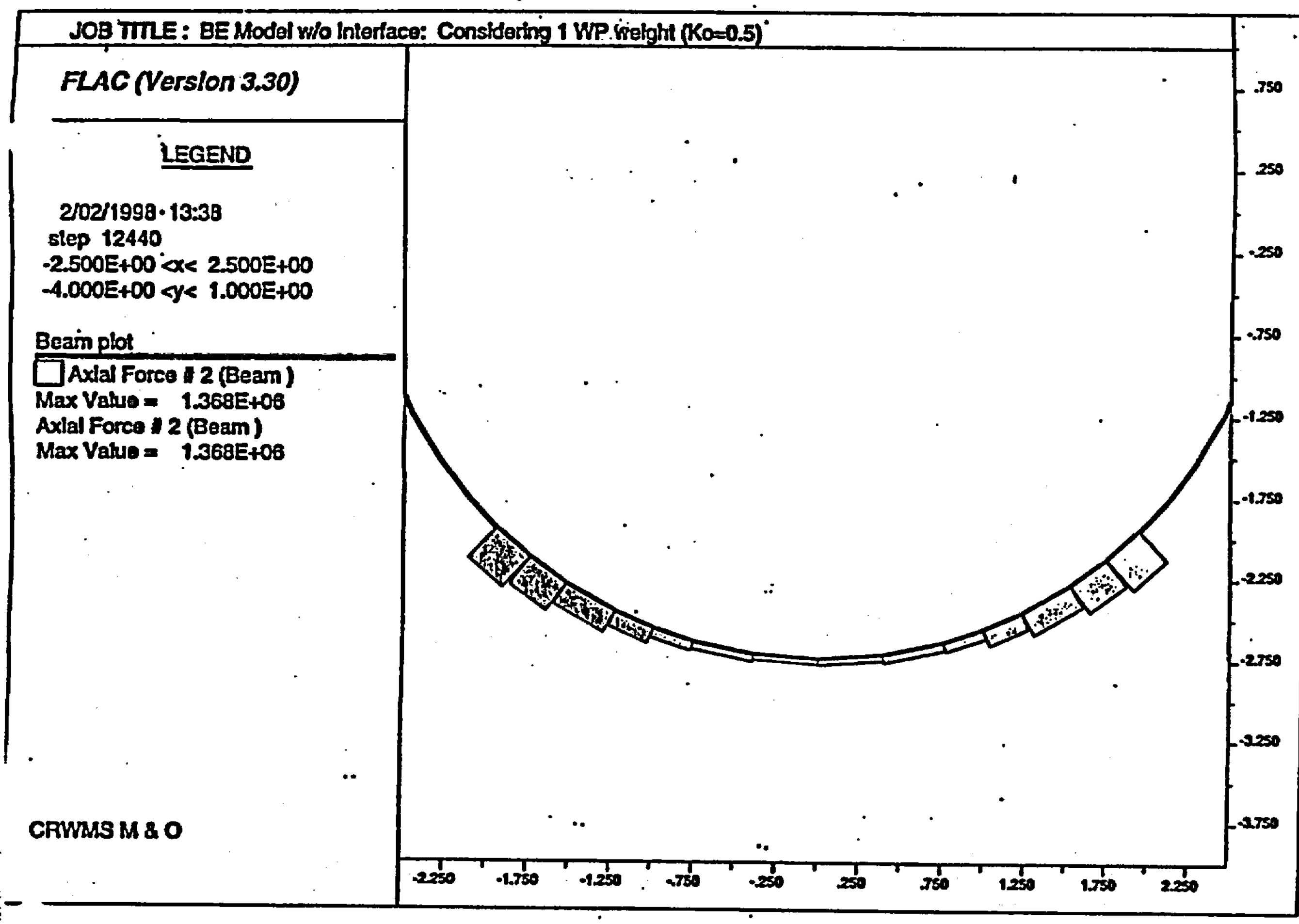


Figure II-24. BB Model without Interface: Axial Force Distribution in the Invert under the Emplaced Waste Package Weight. Forces Are in Newtons/m and Dimensions Are in Meters.

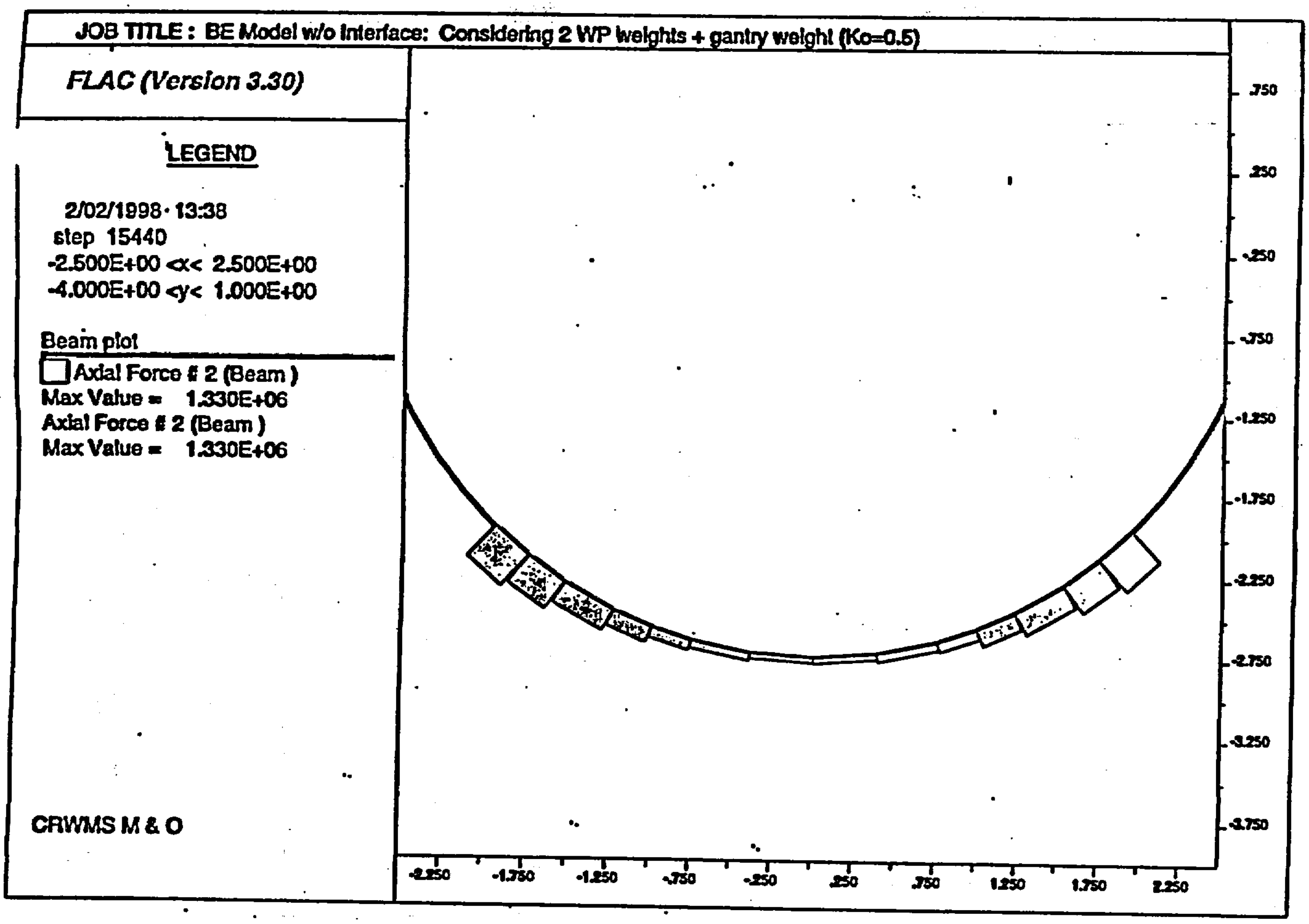


Figure II-25. BE Model without Interface: Axial Force Distribution in the Invert under the Emplaced Waste Package Weight plus the Loaded Gantry Weight. Forces Are in Newtons/m and Dimensions Are in Meters.

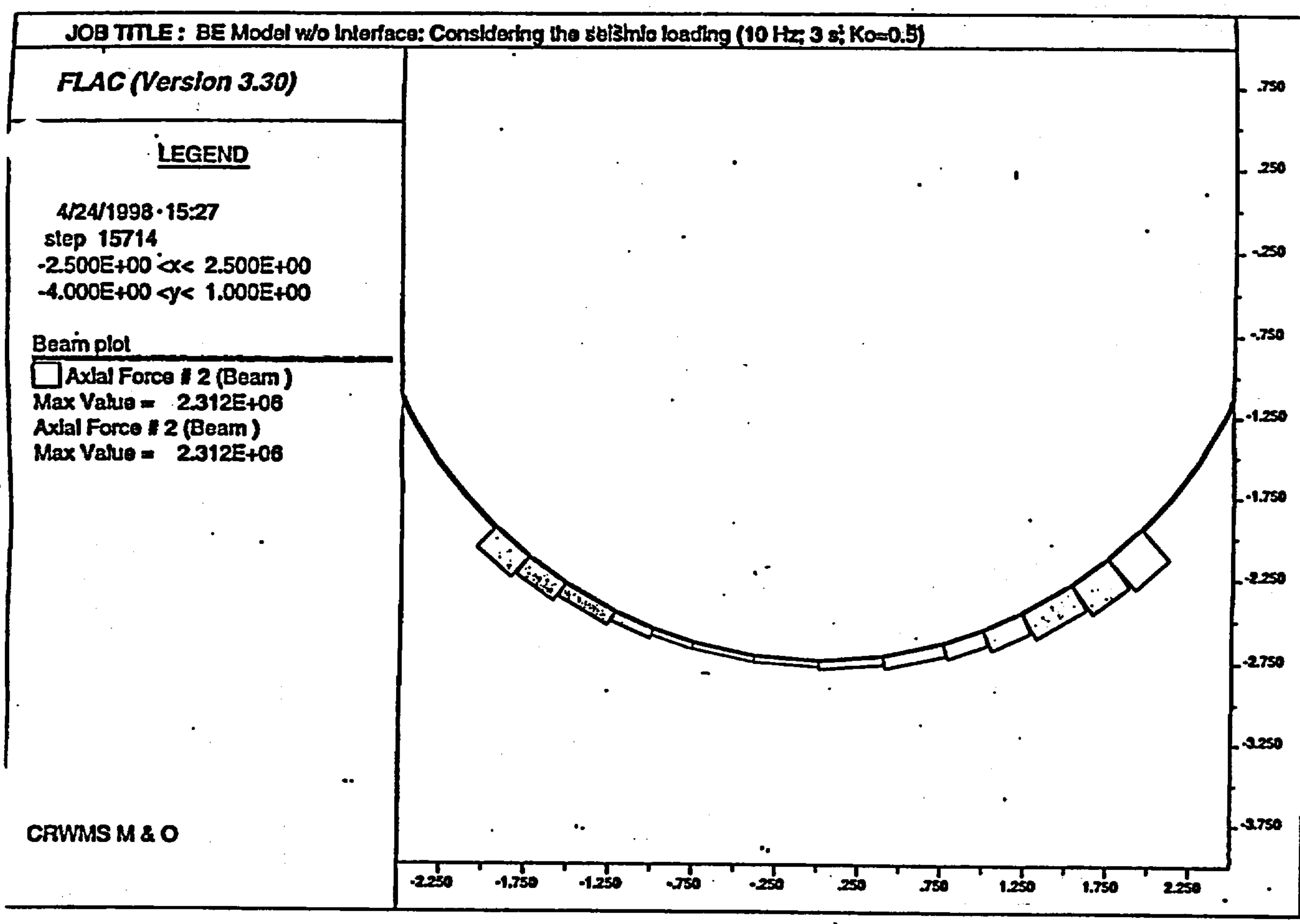


Figure II-26. BE Model without Interface: Axial Force Distribution in the Invert under the Emplaced Waste Package Weight plus the Seismic Load. Forces Are in Newtons/m and Dimensions Are in Meters.

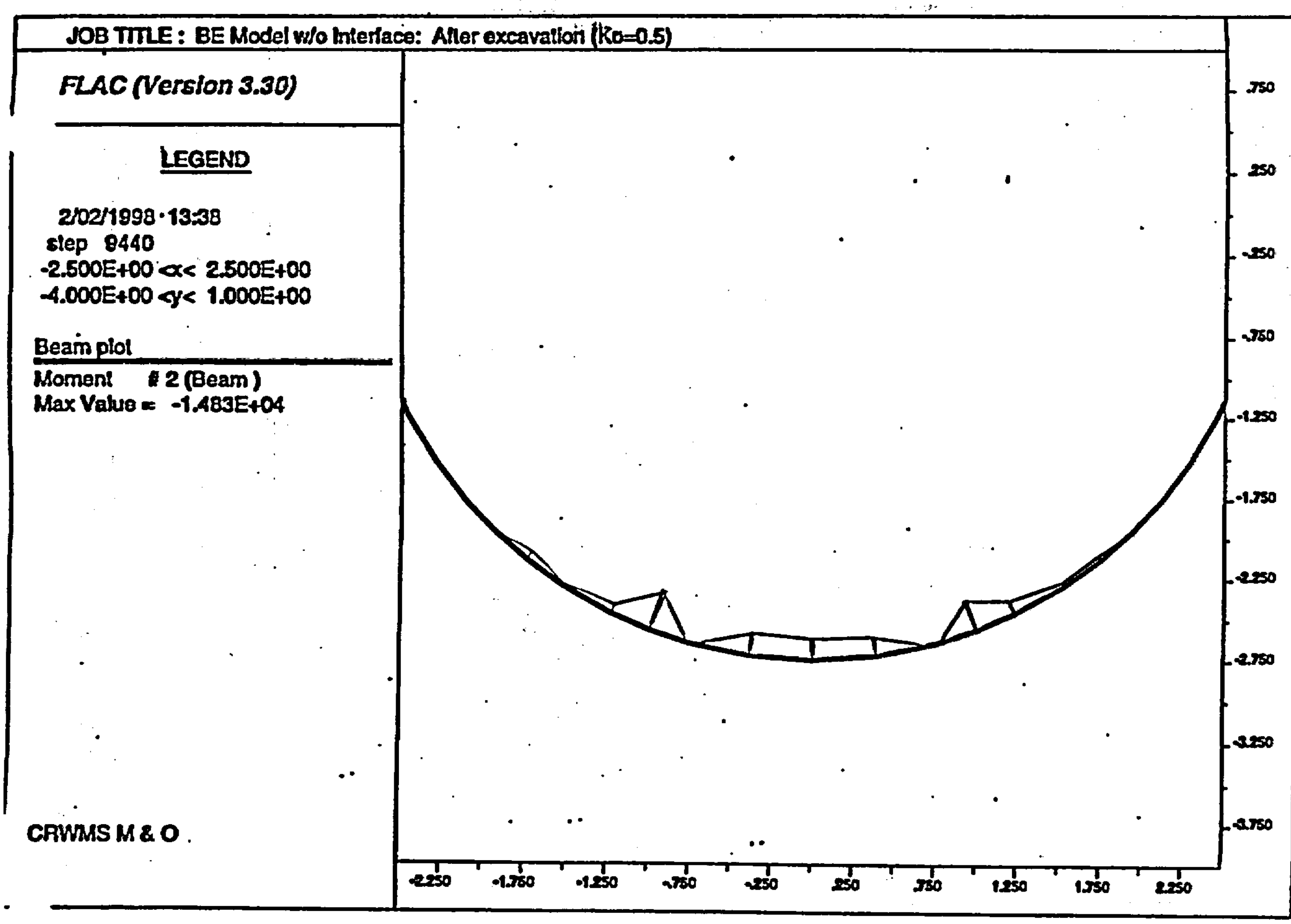


Figure II-27. BE Model without Interface: Bending Moment Distribution in the Invert under the In Situ Load. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

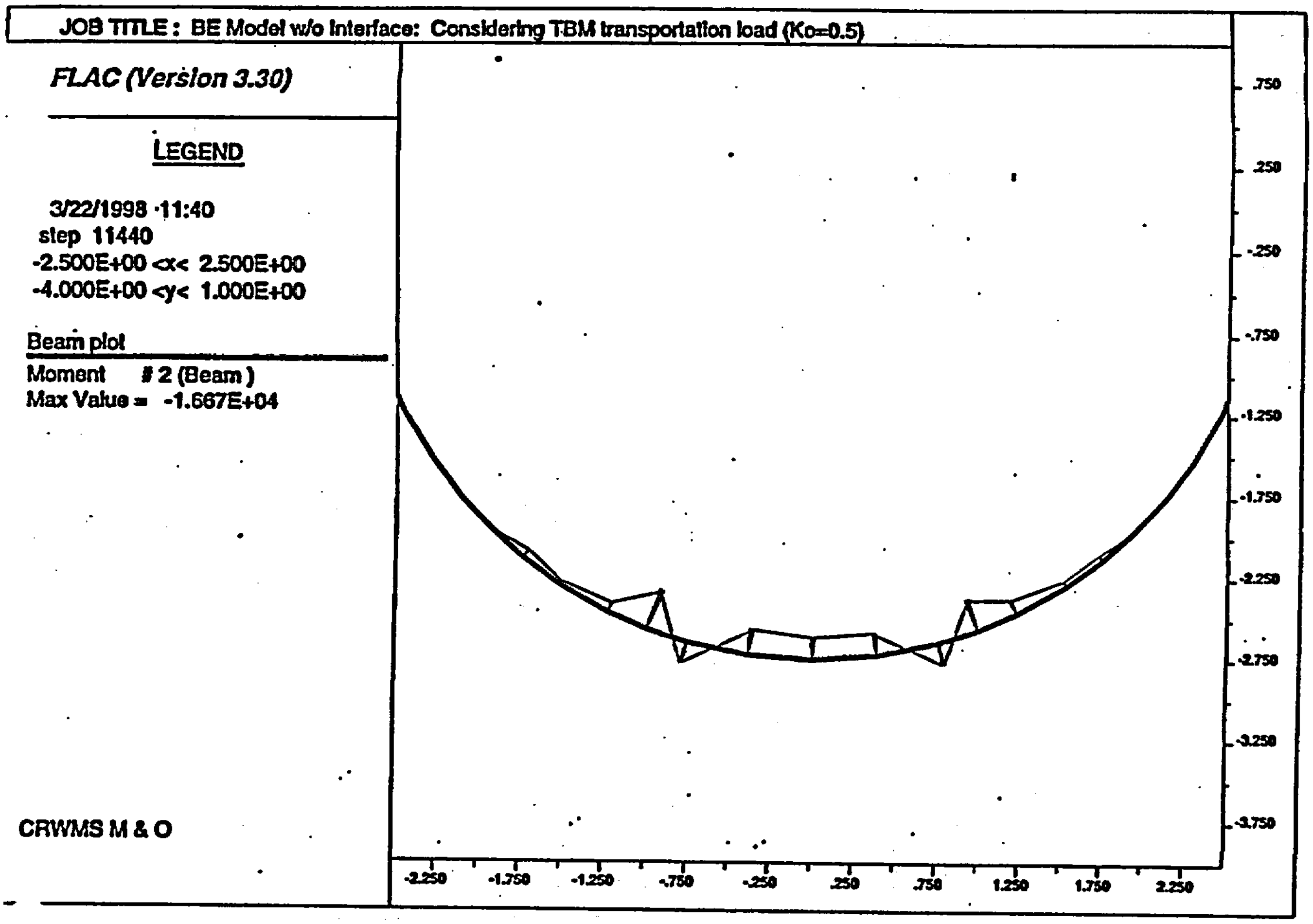


Figure II-28. BE Model without Interface: Bending Moment Distribution in the Invert under the TBM Transportation Load. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

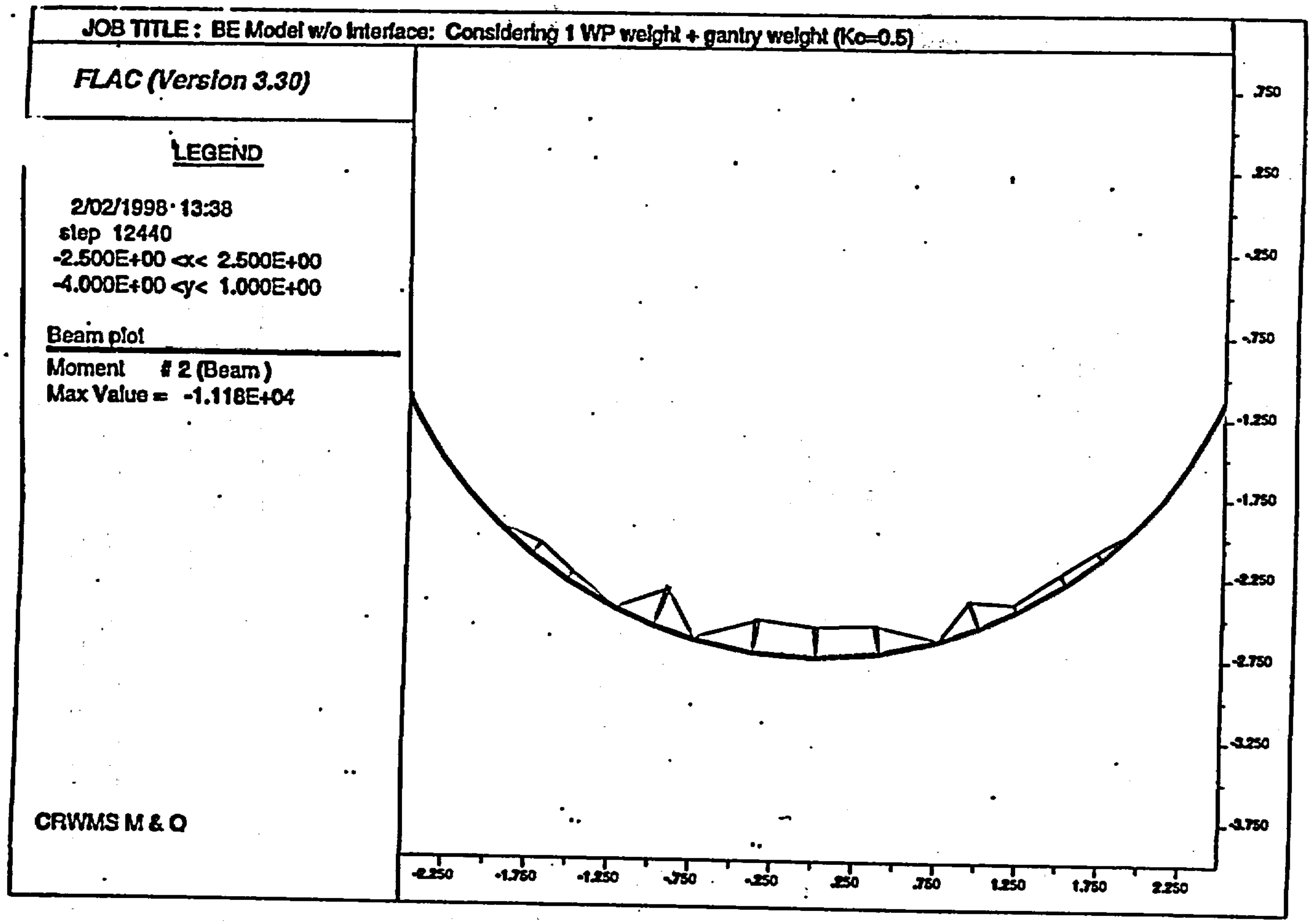


Figure II-29. BE Model without Interface: Bending Moment Distribution in the Invert under the Loaded Gantry Weight. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

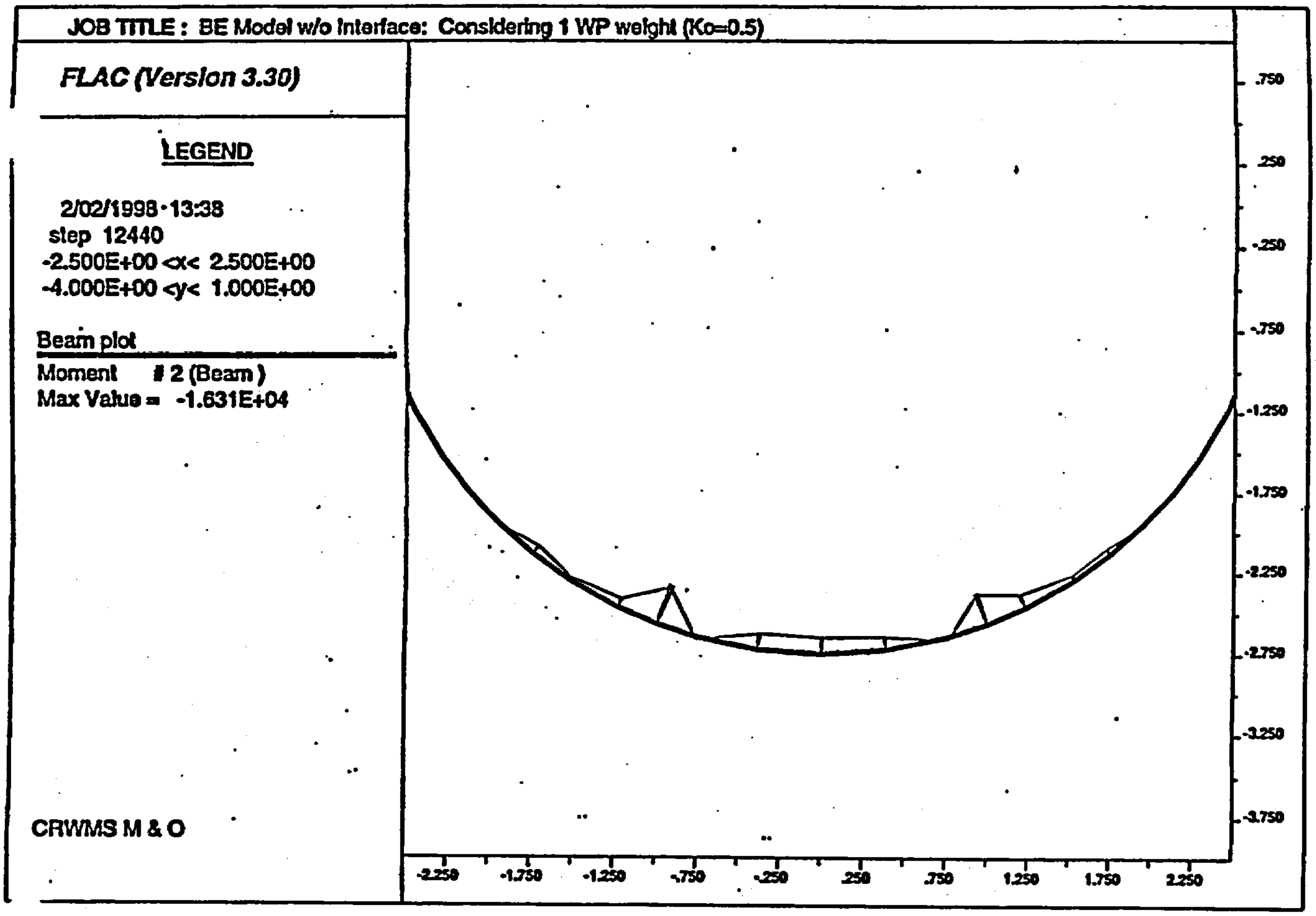


Figure II-30. BE Model without Interface: Bending Moment Distribution in the Invert under the Emplaced Waste Package Weight. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

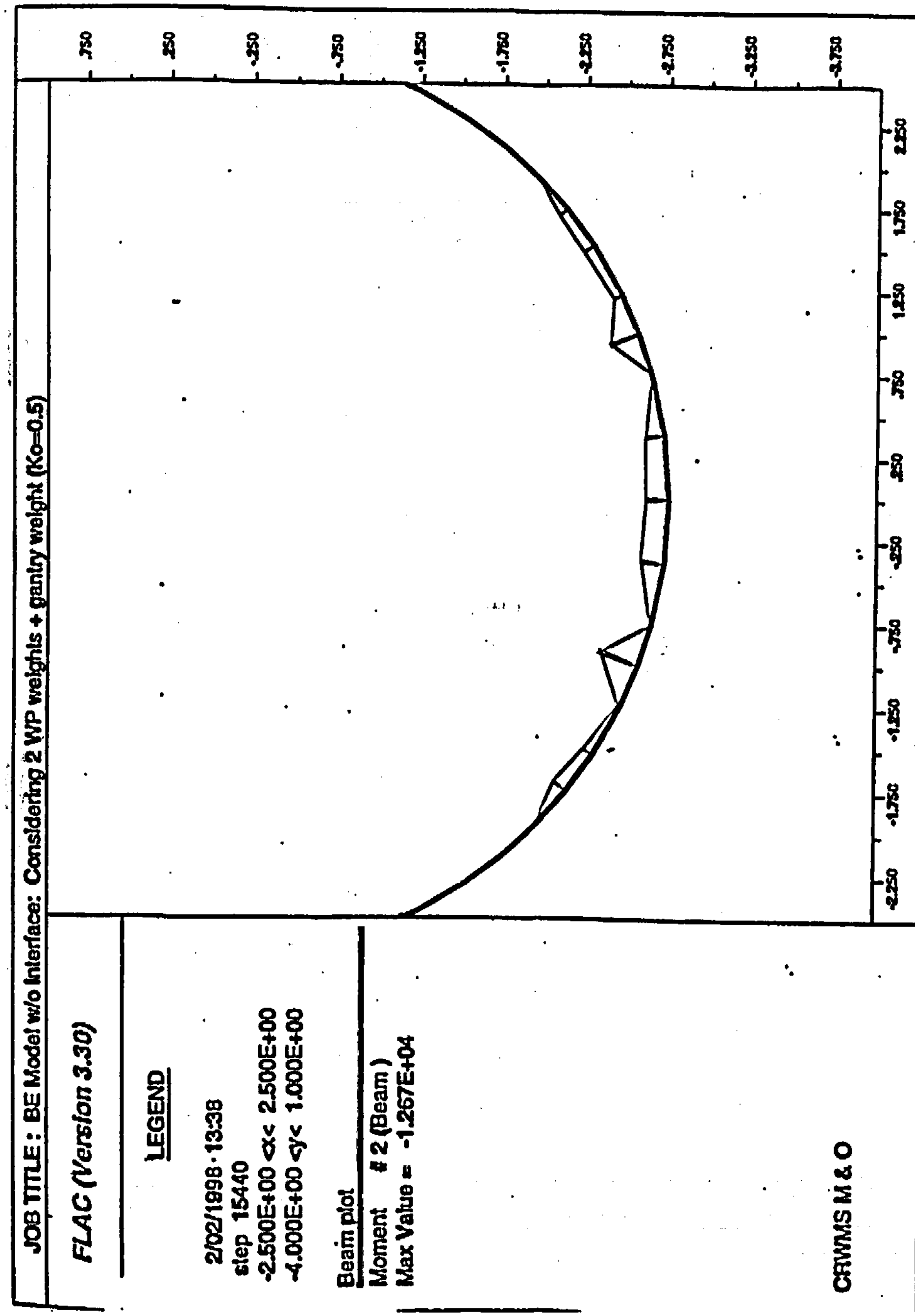


Figure II-31. BE Model without Interface: Bending Moment Distribution in the Invert under the Emplaced Waste Package Weight plus Loaded Gantry Weight. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

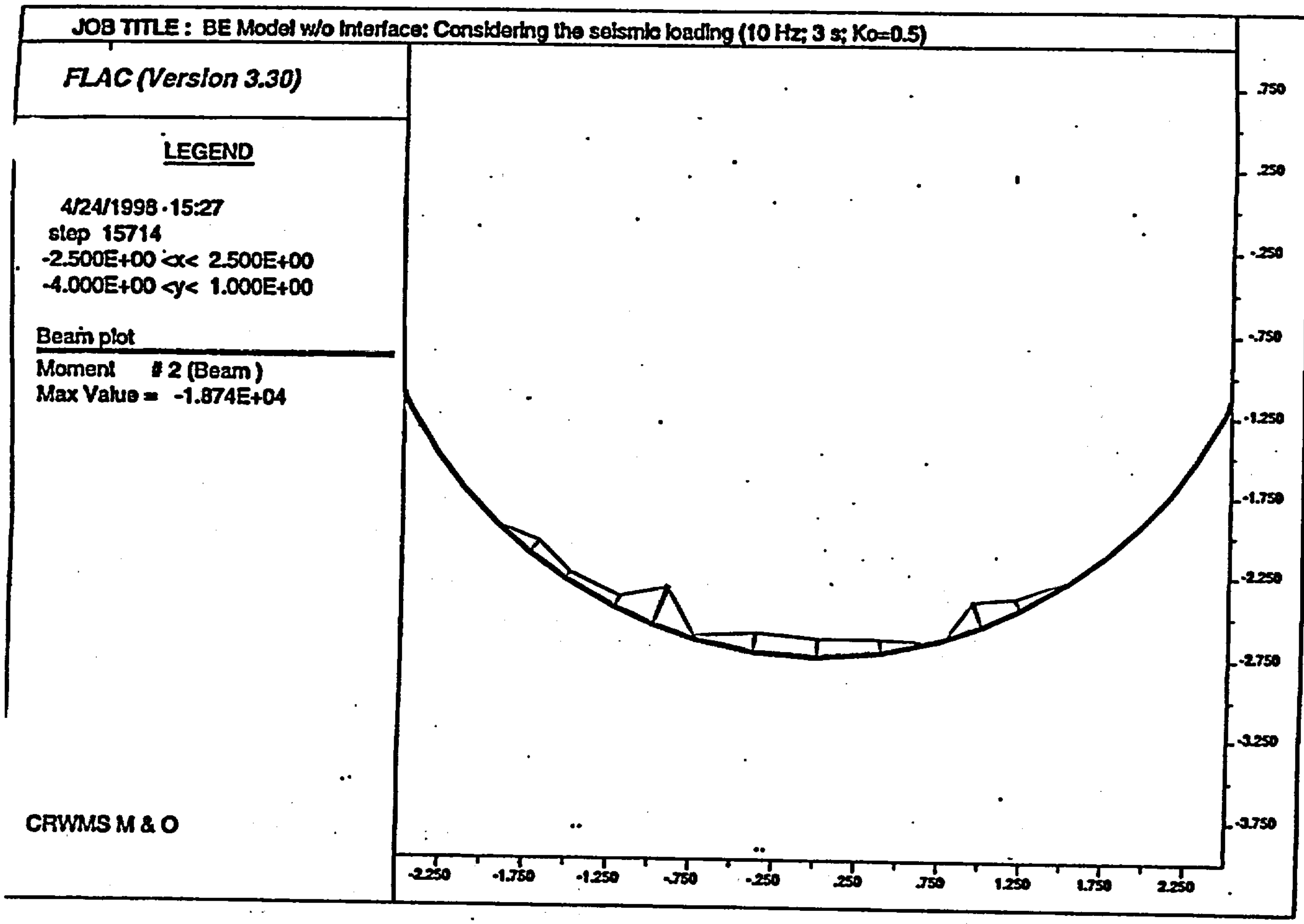


Figure II-32. BE Model without Interface: Bending Moment Distribution in the Invert under the Emplaced Waste Package Weight plus the Seismic Load. Bending Moments Are in (Newtons-m)/m and Dimensions Are in Meters.

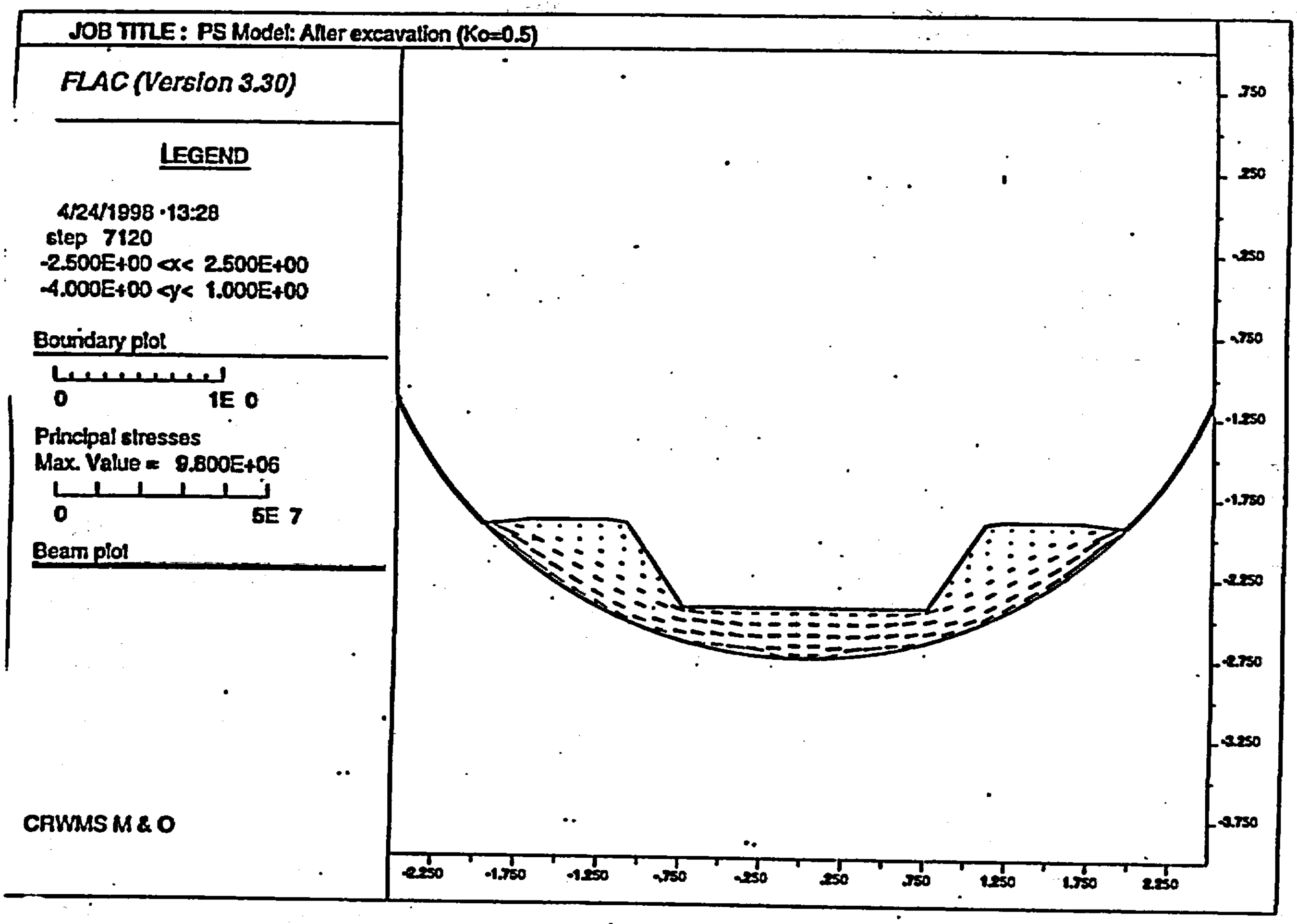


Figure II-33. PSE Model: Principa Stress Distribution in the Invert under the In Situ Load. Stresses Are in Pa and Dimensions Are in Meters.

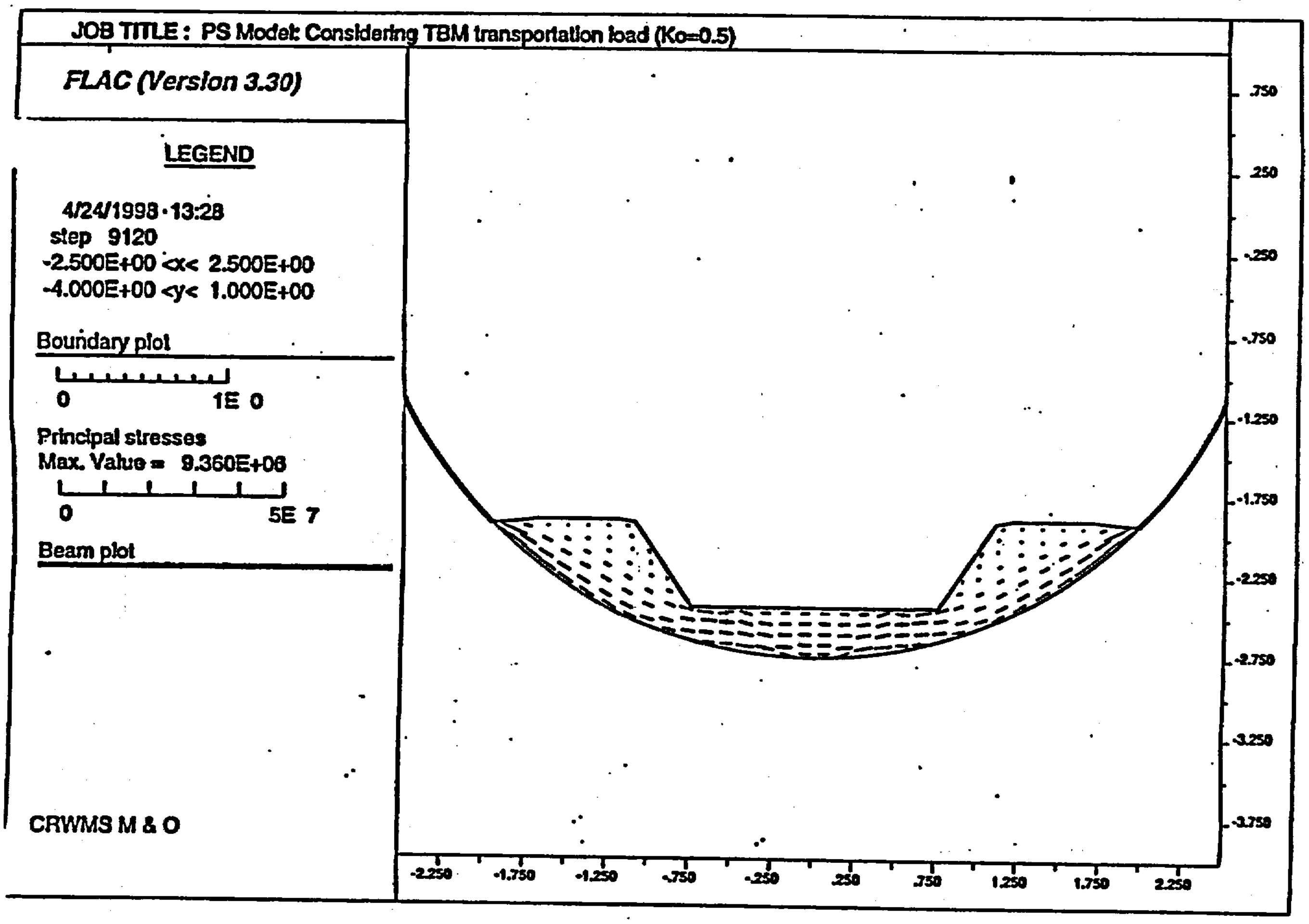


Figure II-34. PSB Model: Principal Stress Distribution in the Invert under the TBM Transportation Load. Stresses Are in Pa and Dimensions Are in Meters.

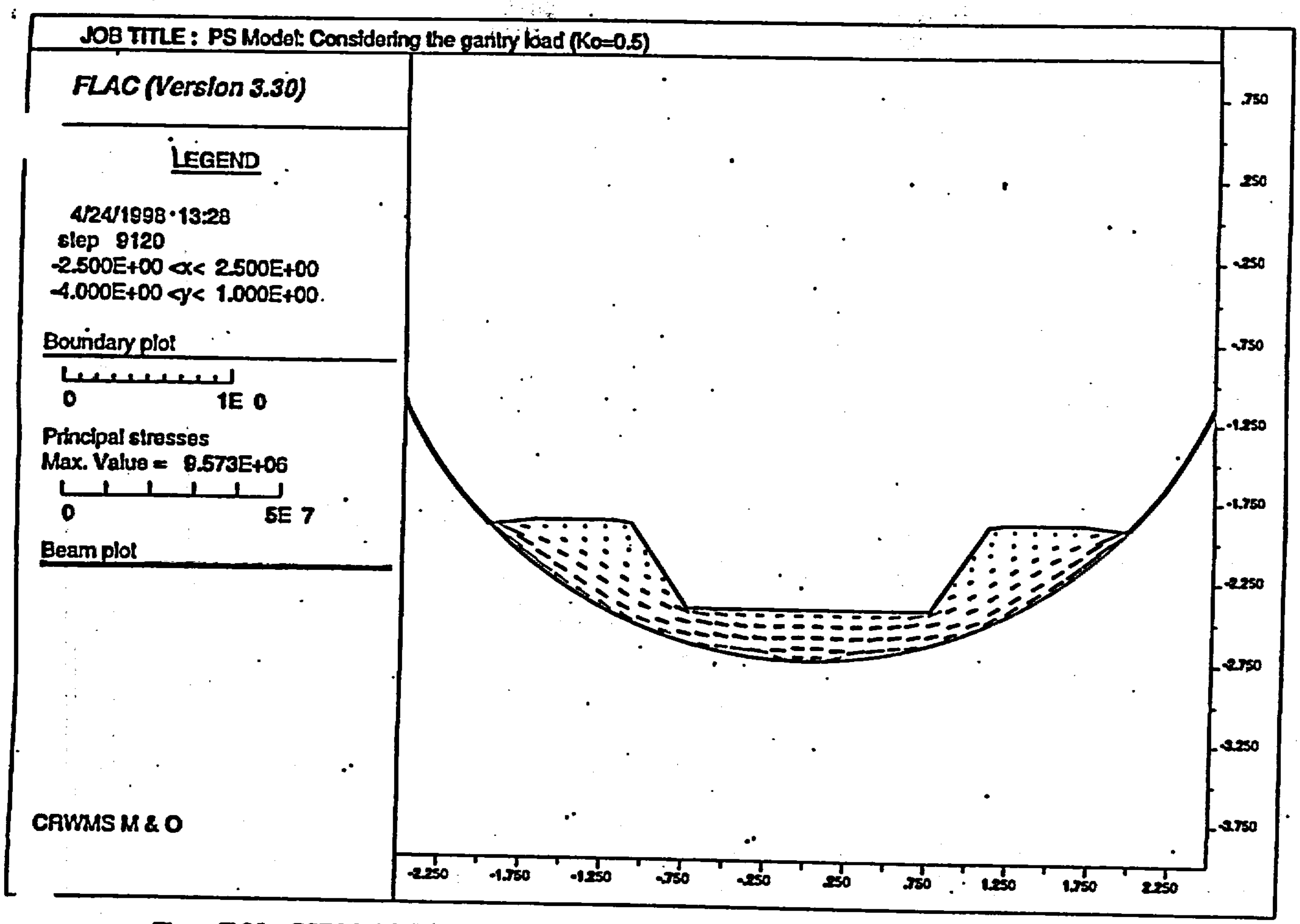


Figure II-35. PSE Model: Principal Stress Distribution in the Invert under the Loaded Gantry Weight. Stresses Are in Pa and Dimensions Are in Meters.

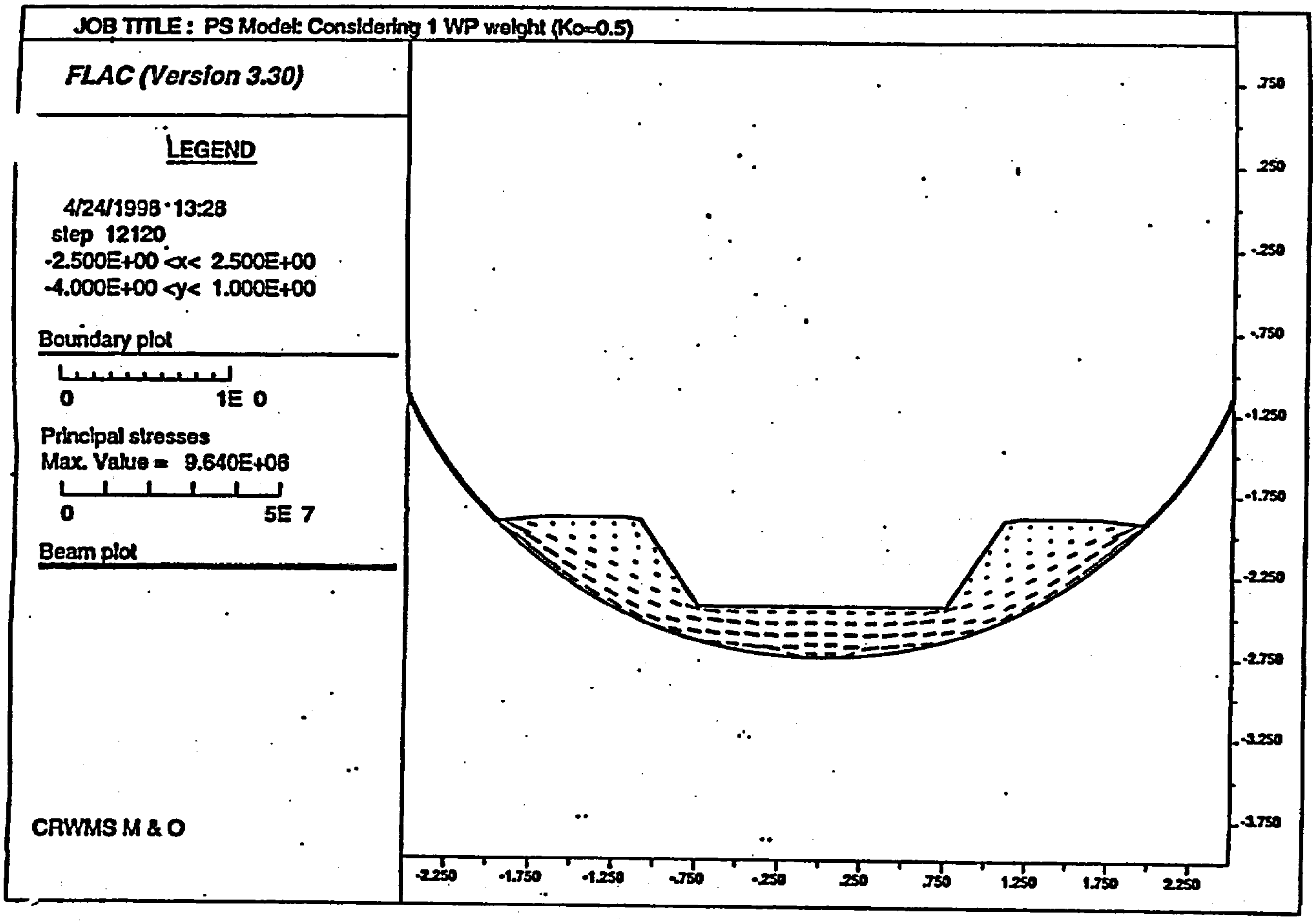


Figure II-36. PSB Model: Principal Stress Distribution in the Invert under the Emplaced Waste Package Weight. Stresses Are in Pa and Dimensions Are in Meters.

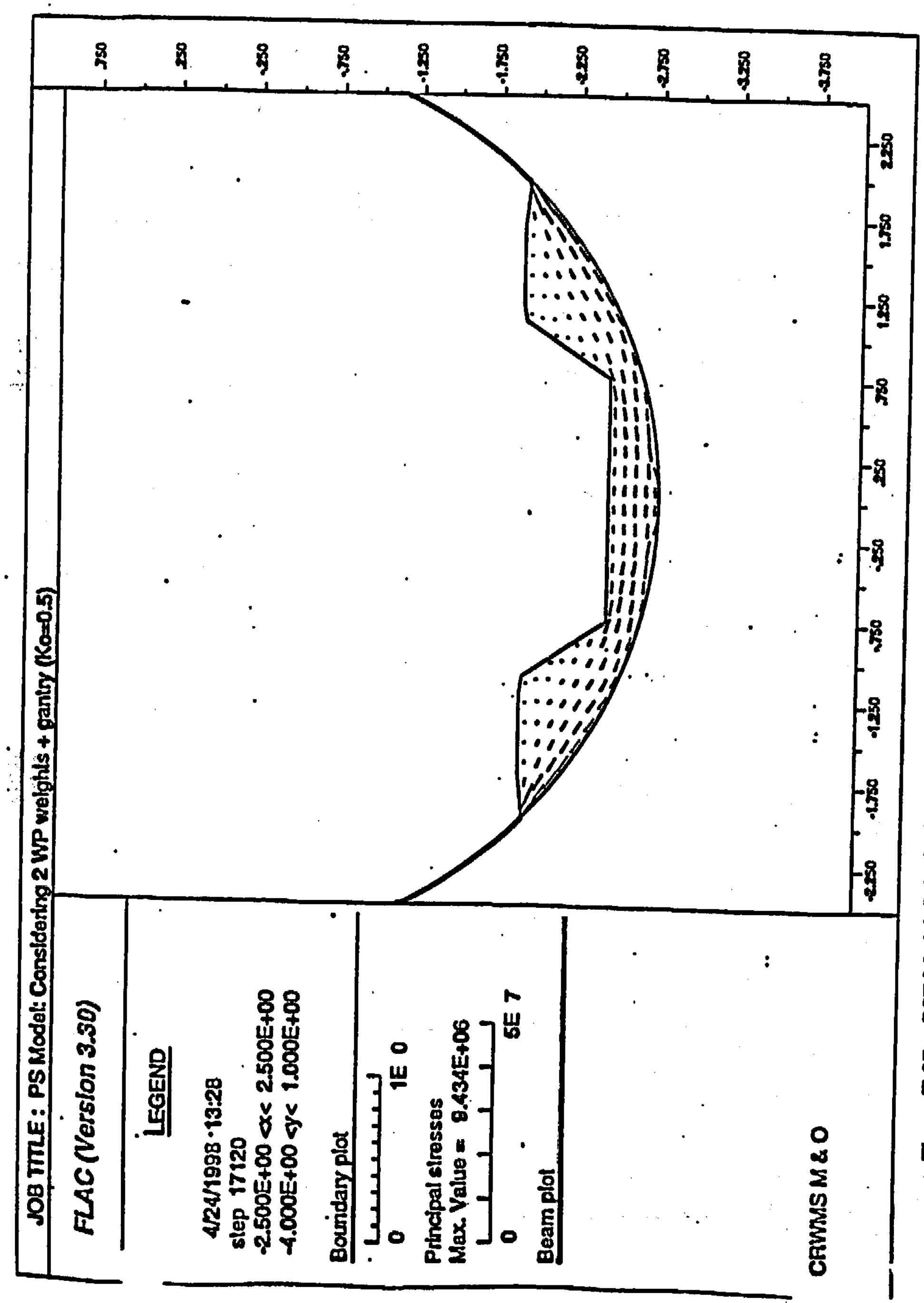


Figure II-37. PSE Model: Principal Stress Distribution in the Invert under the Emplaced Waste Package Weight plus the Loaded Gantry Weight. Stresses Are in Pa and Dimensions Are in Meters.

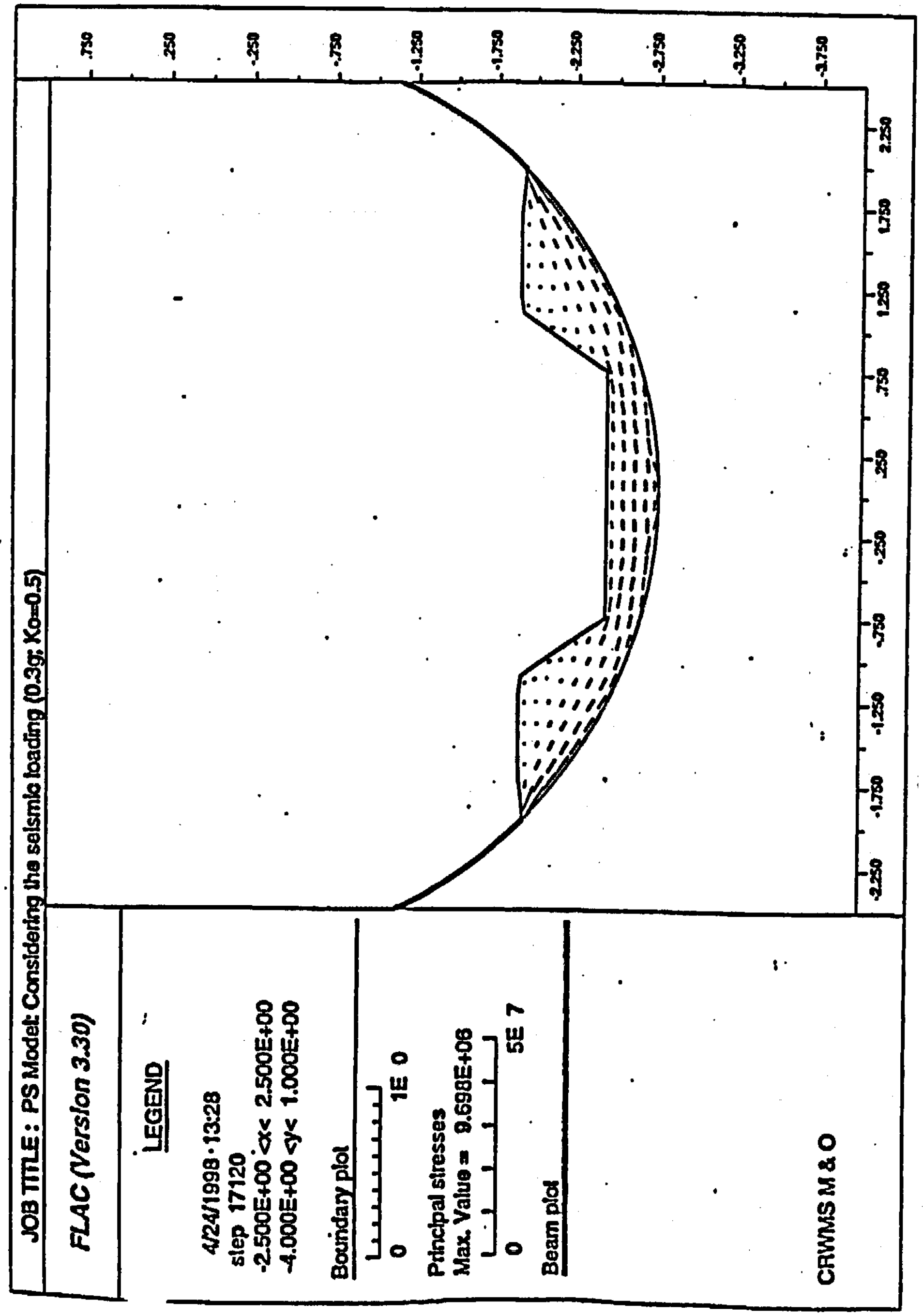


Figure II-38. PSB Model: Principal Stress Distribution in the Invert under the Emplaced Waste Package Weight plus the Seismic Load. Stresses Are in Pa and Dimensions Are in Meters.

6. FLAC INPUT DATA FILES

6.1 INPUT DATA FILE FOR THE BEAM ELEMENT MODEL WITH THE INTERFACE PRESENT

```
new
****************************************************************
*        Input File Name: inv_intf.dat
*
*        A FLAC model for analyzing a concrete invert to be placed
*        in an emplacement drift.
*        Drift diameter:     5.5 m
*        Drift spacing:      28 m
*
*        Loadings:           Static + seismic + thermal
*                            Static: ko=0.5
*                                  sigv_rep = 10 Mpa
*                            Seismic: PGV=16 cm/s
*                                  f=10 Hz
*                                  duration: 3 sec
*
*                            Thermal: Estmated based on
*                                  ground support analysis
****************************************************************
*
* Fish function to initialize scale factors
*
def kc_value
 ko_m=float(ko)
 cat_mat = int(cat_m)
 sig_yy=-1.*sigv_rep
 e_sc=1.
 ch_sc=1.
 fr_sc=1.
 dila_sc=1.
 dn_sc=1.
end
*
* Fish function to list mechanical properties
*
def tsw2_new
case_of cat_mat
;
case 1
command
set e_m=7.76e9 v_m=0.21 ch_m=1.5e6 fr_m=43 dila_m=0 dn_m=2274 tn_m=1.32e6
end_command
;
case 2
command
set e_m=12.18e9 v_m=0.21 ch_m=2.1e6 fr_m=45 dila_m=0 dn_m=2274 tn_m=1.78e6
end_command
;
case 3
command
set e_m=15.75e9 v_m=0.21 ch_m=2.6e6 fr_m=45 dila_m=0 dn_m=2274 tn_m=2.17e6
end_command
;
case 4
command
set e_m=22.99e9 v_m=0.21 ch_m=3.7e6 fr_m=46 dila_m=0 dn_m=2274 tn_m=3.00e6
end_command
;
case 5
command
set e_m=32.61e9 v_m=0.21 ch_m=5.2e6 fr_m=46 dila_m=0 dn_m=2274 tn_m=4.21e6
```

```
end_command
;
end_case
end
*
set ko=0.5 cat_m=1 sigv_rep=10e6
*
****************************************************************
* Description: Long parallel 5.5 m drifts on 28 m spacing  *
****************************************************************
*
config dyn extra 3
*
* MESH CONSTRUCTION
*
def mesh_grd
 r1=1.0675
 r2=1./r1
end
mesh_grd
*
gr 144 82 * Mesh dimensions 140 m (wide) x 100 m (high)
m m      * Mohr-Coulomb Yield Criterion
*
gen -70,-50.0 -70,50.0  70, 50.0  70,-50.0
*
gen -70,-50.0 -70,-8.25 -36.25,-8.25 -36.25,-50.0   R=r2,r2 I=1,21 J=1,24
gen -70,-8.25 -70, 8.25 -36.25,8.25 -36.25,-8.25 R=r2,1  I=1,21 J=24,60
gen -70,8.25 -70,50.0  -36.25,50.0 -36.25, 8.25  R=r2,r1 I=1,21 J=60,83
*
gen -36.25,8.25 -36.25,50.0 36.25,50.0 36.25,8.25 R=1,r1 I=21,125 J=60,83
gen -36.25,-8.25 -36.25,8.25 36.25,8.25 36.25,-8.25 R=1,1 I=21,125 J=24,60
gen -36.25,-50.0 -36.25,-8.25 36.25,-8.25 36.25,-50.0 R=1,r2 I=21,125 J= 1,24
*
gen  36.25, 8.25 36.25,50.0  70,50.0  70, 8.25  R=r1,r1 I=125,145 J=60,83
gen  36.25,-8.25 36.25, 8.25 70, 8.25 70,-8.25  R=r1,1  I=125,145 J=24,60
gen  36.25,-50.0 36.25,-8.25 70,-8.25 70,-50.0     R=r1,r2 I=125,145 J= 1,24
*
ini x add 70   * Put the origin at the lower left corner
ini y add 50.0   * for doing mesh refinement
*
def h_drift
 loop i (istart, iend)
  xstart=x(istart,1)
  loop j (1,jgp)
   x(i,j)=xstart+x_int*(i-istart)
  end_loop
end_loop
end
*
def v_drift
 loop j (jstart,jend)
  ystart=y(1,jstart)
  loop i (1,igp)
   y(i,j)=ystart+y_int*(j-jstart)
  end_loop
 end_loop
end
*
*****************************************
* Adjust the x-cord for the left drift
*****************************************
*
set istart=21 iend=45 x_int=0.6875
h_drift
*
**********************************************************************
* Adjust the x-cord for the pillar between the left and middle drifts
**********************************************************************
*
```

```
set istart=45 iend=56 x_int=1.0
h_drift
*
*****************************************************
* Adjust the x-cord and y-cord for the middle drift
*****************************************************
*
set istart=56 iend=63 x_int=0.70
h_drift
set istart=63 iend=83 x_int=0.385
h_drift
set istart=83 iend=90 x_int=0.70
h_drift
set jstart=24 jend=32 y_int=0.55
v_drift
set jstart=32 jend=52 y_int=0.385
v_drift
set jstart=52 jend=60 y_int=0.55
v_drift
*
*************************************************************************
* Adjust the x-cord for the pillar between the middle and right drifts
*************************************************************************
*
set istart=90 iend=101 x_int=1.0
h_drift
*
*****************************************
* Adjust the x-cord for the right drift
*****************************************
*
set istart=101 iend=125 x_int=0.6875
h_drift
*
ini x add -70 * Set the origin back to the center of the center drift.
ini y add -50.0
*
ini x -60 i=5
ini x -58 i=6
ini x -56 i=7
ini x -54 i=8
ini x -52 i=9
*
ini x 52 i=137
ini x 54 i=138
ini x 56 i=139
ini x 58 i=140
ini x 60 i=141
*
gen circle  0  0 2.75 * center drift
gen circle -28.0 0 2.75 * immediate left drift
gen circle  28.0 0 2.75 * immediate right drift
gen circle -56.0 0 2.75 * remote left drift
gen circle  56.0 0 2.75 * remote right drift
gen adjust
*
* FISH function to find minima and maxima of x- and y-coordinates
*
def max_min
 xmax=0.
 xmin=10e5
 ymax=0.
 ymin=10e5
 loop i (1,igp)
  loop j (1,jgp)
   if x(i,j) > xmax then
    xmax=x(i,j)
   end_if
   if x(i,j) < xmin then
    xmin=x(i,j)
```

```
      end_if
      if y(i,j) > ymax then
        ymax=y(i,j)
      end_if
      if y(i,j) < ymin then
        ymin=y(i,j)
      end_if
    end_loop
  end_loop
end
max_min
*
****************************************************
* Assign material properties and initial stresses
****************************************************
ko_value  * execute the scale factors
*
def mat_ini
  sigv_top = sigv_rep-9.81*dn_m*ymax
  sigy_top = -sigv_top
  e_m=e_sc*e_m
  ch_m=ch_sc*ch_m
  fr_m=fr_sc*fr_m
  dila_m=dila_sc*dila_m
  sh_m=0.5*e_m/(1.+v_m)
  bk_m=(1./3.)*e_m/(1.-2.*v_m)
  ang=fr_m*pi/180.
  tn_m=1.*tn_m
  grad_v=9.81*dn_m*(ymax-ymin)
  grad_h=ko_m*grad_v
  sigv=-(sigv_top+grad_v)
  sigh=ko_m*sigv
   command
     prop s=sh_m b=bk_m d=dn_m
     prop coh=ch_m fric=fr_m dila=dila_m ten=tn_m
     ini syy sigv var 0 grad_v
     ini sxx sigh var 0 grad_h
     ini szz sigh var 0 grad_h
     apply syy sigy_top j=83
   end_command
end
*
set grav=9.81
tsw2_new
mat_ini
*
*************************************
* Fix the boundary conditions
*************************************
fix x i=1
fix x i=145
fix y j=1
*
set dyn off
his unbal
step 200        *initial equilibrium
*
********************************************************
*        Stage 1.     Excavate the Center Drift
********************************************************
*
ini xdisp=0 ydisp=0
*
* Monitor the closures for the center drift
*
def vc_c
 vc_c=ydisp(73,35)-ydisp(73,49)
end
def hc_c
 hc_c=xdisp(66,42)-xdisp(80,42)
```

```
end
his vc_c
his hc_c
*
* Monitor the closures for the left drift
*
def vcl_l
 vcl_l=ydisp(33,35)-ydisp(33,49)
end
def hcl_l
 hcl_l=xdisp(29,42)-xdisp(37,42)
end
his vcl_l
his hcl_l
*
* Monitor the closures for the right drift
*
def vcl_r
 vcl_r=ydisp(113,35)-ydisp(113,49)
end
def hcl_r
 hcl_r=xdisp(109,42)-xdisp(117,42)
end
his vcl_r
his hcl_r
*
* Monitoring at particular points around the center drift
*
his xdisp i 68 j 36
his ydisp i 68 j 36
his xdisp i 70 j 35
his ydisp i 70 j 35
his xdisp i 73 j 35
his ydisp i 73 j 35
his xdisp i 76 j 35
his ydisp i 76 j 35
his xdisp i 78 j 36
his ydisp i 78 j 36
*
set ncw 100
m n reg 73, 41  * the center drift
step 240  * 60% ground relaxation upon drift excavation

*
*******************************************************
*          Stage 2.     Place the invert and lining
*******************************************************
def setup
 xliner=0.
 yliner=0.
 rtunnel=2.75
 rliner=2.746
 nbeam=56
end
setup
def liner
 dang=2.*pi/float(nbeam)
 ang=-pi/4.0
 x1= rliner*cos(ang)
 y1= rliner*sin(ang)
 loop n (1,nbeam)
  ang=ang+dang
  x2=rliner*cos(ang)+xliner
  y2=rliner*sin(ang)+yliner
  np=1
  if n > 42 then
    np = 2
  end_if
  command
   struct beam beg x1 y1 end x2 y2 prop np
```

```
  end_command
  x1=x2
  y1=y2
 end_loop
end
liner
struct prop 1 e=28.85e9 height=0.20 width=1.0
struct prop 2 e=28.85e9 height=0.20 width=1.0
int 1 aside from node 1,nbeam to node 1 bside long from 78.37 to 78.37
int 1 ks 50e9 kn 10e9 fric 35
struct node 1 pin
struct node 43 pin

step 2000
sav c1k1g1d0.sav
*
m n reg 35,41  * 1st left drift
m n reg 111,41 * 1st right drift
step 2000
*
m n reg 6 41  * 2nd left drift
m n reg 139,41 * 2nd right drift
step 5000
sav c1k1g1n0.sav     *contain lining stress du to excavation
*
****************************************
*       Stage 3.     TBM Load
****************************************
struct node 48 load 0 -196200 0
struct node 52 load 0 -196200 0
step 2000
sav b1k1_tbm.sav
*
************************************************************
*       Stage 4.     One Wast Package Weight plus Gantry Weight
************************************************************
*
struct node 45 load 0 -91968.75 0
struct node 46 load 0 -91968.75
struct node 54 load 0 -91968.75
struct node 55 load 0 -91968.75 0
step 3000
sav c1k1_wpg.sav
*
**********************************************
*       Stage 5.     Single Waste Package Weight
**********************************************
res c1k1g1n0.sav
* Vertical nodal load is calculated as(1/2)*(90 MT)*(9810 N/MT)
struct node 47 load 0 -36787.5 0
struct node 48 load 0 -73575. 0
struct node 49 load 0 -73575. 0
struct node 50 load 0 -73575. 0
struct node 51 load 0 -73575. 0
struct node 52 load 0 -73575. 0
struct node 53 load 0 -36787.5 0
step 3000
sav c1k1_1wt.sav
*
************************************************************
*       Stage 6.     Two Wast Package Weights plus Gantry Weight
************************************************************
*
struct node 45 load 0 -91968.75 0
struct node 46 load 0 -91968.75
struct node 54 load 0 -91968.75
struct node 55 load 0 -91968.75 0
step 3000
sav c1k1_2wt.sav
*
```

```
*
***********************************************************
*    Stage 8: Start for Seismic Loading
***********************************************************
* turn on dynamic analysis
res e1k1_1wt.sav
set dyn on
set large
set st 3000000

apply syy sigy_top j=83
apply xquiet yquiet j=83        *top
apply xquiet  i=1              *left vertical
apply xquiet  i=145             *right vertical
*
def p_wave
 freq=freq_set
 dura=dura_set
  p_wave=1.*sin(2.*pi*freq*dytime)
 if dytime > dura_set then
  p_wave=0.0
 end_if
end
*
def s_wave
 freq=freq_set
 dura=dura_set
  s_wave=1.*sin(2.*pi*freq*dytime)
 if dytime > dura_set then
  s_wave=0.0
 end_if
end
*
set freq_set=10 dura_set=3
*
apply yvel  0.16 his p_wave j=1
apply xvel  0.16 his s_wave j=1
*
def run_time
 wave_on=dura_set
 wave_lon=2.*dura_set
end
run_time
*
set dytime 0
*
ini xdisp 0 ydisp 0
ini xvel=0 yvel=0

his reset
his nstep 400
his unbal
his dytime

* Horizontal velocity monitoring at the base line
his xvel i=1  j=1       * at the left corner
his xvel i=73 j=1       * at the center
his xvel i=145 j=1      * at the right corner
* Horizontal velocity monitoring at the top
his xvel i=1  j=83      * at the left corner
his xvel i=73 j=83      * at the center
his xvel i=145 j=83     * at the right corner
* Vertical velocity monitoring at the base line
his yvel i=1  j=1       * at the left corner
his yvel i=73 j=1       * at the center
his yvel i=145 j=1      * at the right corner
* Vertical velocity monitoring at the top
his yvel i=1  j=83      * at the left corner
his yvel i=73 j=83      * at the center
his yvel i=145 j=83     * at the right corner
```

```
his xdisp i 70 j 35
his ydisp i 70 j 35
his xdisp i 73 j 35
his ydisp i 73 j 35
his xdisp i 76 j 35
his ydisp i 76 j 35

his vc_c
his hc_c

his sxx i=72 j=34
his sxx i=73 j=34
his syy i=72 j=34
his syy i=73 j=34
his sxy i=72 j=34
his sxy i=73 j=34

his sig1 i=69 j=35
his sig2 i=69 j=35
his sxy  i=69 j=35
solve dytime 0.0125
sav k1t0125.sav
solve dytime 0.025
sav k1t0250.sav
solve dytime 0.0375
sav k1t0375.sav
solve dytime 0.050
sav k1t0500.sav
solve dytime 0.0625
sav k1t0625.sav
solve dytime 0.0750
sav k1t0750.sav
solve dytime 0.0875
sav k1t0875.sav
solve dytime 0.1000
sav k1t1000.sav
solve dytime 0.1125
sav k1t1125.sav
solve dytime 0.1250
sav k1t1250.sav
solve dytime 0.1375
sav k1t1375.sav
solve dytime 0.1500
sav k1t1500.sav
solve dytime 0.1625
sav k1t1625.sav
solve dytime 0.1750
sav k1t1750.sav
solve dytime 0.1875
sav k1t1875.sav
solve dytime 0.2000
sav k1t2000.sav
solve dytime 0.2125
sav k1t2125.sav
solve dytime 0.225
sav k1t2250.sav
solve dytime 0.2375
sav k1t2375.sav
solve dytime 0.250
sav k1t2500.sav
solve dytime 0.2625
sav k1t2625.sav
solve dytime 0.2750
sav k1t2750.sav
solve dytime 0.2875
sav k1t2875.sav
solve dytime 0.3000
sav k1t3000.sav
solve dytime 0.3125
sav k1t3125.sav
```

```
solve dytime 0.3250
sav k1t1250.sav
solve dytime 0.3375
sav k1t3375.sav
solve dytime 0.3500
sav k1t3500.sav
solve dytime 0.3625
sav k1t3625.sav
solve dytime 0.3750
sav k1t3750.sav
solve dytime 0.3875
sav k1t3875.sav
solve dytime 0.4000
sav k1t4000.sav
solve dytime wave_on
sav c1k1g1db.sav
solve dytime wave_lon
sav c1k1g1d2.sav
ret
*
```

6.2 INPUT DATA FILE FOR THE BEAM ELEMENT MODEL WITHOUT THE INTERFACE PRESENT

```
new
******************************************************************
*        Input File Name: inv_bond.dat
*
*        A FLAC model for analyzing a concrete invert to be placed
*        in an emplacement drift.
*        Drift diameter:      5.5 m
*        Drift spacing:                 28 m
*
*        Loadings:   Static + seismic + thermal
*
*                    Static: ko=0.5
*                            sigv_rep = 10 MPa
*
*                    Seismic: PGV=16 cm/s
*                             f= 10 Hz
*                             duration: 3 sec
*
*                    Thermal: Estimated based on
*                             ground support analysis
******************************************************************
*
* Fish function to initialize scale factors
*
def ke_value
 ko_m=float(ko)
 cat_mat = int(cat_m)
 sig_yy=-1.*sigv_rep
 e_sc=1.
 ch_sc=1.
 fr_sc=1.
 dila_sc=1.
 dn_sc=1.
end
*
* Fish function to list mechanical properties
*
def tsw2_new
case_of cat_mat
;
case 1
command
set e_m=7.76e9 v_m=0.21 ch_m=1.5e6 fr_m=43 dila_m=0  dn_m=2274 tn_m=1.32e6
end_command
;
case 2
command
set e_m=12.18e9 v_m=0.21 ch_m=2.1e6 fr_m=45  dila_m=0 dn_m=2274 tn_m=1.78e6
end_command
;
case 3
command
set e_m=15.75e9 v_m=0.21 ch_m=2.6e6 fr_m=45  dila_m=0 dn_m=2274 tn_m=2.17e6
end_command
;
case 4
command
set e_m=22.99e9 v_m=0.21 ch_m=3.7e6 fr_m=46  dila_m=0 dn_m=2274 tn_m=3.00e6
end_command
;
case 5
command
set e_m=32.61e9 v_m=0.21 ch_m=5.2e6 fr_m=46 dila_m=0 dn_m=2274 tn_m=4.21e6
```

```
end_command
;
end_case
end
*
set ko=0.5 cat_m=1 sigv_rep=10e6
*
************************************************************
* Description: Long parallel 5.5 m drifts on 28 m spacing  *
************************************************************
*
config dyn extra 3
*
* MESH CONSTRUCTION
*
def mesh_grd
 r1=1.0675
 r2=1./r1
end
mesh_grd
*
gr 144 82 * Mesh dimensions 140 m (wide) x 100 m (high)
m m      * Mohr-Coulomb Yield Criterion
*
gen -70,-50.0 -70,50.0  70, 50.0  70,-50.0
*
gen -70,-50.0 -70,-8.25 -36.25,-8.25 -36.25,-50.0    R=r2,r2 I=1,21 J=1,24
gen -70,-8.25 -70, 8.25 -36.25,8.25 -36.25,-8.25 R=r2,1  I=1,21 J=24,60
gen -70,8.25 -70,50.0  -36.25,50.0 -36.25, 8.25  R=r2,r1 I=1,21 J=60,83
*
gen -36.25,8.25 -36.25,50.0 36.25,50.0 36.25,8.25 R=1,r1 I=21,125 J=60,83
gen -36.25,-8.25 -36.25,8.25 36.25,8.25 36.25,-8.25 R=1,1 I=21,125 J=24,60
gen -36.25,-50.0 -36.25,-8.25 36.25,-8.25 36.25,-50.0 R=1,r2 I=21,125 J= 1,24
*
gen  36.25, 8.25 36.25,50.0  70,50.0  70, 8.25  R=r1,r1 I=125,145 J=60,83
gen  36.25,-8.25 36.25, 8.25 70, 8.25 70,-8.25  R=r1,1  I=125,145 J=24,60
gen  36.25,-50.0 36.25,-8.25 70,-8.25 70,-50.0     R=r1,r2 I=125,145 J= 1,24
*
ini x add 70   * Put the origin at the lower left corner
ini y add 50.0    * for doing mesh refinement
*
def h_drift
 loop i (istart, iend)
  xstart=x(istart,1)
  loop j (1,jgp)
   x(i,j)=xstart+x_int*(i-istart)
  end_loop
end_loop
end
*
def v_drift
 loop j (jstart,jend)
  ystart=y(1,jstart)
  loop i (1,igp)
   y(i,j)=ystart+y_int*(j-jstart)
  end_loop
 end_loop
end
*
****************************************
* Adjust the x-cord for the left drift
****************************************
*
set istart=21 iend=45 x_int=0.6875
h_drift
*
**********************************************************************
* Adjust the x-cord for the pillar between the left and middle drifts
**********************************************************************
*
```

```
set istart=45 iend=56 x_int=1.0
h_drift
*
****************************************************
* Adjust the x-cord and y-cord for the middle drift
****************************************************
*
set istart=56 iend=63 x_int=0.70
h_drift
set istart=63 iend=83 x_int=0.385
h_drift
set istart=83 iend=90 x_int=0.70
h_drift
set jstart=24 jend=32 y_int=0.55
v_drift
set jstart=32 jend=52 y_int=0.385
v_drift
set jstart=52 jend=60 y_int=0.55
v_drift
*
*********************************************************************
* Adjust the x-cord for the pillar between the middle and right drifts
*********************************************************************
*
set istart=90 iend=101 x_int=1.0
h_drift
*
******************************************
* Adjust the x-cord for the right drift
******************************************
*
set istart=101 iend=125 x_int=0.6875
h_drift
*
ini x add -70 * Set the origin back to the center of the center drift.
ini y add -50.0
*
ini x -60 i=5
ini x -58 i=6
ini x -56 i=7
ini x -54 i=8
ini x -52 i=9
*
ini x 52 i=137
ini x 54 i=138
ini x 56 i=139
ini x 58 i=140
ini x 60 i=141
*
gen circle  0  0 2.75 * center drift
gen circle -28.0 0 2.75 * immediate left drift
gen circle  28.0 0 2.75 * immediate right drift
gen circle -56.0 0 2.75 * remote left drift
gen circle  56.0 0 2.75 * remote right drift
gen adjust
*
* FISH function to find minima and maxima of x- and y-coordinates
*
def max_min
 xmax=0.
 xmin=10e5
 ymax=0.
 ymin=10e5
 loop i (1,igp)
  loop j (1,jgp)
   if x(i,j) > xmax then
    xmax=x(i,j)
   end_if
   if x(i,j) < xmin then
    xmin=x(i,j)
```

```
      end_if
      if y(i,j) > ymax then
        ymax=y(i,j)
      end_if
      if y(i,j) < ymin then
        ymin=y(i,j)
      end_if
    end_loop
  end_loop
end
max_min
*
**************************************************
* Assign material properties and initial stresses
**************************************************
kc_value  * execute the scale factors
*
def mat_ini
  sigv_top = sigv_rep-9.81*dn_m*ymax
  sigy_top = -sigv_top
  e_m=e_sc*e_m
  ch_m=ch_sc*ch_m
  fr_m=fr_sc*fr_m
  dila_m=dila_sc*dila_m
  sh_m=0.5*e_m/(1.+v_m)
  bk_m=(1/3.)*e_m/(1.-2.*v_m)
  ang=fr_m*pi/180.
  tn_m=1.*tn_m
  grad_v=9.81*dn_m*(ymax-ymin)
  grad_h=ko_m*grad_v
  sigv=-(sigv_top+grad_v)
  sigh=ko_m*sigv
   command
    prop s=sh_m b=bk_m d=dn_m
    prop coh=ch_m fric=fr_m dila=dila_m ten=tn_m
    ini syy sigv var 0 grad_v
    ini sxx sigh var 0 grad_h
    ini szz sigh var 0 grad_h
    apply syy sigy_top j=83
  end_command
end
*
set grav=9.81
tsw2_new
mat_ini
*
***********************************
* Fix the boundary conditions
***********************************
fix x i=1
fix x i=145
fix y j=1
*
set dyn off
his unbal
step 200        *initial equilibrium
*
*****************************************************
*       Stage 1.     Excavate the Center Drift
*****************************************************
*
ini xdisp=0 ydisp=0
*
* Monitor the closures for the center drift
*
def vc_c
 vc_c=ydisp(73,35)-ydisp(73,49)
end
def hc_c
 hc_c=xdisp(66,42)-xdisp(80,42)
```

```
end
his vc_c
his hc_c
*
* Monitor the closures for the left drift
*
def vcl_l
 vcl_l=ydisp(33,35)-ydisp(33,49)
end
def hcl_l
 hcl_l=xdisp(29,42)-xdisp(37,42)
end
his vcl_l
his hcl_l
*
* Monitor the closures for the right drift
*
def vcl_r
 vcl_r=ydisp(113,35)-ydisp(113,49)
end
def hcl_r
 hcl_r=xdisp(109,42)-xdisp(117,42)
end
his vcl_r
his hcl_r
*
* Monitoring at particular points around the center drift
*
his xdisp i 68 j 36
his ydisp i 68 j 36
his xdisp i 70 j 35
his ydisp i 70 j 35
his xdisp i 73 j 35
his ydisp i 73 j 35
his xdisp i 76 j 35
his ydisp i 76 j 35
his xdisp i 78 j 36
his ydisp i 78 j 36
*
set ncw 100
m n reg 73, 41  * the center drift
step 240  * 60% ground relaxation upon drift excavation

*
*********************************************************
*        Stage 2.     Place the invert and lining
*********************************************************
*
* FISH function to find all boundary grid points
*
def boung
loop i (1,igp)
 loop j (1,jgp)
  ex_1(i,j)=0
   if and(flags(i,j),8)=0 then
    loop ii (i-1,i+1)
     loop jj (j-1,j+1)
      if ii>0 then
      if ii<igp+1 then
      if jj>0 then
      if jj<jgp+1 then
        if and(flags(ii,jj),8)=8 then
         ex_1(i,j)=1
        end_if
      end_if
      end_if
      end_if
      end_if
     end_loop
    end_loop
```

```
      end_if
    end_loop
  end_loop
  loop i (1,igp)
    if and(flags(i,1),8)=0 then
      ex_1(i,1)=1
    end_if
    if and(flags(i,jgp),8)=0 then
      ex_1(i,jgp)=1
    end_if
  end_loop
  loop j (1,jgp)
    if and(flags(1,j),8)=0 then
      ex_1(1,j)=1
    end_if
    if and(flags(igp,j),8)=0 then
      ex_1(igp,j)=1
    end_if
  end_loop
end
*
* FISH function to create list of STRUCT BEAM commands for segment of tunnel
*
def beam
  if ie=0 then
    ie=ib
  end_if
  if je=0 then
    je=jb
  end_if
  if nprop=0 then
    nprop=1
  end_if
  iet=ib
  jet=jb
 ibt=ib
 jbt=jb
  loop k (1,2)
  if iet=ie then
    if jet=je then
      if count>0 then
        exit
      end_if
    end_if
  end_if
  if err1=1 then
      command
        print err1
      end_command
      exit
  end_if
  beam1
  k=k-1
  end_loop
end
def beam1
  ibp=ibt
  jbp=jbt
  ibt=iet
  jbt=jet
  if ex_1(ibt,jbt+1)=1 then
    iet=ibt
    jet=jbt+1
    flag1=0
    beam2
    if flag1=0 then
    exit
    end_if
  end_if
  if ex_1(ibt-1,jbt)=1 then
```

```
   iet=ibt-1
   jet=jbt
   flag1=0
   beam2
   if flag1=0 then
   exit
   end_if
  end_if
  if ex_1(ibt,jbt-1)=1 then
   iet=ibt
   jet=jbt-1
   flag1=0
   beam2
   if flag1=0 then
    exit
   end_if
  end_if
  if ex_1(ibt+1,jbt)=1 then
   iet=ibt+1
   jet=jbt
   flag1=0
   beam2
   if flag1=0 then
   exit
   end_if
  end_if
  err1=1
 end
 def beam2
 if iet=ibp then
  if jet=jbp then
  flag1=1
  exit
  end_if
 end_if
  command
   stru beam beg grid ibt jbt end grid iet jet prop nprop
  end_command
  count=count+1
  ex_1(iet,jet)=2
 end
 set ib =78 jb=37 ie=68 jb=37 nprop=1
 boung
 beam
 set ib=68 jb=37 ie=78 jb=37 nprop=2
 boung
 beam

 struct prop 1 e=28.85e9 height=0.20 width=1.0
 struct prop 2 e=28.85e9 height=0.20 width=1.0

 struct node 1 pin
 struct node 15 pin
 step 2000
 sav b1k1g1d0.sav
 *
 m n reg 35,41  * 1st left drift
 m n reg 111,41 * 1st right drift
 step 2000
 *
 m n reg 6 41   * 2nd left drift
 m n reg 139,41 * 2nd right drift
 step 5000
 sav b1k1g1n0.sav     *contain lining stress du to excavation

 *
 ***********************************************
 *        Stage 3.    TBM Load
 ***********************************************
 struct node 10 load 0 -196200 0
```

```
struct node 6 load 0 -196200 0
step 2000
sav blk1_tbm.sav
*
*****************************************************************
*       Stage 4.     One Wast Package Weight plus Gantry Weight
*****************************************************************
*
res blk1g1n0.sav
struct node 4 load 0 -91968.75 0
struct node 5 load 0 -91968.75 0
struct node 11 load 0 -91968.75 0
struct node 12 load 0 -91968.75 0
step 3000
sav blk1_wpg.sav
*****************************************************
*       Stage 5.     Single Waste Package Weight
*****************************************************
res blk1g1n0.sav
* Vertical nodal load is calculated as (90 MT)*(9810 N/MT)/2
struct node 6 load 0 -55181.25 0
struct node 7 load 0 -110362.5 0
struct node 8 load 0 -110362.5 0
struct node 9 load 0 -110362.5 0
struct node 10 load 0 -55181.25 0
step 3000
sav blk1_1wt.sav
*
*****************************************************************
*       Stage 6.     Two Wast Package Weights plus Gantry Weight
*****************************************************************
*
struct node 4 load 0 -91968.75 0
struct node 5 load 0 -91968.75 0
struct node 11 load 0 -91968.75 0
struct node 12 load 0 -91968.75 0
step 3000
sav blk1_2wt.sav

*
*****************************************************
*     Stage 8: Start for Different Seismic Loading
*****************************************************
* turn on dynamic analysis
res blk1_1wt.sav
set dyn on
set large
set st 3000000

apply syy sigy_top j=83
apply xquiet yquiet j=83        *top
apply xquiet i=1               *left vertical
apply xquiet i=145              *right vertical
*
def p_wave
 freq=freq_set
 dura=dura_set
  p_wave=1.*sin(2.*pi*freq*dytime)
 if dytime > dura_set then
  p_wave=0.0
 end_if
end
*
def s_wave
 freq=freq_set
 dura=dura_set
  s_wave=1.*sin(2.*pi*freq*dytime)
 if dytime > dura_set then
  s_wave=0.0
 end_if
```

```
end
*
set freq_set=10 dura_set=3
*
apply yvel 0.16 his p_wave j=1
apply xvel 0.16 his s_wave j=1
*
def run_time
 wave_on=dura_set
 wave_lon=2.*dura_set
end
run_time
*
set dytime 0
*
ini xdisp 0 ydisp 0
ini xvel=0 yvel=0

his reset
his nstep 400
his unbal
his dytime

* Horizontal velocity monitoring at the base line
his xvel i=1  j=1      * at the left corner
his xvel i=73 j=1      * at the center
his xvel i=145 j=1     * at the right corner
* Horizontal velocity monitoring at the top
his xvel i=1  j=83     * at the left corner
his xvel i=73 j=83     * at the center
his xvel i=145 j=83    * at the right corner
* Vertical velocity monitoring at the base line
his yvel i=1  j=1      * at the left corner
his yvel i=73 j=1      * at the center
his yvel i=145 j=1     * at the right corner
* Vertical velocity monitoring at the top
his yvel i=1  j=83     * at the left corner
his yvel i=73 j=83     * at the center
his yvel i=145 j=83    * at the right corner

his xdisp i 70 j 35
his ydisp i 70 j 35
his xdisp i 73 j 35
his ydisp i 73 j 35
his xdisp i 76 j 35
his ydisp i 76 j 35

his vc_c
his hc_c

his sxx i=72 j=34
his sxx i=73 j=34
his syy i=72 j=34
his syy i=73 j=34
his sxy i=72 j=34
his sxy i=73 j=34

his sig1 i=69 j=35
his sig2 i=69 j=35
his sxy  i=69 j=35

solve dytime 0.0125
sav bk1t0125.sav
solve dytime 0.025
sav bk1t0250.sav
solve dytime 0.0375
sav bk1t0375.sav
solve dytime 0.050
sav bk1t0500.sav
solve dytime 0.0625
```

```
sav bk1t0625.sav
solve dytime 0.0750
sav bk1t0750.sav
solve dytime 0.0875
sav bk1t0875.sav
solve dytime 0.1000
sav bk1t1000.sav
solve dytime 0.1125
sav bk1t1125.sav
solve dytime 0.1250
sav bk1t1250.sav
solve dytime 0.1375
sav bk1t1375.sav
solve dytime 0.1500
sav bk1t1500.sav
solve dytime 0.1625
sav bk1t1625.sav
solve dytime 0.1750
sav bk1t1750.sav
solve dytime 0.1875
sav bk1t1875.sav
solve dytime 0.2000
sav bk1t2000.sav
solve dytime 0.2125
sav bk1t2125.sav
solve dytime 0.225
sav bk1t2250.sav
solve dytime 0.2375
sav bk1t2375.sav
solve dytime 0.250
sav bk1t2500.sav
solve dytime 0.2625
sav bk1t2625.sav
solve dytime 0.2750
sav bk1t2750.sav
solve dytime 0.2875
sav bk1t2875.sav
solve dytime 0.3000
sav bk1t3000.sav
solve dytime 0.3125
sav bk1t3125.sav
solve dytime 0.3250
sav bk1t1250.sav
solve dytime 0.3375
sav bk1t3375.sav
solve dytime 0.3500
sav bk1t3500.sav
solve dytime 0.3625
sav bk1t3625.sav
solve dytime 0.3750
sav bk1t3750.sav
solve dytime 0.3875
sav bk1t3875.sav
solve dytime 0.4000
sav bk1t4000.sav
solve dytime wave_on
sav b1k1g1db.sav
solve dytime wave_lon
sav b1k1g1d2.sav
ret
```

6.3 INPUT DATA FILE FOR THE PLANE STRAIN ELEMENT MODEL

```
*******************************************************
*       Input file name: inv_plsm.dat          *
*                              *
*    A PLANE STRAIN ELEMENT FLAC MODE
*
*    FOR THE CONCRETE INVERT IN EMPLACEMENT DRIFT *
*******************************************************
config dyn extra 1
**************
*kc_value.fis
**************
def kc_value
 ko_m=float(ko)
 cat_mat = int(cat_m)
 e_sc=1.
 ch_sc=1.
 fr_sc=1.
 dila_sc=1.
 dn_sc=1.
end
********************
*tsw2.fis
********************
def tsw2
case_of cat_mat
;
case 1
command
set e_m=7.76e9 v_m=0.21 ch_m=1.5e6 fr_m=43 dn_m=2274 tn_m=1.32e6
end_command
;
case 2
command
set e_m=12.18e9 v_m=0.21 ch_m=2.1e6 fr_m=45 dn_m=2274 tn_m=1.78e6
end_command
;
case 3
command
set e_m=15.75e9 v_m=0.21 ch_m=2.6e6 fr_m=45 dn_m=2274 tn_m=2.17e6
end_command
;
case 4
command
set e_m=22.99e9 v_m=0.21 ch_m=3.7e6 fr_m=46 dn_m=2274 tn_m=3.00e6
end_command
;
case 5
command
set e_m=32.61e9 v_m=0.21 ch_m=5.2e6 fr_m=46 dn_m=2274 tn_m=4.21e6
end_command
;
end_case
end
set ko=0.50 cat_m=1
*
* Mesh construction
def mesh_grd
rat1=1.1
rat2=1./rat1
end
mesh_grd

GR 90 90
m m *
GEN -30,-30 -30, 30  30, 30  30,-30 * mesh dimensions 60m x 60m
GEN -30,-30 -30,-10 -10,-10 -10,-30 R=rat2,rat2 I= 1,11 J= 1,11
GEN -30,-10 -30, 10 -10, 10,-10,-10 R=rat2,1   I= 1,11 J=11,21
```

```
GEN -30, 10 -30, 30 -10, 30 -10, 10 R=rat2,rat1 i= 1,11 j=81,91
GEN -10, 10 -10, 30  10, 30  10, 10 R=1,  rat1 i=11,81 j=81,91
*
GEN -10,-10 -10, 10  10, 10  10,-10          i=11,81 j=11,81
*
GEN -10,-30 -10,-10  10,-10  10,-30 R=1,  rat2 i=11,81 j= 1,11
GEN  10, 10  10, 30  30, 30  30, 10 R=rat1,rat1 i=81,91 j=81,91
GEN  10,-10  10, 10  30, 10  30,-10 R=rat1,1    i=81,91 j=11,81
GEN  10,-30  10,-10  30,-10  30,-30 R=rat1,rat2 i=81,91 j= 1,11
*
def xy_adj_l
 loop i (12,21)
  loop j (1,jgp)
   x(i,j)=x(i-1,j)+(7./10)
  end_loop
 end_loop
end
xy_adj_l
*
def xy_adj_c
 loop i (22,71)
  loop j (1,jgp)
   x(i,j)=x(i-1,j)+(6./50)
  end_loop
 end_loop
end
xy_adj_c
*
def xy_adj_r
 loop i (72,81)
  loop j (1,jgp)
   x(i,j)=x(i-1,j)+(7./10)
  end_loop
 end_loop
end
xy_adj_r
*
def xy_adj_u
 command
  ini y=0 j=61
 end_command
loop j (62,71)
  loop i (1,igp)
   y(i,j)=y(i,j-1)+(3./10)
  end_loop
 end_loop
end
xy_adj_u
*
def xy_adj_v
 loop j (72,81)
  loop i (1,igp)
   y(i,j)=y(i,j-1)+(7/10.)
  end_loop
 end_loop
end
xy_adj_v
*
def xy_adj_d
 loop j (12,26)
  loop i (1,igp)
   y(i,j)=y(i,j-1)+(6.5/15.)
  end_loop
 end_loop
end
xy_adj_d
*
def xy_adj_m
 loop j (27,51)
  loop i (1,igp)
```

```
    y(i,j)=y(i,j-1)+(2/25.)
   end_loop
  end_loop
end
xy_adj_m
;
def xy_adj_o
  loop j (52,61)
   loop i (1,igp)
    y(i,j)=y(i,j-1)+(1.5/10.)
   end_loop
  end_loop
end
xy_adj_o

GEN CIRCLE 0 0 2.75
gen circle 0.0 -0.815 1.0
;
gen adj
ini y -2.437 i=40,52 j=40
ini y -2.5  i=40,52 j=39
;
gen line -0.6166 -2.637 -1.244 -1.695
gen line 0.6166 -2.637 1.244 -1.695
;
gen line 1.111 -1.895 1.706 -1.895
gen line -1.111 -1.895 -1.706 -1.895
;
gen line -0.75 -2.437 0.75 -2.437
gen line -1.05 -1.987 1.05 -1.987
;
mark i=61 j=46
mark i=31 j=46
;
mark i=50 j=45
mark i=51 j=46,49
mark i=42 j=45
mark i=41 j=46,49
;
gen arc 0 0 -2.654, -1.039, 137.24
gen adjust
;
def umark_gn
  t_r=2.85
  tol_r=0.005
  loop ii (1,igp)
   loop jj (1,jgp)
    xx=x(ii,jj)
    yy=y(ii,jj)
    dd=sqrt(xx*xx + yy*yy)
    dr=abs(dd-t_r)
    if dr <= tol_r then
     command
      unmark i=ii j=jj
     end_command
    end_if
   end_loop
  end_loop
 end
 umark_gn
 def max_min
  xmax=0.
  xmin=10e5
  ymax=0.
  ymin=10e5
  loop i (1,igp)
   loop j (1,jgp)
    if x(i,j) > xmax then
     xmax=x(i,j)
    end_if
```

```
    if x(i,j) < xmin then
      xmin=x(i,j)
    end_if
    if y(i,j) > ymax then
      ymax=y(i,j)
    end_if
    if y(i,j) < ymin then
      ymin=y(i,j)
    end_if
   end_loop
  end_loop
end
max_min
***********************************************************
* Assign material properties,
*      initial stresses and
*      initial temperature according to ground strata
***********************************************************
kc_value  * execute the scale factors
*
def mat_ini
 sigv_top = 9.33076e6
 sigy_top = -1*sigv_top
 e_m=e_sc*e_m
 ch_m=ch_sc*ch_m
 fr_m=fr_sc*fr_m
 dila_m=dila_sc*dila_m
 sh_m=0.5*e_m/(1.+v_m)
 bk_m=(1/3.)*e_m/(1.-2.*v_m)
 ang=fr_m*pi/180.
 tn_m=1.*tn_m
 grad_v=9.81*dn_m*(ymax-ymin)
 grad_h=ko_m*grad_v
 sigv=-(sigv_top+grad_v)
 sigh=ko_m*sigv
  command
   prop s=sh_m b=bk_m d=dn_m
   prop coh=ch_m fric=fr_m dila=dila_m ten=tn_m
   ini syy sigv var 0 grad_v
   ini sxx sigh var 0 grad_h
   ini szz sigh var 0 grad_h
   apply syy sigy_top j=91
  end_command
end
*
set grav=9.81
tsw2
mat_ini
*

fix x i=1
fix x i=91
fix y j=1
set dyn off
his unbal
step 500    * initial setting
ini xdisp=0 ydisp=0
* monitor the closures for the center drift
def vcl
 vcl=ydisp(46,35)-ydisp(46,70)
end
def hcl
 hcl=xdisp(23,61)-xdisp(69,61)
end
his vcl
his hcl
his ydisp i 46 j 35
set ncw 100
*****************************************
*     Stage 1: Excavate the drift
```

```
****************************************
m n reg 30,56 *rock annulus
m n reg 40,56 *package
m n reg 45,45 *pestal
m n reg 46,42 *pier
m n reg 45,37 *pit
m n reg 34,42 *left shoulder
m n reg 56,42 *right shoulder
step 620  *relax the ground to 60%
*sav p_excav1.sav
**********************************************************
*     Stage 2: Soften the invert foundation rock
**********************************************************
def mark_gn
 t_r=2.85
 tol_r=0.005
 loop ii (1,igp)
  loop jj (1,jgp)
   xx=x(ii,jj)
   yy=y(ii,jj)
   dd=sqrt(xx*xx + yy*yy)
   dr=abs(dd-t_r)
   if dr <= tol_r then
    command
     mark i=ii j=jj
    end_command
   end_if
  end_loop
 end_loop
end
mark_gn
*
m e reg 35,38
m e reg 63,45
m e reg 45,34
m e reg 43,34
m e reg 48,34
prop b=4.4598e8 s=3.2066e8 den=2274 reg 35,38
prop b=4.4598e8 s=3.2066e8 den=2274 reg 63,45
prop b=4.4598e8 s=3.2066e8 den=2274 reg 45,34
prop b=4.4598e8 s=3.2066e8 den=2274 reg 43,34
prop b=4.4598e8 s=3.2066e8 den=2274 reg 48,34
*
******************************************************
*     Stage 3: Install the invert, pestal and lining
******************************************************
*
m e reg 45 37
m e reg 34,42
m e reg 56,42
m n i=36 j=46
m n i=37 j=44
m n i=54 j=44
m n i=55 j=46
prop b=15.85e9 s=11.40e9 den=2000 reg 45,37
prop b=15.85e9 s=11.40e9 den=2000 reg 34,42
prop b=15.85e9 s=11.40e9 den=2000 reg 56,42
call boung.fis
call beam.fis
set ib=30 jb=46 ie=62 jb=46 nprop=1
beam
struct prop 1 e=28.85e9 height=0.20 width=1.0
struct prop 2 e=28.85e9 height=0.20 width=1.0
step 6000
sav pk1_g1d0.sav
*
****************************************************
*     Stage 4: TBM Transportation Load
****************************************************
apply yf -196200 i=42 j=40
```

```
apply yf -196200 i=50 j=40
step 2000
sav pk1_tbm.sav
*************************************************
*    Stage 5: Gantry load + WP
*************************************************
res pk1_g1d0.sav
app pressure 502560 i=33,36 j=46
app pressure 502560 i=56,59 j=46    *based on (150/4)*9810/(0.366*1)
step 2000
sav pk1_gtry.sav
*
*************************************************
*    Stage 6: Single W.P. Weight
*************************************************
res pk1_g1d0.sav
m e reg 40,56
m e reg 45,45
m e reg 46,42
prop b=114.94e9 s=82.64e9 den=5370 reg 40,56  *based on (90)*1000/(pi)/5.335
prop b=114.94e9 s=82.64e9 den=7800  reg 44,45
prop b=15.85e9  s=11.40e9 den=2000  reg 46,42
step 5000
sav pk1_1wt.sav
*
*************************************************
*    Stage 7: Two W.P. Weights + Gantry
*************************************************
app pressure 502560 i=33,36 j=46
app pressure 502560 i=56,59 j=46    *based on (150/4)*9810/(0.366*1)
step 5000
sav pk1_2wt.sav
*
*******************************************
*    Stage 8: Seismic Loading
*******************************************
res pk1_1wt.sav
interior xf  132435 i=46 j=55
interior yf -132435 i=46 j=55
step 5000
sav pk1_g1da.sav
ret
```

ATTACHMENT III
MISCELLANEOUS REFERENCE DATA

MIDWEST RAILS, TRACKWORK AND ACCESSORIES

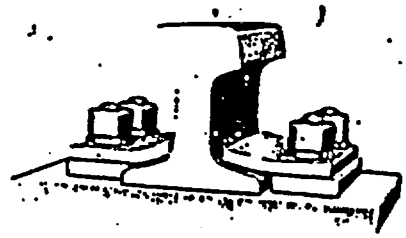
MIDWEST STEEL DIVISION

General Offices
510 Capitol Street
Charleston, W. Va. 25321
(304) 343-8874

Branches
1107 22nd Street
Granite City, Illinois 62040
(314) 241-6061

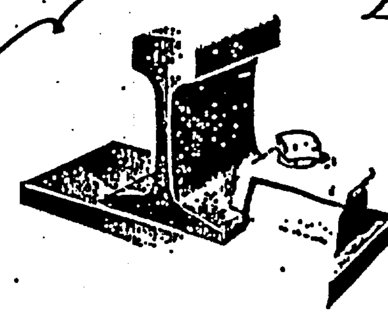
736 East Main Street
Pomeroy, Ohio 45769
(614) 992-3265

MIDWEST CORPORATION



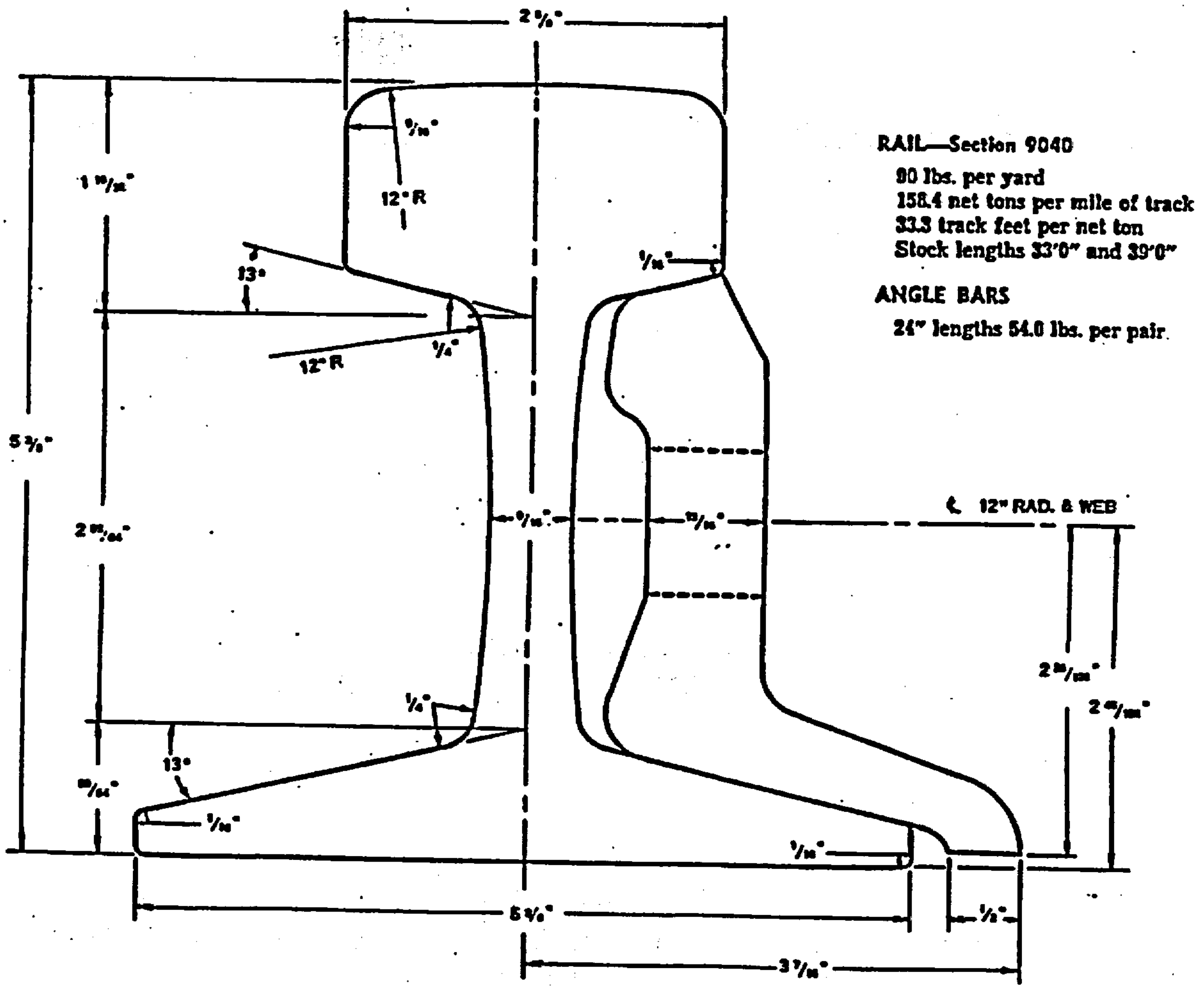
SINGLE CLAMP AND FILLER NO. K-301
DOUBLE CLAMP AND FILLER NO. K-302

These pressed steel clamps are furnished with reversible fillers. They are usually furnished for a "tight" fit on rail base unless specified otherwise.



SINGLE CLAMP AND HOLDER No. K-107
DOUBLE CLAMP AND HOLDER No. K-108

These heavy fasteners are used to anchor crane rails to supporting beams. The holder is usually welded to supporting member, but can be bolted when specified. Furnished complete with bolts and lock washers.



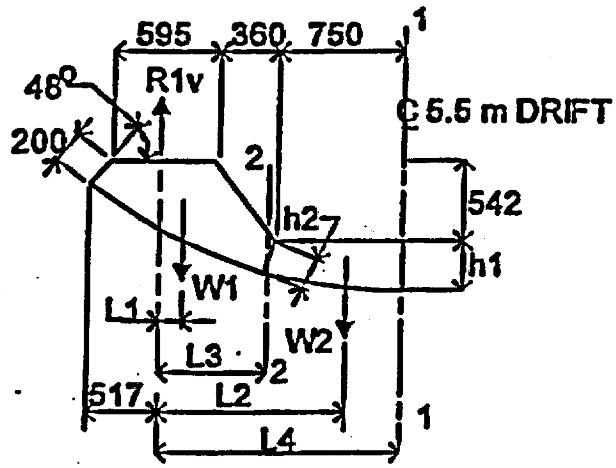
MIDWEST STEEL DIVISION

MIDWEST CORPORATION

ATTACHMENT IV
REINFORCED CONCRETE DESIGN

DOE policy requires the subsurface design be performed using metric units. Much source information (e.g., vendor data/steel member sizes) used for design, however, is available only in English units. Because of this, calculations are generally performed in English units. The results are converted to metric units in the main body of the analysis, followed by the corresponding English values in parenthesis.

Check capacity of precast concrete invert due to lifting loads. Lifting lugs assumed to be placed at centerline of gantry rail.Refer to Attchment II figure II-3 for data shown in sketch below. Note: Where ACI is shown in this attachment it refers to ACI-318-95.



Precast Concrete Invert symetrical about center line of tunnel

Data:

$k = 1000 \cdot lb$ $\quad psi = \frac{lb}{in^2}$

$f_c := 5000 \cdot \frac{lb}{in^2}$ Concrete compressive strength @ 28 days
Note: fc = f'c, typical pages IV-2 thru IV-19

$L_1 := 2.82 \cdot in$ (71.7 mm) $h_1 := 12.32 \cdot in$ (313 mm) $L_2 := 37.77 \cdot in$ (959.3 mm) $b := 12 \cdot in$ (304.8 mm)

$L_3 := 21.4 \cdot in$ (543.6 mm) $h_2 := 7.874 \cdot in$ (200 mm) $L_4 := 52.09 \cdot in$ (1323 mm)

$f_y := 60000 \cdot \frac{lb}{in^2}$ Reinforcing steel yield strength

$\gamma_c := 150 \cdot \frac{lb}{ft^3}$ Unit weight of concrete

Concrete Areas and centroid locations calculated from Micro Station measure command.

$V_1 := 4.69 \cdot ft^3$ Volume of concrete per foot bounded to the left of section 2-2

$V_2 := 2.31 \cdot ft^3$ Volume of concrete per foot bounded to the right of section 2-2

$W_1 := \gamma_c \cdot V_1$ $\quad W_1 = 703.5 \cdot lb$ concrete weight of element 1

$W_2 := \gamma_c \cdot V_2$ $\quad W_2 = 346.5 \cdot lb$ concrete weight of element 2

$R_{1v} := W_1 + W_2$ $\quad R_{1v} = 1050 \cdot lb$ Total concrete weight of elements 1 + 2 and construction live load.

Find moment and shear at section 1-1:

Sum moments at section 1-1

$M_1 := R_{1v} \cdot L_4 - W_1 \cdot (L_4 - L_1) - W_2 \cdot (L_4 - L_2)$ Moment at section 1-1

$M_1 = 15071.18 \cdot in \cdot lb$

Sum moments at section 2-2

$M_2 := \left[(R_{1v} \cdot L_3) - W_1 \cdot (L_3 - L_1) \right]$ Moment at section 2-2

$M_2 = 9398.97 \cdot in \cdot lb$

Try ACI Structural Plain Concrete, chapter 22, strength design procedure.

$\phi := 0.65$ Strength reduction factor ACI, section 9.3.5

$U := 1.4$ Strength factor ACI, section 9.2, deadload

$M_{u1} := U \cdot M_1$ $M_{u1} = 21099.645 \cdot in \cdot lb$ Factor moment at section 1-1

$M_{u2} := U \cdot M_2$ $M_{u2} = 13158.558 \cdot in \cdot lb$ Factor moment at section 2-2

$A_1 := b \cdot h_1$ $A_1 = 147.84 \cdot in^2$ Area at section 1-1 per foot

$S_1 := \frac{b \cdot h_1^2}{6}$ $S_1 = 303.5648 \cdot in^3$ Section modulus at section 1-1 per foot

$A_2 := b \cdot h_2$ $A_2 = 94.488 \cdot in^2$ Area at section 2-2 per foot

$S_2 := \frac{b \cdot h_2^2}{6}$ $S_2 = 123.99975 \cdot in^3$ Section modulus at section 2-2 per foot

$$\phi M_{n1} := \phi \cdot 5 \cdot S_1 \cdot \sqrt{\frac{f_c}{\left(\frac{lb}{in^2}\right)}} \cdot \frac{lb}{in^2} \qquad \phi M_{n1} = 69762.1368 \cdot in \cdot lb$$

Factor nominal moment strength at section 1-1, ACI, equation 22-2,

$$M_{u1} = 21099.645 \cdot in \cdot lb$$

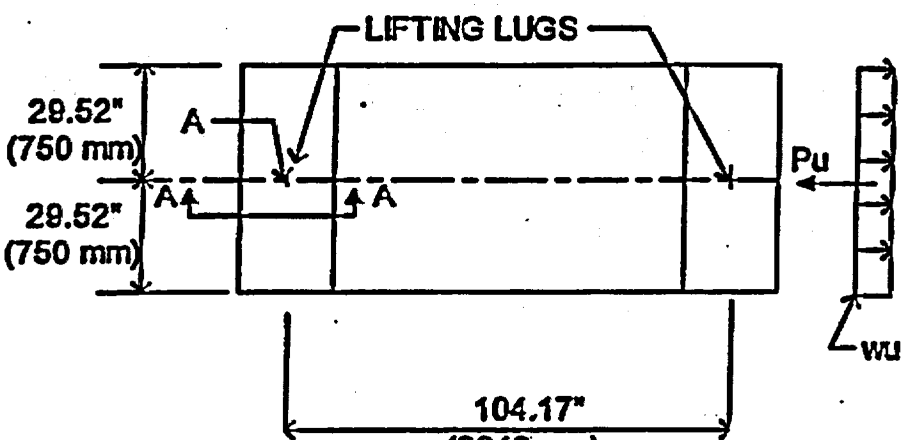
Factor moment at section 1-1 less than factored nominal moment strength

$$\phi M_{n2} := \phi \cdot 5 \cdot S_2 \cdot \sqrt{\frac{f_c}{\left(\frac{lb}{in^2}\right)}} \cdot \frac{lb}{in^2} \qquad \phi M_{n2} = 28496.34629 \cdot in \cdot lb$$

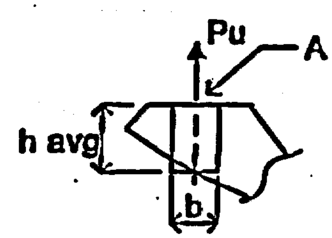
Allowable factor moment at section 2-2, ACI, equation 22-2,

$$M_{u2} = 13158.558 \cdot in \cdot lb$$

Factor moment at section 2-2 less than factored nominal moment strength



PRECAST INVERT PLAN



SECTION A-A

Calculate bending moment in short direction of panel about lifting lug. Use an equivalent rectangular beam to resist bending, see Precast Invert Plan and Section above.

$w_u := 1.4 \cdot \frac{R_{1v}}{12 \cdot in}$ $w_u = 122.5 \cdot \frac{lb}{in}$ Factor load ACI, section 9.2, see first page of calculation for R1v.

Radius of Drift = 108.267" $b := 16 \cdot in$ Conservatively use 16" beam width center about lifting lug, see section A-A above.

$h_{avg} := \sqrt{(108.267 \cdot in)^2 - L_4^{\,2}} - (108.267 \cdot in - 21.339 \cdot in - h_1)$ Depth of concrete at centerline of lifting lug, see section A-A above.

$h_{avg} = 20.30446 \cdot in$

$S_{bm} := \frac{b \cdot h_{avg}^{\,2}}{6}$ $S_{bm} = 1099.38966 \cdot in^3$ Section modulus of equivalent rectangular beam, see section A-A.

$M_u := w_u \cdot \frac{(29.52 \cdot in)^2}{2}$ $M_u = 53375.112 \cdot in \cdot lb$ Cantilever Bending moment about lifting lug, see Precast Invert Plan for dimension = 29.52".

$f_t := \frac{M_u}{S_{bm}}$ $f_t = 48.54977 \cdot \frac{lb}{in^2}$ Required concrete tensile stress at point A. see Precast Invert Plan and Section A-A.

$f_{ct} := 5 \cdot \phi \cdot \sqrt{\frac{f_c}{\left(\frac{lb}{in^2}\right)}} \cdot \frac{lb}{in^2}$ $f_{ct} = 229.8097 \cdot \frac{lb}{in^2} >_{ok} f_t = 48.54977 \cdot \frac{lb}{in^2}$ Allowable tensile stress, ACI, section 22.5.3 greater than required tensile stress.

Conclusion: Plain concrete adequate to carry lifting load at 28 day compressive strength of fc = 5000 psi.

Design Invert using the results from the FLAC analysis, Attachment II, Table II-2. Inspection of Table II-2 indicates two controlling load cases:

Load Case 1 (During WP's sitting on invert) Design is based on contact grout between invert and tunnel as dicussed in Attachment II, section 5.

Load Case 2 (During Earthquake: WP's in place) Design is based on contact grout between invert and tunnel as discussed in Attachment II, section 5.

The 15 MPa hoop stress due to thermal loading from disposal containers is required to be added to the above
Try: 200 mm minimum invert thicknes = 7.874 inches. Design Invert using one foot beam strip along tunnel length.

$h := 7.874 \cdot in$	Thickness of invert at haunch	$KN \equiv 224.809 \cdot lb$	Conversion kilonewton to pound
$b := 12 \cdot in$	Design width of invert	$m \equiv 3.28084 \cdot ft$	Conversion meter to feet
		$MPa \equiv 145.038 \cdot psi$	Conversion from megapascal to pounds per square inch

Load Case 1 (During WP's sitting on invert):

$P_{thrustdl} := 1368 \cdot \frac{KN}{m} \cdot ft$	$P_{thrustdl} = 93737.79642 \cdot lb$	Thrust load per foot of invert due to ground support, Attachment II, Table II-2
$\rho_{ground} := \frac{P_{thrustdl}}{b \cdot h}$	$\rho_{ground} = 992.06033 \cdot psi$	Hoop compressive stress due to ground loading
Use: $\rho_{thermal} := 15 \cdot MPa$	$\rho_{thermal} = 2175.57 \cdot psi$	Hoop maximum compressive stress due to thermal loading Attachment II, page II-5.
Use: $M_{ground} := \frac{16.3 \cdot KN \cdot m}{m} \cdot 1 \cdot ft$	$M_{ground} = 43972.6404 \cdot lb \cdot in$	Moment per foot of invert due to ground loading, Attachment II, Table II-2.

Factor loads use ACI:

$A := b \cdot h$	$A = 94.488 \cdot in^2$	Area of invert per foot of tunnel
$D_{gr} := \rho_{ground} \cdot A$	$D_{gr} = 93737.79642 \cdot lb$	Axial dead load due to ground support
$T := \rho_{thermal} \cdot A$	$T = 205565.25816 \cdot lb$	Axial thermal load due to ground support
$P_{ugr} := (1.4 \cdot T + 1.4 \cdot D_{gr})$	$P_{ugr} = 419024.27641 \cdot lb$	Factored axial load, ACI, 9.2.7 equation 9-6
$M_{ugr} := (1.4 \cdot M_{ground})$	$M_{ugr} = 5130.14138 \cdot lb \cdot ft$	Factored moment due to ground support ACI, 9.2:7 equation 9-5

Load Case 2 (During Earthquake: WP's in Place), Attachment II-2, Table II-2, Use:

$P_{thrustEq} := 2312 \cdot \frac{KN}{m} \cdot ft$ $P_{thrustEq} = 158422.35769 \cdot lb$ Thrust load per foot of invert due to ground support,

$\rho_{groundEq} := \frac{P_{thrustEq}}{b \cdot h}$ $\rho_{groundEq} = 1676.63997 \cdot psi$ Hoop compressive stress due to ground loading

$M_{groundEq} := \frac{18.7 \cdot KN \cdot m}{m} \cdot 1 \cdot ft$ $M_{groundEq} = 50447.1396 \cdot lb \cdot in$ Moment per foot of invert due to ground loading

Factor loads use ACI:

$E := \rho_{groundEq} \cdot A$ $E = 158422.35769 \cdot lb$ Axial dead load due to ground support

$T := \rho_{thermal} \cdot A$ $T = 205565.25816 \cdot lb$ Axial thermal load due to ground support

$P_{uEq} := .75 \cdot (1.4 \cdot T + 1.7 \cdot 1.1 \cdot E)$ $P_{uEq} = 438030.87773 \cdot lb$ Factored axial load, ACI, 9.2.2 equation 9-2

$M_{uEq} := .75 \cdot \left(1.7 \cdot 1.1 \cdot M_{groundEq}\right)$ $M_{uEq} = 5896.00944 \cdot lb \cdot ft$ Factored moment due to ground support ACI, 9.2.2 equation 9-2

Compare Load Cases 1&2:

$P_{uEq} = 438030.87773 \cdot lb >$ $P_{ugr} = 419024.27641 \cdot lb$

$M_{uEq} = 5896.00944 \cdot lb \cdot ft >$ $M_{ugr} = 5130.14138 \cdot lb \cdot ft$

Conclusion : Load Case 2 controls Invert Design. Set PuEq and MuEq equal to Pu and Mu respectively.

$P_u := P_{uEq}$

$M_u := M_{uEq}$

Design Invert section for: $P_u = 438030.87773 \cdot lb$ $M_u = 5896.00944 \cdot lb \cdot ft$

Find eccentricity: $e := \frac{M_u}{P_u}$ $e = 0.16152 \cdot in$

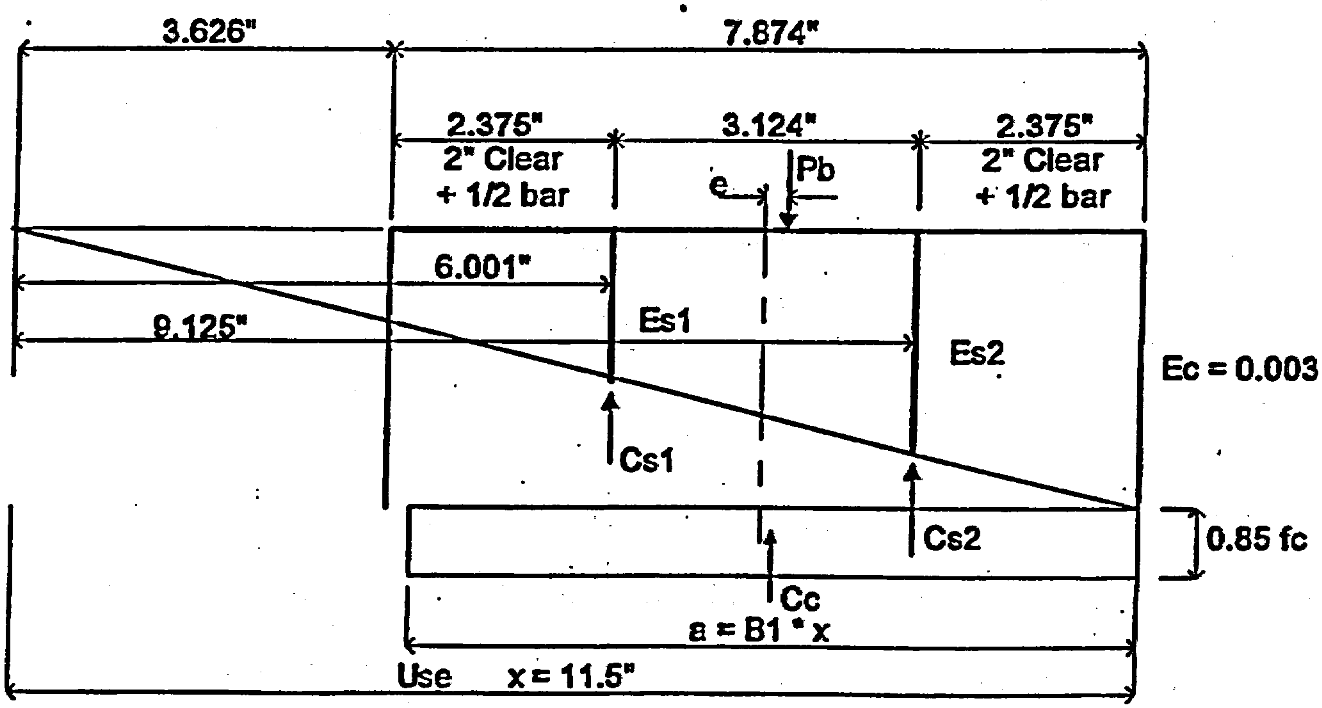
Try reinforcing with #6 @ 8" o.c. each face. Try:

$f_c := 9000 \cdot psi$	Trial concrete compressive strength
$A_{st} := 1.32 \cdot in^2$	Area of steel per foot.
$A_g := b \cdot h$	Gross area of concrete
$\phi := 0.7$	Strength reduction factor, ACI 9.3.2.2.b, Other reinforced members.
$\phi P_{nmax} := 0.8 \cdot \phi \cdot \left[0.85 \cdot f_c \cdot \left(A_g - A_{st}\right) + f_y \cdot A_{st}\right]$	ACI 10.3.5.2, Equation (10-2)

$\phi P_{nmax} = 443483.712 \cdot lb$

Check section for eccentricity equal to approximately 0.16 inches. Use trial strain diagram to develop axial and bending loads with eccentricity greater than or equal to e = 0.16" and compare design axial strength with required axial strength.

$f_y := 60000 \cdot psi$	Yield stress of reinforcing
$E := 29000000 \cdot psi$	Modulus of elasticity of reinforcing
$A_s := .66 \cdot in^2$	Area of reinforcing per face per foot (#6 @ 8" o.c.)



$\beta_1 := 0.65$ — Factor for calculating the length of the equivalent stress block,ACI, 10.2.7.3

Try: $x := 11.5 \cdot in$ — Length of triangle base for the above trial strain diagram. Purpose: To approximate 0.16" eccentricity

$b := 12 \cdot in$ — Width of column strip

$\xi_{s1} := .003 \cdot \frac{6.001}{11.5}$ $\xi_{s1} = 0.00157$ — Strain in reinforcing located at Cs1

$\xi_{s2} := .003 \cdot \frac{9.125}{11.5}$ $\xi_{s2} = 0.00238$ $> e_y := \frac{f_y}{E}$ $e_y = 0.00207$ — Strain in reinforcing located at Cs2. Compression steel has yielded, use fy = 60 ksi, ACI, section 10.2.4

$C_{s1} := A_s \cdot \frac{\xi_{s1}}{\left(\frac{f_y}{E}\right)} \cdot \left(f_y - .85 \cdot f_c\right)$ $C_{s1} = 26142.93904 \cdot lb$ — Force developed in compression steel Cs1

$C_{s2} := A_s \cdot \left(f_y - .85 \cdot f_c\right)$ $C_{s2} = 34551 \cdot lb$ — Force developed in compression steel Cs2

$C_c := 0.85 \cdot f_c \cdot \beta_1 \cdot x \cdot b$ $C_c = 686205 \cdot lb$ — Force developed by concrete

$P_b := C_{s1} + C_{s2} + C_c$ $P_b = 746898.93904 \cdot lb$ — Total force developed by liner cross section per foot

Calculate corresponding eccentricity associated with Pb. Sum moment about centerline of invert cross section:

$$e := \frac{-C_{s1} \cdot 1.562 \cdot in + C_{s2} \cdot 1.562 \cdot in + C_c \cdot \left(\frac{7.874 \cdot in}{2} - \frac{\beta_1 \cdot x}{2}\right)}{P_b}$$

Eccentricty corresponding to maximum axial load Pb

$e = 0.2 \cdot in$ $>$ $e = 0.16$ in. ok Use: $e := 0.2 \cdot in$

$\phi P_n := \phi \cdot P_b$ Ref. 5.23, Exp 13.19.1, page 433 $\phi P_n = 522829.25733 \cdot lb$ Column design strength with $e = 0.2$ in.

However, $\phi P_n > \phi P_{nmax}$, Use ϕP_{nmax} (ACI, 10.3.5.2) with e = 0.2 in. eccentricity for design axial load and calculate design moment, ACI ,10.3.5.2, Eq. 10-2. Requires Ties per ACI, 7.10.5 .

$\phi P_n := \phi P_{nmax}$ $\phi P_n = 443483.712 \cdot lb > P_u = 438030.87773 \cdot lb$ Design axial load greater than Required axial load

$\phi M_n := \phi P_n \cdot e$ Ref. 5.23, Exp 13.19.1, page 433

$\phi M_n = 7391.4 \cdot lb \cdot ft$ $> M_u = 5896.00944 \cdot lb \cdot ft$ Design Moment greater than Required Moment.

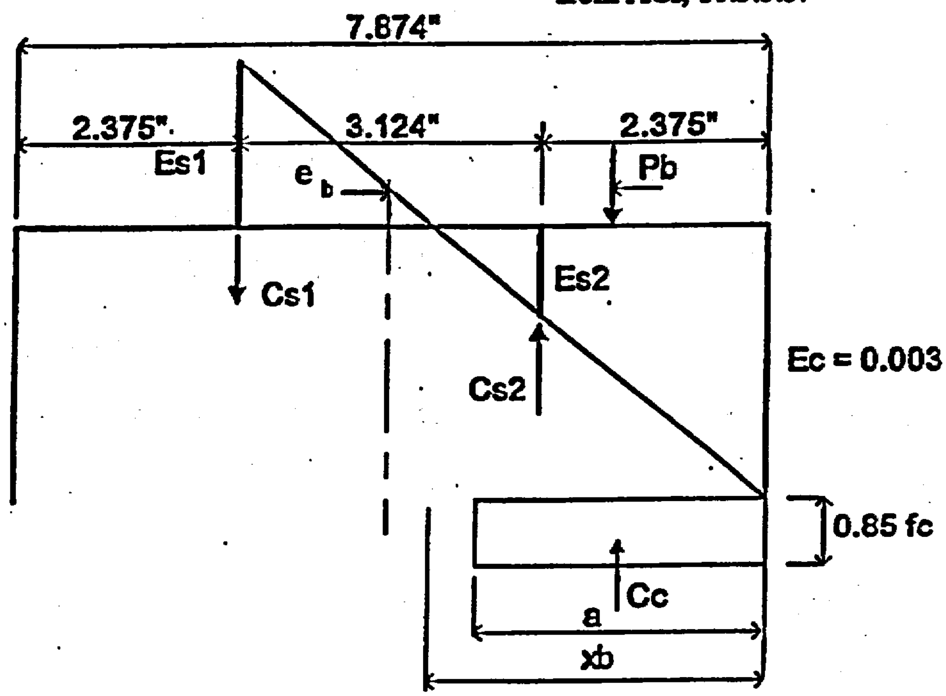
Cross section adequate for maximum axial load and moment.

Plot interaction diagram, calculate $f_c = 9000 \cdot psi$ $f_y = 60000 \cdot psi$

$$P_o := \frac{\phi P_{nmax}}{\phi \cdot 0.8}$$

$P_o = 791935.2 \cdot lb$ Nominal Axial load strength with zero eccentricity, ACI, 10.3.5.2, Eq. 10-2. divided by strength reduction factor and 0.8 factor from ACI, 10.3.5.3.

Calculate balanced condition (Po & Mo):



$$\xi_{s1} := \frac{f_y}{E}$$

$\xi_{s1} = 0.00207$ Strain in tension steel, fy = 60000 psi

$\xi_c := 0.003$ Strain in concrete at extreme fiber

$$x_b := 5.499 \cdot in \cdot \left(\frac{\xi_c}{\xi_{s1} + \xi_c} \right)$$

$x_b = 3.25451 \cdot in$ Distance to nuetral axis, see diagram

$\xi_{s2} := \xi_c \cdot \left(\frac{x_b - 2.375 \cdot in}{x_b} \right)$ $\xi_{s2} = 0.00081$ strain in compression steel

$f_{s2} := 60000 \cdot psi \cdot \frac{\xi_{s2}}{\xi_{s1}}$ $f_{s2} = 23511.18385 \cdot psi$ Stress in compression steel

$C_{s1} := 60000 \cdot psi \cdot A_s$ $C_{s1} = 39600 \cdot lb$ Force in tension steel

$C_{s2} := f_{s2} \cdot A_s$ $C_{s2} = 15517.38134 \cdot lb$ Force in compression steel

$C_c := .85 \cdot f_c \cdot \beta_1 \cdot x_b \cdot b - 0.85 \cdot f_c \cdot A_s$ $C_c = 189147.62388 \cdot lb$ Force in concrete

$P_b := -C_{s1} + C_{s2} + C_c$ $P_b = 165065.00522 \cdot lb$ Axial load at balance condition

Determine moment at balance condition by summing moments about plastic centroid:

$$M_b := C_{s1} \cdot 1.562 \cdot in + C_{s2} \cdot 1.562 \cdot in + C_c \cdot \left(\frac{7.874 \cdot in}{2} - \frac{x_b \cdot \beta_1}{2} \right)$$

$M_b = 52558.59262 \cdot lb \cdot ft$ Moment at balance condition

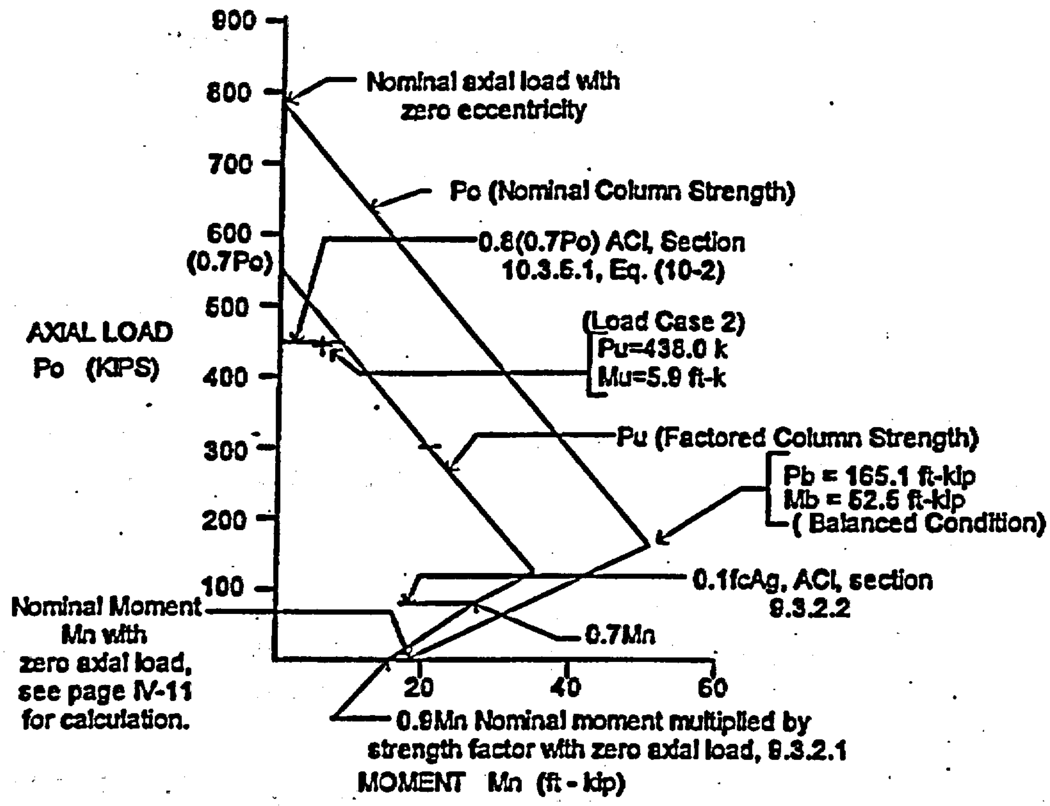
Determine moment with zero axial load, neglect compression steel:

$C_{s1} = 39600 \cdot lb$ $d := 5.499 \cdot in$

$a := \frac{C_{s1}}{.85 \cdot f_c \cdot b}$ $a = 0.43137 \cdot in$ Depth of stress block, ACI, section 10.2.7

$M_n := C_{s1} \cdot \left(d - \frac{a}{2} \right)$ $M_n = 17434.93529 \cdot ft \cdot lb$ Moment with zero axial load

Draw interaction diagram, Using Axial loads and Moments calculated above. Sketch performed using MathCad Drawing program :



CONCRETE COLUMN INTERACTION DIAGRAM

Conclusion: Comparison of Table II-2 loading scenario:

> During Earthquake: WP's in Place + Thermal

with concrete interaction diagram. Indicates 200 mm minimum invert thickness with longitudinal bars #6 @ 8" each face and fc = 9000 psi is adequate to support Table II-2 loading.

Using ACI, Structural Plain Concrete, chapter 22, design invert thickness and concrete strength
Try 16" minimum thickness of invert at section 2-2, see sketch on page to of calculations:
By inspection and calculation of table II-2, Load case 2 controls design of Invert.
Load Case 2: During Earthquake: WP's in place, see calculation above for factored load calculation.

$P_u = 438030.87773 \cdot lb$ Factored axial load due to load case 2

$M_u = 5896.00944 \cdot lb \cdot ft$ Factored moment due to load case 2

$b := 12 \cdot in$ $h := 16 \cdot in$ $\phi := 0.65$ $l_c := 0 \cdot in$

Try: $f_c := 10500 \cdot psi$

$A_1 := b \cdot h$ $A_1 = 192 \cdot in^2$ Area at section 2-2 per foot

$S_1 := \frac{b \cdot h^2}{6}$ $S_1 = 512 \cdot in^3$ Section modulus at section 2-2 per foot

$\phi P_{n1} := \phi \cdot 0.6 \cdot f_c \cdot \left[1 - \left(\frac{l_c}{32 \cdot h_1}\right)^2\right] \cdot A_1$ $\phi P_{n1} = 786240 \cdot lb$ Allowable factor axial load at section 2-2, ACI, equation 22-4.

$\phi M_{n1} := \phi \cdot 5 \cdot S_1 \cdot \sqrt{\frac{f_c}{\left(\frac{lb}{in^2}\right)}} \cdot \frac{lb}{in^2}$ $\phi M_{n1} = 14209.10506 \cdot ft \cdot lb$ Allowable factor moment at section 2-2, ACI, equation 22-2,

Check combined flexure and compression on the compression face at section 2-2, ACI section 22.5.3:

$C_{1c} := \frac{P_u}{\phi P_{n1}} + \frac{M_u}{\phi M_{n1}}$ $C_{1c} = 0.97$ < 1.0 ok ACI, equation 22-5

Check combined flexure and compression on the tension face at section 2-2, ACI section 22.5.3:

$f_{ct} := 5 \cdot \phi \cdot \sqrt{\frac{f_c}{\left(\frac{lb}{in^2}\right)}} \cdot \frac{lb}{in^2}$ $f_{ct} = 333.0259 \cdot \frac{lb}{in^2}$ Allowable tensile stress

$C_{1t} := \frac{M_u}{S_1} - \frac{P_u}{A_1}$ $C_{1t} = -2143.2231 \cdot \frac{lb}{in^2} < f_{ct} = 333.0259 \cdot \frac{lb}{in^2}$ ACI, equation 22-6
Ok, All compression no tension

Conclusion: 16" minimum invert thickness with fc = 10500 psi is adequate.

Design gantry rail connection to concrete using scaled up, Staad III, file Gantry-H, Ref 5.18., support reactions:

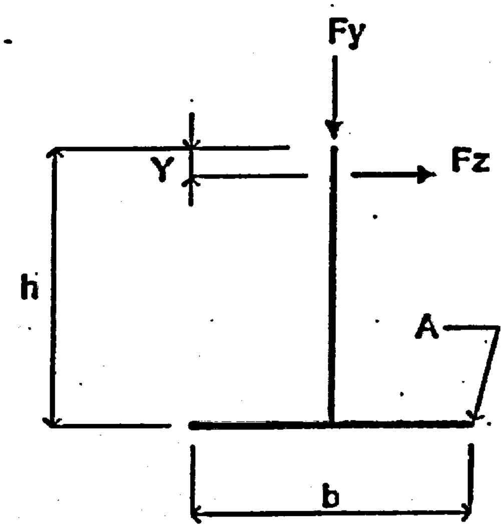
$SF := 1.272$ Scale factor calculated in Attachment I

Controlling Load Cases from , Ref 5.18, Joint 118; scaled up and divided by 2 wheels

Load Case 102 $F_{x102} := SF \cdot \left(\frac{-38740}{2} \cdot lb\right)$ $F_{y102} := SF \cdot \frac{145740}{2} \cdot lb$ $F_{z102} := SF \cdot \frac{34140}{2} \cdot lb$

Load Case 105 $F_{x105} := SF \cdot \left(-\frac{19140}{2} \cdot lb\right)$ $F_{y105} := SF \cdot \frac{92460}{2} \cdot lb$ $F_{z105} := SF \cdot \frac{24050}{2} \cdot lb$

Determine overturning stablility of 135 lb crane rail for the above load cases, (see Attachment V for rail geometry and design).



$b := 5.1875 \cdot in$

$h := 5.75 \cdot in$

$Y := .787 \cdot in$ (20 mm), (Attachment V, page V-2)

Gantry Rail Elevation

Load Case 102

Sum Resisting Moment about point A, see Gantry Rail Elevation.

$M_r := F_{y102} \cdot \frac{b}{2}$ Resisting Moment

Sum overturning Moment about bottom of rail. see Gantry Rail Elevation.

$M_o := F_{z102} \cdot (h - Y)$ Overturning Moment

$FSO := \frac{M_r}{M_o}$ $FSO = 2.231$ Factor of safety overturning

Load Case 105

Sum Resisting Moment about point A, see Gantry Rail Elevation.

$M_r := F_{y105} \cdot \frac{b}{2}$ Resisting Moment

Sum overturning Moment about bottom of rail. see Gantry Rail Elevation.

$M_o := F_{z105} \cdot (h - Y)$ Overturning Moment

$FSO := \frac{M_r}{M_o}$ $FSO = 2.0092$ Factor of safety overturning

Conclusion: No uplift, design rail connection for shear only, using Load Case 102.

$C := 1.3$

$V_u := C \cdot 1.7 \cdot F_{z102}$ $V_u = 47985.8184 \cdot lb$ Factor shear, ACI, equation 9-1 and UBC 1923.2, C=1.3 special inspection factor. Provide special inspection for anchor bolt installation.

Design anchor bolts UBC-97, section 1923.3.3

$\phi := 0.65$ Strength reduction factor

$f_c := 9000 \cdot psi$ Concrete compressive strength from reinforced concrete/ Precast design.

$f_{ut} := 60000 \cdot psi$ Minimum anchor bolt tensile strength, Use ASTM A307, AISC, Table, page 4-4

Try 2 - 1" diameter bolts:

$d := 1.0 \cdot in$ Diameter of anchor bolt

$A_b := \frac{\pi \cdot d^2}{4}$ $A_b = 0.785398163 \cdot in^2$ Gross Area of anchor bolt, Used for concrete design.

$A_t := 0.606 \cdot in^2$ Tensile area for tension and shear- threads included in shear plane, AISC, page 4-147.

$v_u := \frac{V_u}{2}$ $v_u = 23992.9092 \cdot lb$ Required anchor bolt shear strength per anchor bolt. Using 2 anchor bolts

Steel design strength:

$V_{ss} := 0.75 \cdot A_t \cdot f_{ut}$ $V_{ss} = 27270 \cdot lb$ Shear strength for steel per anchor

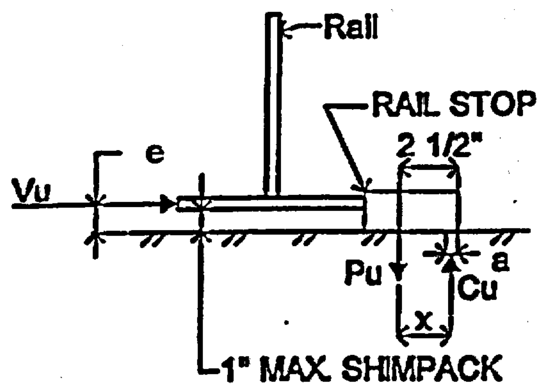
Concrete design strength:

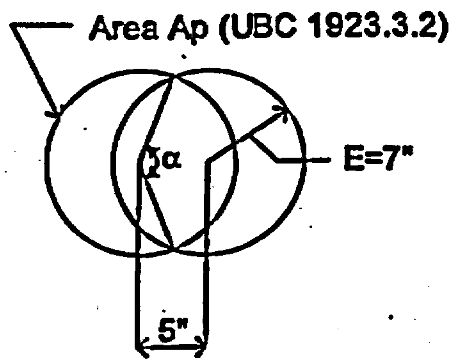
$\phi V_c := \phi \cdot 800 \cdot A_b \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $\phi V_c = 38744.89 \cdot lb$ Shear strength for concrete per anchor

Determine Anchor bolt tension with 1" maximum shim pack thickne:Try $a := .4375 \cdot in$

$e := 1.125 \cdot in$ Distance between T.O.C. & centerline of load

$x := 2.5 \cdot in - \frac{a}{2}$ Trial distance between force resultants, a = 7/16 ", check "a" at end of calculation for Pu.





$P_u := \frac{v_u \cdot e}{x}$ $P_u = 11832.11961 \cdot lb$ Required pullout strength per anchor

$b := 4 \cdot in$ effective width of base plate = 1/2 base plate length. L = 8".

$a := \frac{P_u}{.9 \cdot (0.85 \cdot f_c \cdot b)}$ $a = 0.42963 \cdot in <$ $a = .4375"$ ok Ref. 5.23, section 3.3, page 38 and 39.

Find steel tensile strength of anchor, UBC, Section 1923.3.2

$P_{ss} := 0.9 \cdot A_t \cdot f_{ut}$ $P_{ss} = 32724 \cdot lb$ Steel tensile strength per anchor

Design concrete tensile strength of anchor

$E := 7 \cdot in$ Embedment depth of anchor

$s := 5 \cdot in$ Anchor spacing

Projected Area of cone on surface of concrete, UBC, Sect. 1923.3.2 for two anchor spaced 5" with 7" embedment, see plan sketch above..

$\alpha := 2 \cdot acos\left(\frac{1}{2} \cdot \frac{s}{E}\right)$ $\alpha = 2.41118 radians$

$A_p := 2 \cdot \pi \cdot E^2 - (\alpha - \sin(\alpha)) \cdot E^2$ $A_p = 222.42009 \cdot in^2$ Projected area of anchor at suface of concrete

$\phi P_c := \phi \cdot 4 \cdot A_p \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $\phi P_c = 54861.61833 \cdot lb$ Concrete design tension stength

Check combined tension and shear, UBC-97, section 1923.3.4

Check steel combined tension and shear:

$$C_s := \left(\frac{P_u}{P_{ss}}\right)^2 + \left(\frac{v_u}{V_{ss}}\right)^2 \qquad C_s = 0.90483 \quad < 1.0 \text{ ok}$$

Check concrete combined tension and shear:

$$C_c := \frac{1}{\phi}\cdot\left[\left[\frac{2\cdot P_u}{\left(\frac{\phi P_c}{\phi}\right)}\right]^{\frac{5}{3}} + \left[\frac{v_u}{\left(\frac{\phi V_c}{\phi}\right)}\right]^{\frac{5}{3}}\right] \qquad C_c = 0.52237 \quad < 1.0 \text{ ok}$$

Check shear and tensile strength individually:

$$c_v := \frac{v_u}{\phi V_c} \qquad c_v = 0.61925 < 1.0 \text{ ok}$$

$$p_u := \frac{2\cdot P_u}{\phi P_c} \qquad p_u = 0.43134 < 1.0 \text{ ok}$$

Minimum Anchor bolt edge distance, UBC 1923.3.3

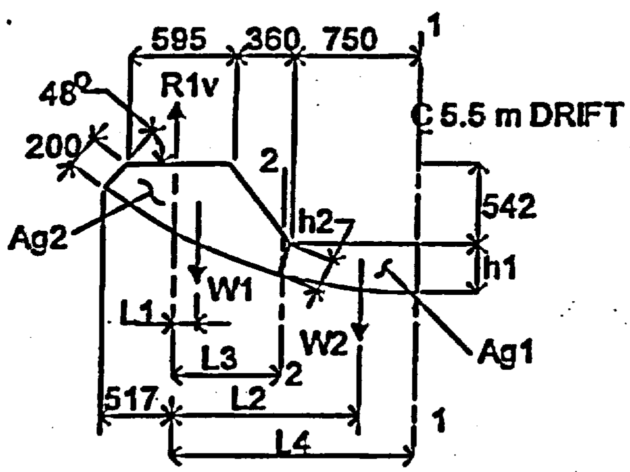
$d_{away} := 4\cdot d$ $\quad d_{away} = 4\cdot in$ $\quad$ Minimum anchor bolt edge distance = 4xd

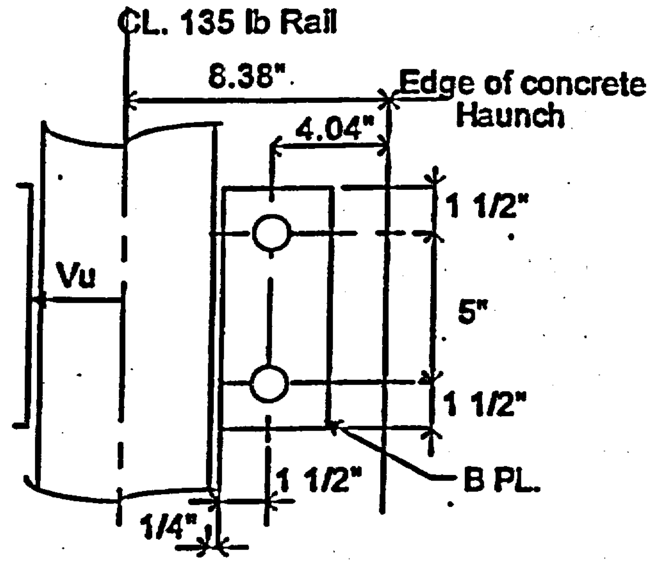
$d_{toward} := 10\cdot d$ $\quad d_{toward} = 10\cdot in$ $\quad$ Minimum edge distance for loading toward a free edge. Inorder to use full design load.

Inspection of Figure II-3 and Base plate sketch page IV- 18, Precast invert shows edge distances are adequate.

Conclusion: Use 2 - 1" diameter anchors, spaced 5" o.c., embedment depth = 7" and f'c = 9000 psi

Design temperature steel for concrete invert:





Precast Concrete Invert symetrical about center line of tunnel

Data:

$k \equiv 1000 \cdot lb$ $\quad psi \equiv \frac{lb}{in^2}$ $\quad f_c := 9000 \cdot \frac{lb}{in^2}$ Concrete compressive strength @ 28 days

$L_1 := 2.82 \cdot in$ (71.7 mm) $h_1 := 12.32 \cdot in$ (313 mm) $L_2 := 37.77 \cdot in$ (959.3 mm) $b := 12 \cdot in$ (304.8 mm)

$L_3 := 21.4 \cdot in$ (543.6 mm) $h_2 := 7.874 \cdot in$ (200 mm) $L_4 := 52.09 \cdot in$ (1323 mm)

Invert temperature steel parallel to tunnel:

$\rho_{temp} := 0.0018$ Temperture steel ACI, 7.12

$A_{g1} := \frac{V_2}{1 \cdot ft}$ $\quad A_{g1} = 332.64 \cdot in^2$ Gross concrete area per foot of tunnel between section 1-1 and 2-2

$As_{temp} := \rho_{temp} \cdot A_{g1}$ $\quad As_{temp} = 0.5988 \cdot in^2$ Tempature steel ACI, 7.12

Try #3 each face

$A_{s3} := 0.11 \cdot in^2$

Find quanity of #3 bars:

$Q := \frac{As_{temp}}{A_{s3}}$ $\quad Q = 5.4432$ #3 bars

Use 3 #3 top and bottom equally spaced in area between section 1-1 and 2-2.

Invert temperature steel parallel to tunnel, continued:

$\rho_{temp} := 0.0018$ — Temperture steel ACI, 7.12

$A_{g2} := \dfrac{V_1}{1 \cdot ft}$ $\quad A_{g2} = 675.36 \cdot in^2$ — Gross concrete area per foot of tunnel to the left of section 2-2.

$As_{temp} := \rho_{temp} \cdot A_{g2}$ $\quad As_{temp} = 1.216 \cdot in^2$ — Tempature steel ACI, 7.12

Try #3 each face

$A_{s3} := 0.11 \cdot in^2$

Find quanity of #3 bars:

$Q := \dfrac{As_{temp}}{A_{s3}}$ $\quad Q = 11.05$ — Use 12 #3 spaced in 3 layers equally spaced in area to the left of section 2-2.

Determine area of temperature steel perpendicular to centerline of tunnel located in haunch at inside face:

$b := 12 \cdot in$

$h := 12.89 \cdot in$ — Maximum depth of concrete in haunch above 200 mm liner thickness.

$A_{gh} := b \cdot h$ $\quad A_{gh} = 154.68 \cdot in^2$ — Area of concrete per foot at maximum depth in haunch above 200 mm liner thickness.

$As_{temp} := 0.0018 \cdot \dfrac{A_{gh}}{2}$ $\quad As_{temp} = 0.13921 \cdot in^2$ — Area of temperature steel at inside face of haunch.

$s := \left(\dfrac{A_{s3}}{As_{temp}}\right) \cdot 12 \cdot in$ $\quad s = 9.48194 \cdot in$ — Use #3@9" o.c. inside face, area of steel provided = .15 sqin per foot.

Determine Column Tie Reinforcement to qualify Invert as a reinforced compression member. Use ACI, 7.10.5.1

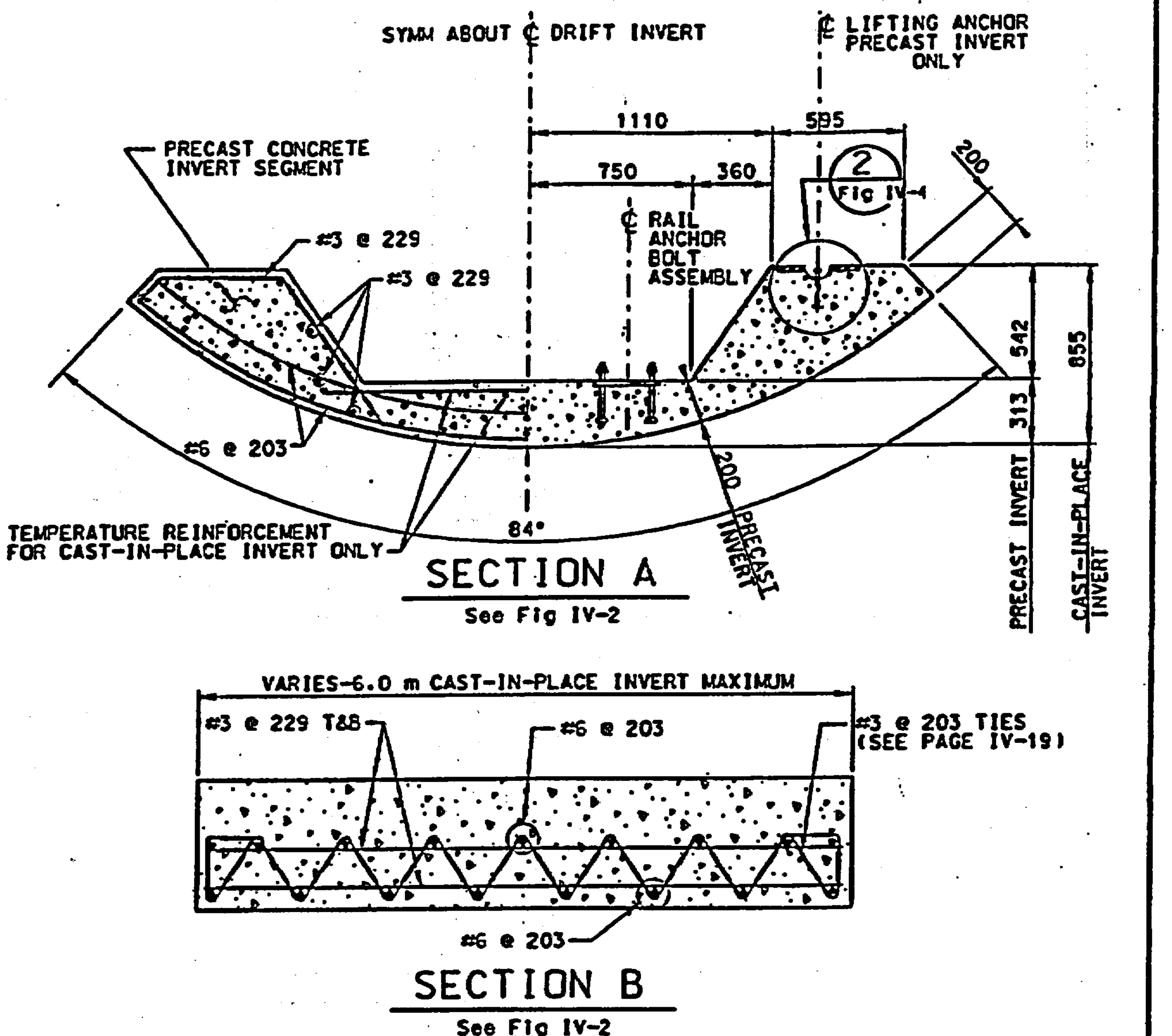
Space Lateral tie members less than or equal to : s<= 16 longitudinal bar dia. = 0.75*16= 12"
48 Tie bar dia.= .375 x 48 = 18"
least dimension of compression member = 8"

Use : #3 ties @ 8" o.c.

CALCULATION SUMMARY: Recommend using reinforced concrete invert:

f'c = 9000 psi
Longitudinal bar: #6 @ 8" each face
Compression ties #3 @ 8" on center
Tempature Reinforcing #3 see sketch page IV-20 for spacing

Plain concrete requires a minimum invert thickness equal to 16 inches which may interfer with TBM and disposal clearance requirements.



NOTES:

1. ALL CAST-IN-PLACE CONCRETE AND ALL CONCRETE FOR PRECAST INVERTS SHALL BE IN ACCORDANCE WITH:
   ACI 301, ACI 318, ACI 305R, ACI 306R, ACI 117R, AND ACI 211.1.
   ASTM C31, ASTM C33, ASTM C39, ASTM C94, ASTM C109, ASTM 138, ASTM 143, ASTM 150,
   ASTM 171, ASTM 172, ASTM 173, ASTM 231, ASTM C260, ASTM C309, ASTM C494,
   ASTM C1064, ASTM C1107, AND ASTM D75.
2. PRECAST CONCRETE SHALL BE IN ACCORDANCE WITH PCI MNL116.
3. CONCRETE REINFORCEMENT SHALL BE IN ACORDANCE WITH:
   ACI 301 AND ACI 318.
   ASTM A510 AND ASTM A615.
   CRSI-DA4.
4. CONCRETE SHALL HAVE A COMPRESSIVE STRENGTH OF 62.1 MPa.

FIGURE IV-1
CONCRETE REINFORCEMENT - PRECAST OR CAST-IN-PLACE INVERT SECTION

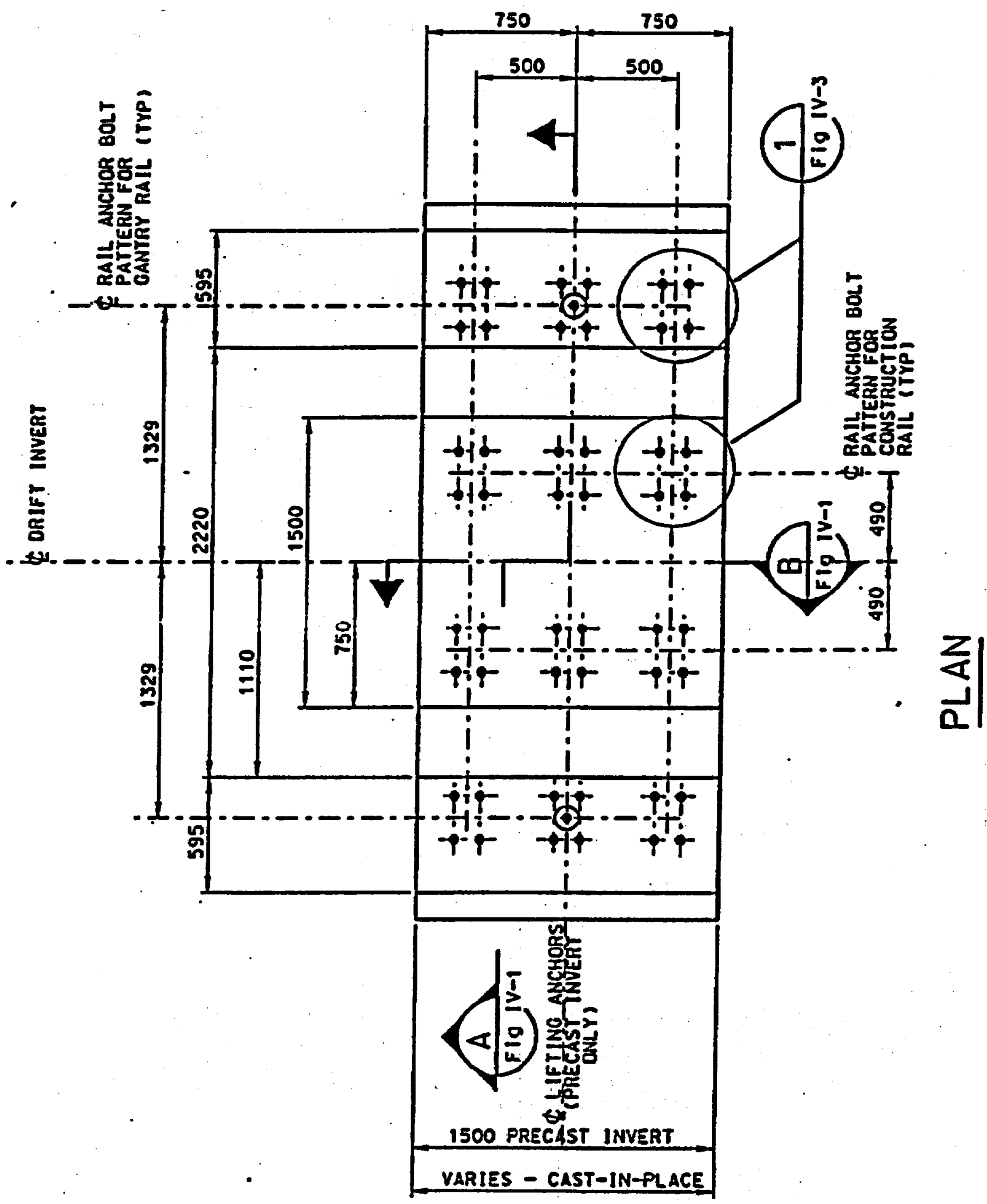
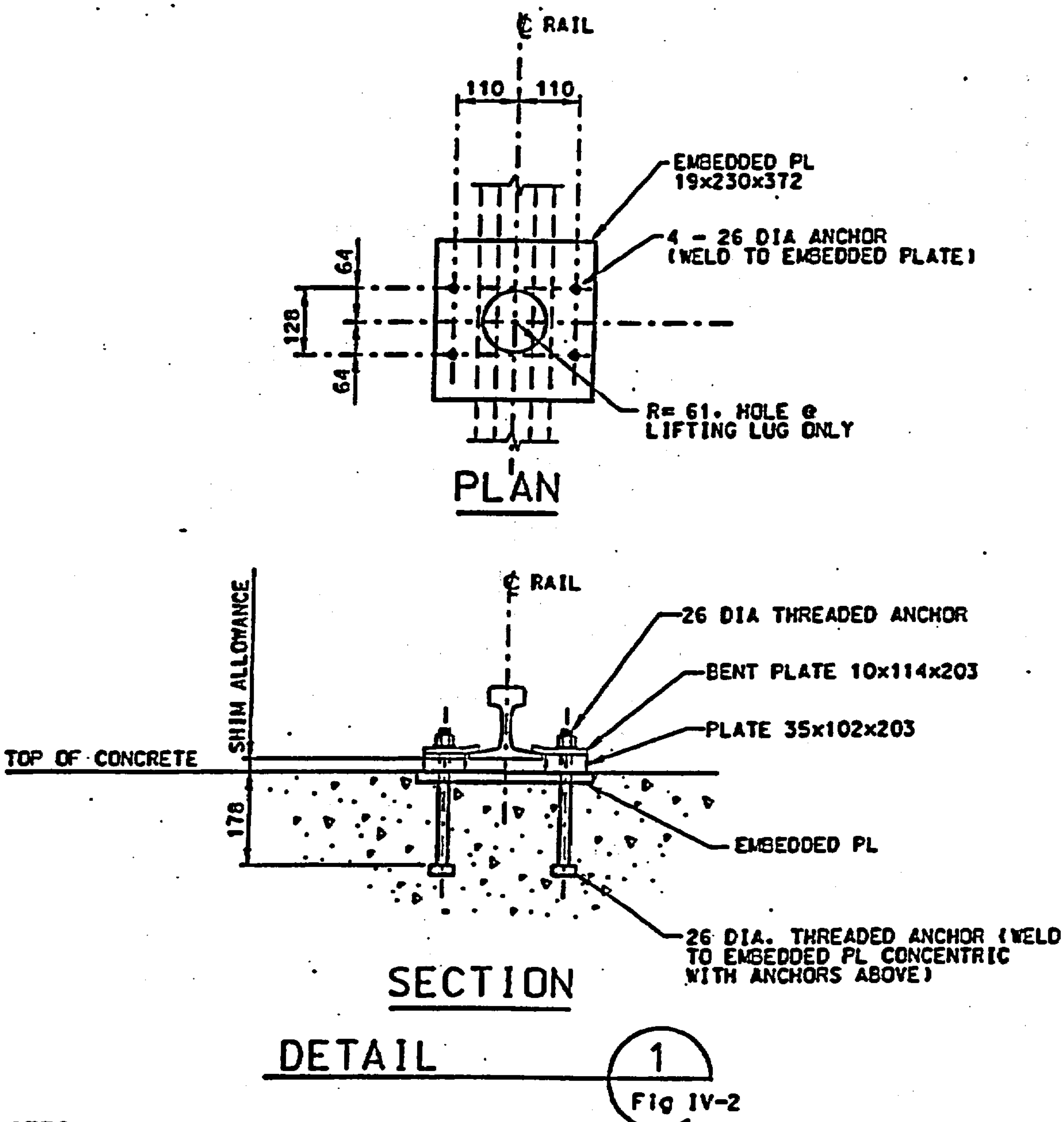


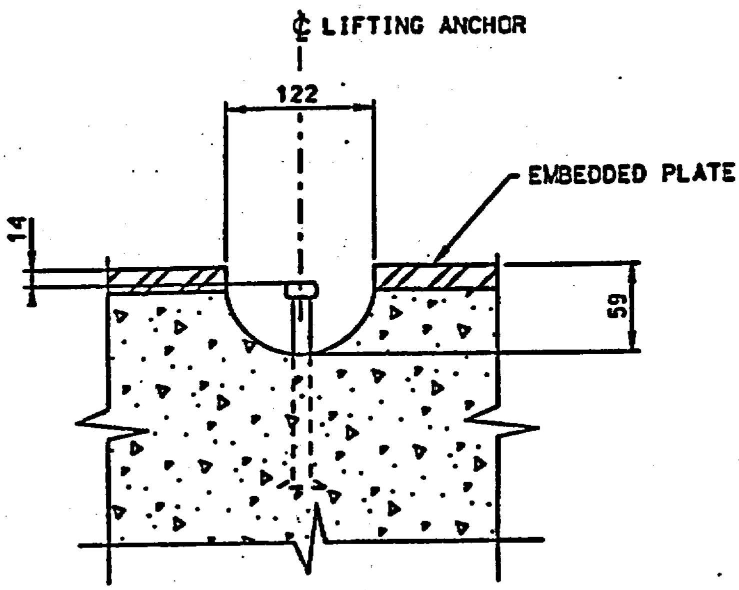
FIGURE IV-2
CAST-IN-PLACE OR PRECAST
CONCRETE INVERT PLAN



NOTES:

1. ALL STEEL ANCHORS SHALL BE IN ACCORDANCE WITH ASTM A307. MINIMUM.
2. ALL EXPOSED CONCRETE CORNERS SHALL BE CHAMFERED 19 mm (3/4").
3. ALL UNFORMED CONCRETE SURFACES SHALL RECEIVE A BROOM FINISH.
4. ALL STEEL SHAPES AND PLATES SHALL BE ASTM A36 IN ACCORDANCE WITH AISC M016.

FIGURE IV-3
CONCRETE INVERT DETAILS



DETAIL 2 Fig IV-1

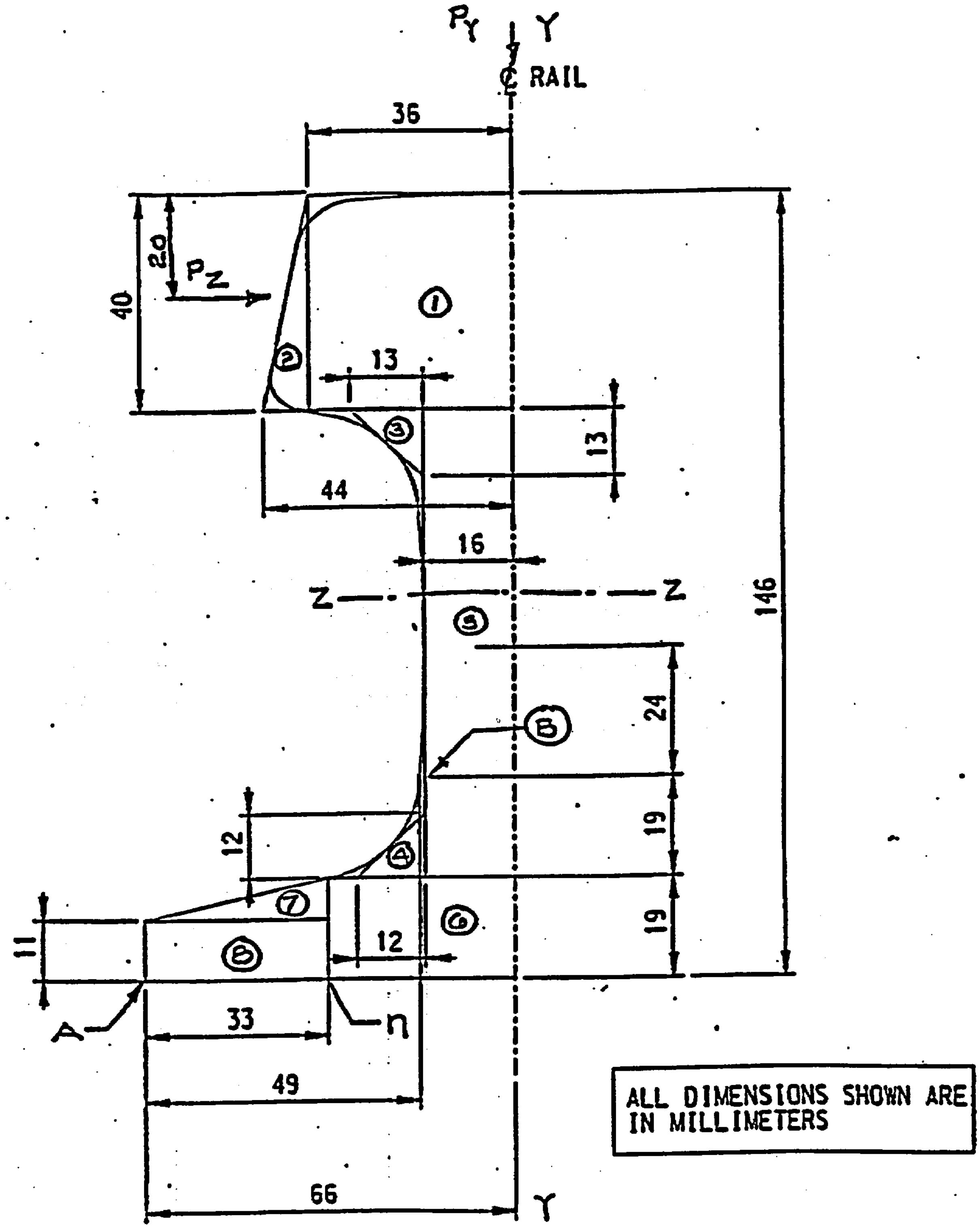
NOTE:

1. LIFTING ANCHORS TO BE "SWIFT LIFT SYSTEM" DAYTON SUPERIOR. TYPE P-52SL. STANDARD. RATED LOAD 8 TONS. SAFE WORKING LOAD 16000 LBS AND P-50SL UNIVERSAL LIFTING EYE. 8 TON RATED LOAD. OR APPROVED EQUAL. (REFERENCE 5.28)

FIGURE IV-4
CONCRETE INVERT DETAIL

ATTACHMENT V
GANTRY RAIL DESIGN

DOE policy requires the subsurface design be performed using metric units. Much source information (e.g., vendor data/steel member sizes) used for design, however, is available only in English units. Because of this, calculations are generally performed in English units. The results are converted to metric units in the main body of the analysis, followed by the corresponding English values in parenthesis.



.135 LB RAIL USED FOR CALCULATING SECTION PROPERTIES AND DISTANCES TO CRITICAL SECTION FOR FLEXURAL CALCULATIONS. ALL DIMENSION SHOWN ARE INTERNALLY GENERATED BY MICROSTATION 95 (AISC, Table, page 1-113)

Calculate: Section modulus of 135 lb rail. use attach sketch for analysis.

ITEM	Qty A (in^2)	Ad^2 (in^4)	I_y (in^4)
1	4.46	–	$\frac{1}{12}(40)(72)^3/25.4^4 = 2.59$
2	0.456	1.15	–
3	0.26	0.17	–
4	0.22	0.13	–
5	4.31	–	$\frac{1}{12}(3.42)(1.26)^3 = 0.57$
6	1.95	–	$\frac{1}{12}(0.75)(2.6)^3 = 1.09$
7	0.41	1.23	–
8	1.125	4.27	–
	13.23	6.95	4.65

≅ 13.2 (AISC, TABLE, pg 1-113) ok compares closely

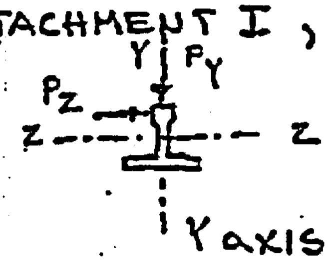
$I_{y\ TOTAL} = 6.95 + 4.65 = 11.60$ (MOMENT OF INERTIA ABOUT Y AXIS)

$S_Y = 11.60 / 2.55 = 4.48\ in^3$ (SECTION MODULUS ABOUT Y AXIS @ BOTT., PT A

$S_{Z\ base} = 18.1$ (AISC TABLE, PAGE 1-113
$S_{Z\ head} = 17.3$ STRONG AXIS SECTION MODULUS)

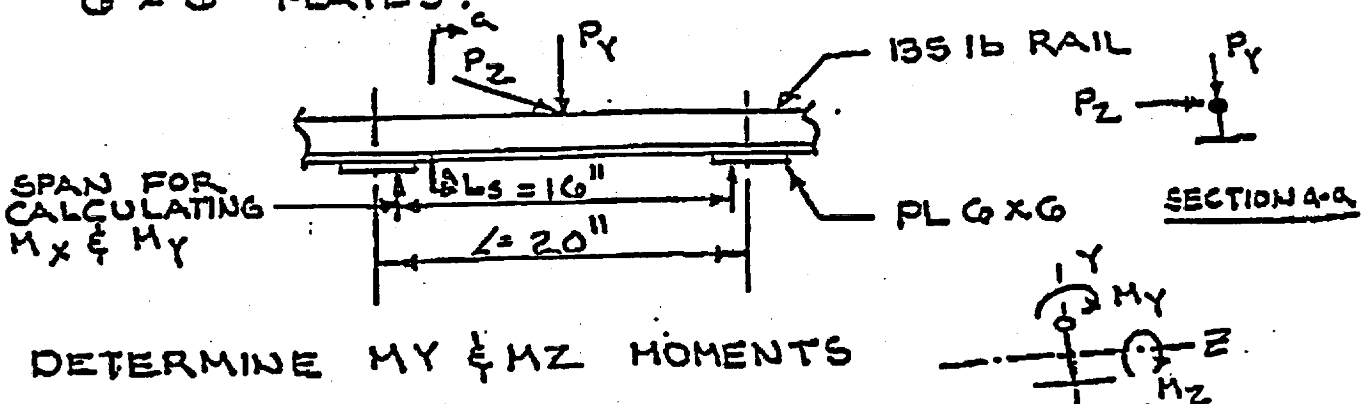
MAXIMUM WHEEL LOADS

$P_Y = 92{,}691$ lbs ↓ } ATTACHMENT I, Page I-28
$P_Z = 21{,}718$ lbs → }



Rail Section

TRY SUPPORTING RAIL @ 20" O.C WITH 6" x 6" PLATES.



DETERMINE MY & MZ MOMENTS

$M_Z = P_Y L_s / 4 = 92{,}691(16)/4 = 370{,}764$ in-lbs

$M_Y = P_Z L / 4 = 21{,}713(20)/4 = 108{,}565$ in-lbs

CALCULATE STRESS @ TOP FLANGE

$f_{bZ} = \frac{M_Z}{S_{Z\,Head}} = (370{,}764 / 17.3 / 1000) = 21.4$ ksi

$f_{bY} = \frac{M_Y}{S_Y} = (108{,}565 / 4.48 / 1000) = 24.2$ ksi

$f_{b\,TOTAL} = f_{bZ} + f_{bY} = 21.4 + 24.2 = 45.6$ ksi

RAIL MATERIAL PROPERTY (AREA, Chapter 4, PART 2 SPECIFICATIONS, TABLE 2-3, Page 4-2-10.

$F_Y = 70$ ksi (YIELD STRENGTH)

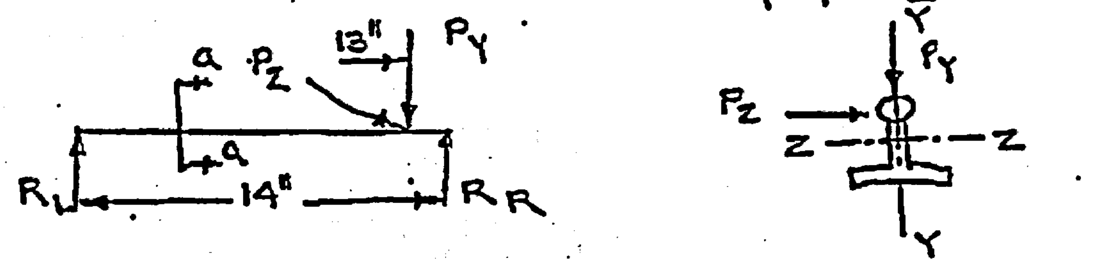
$F_T = 140$ ksi (TENSILE STRENGTH)

$F_b = 0.66\,F_Y$ ALLOWABLE BENDING STRESS AISC Eq. F1-1, Pg 5-45

$= 46.2$ ksi

$> f_{b\,TOTAL} = 45.6$ ksi <u>OK</u>

CHECK SHEAR SET LOAD 1" OFF EDGE OF BASE PL, FIND MAXIMUM V_Y & V_Z SHEAR



$V_{Y\,MAX} = P_Y\left(\frac{13}{14}\right) = 86{,}070$ lb

$V_{X\,MAX} = F_X\left(\frac{13}{14}\right) = 20{,}162$ lb

FOR V_Y SHEAR USE WEB TO RESIST VERTICAL SHEAR:

$t_{min} = 1.25$ in.

$h = 5.75$ in.

$A_w = 7.188\ in^2$

FOR V_X SHEAR USE HEAD OF RAIL OUTSIDE OF WEB TO RESIST SHEAR, SEE SKETCH, Pg 1 for ITEMS 1&2.

$A_{HEAD\ EFF.} = [\text{ITEM}①] + [\text{ITEM}②] - 1.25(1.57)$

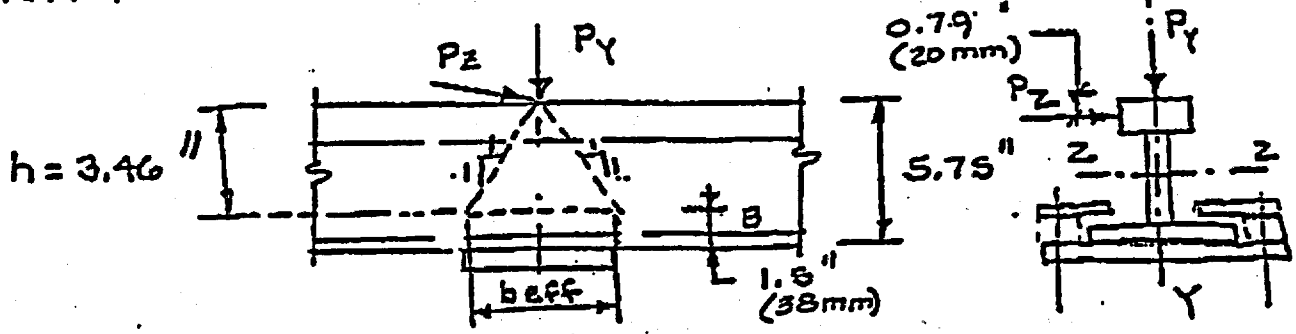
$= 4.46 + 0.496 - 1.96 = 3.0\ in^2$

$F_V = 0.4\,F_y = 0.4(70) = 28$ Ksi AISC EQ. F4-1 Pg. 5-49

$f_{v\,web} = 86.1/7.188 = 11.98$ Ksi WEB SHEAR $< F_V = 28$ Ksi OK

$f_{v\,Head} = 20.16/3.0 = 6.72$ Ksi HEAD SHEAR $< F_V = 28$ Ksi OK

CHECK WEB BENDING STRESS @ SUPPORT
WITH LOAD CENTERED ON BASE PLATE



ΣMOMENT @ TOE OF FILLET ON WEB @ ELEVATION B
(SEE SKETCH ABOVE AND SKETCH ON PAGE V-2 FOR LOCATION)

$M = P_z h_B = 21{,}713\,(5.75 - \overset{3.46}{1.5} - 0.79) = 75{,}127$ in-lb

$b_{eff} = 2\,(3.467) = 6.92$

AREA OF WEB EFFECTIVE TO RESIST AXIAL & BENDING, (SEE SKETCH FOR GEOMETRY, page V-2)

$t_w = (66-45)2/25.4 = 1.33$ in.

$b_{eff} = 2\,(3.46) = 6.92''$

$A_{eff} = 1.33(6.92) = 9.20\ in^2$

$S_{z\,eff} = \frac{1}{6}(6.92)(1.33)^2 = 2.04\ in^3$

$f_a = P_Y / A_{eff} = 92{,}691 / 9.22 = 10{,}053$ psi

$f_{b_z} = M / S_{x\,eff} = 75{,}127 / 2.04 = 36{,}826$ psi

$F_b = 0.75\,(F_Y) = 0.75(70) = 52.5$ ksi — AISC, EQ F2-1, Pg 5-48, used in interaction Eq H1-1 & H1-2

FIND ALLOWABLE AXIAL STRESS F_a

$r_y = \sqrt{\frac{I_y}{A}} = [(1/12(6.92)(1.33)^3) / 9.20]^{1/2} = 0.384$ in

$L = h_B = 5.75 - 1.5 - 0.75 = 3.46$ in.

$K = 2.1$ AISC TABLE C-C2.1, Pg 5-135

$\frac{KL}{r_y} = \frac{2.1(3.46)}{0.384} = 18.922$

$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \left(\frac{2 \cdot \pi^2 \cdot 25{,}000{,}000}{70{,}000}\right)^{1/2} = 90.43 > \frac{Kl}{r_y}$ (AISC, E2, Pg 5-42)

$F_a = \frac{\left[1 - \frac{18.922^2}{2(90.43)^2}\right] 70}{\frac{5}{3} + \frac{3(18.922)}{8(90.43)} - \frac{(18.922)^3}{8(90.43)^3}}$ (annotations: 0.978, 67.576, .0011) AISC, Equation E2-1, Pg 5-42

$= \frac{68.46}{1.66 + 0.0785 - 0.0011} = 39.431$ ksi

$\frac{f_a}{F_a} = \frac{10.053}{39.431} = 0.25 > 0.15$ use EQ. (H1-1 & H1-2)

$C_m = 1.0$ AISC H1.C, Pg 5-55

$F'_{e z} = \frac{12 \pi^2 E}{23(18.922)^2} = 417.078$ ksi (E = 29,000 ksi) AISC H1 Pg 5-54

CHECK AISC EQ H1-1, page 5-54

$$\frac{f_a}{F_a} + \frac{C_{mz} f_{bz}}{\left(1 - \frac{f_a}{F'_{ez}}\right) F_{bz}} \leq 1.0$$

$$0.25 + \frac{(1.0)(36.826)}{\left(1 - \frac{16.053}{417.078}\right)(.75)(70)} = 0.97 < 1.0 \text{ OK}$$

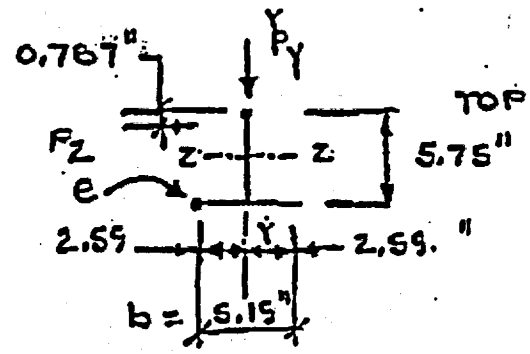
(.72 written above the second term)

CHECK AISC (EQ H1-2), page 5-54

$$\frac{f_a}{F_a} + \frac{f_{bz}}{F_{bz}} \leq 1.0$$

$$0.25 + \frac{36.826}{.75(70)} = 0.95 < 1.0 \text{ OK}$$

SEE NEXT PAGE FOR RAIL BOTTOM FLANGE CALCULATION.



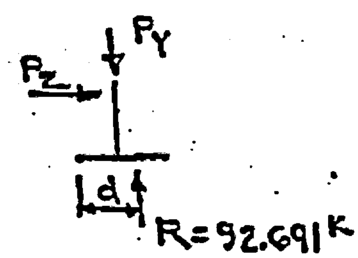
P_Y = 92.691 Kips
P_Z = 21.713 Kips

ΣM @ e (moment @ HEEL OF SECTION)

$M = P_Y(2.59) + P_Z(5.75 - 0.787)$

$= 92.691(2.59) + 21.713(4.963)$

$= 347.831$ Kip-in



DISTANCE OF RESULTANT FROM HEEL

$R = P_Y = 92.691$ kips

$d = \frac{M}{R} = \frac{347.831}{92.691} = 3.75''$

$e = d - \frac{b}{2} = 3.75 - 2.59 = 1.16 > \frac{b}{6} = 0.865$

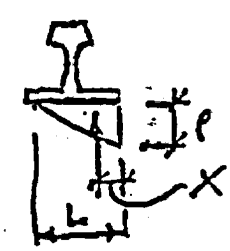
Resultant outside of Kern distance, (MIDDLE THIRD OF SECTION)

Determine pressure under base of Rail

$X = b - d = 1.44''$, $L = 3 \cdot X = 3(1.44) = 4.32''$

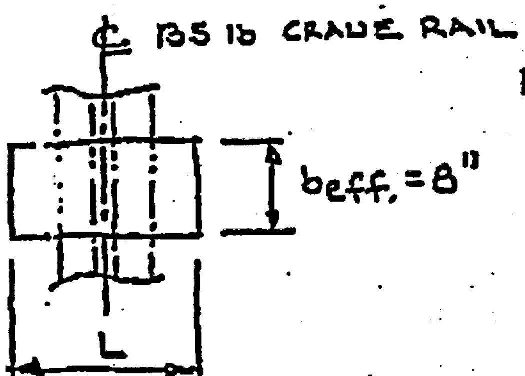
$R = \frac{1}{2} \rho L$, $R = 92.691^k$

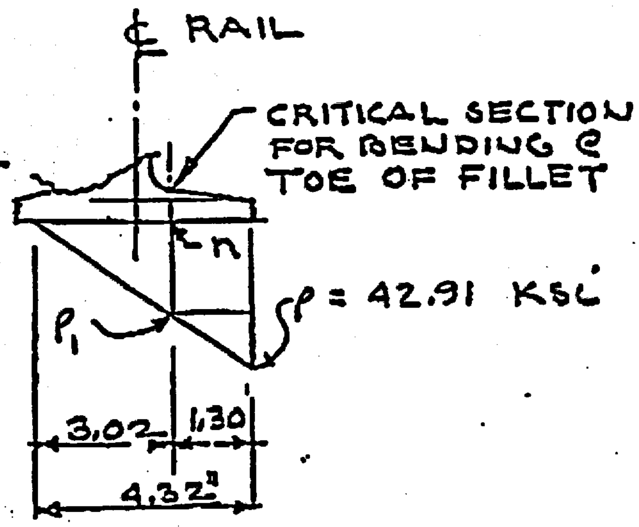
$\rho = \frac{2R}{L} = \frac{92.691(2)}{4.32} = 42.91$ ksi



Σ MOMENT @ TOE OF FILLET, SEE 135 lb RAIL SKETCH, POINT n

$\rho_1 = 42.91 \left(\frac{3.02}{4.32}\right) = 30.0$ KSI





NOTE: L to be determine IN FUTURE (L ≥ 6")

$M_{en} = 30.0(1.3)^2/2 + \frac{1}{3}(1.3)^2(42.91 - 30.0)$

$= 25.35 + 7.27 = 32.62$ in-K [MOMENT @ PT n @ base of Fillet]

FIND MOMENT PER INCH OF FLANGE

USE: $b_{eff} = 8"$ (USE: WIDTH OF BASE PLATE)

$m = \frac{32.62}{8} = 4.08$ in-K MOMENT PER INCH OF FLANGE

REQ FLANGE THICKNESS @ TOE OF FILLET

$t = \left(\frac{6M}{F_b}\right)^{1/2} = \sqrt{\frac{6(4.08)}{.75(70)}} = 0.68$ inch

THICKNESS @ TOE OF FILLET $= \frac{19}{25.4} = 0.748" > 0.68$ OK

USE: 135 lb RAIL FOR BOTH CONCRETE & STEEL INVERTS, with an 8" B℗ WIDTH @ 20" centers