Appendix C: Project #6 Construction Methods and Coordination

Project #6 - Bridge Rehabilitation: Runway 16L/34R and Taxiway Bravo

Description and Agency Consultation

12/21/2025

The purpose of this appendix is to document the construction methods proposed for Project #6 - Bridge Rehabilitation: Runway 16L/34R and Taxiway Bravo (Bridge Rehabilitation Project). This includes:

- Description of the construction methods proposed
- Agency Consultation with U.S. Army Corps of Engineers (USACOE) and Virginia Department of Environmental Quality (VADEQ)

1. Description of Construction Method

The Bridge Rehabilitation Project is needed to strengthen the existing Runway 16L/34R Bridge and the Taxiway Bravo Bridge for routine operations by Group III aircraft (737-800 and A320). **Figure A11, Appendix A**[1] illustrates the locations that would be subject to the Bridge Rehabilitation Project which would occur in 2026-2027. The Bridge rehabilitation was determined through engineering investigations performed by the engineering firm, Walter P. Moore and Associates (WPM). Their report is titled*: HEF Manassas Airport Bridge Assessments, Structural Evaluation Results, Walter P Moore Dated August 22, 2025,* and is included as **Attachment #1** in this Appendix.

In summary, the recommendations provided by WPM for the Bridge Strengthening are shown in **Figure A12, Appendix A** and involve the stabilization of the earth (soil) supporting each bridge to increase the soil bearing capacity for each bridge. Soil Stabilization involves the installation of soil nails[2] under each bridge. The soil nails are steel rods that would be driven into the soil under each side of the bridges. Once the soil nails are installed, a concrete retaining wall is to be poured to secure the ends of the soil nails and thereby stabilize the soil and increase the load bearing capacity of the bridges. The soil nails coupled with the concrete retaining wall will provide the added bridge strengthening to support routine operations by Group III aircraft.

[1] The Figures referenced in this Appendix are located in Appendix A of this EA.

[2] Soil nailing is a common ground improvement technique used to enhance the stability of soil or rock masses. It involves the insertion of slender reinforcing elements, typically steel bars or rods, into the ground at a predetermined angle.

In order to complete the proposed stream bank and under bridge work, the stream water from Broad Run would need to be maintained at the seasonal low flow level. Maintaining Broad Run at low flow would keep the water level below the bridge footings, allowing construction crews to full access to the Bridge structure. In addition, maintaining low flow will preserve mussel species downstream of the Bridges. In this Appendix, see the memo titled "*Memorandum – Preliminary Concept for Maintaining Broad Run Minimum Water Flow During Construction at HEF Manassas Airport*" December 12, 2025, from Walter P. Moore for details on construction methods. This memo is included as **Attachment #2** in this Appendix. The primary work area is marked by the yellow and orange stripe on **Figure A11, Appendix A**. It would require contractor workspace on the north and south sides of Broad Run, with a width of 75-100 feet. Within these workspaces, there would be minor grading to create stable, level pads for construction equipment and to prevent ruts/ponding. Timber mats/rig mats may be used to minimize ground disturbance. The workspaces would be bounded by super silt fence. There would be stabilized construction entrances and, if needed, turbidity curtains may be set in wet areas to preserve water quality. The contractor's Stormwater Pollution Prevention Plan (SWPPP) would include a Spill Prevention and Response Plan as heavy equipment would be in use. The construction equipment may include a 20 to 30-ton excavator (plus one long-reach), a D4/D5 dozer, a wheeled loader, a skid steer, six to twelve inch diesel pumps (primary + standby), a water truck, tandem dumps, and a small crane or 60–100-ton crane if picks are needed.

A contractor's yard would be in place at the south end of Observation Road (**Figure A11, Appendix A**). This area also would contain construction support equipment. The workers and equipment would enter into the construction area using an existing Airport roadway and that road would not need to be enhanced. The existing riprap at the base of Broad Run would be removed and stockpiled within the construction area. Upon completion of the stream bank work and the under bridge strengthening, the bypass channel's north diversion opening would be closed, and the temporary dam would be removed restoring full flow to Broad Run.

2. Agency Consultation

When the construction methods were defined for the Bridge Rehabilitation project, the ASG team conducted a series of coordination calls with the interested agencies – U.S. Army Corps of Engineers (USACE) and Virginia Department of Environmental Quality (VADEQ). These meetings were focused on the Pre-Application process with USACE and VADEQ. The conference calls occurred on October 23, 2025 (initial meeting) and November 13, 2025 (follow-up meeting).

In summary, the meetings discussed the permitting process and the other interested agencies, including a discussion that the Virginia Marine Resources Commission (VMRC) would likely issue a statement that the project is exempt from permitting from their agency as no permanent changes to Broad Run are proposed. The bridge rehabilitation would result in two potential impacts:

1. Temporary placement of dredged and fill material within Broad Run to the proposed dams that will help to moderate flow through the construction zone.
2. One PEM wetland would be temporarily impacted when overflow waters from Broad Run are diverted through it, so that Broad Run maintains only seasonal low-flow conditions during the bridge work.

Both of these impacts would be permitted under the VADEQ Individual Permit (IP) and would additionally be authorized under a USACE State Programmatic General Permit (SPGP) or Nationwide Permit (NWP), depending on final impact amounts. Specific conditions of these permits would allow for the aforementioned impacts and would outline the conditions required to generate no water quality issues, including but not limited to the previously discussed SMMP, CHASP, PPC plans, and the existing VPDES permit (VAR050985), etc. The USACE and VADEQ see no reason that the above-mentioned permits would not be authorized so long as all permit conditions are met and proper mitigation is provided. Detailed minutes from the pre-application meetings held with VADEQ and USACE on October 23, 2025, and November 13, 2025, are included in **Appendix Q**.

Furthermore, concurrence with this methodology for limiting impacts was received from VADWR on December 18, 2025 (**Attachment #3**). As part of their concurrence, VADWR requested to have an opportunity to review and comment on the final Project #6 plan before construction and to ensure that a flow monitoring plan would be developed. Given this, no significant impacts to the mussel population in Broad Run would occur.

Attachment #1 – HEF Manassas Airport Bridge Assessments, Structural Evaluation Results, Walter P Moore

202.481.7685 main

August 22, 2025

Abel Garcia
President
V-1 Consulting, LLC
528 Clayton St
Denver, CO 80206

RE: **HEF Manassas Airport Bridge Assessments**
Structural Evaluation Results
Walter P Moore Project No. D01.24004.00

Dear Abel:

We have completed and compiled the results of our HEF Manassas Airport Bridges structural assessment, load testing, and geotechnical investigation.

Executive Summary

The runway and taxiway bridges were originally designed for aircraft with a maximum total take-off weight of 108,000-pounds, or 108 kips. V-1 Consulting requested both runway and taxiway bridges be evaluated for the potential to structurally support Group III aircraft with a maximum total take-off weight of 210 kips.

The bridges are composed of the following assemblies that required separate analysis of each component:

1. Superstructure (multiple concrete box beams);
2. Substructure (foundation walls and Mechanically Stabilized Earth (MSE) retaining walls); and
3. Connection between Superstructure and Substructure (dowels and pads).

SUPERSTRUCTURE

The limiting factor of the superstructure to support 210 kips was found to be the distribution of the aircraft live load onto the beams, called the live load Distribution Factor (DF). The live load DF is the maximum portion of the total live load assigned to any one individual box beam from each strut of the rear wheels. The original design designated a DF of 0.50.

Walter P Moore performed field load testing of each bridge to justify a DF of 0.35 (the live load is actually distributed onto approximately three adjacent beams instead of the assumed original design of two beams). The demonstrated increased load distribution enables the superstructure to support Group III aircraft of 210 kips gross total weight.

SUBSTRUCTURE

The substructure originally had two limiting factors as outlined below to support 210 kips, but further analysis eliminated one limiting factor. The second limiting factor listed below can be remediated as explained.

1. The original drawings indicated a soil bearing resistance of 3500 pounds per square foot (psf) at the base of the existing abutment footings at both bridges, which was inadequate to support 210 kips. However additional geotechnical investigations and analysis were performed and the in-situ soil bearing resistance was determined to be 4000 psf, which is sufficient to support 210 kips aircraft.
2. The Mechanically Stabilized Earth (MSE) retaining walls were analyzed and determined to have the capacity to support up to 165 kips aircraft, or approximately 20% deficient of the strength required to support 210 kips aircraft. This condition can be remediated as recommended below.

RECOMMENDATIONS

1. **Option 1: Take No Remedial Action**
    a. The Mechanically Stabilized Earth (MSE) retaining walls were analyzed and determined to be approximately 20% deficient of the strength required to support 210 kips aircraft. However, both the taxiway and runway bridges are expected to withstand an aircraft weight of 165 kips without remedial action for a period of 5 years from today (for a 50-year lifespan from original construction).

2. **Option 2: Perform Remedial Action on the MSE Wall**
    a. Strengthen the MSE Retaining Walls with a soil nail system. The design of this system will take the existing soil bearing capacity into account and enable the system to support 210 kips aircraft.
    b. The total cost of the recommended remediations is expected to be in the $3.75MM to $6.025MM range.

Additional evaluations are recommended for both bridges, such as investigating and/or installing box beam drain holes, performing top side test pits and underside material testing to assess the box beams condition, and hydraulic analysis for narrowed flow area as outlined further in the Recommendations section of this report.

The following components were beyond our scope and were not analyzed as part of this study:

1. Safety areas (i.e., portions of bridges not directly underneath the runway and taxiway pavement); and
2. Pavement concrete and subbase performance over time.

Background

The existing runway and taxiway bridges at the HEF Manassas Airport span over a 40-ft-wide stream crossing under the runway. The bridges were originally constructed circa 1983, and an extension was added circa 2013 to widen each bridge. The original drawings specify that the bridges were designed for a 737-200 aircraft with 108-kip total gross weight (dual rear gear with 48.5 kips per strut). The intent of the current study was to analyze, and load test the structural capacity of the existing bridge components to determine if the reserve capacity (if any) is adequate to support Group III aircraft with a maximum gross weight of 210 kips and dual rear gear strut loads of 100 kips per strut.

Based on the original structural drawings by HDR dated July 1983, the existing runway and taxiway bridge structures generally consist of the following structural components and design:

- Prestressed concrete box beams (2 ft-9in. deep x 4 ft-0 in. wide),
- Elastomeric bearing pads (6 in.x12 in.x1/2in., 70 durometer, 2 per beam end),
- Steel Dowel pin connection (1.5” Dia. X 2’-6” long, 1 per box beam & 1 side only)
- Foundation stem wall (18-in. thick) supporting each end of each box beam, with dowel (1-1/2 in. diameter embedded steel rod) – one end only with the other end free,
- Strip footing (2 ft-6 in. thick x 6 ft-0 in. wide with 3,500 psf soil bearing capacity (to be further verified),
- Mechanically Stabilized Earth (MSE) retaining wall (5-1/2 in. thick, 4 ksi precast panels anchored into fill with 60mm wide x 5mm thick x 23 ft long 36 ksi steel reinforcing strips).

Extracted partial drawings of the runway bridge from the original structural drawings are provided in Figures 1 through 7. Refer to the full set of original structural drawings for further runway and taxiway bridge information.

At the runway bridge, two different box beam configurations were originally designed. Only the portions of the bridge where the box beams are directly below the runway pavement have been analyzed. Box beams within the safety zone (outside of the runway and taxiway concrete pavement portions) have not been analyzed as part of this study.

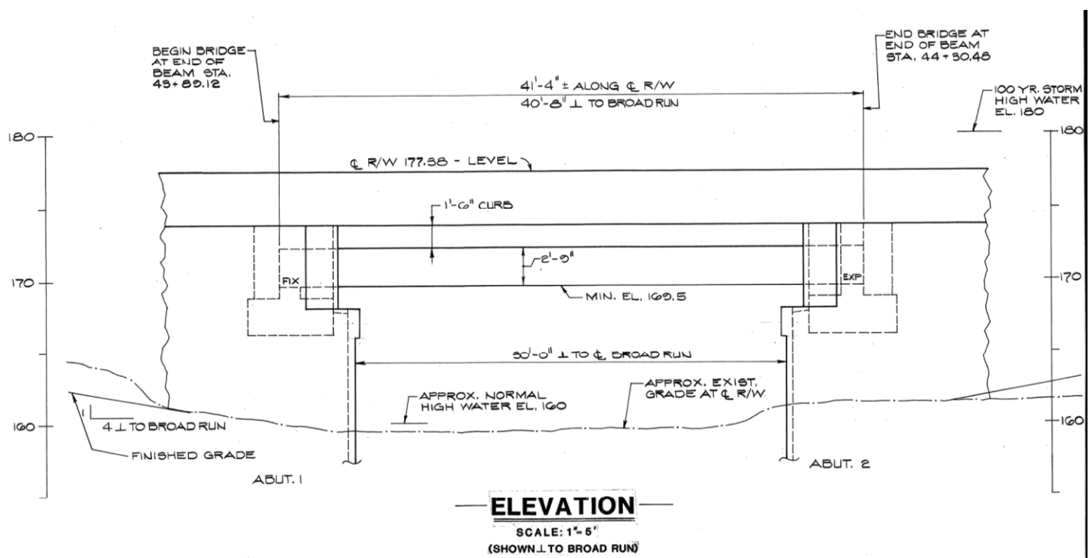


Figure 1 – Elevation overview of runway bridge.

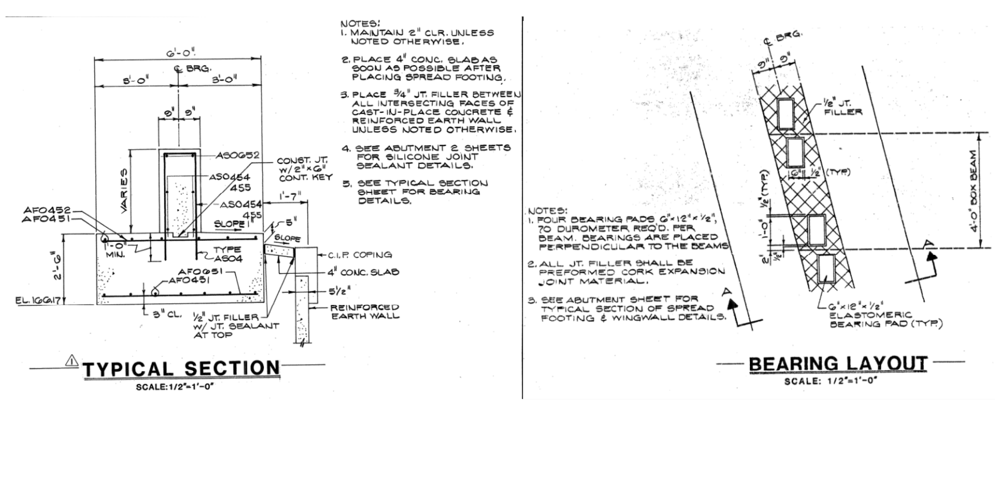
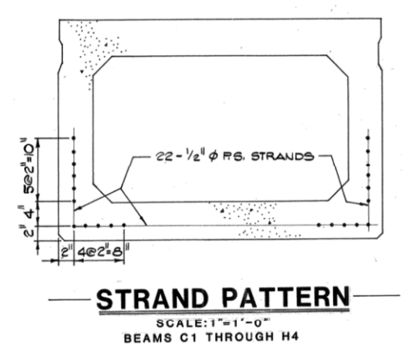
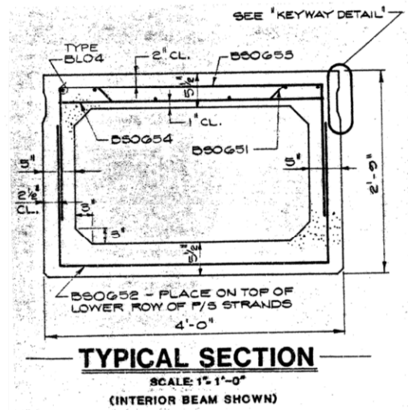


Figure 2 (left) – Cross sectional view of strip footing at runway bridge.
Figure 3 (right) – Plan view of beam ends showing 2 bearing pads per beam.





Figures 4 (left) and 5 (right) – Cross sectional views of runway bridge box beams (directly below runway) showing prestressing strands and reinforcement layout.

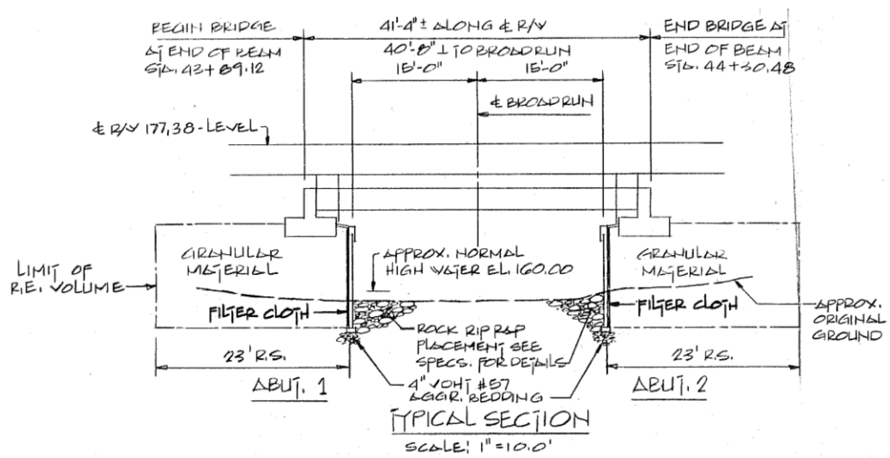


Figure 6 – Elevation overview of bridge abutment composition and extens of MSE retaining walls.

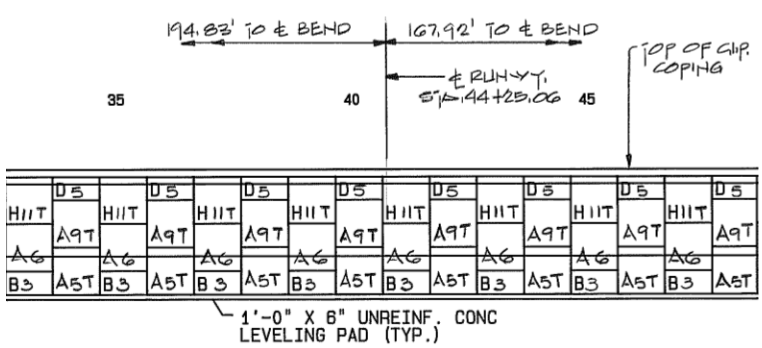


Figure 7 – Elevation of typical MSE wall panel layout at runway. Number after letter indicates number of tie strips attached to each panel e.g., “A6” indicates 6 tie strips.

Load Testing and Subsurface Investigations

From April 28, 2025 to May 9, 2025 representatives from Walter P Moore performed load testing and visual condition assessment of the bridges, from above and below, to evaluate the following:

- **Distribution Factor (DF):** The live load distribution factor is the maximum portion of the total live load assigned to any one individual box beam. The original design guidelines and our initial calculations followed a prescribed assumption of DF = 0.50 (for one strut, point load). In other words, 50% of the live load (100 kips) from one airplane strut was assigned to the design of each box beam. Load testing was performed by Walter P Moore using a 70-kip forklift simulating the rear strut of an aircraft and the resulting box beam deflections were measured with redundant instrumentation to determine an appropriate distribution factor. Based on field load testing, the DFs were determined to be 0.22 for the runway bridge and 0.32 for the taxiway bridge which are significantly lower than the previously used value of 0.50. To account for the uncertainty of existing conditions and the age of the bridges, for purposes of this study, a conservative DF of 0.35 for one strut point load was assumed for both the runway and the taxiway.

- **Visual Observations Summary**: The following summarizes visual observations made during the April–May 2025 field assessment. The majority of structural components were in “Good” condition, with isolated areas in “Fair” condition as outlined below:
  - Box beams do not appear to have drain holes on the underside, as was specified in the original drawings. Interior conditions of the box beams were not directly observed, and it remains unknown whether water or condensation has accumulated inside the beam cavities. Water accumulation within closed box beams is common and undesirable.
  - Minor concrete spalls were observed on the exposed faces of the MSE wall panels. Based on visual assessment, none of the spalls appear to impact the structural load-carrying capacity at this time.
  - Active water leakage and prior staining were observed at several beam-to-beam joints, particularly near the edges of the paved runway.
  - Vertical steel dowels connecting adjacent MSE wall panels were generally found to be in good condition. However, two dowels exhibited visible surface corrosion at exposed sections and may warrant further evaluation to determine the extent of deterioration.
  - Minor displacement and rotation of the retaining wall top coping was observed, along with localized soil settlement behind the wall, some soil washout, joint widening, and visible misalignment of panels. The amount of movement observed is consistent with MSE retaining walls of this age.

The original drawings indicate a maximum allowable soil bearing resistance of 3,500 psf. Additional geotechnical assessment was performed by Walter P Moore’s subconsultant and included eight additional borings. The additional geotechnical assessment determined that the in situ allowable soil bearing resistance is 4,000 psf. Walter P Moore utilized the new allowable resistance of 4,000 psf in the revised structural analysis of the MSE walls.

The load testing results and geotechnical reports are provided in the Appendices for more detail.

Analysis Methodology

In general, Walter P Moore analyzed the structural capacity of the bridges and abutments to withstand design aircraft loads.

The following bridge guidelines and industry guidelines were used:

- AASHTO LRFD Bridge Design Specifications 9th Edition 2020,
- AASHTO The Manual for Bridge Evaluation 3rd Edition, 2018 (AASHTO MBE), and
- ACI 343R-95 Analysis and Design of Reinforced Concrete Bridge Structures
    a. Section 5.10 – Airport runway bridge loads.

Per the AASHTO Manual of Bridge Evaluation (MBE), the load capacity evaluation of in-service bridges consists of three load-rating procedures at three different levels. The Design Load Rating at Inventory level (Live Load factor of 1.75) of reliability was selected for this study, as this is the same reliability adopted for new bridges per AASHTO LRFD Bridge Design Specifications.

While the AASHTO load rating procedures were developed for vehicular traffic, similar principles can be applied to aircraft loads. The following components were reviewed in our analysis:

1. Superstructure
    a. PGSuper software was used to analyze the Prestressed concrete box beams;
    b. The top lid of the concrete box beams was analyzed separately with hand calculations; and
    c. Superstructure to Substructure connections (dowel and bearing pads) were reviewed with hand calculations.
2. Substructure
    a. Abutment (Strip footing and stem walls) were reviewed with hand calculations; and
    b. Mechanically Stabilized Earth retaining walls were analyzed using MSE Wall GEO5 software.

The following assumptions were made during our analysis:

- The two Group III aircraft considered are the Airbus A321 Neo and the Boeing B737 Max 9, per the FAA Aircraft Characteristics Database and the Aircrafts Characteristics Airport and Maintenance Planning documents. Refer to Figures 8 and 9 for geometric data on the two aircraft.
- The following loading assumptions were made based on A321 Neo (WV080) specifications (which is the heavier of the two aircraft examined) unless noted otherwise:
    a. Maximum Takeoff Weight (MTOW): 209.4 kips
    b. Maximum Landing Weight (MALW): 174.6 kips
    c. Braking Force at Steady Braking: 32.7 kips (per strut)
    d. Braking Force at Instant Braking: 80.0 kips (per strut)
    e. Axle configuration,
        i. Wheelbase Length (centerline to centerline of front and back wheel axles): 55.4 ft
        ii. Centerline distance between rear axle wheels: 24.9 ft

1. B737 Max 9 rear axle wheels are spaced at 18 ft-9 in. instead. A center to center spacing of 17 ft-0 in. was conservatively assumed for this study.

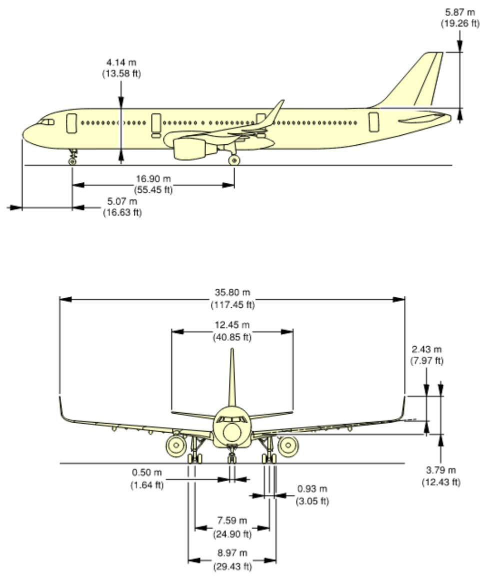


Figure 8. Axle and Tires Configuration for Airbus A321 Neo

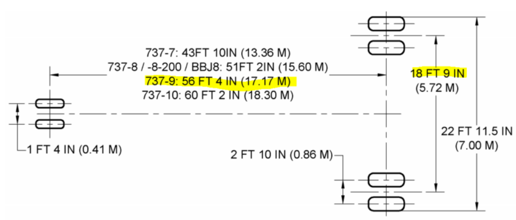


Figure 9. Axle and Tire Configuration for Boeing B737 Max 9

- The overburden assembly consists of the following layers, and associated dimensions, based on the provided drawings:
    a. Runway total overburden depth = 3.13 ft
        i. Original pavement/select fill = 2.38 ft
        ii. 9-in. overlay (installed in 2000s)
    b. Taxiway total original overburden depth = 2.76 ft
        i. Original pavement/select fill = 2.01 ft
        ii. 9-in. overlay (installed in 2000s)
- Impact Factors per ACI 343R-95 Section 5.10.2
    a. Taxiway Impact Factor = 40%
    b. Runway Impact Factor = 100%
- Load Factors
    a. DC (dead load of structural components and nonstructural components) and DW (dead load of wearing surfaces and utilities) Dead Loads: $\gamma_{DC} = \gamma_{DW} = 1.25$
        i. Per AASHTO MBE Section 6A.2.2.3 if the wearing surface thickness is field measured the LRFD load factor for wearing surfaces and utilities may be taken as 1.25

ii. The runway and taxiway pavements and overburden over the top of box beams were treated as DW for our analysis.

b. Live Loads (LL): Evaluations of existing bridges per AASHTO MBE concerns three load ratings. The following ratings were considered:
   i. Design Load Rating: Assess performance of existing bridges based on design standards. Per table 6A.4.3.2.2.1 of AASHTO MBE:
      1. Strength I Limit State
      2. Load Factor based on Evaluation Level
         a. Inventory: $\gamma_L = 1.75$
         b. Operating: $\gamma_L = 1.25$ (not used in this study)

- Condition and System Factors: Per AASHTO MBE the following was assumed:
   a. Condition Factor $\Phi_c = 1.00$ (Good or Satisfactory per Table 6A.4.2.3-1)
   b. System Factor $\Phi_s = 1.00$ ("All other girder bridges" per Table 6A.4.2.4-1)
- Load distribution
   a. Based on testing performed as part of this study, a conservative DF of 0.35 was established for both runway and taxiways based on load tests performed by Walter P Moore.

Results and Discussion

The allowable Design Load Weight Limit for Group III aircraft of 210 kips gross total weight is permissible for the runway and taxiway bridges, other than the item listed below.

- **Substructure:** The allowable MSE retaining wall capacity is 20% less than total loads imposed by Group III aircraft using code required factors of safety. While the walls still meet a lower factor of safety for potential short-term use, this does not satisfy current factors of safety required by AASHTO for typical bridge designs and ultimately would need to be strengthened.

Other items of concern identified during our evaluation include:

- Box Beam Stirrup Reinforcement Detailing: The existing box beam stirrups are spaced at 12 in. on center. While this spacing meets AASHTO requirements from the time of construction, it does not meet current AASHTO maximum spacing requirements at the box beam ends. However, the current spacing provides adequate design strength by calculations. This will require further evaluations should there be any future modifications to the bridges.

- Bearing Pads: While the existing neoprene bearing pads can withstand the proposed design airplane loads, they may require further investigation and evaluation based on shear deformation requirement. This AASHTO requirement is primarily dependent on the horizontal thermal expansion and contraction of the bridge and not on airplane load. Based on limited field visual observations by Walter P Moore, the exposed edge of these bearing pads generally appeared to be intact (free of cracks or tears when observed with a borescope), and still performing well after over likely 40 years in service. As they near the end of their service life, they should be monitored periodically to determine when remedial actions or replacement are required.

Recommendations

Continued use of the existing bridges will need to follow one of the following options :

1. **Option 1: Take No Remedial Action**
    a. The Mechanically Stabilized Earth (MSE) retaining walls were analyzed and determined to be approximately 20% deficient of the strength required to support 210 kips aircraft. However, both the taxiway and runway bridges can withstand an aircraft weight of 165 kips without remedial action for a period of 5 years.

2. **Option 2: Perform Remedial Action on the MSE Wall to Increase the Aircraft Capacity to 210 kips Aircraft.**
    a. **Strengthening of MSE Retaining Walls:** Install a soil nail retaining wall system, including reinforced shotcrete facing along both sides of the existing MSE wall under the runway and taxiway pavement. This approach abandons the existing MSE wall in place and uses the soil nail wall system as a new, permanent retaining wall system. The design of this system will take the 4,000 psf soil bearing resistance into account.

    b. The above strengthening measures are required to achieve 210 kips Aircraft capacity of the entire system for a 100 year lifespan.

    c. Walter P Moore prepared conceptual reinforcing sketches, a preliminary opinion of probable costs, and a preliminary construction schedule in the June 13, 2025 slide deck included in the Appendices of this report. The total cost of the recommended remediations is expected to be in the $3.75MM to $6.025MM range.

The following items will be required, regardless of which options above are selected:

1. **Structural Health Monitoring:** A robust Structural Health Monitoring system is installed and should be maintained to monitor changes over time. Periodic visual observations will also be required at the underside of the bridges to inspect for signs of deterioration or potential damage from flooding to supplement the structural health monitoring system.

2. **Bearing Pads Maintenance:** Future displacement or deterioration of bearing pads could result in bearing pad failure and damage to the bridge over time due to concrete-on-concrete contact of the box beam superstructure against the foundation wall substructure. The existing ½-in.-thick neoprene pads should be periodically monitored for early detection of deterioration that could compromise bridge performance. This recommendation is independent of airplane design load and applies to the bridge's continued use.

Additional recommended evaluations are still required for both bridges as follows:

1. **Investigate/Install Box Beam Drain Holes:** The original drawings specify drain holes (4 per beam) to allow drainage of any water inside the box beams. These holes were

not observed based on our visual assessment and we were not able to observe the interiors of the box beams. We recommend installing a preliminary number of drain holes at both bridges to attempt to observe interior conditions, and if water drains out, we may recommend additional investigations or installation of additional drain holes.

2. **Top Side Test Pits:** We previously recommended performing top-side test pits to observe the existing condition of the concrete box beam lids, but we were not able to perform this scope of work. We still recommend performing this investigation.

3. **Concrete Box Beam Testing:** We recommend performing laboratory testing on the bottom side of the box beams for determining the depth of Chlorides and Carbonation concentrations. We also recommend Petrographic analysis and strength testing to confirm the composition and strength of the concrete.

Sincerely,

WALTER P. MOORE AND ASSOCIATES, INC.

COMMONWEALTH OF VIRGINIA
AMIR MANAFPOUR
Lic. No. 061979
PROFESSIONAL ENGINEER

Amir Manafpour, PE, SE
Senior Project Manager
Diagnostics Group

Steve Treser, AIA, PE
Managing Director / Principal
Diagnostics Group

Appendices:

- App A – Runway and Taxiway Bridge Load Test Reports by Walter P Moore
- App B – Geotechnical Analysis Report of Soil Bearing Capacity by Walter P Moore's subconsultant
- App C – Bridge Superstructure Structural Analysis Calculations (Part 1 Runway and Part 2 Taxiway) by Walter P Moore
- App D – Bridge Substructure Structural Analysis Calculations by Walter P Moore
- App E – Slide Deck on Conceptual Strengthening, Opinion of Probable Costs, and Construction Schedule by Walter P Moore

Appendix A – Runway and Taxiway Bridge Load Test Reports by Walter P Moore

202.481.7685 main

August 8, 2025

Mr. Abel Garcia
Co-Founder & President
V-1 Consulting
528 Clayton St.
Denver, CO 80206

Re: Runway and Taxiway Bridge Load Tests
HEF Manassas Airport Bridge Assessments
10600 Harry J Parrish Blvd, Manassas, VA 20110
Walter P Moore Project No. D01.24004.00

Dear Abel:

We have completed the Runway and Taxiway Bridge Load Tests of the referenced HEF Manassas Airport Bridge Assessments in accordance with our proposal 24-1089 ASR #3 dated February 6, 2025.

Included in this report is the load test procedure, the load test results, and our conclusions and conceptual recommendations.

We very much appreciate this opportunity to provide these services to you. Please do not hesitate to contact us if we can further assist you with the follow-up evaluation and development of repair documents for the distress conditions described in the following report.

Sincerely,

Walter P. Moore and Associates, Inc.

Shengyi Shi, Ph.D.
Graduate Engineer
Diagnostics Group

Amir Manafpour, P.E., S.E.
Senior Project Manager
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HEF MANASSAS AIRPORT BRIDGE ASSESSMENTS

RUNWAY AND TAXIWAY BRIDGE LOAD TESTS

10600 Harry J Parrish Blvd, Manassas, VA 20110

GENERAL INFORMATION

Year Built	Circa 1983	**Bridge Type**	Precast Concrete Box Girder
Bridge Footprint	Runway Bridge 40 ft x 120 ft Taxiway Bridge 40 ft x 60 ft	**Feature Carried**	Vehicular and Aircraft Access

EXECUTIVE SUMMARY

Walter P. Moore and Associates, Inc. (Walter P Moore) has completed load testing of the referenced Runway and Taxiway bridges located at the Manassas Regional Airport. The overall scope of the bridge assessment project was to analyze the structural capacity of the existing bridge structure to determine its reserve capacity (if any) to withstand dual gear Group III aircraft with up to a maximum gross takeoff weight of 210 kips. As part of the overall effort, load tests were conducted to determine the distribution factors on both bridges. The obtained distribution factors were utilized in the analytical analysis of the bridges in a different phase of the bridge assessment project. This report presents the load test procedures, data analysis and conclusions.

Various types of sensors, including tilt rods, LVITs (Linear Variable Inductive Transducers) and strain transducers, were utilized to instrument the bridges for load testing. These sensors were selected both to capture the flexural response of the bridge near midspan during testing and to remain functional under potential flood conditions at the site. Walter P Moore developed the instrumentation plans and load test plans for both bridges. Temporary scaffolding platforms were constructed by others to provide access beneath the bridges over the river. Sensor installation was carried out by Walter P Moore representatives with assistance from the contractor team. Load testing for the Runway Bridge was conducted on the night of May 5, 2025, during a scheduled runway closure. A forklift loaded with steel plates was used as the test vehicle. A series of static load tests were performed under the supervision of Walter P Moore personnel. The same forklift was used to test the Taxiway Bridge on the night of May 8, 2025.

The load test results collected from both the Runway and Taxiway Bridges were processed and analyzed to determine their live load distribution characteristics. For dual-strut aircraft loading scenarios, the live load distribution factor was conservatively estimated to be approximately 0.35 for both bridges. The conclusions drawn from these load tests are applicable to the current structural conditions of the bridges and to the published aircraft loading scenarios and anticipated use cases. Load testing provides insights into the behavior of the referenced structure but does not provide a guarantee of future performance and does not capture the full magnitude and range of load applications that may occur in real world use under aircraft operations. Walter P Moore recommends implementing a long-term structural health monitoring program and conducting condition assessments of both bridges at six-month intervals. This monitoring strategy, coupled with enhanced visual inspections, is designed to help the client understand the performance of the bridges and the accumulation of representative potential damage if it occurs in the future.

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BACKGROUND

The Runway Bridge and Taxiway Bridge constructed in 1983 share a similar structural design. Both bridges consist of simple-span precast, pretensioned, prestressed concrete box beam superstructures supported by stub abutments on spread footings located behind mechanically stabilized earth (MSE) walls. Figure 1 shows the box beam section and soil fill of the Runway Bridge. Each concrete box beam is 4 feet in width, 2 feet 9 inches in depth, and spans 40 feet 8 inches in length. Although structurally similar, the bridges differ in the depth of overburden (soil) fill. The Runway Bridge has an overburden depth of approximately 3.13 feet, while the Taxiway Bridge has an overburden depth of approximately 2.76 feet. Figure 2 shows the plan view of both bridges.

The purpose of the load test is to determine the distribution factors on both bridges. Walter P Moore conducted an initial condition assessment and performed preliminary structural calculations. However, due to the presence of overburden fill on top of the concrete box beams, accurate estimation of live load distribution factors (ie the percentage of load sharing between adjacent beams) could not be achieved through analytical methods alone. As a result, a series of static load tests were planned to empirically determine the live load distribution factors. Due to an aircraft being unavailable for use in the load test, a forklift was utilized to approximate the effects of a typical 210,000 lb aircraft that would likely use the bridge.

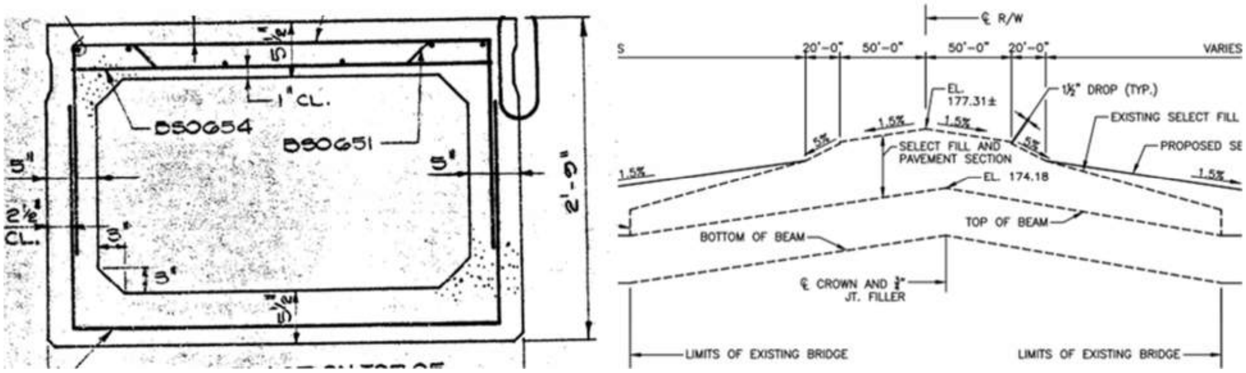


Figure 1. Box Beam Section and Soil Fill of Runway Bridge (Taxiway Bridge Similar).

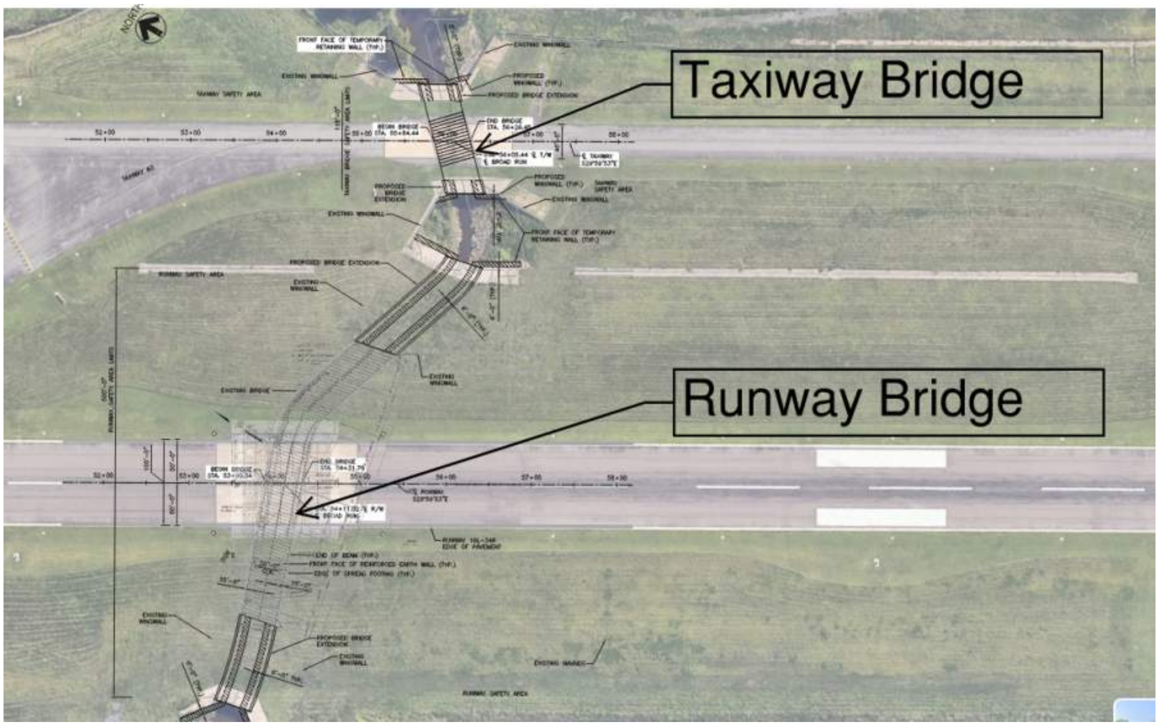


Figure 2. Plan View of the Runway and Taxiway Bridges at the Manassas Airport (Sketches by Walter P Moore Show Approximate Locations of Box Beams).

INSTRUMENTATION PLAN AND TEST VEHICLE

Instrumentation Equipment

Instrumentation plans for the load testing of the Runway and Taxiway bridges were designed based on the objectives of the project and site-specific conditions. Tilt Rods, Linear Variable Inductance Transducers (LVIT), and strain transducers were installed on the test bridges to measure the structural response under test forklift loading. Figure 3 shows selected photos of sensors and the data acquisition system (DAQ).

Tilt Rods

The tilt rods (named as Track Monitoring System per manufacture) utilized tilt meters and metal rods manufactured by RST Instruments to measure angular rotation and detect differential movement across the monitored structure. These rods were constructed using mechanical joints and stainless steel components for durability. Each tilt rod was equipped with an RST biaxial MEMS tilt meter, capable of measuring rotation in two perpendicular axes. The sensor features a measurement range of ±15 degrees, a resolution of 0.0003°, and an accuracy of ±0.003° within the central range. Both 3 meter (~10 ft) and 2 meter (~6.5 ft) rods were utilized for this project,.

LVITs

The linear displacement measurements were collected using the P103 LVIT manufactured by Positek. This sensor is designed for high-precision displacement measurements. The P103 has a stainless steel body with a sealed IP67 rating, allowing for reliable operation in outdoor environments. The sensor used in this setup had a measurement range of 0 to 20 mm (0.78 inches), with a linearity of ±0.25% full-scale and a resolution limited only by the data acquisition system. The Positek P103 was mounted to monitor relative displacement of the girders during load tests.

Strain Gauges

The ST350 strain transducers (referred to as Strain Gauges in this report) produced by BDI were used in the instrumentation. The ST350 reusable strain transducers are durable due to their rugged, waterproof construction. The ST350 internal circuitry consists of a full Wheatstone bridge with four fully active 350Ω foil gauges optimized to provide high electrical output for a given strain magnitude. The strain gauge has a resolution of one microstrain with an accuracy of ±1%.

Data Acquisition (DAQ) Systems

A Campbell Scientific DAQ systems controlled and collected data from the tilt rods, strain transducers, and LVITs setup. The GRANITE 10 all-digital measurement and control DAQ was designed as the core of the data-acquisition network. For each bridge, one GRANITE VOLT 116 measurement module was integrated with the GRANITE 10. The tilt rods were daisy-chained and connected to the datalogger for power supply and measurement records. The LVITs and strain gauges were wired to the VOLT 116 expansion module for power supply, excitation and measurement records. Walter P Moore developed the data acquisition and control programs for both bridge load tests. The system was configured to measure and store sensor data at 30-second intervals throughout the testing period.

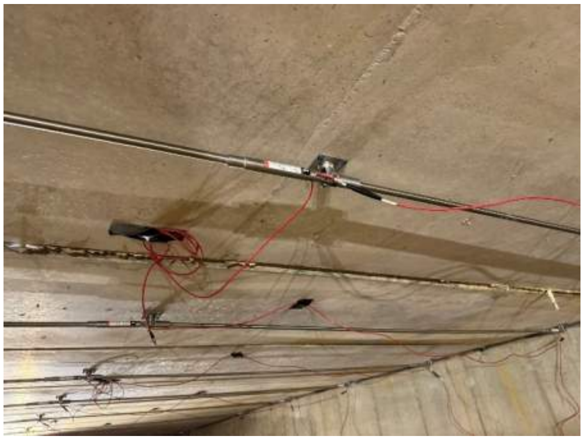
Instrumentation Plan

Runway Bridge Instrumentation Plan

As shown in the instrumentation plan (Appendix A1), two LVITs were installed near the midspan of Beams D8 and D10 to measure vertical deformation during the load test. Prior to sensor installation, scaffolding was erected beneath the beam soffits, with the base of the scaffolding freestanding on a temporary platform. At each selected beam, 12-inch-long mounting tubes were attached to the scaffolding posts to support the LVITs, positioning their measurement tips in contact with the underside of the beams. Tilt rods were installed on beams E1, E3, E5 and E7, and Beams D10 to D3. Each beam was instrumented with two 10 ft long tilt rods to obtain the approximate midspan deformations. The tilt rods in green were detached after the Runway Bridge load test and reused for Taxiway Bridge instrumentation. In addition, two strain gauges were installed on Beam D8 and E3 as comparisons. The data acquisition system and all sensors were powered by a 12-volt battery on the day of testing.

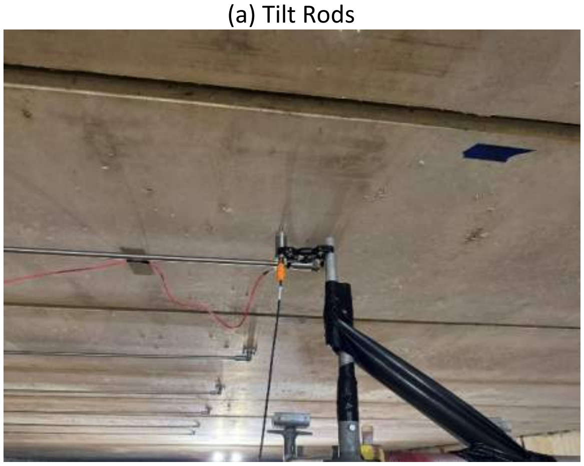
Taxiway Bridge Instrumentation Plan

Tilt rods were installed on Beam A1 to A5, and Beam B1 and B3 on the Taxiway Bridge as shown in Appendix A2. Each beam started with a 6.5 ft long tilt rod and connected to a 10 ft long tilt rod due to the space limitation during installation near the beam ends. Since the scaffolding and the temporary platform were not stable enough to support the LVITs, two LVITs were installed on Beam B1 and A4 to monitor the bearing pad movement (if any). Two strain gauges were installed on Beam B1 and A3.

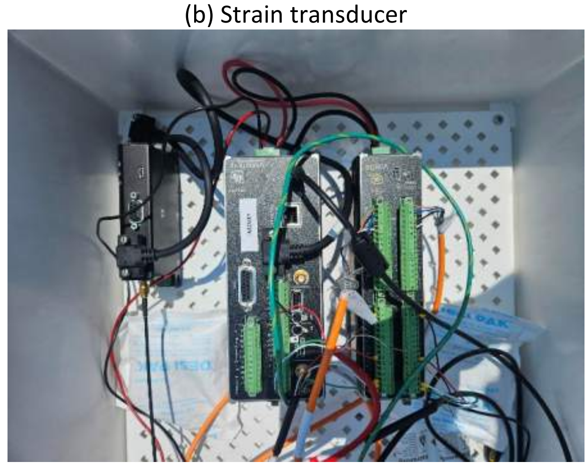


(a) Tilt Rods

(b) Strain transducer



(c) LVIT



(d) Granite 10 Datalogger and Expansion Module

Figure 3. Sensors and DAQ Used During Load Tests.

Test Vehicle

An HTH 24.11 heavy-duty telescopic handler forklift with two axles was used as the test vehicle for the bridge load tests. The front fork of the forklift was loaded with steel plates, and individual axle weights were measured on site using portable scales. The Walz AXW-30 axle scales were used to measure the wheel weights. Each pad of the scale has a 30,000 lb capacity with an accuracy of ±1%. The GVW of the loaded forklift was measured to be approximately 70,000 lb. The axle and wheel spacing information were obtained from the manufacturer specifications, and the measured axle loads of the forklift are shown in Figure 4.

Figure 4. Weight and Axle Information of the Forklift (Dimension Unit is in Millimeters).

LOAD TEST

Runway Bridge Load Test

To evaluate the in-service performance of the bridge, a series of 12 static load tests were conducted using a forklift in various positions. As shown in Figure 5, Tests 1 and 2 shared the same loading configuration, with the forklift unladen in Test 1 and loaded with steel plates in Test 2. For subsequent tests, the loaded forklift was positioned perpendicular to the bridge beams and moved sequentially from west to east. Figure 6 shows the load test pattern at the Runway Bridge. Starting from Beam E9 in Test 3, the forklift was placed with its front axle approximately centered over the beam for about 10 minutes per test. It was then repositioned on every other beam, continuing this pattern until reaching Beam D1.

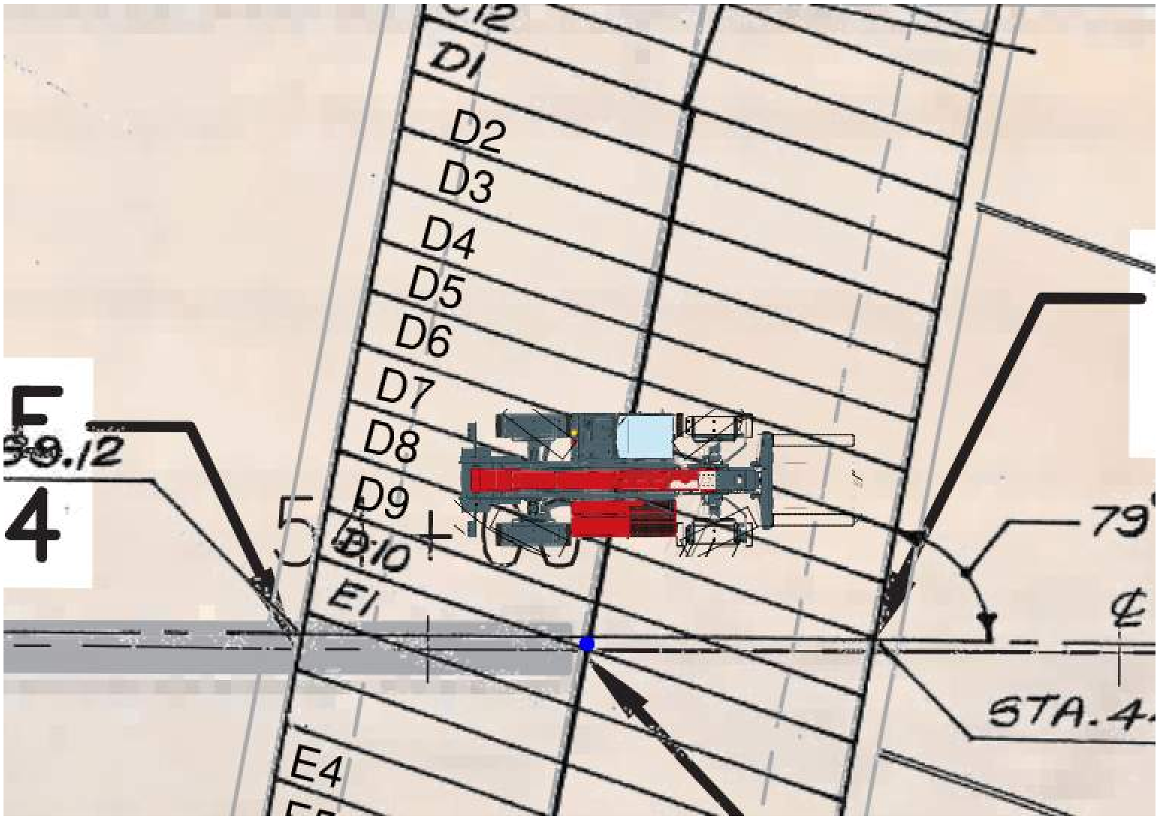


Figure 5. Position for Test 1 and Test 2 at Runway Bridge.

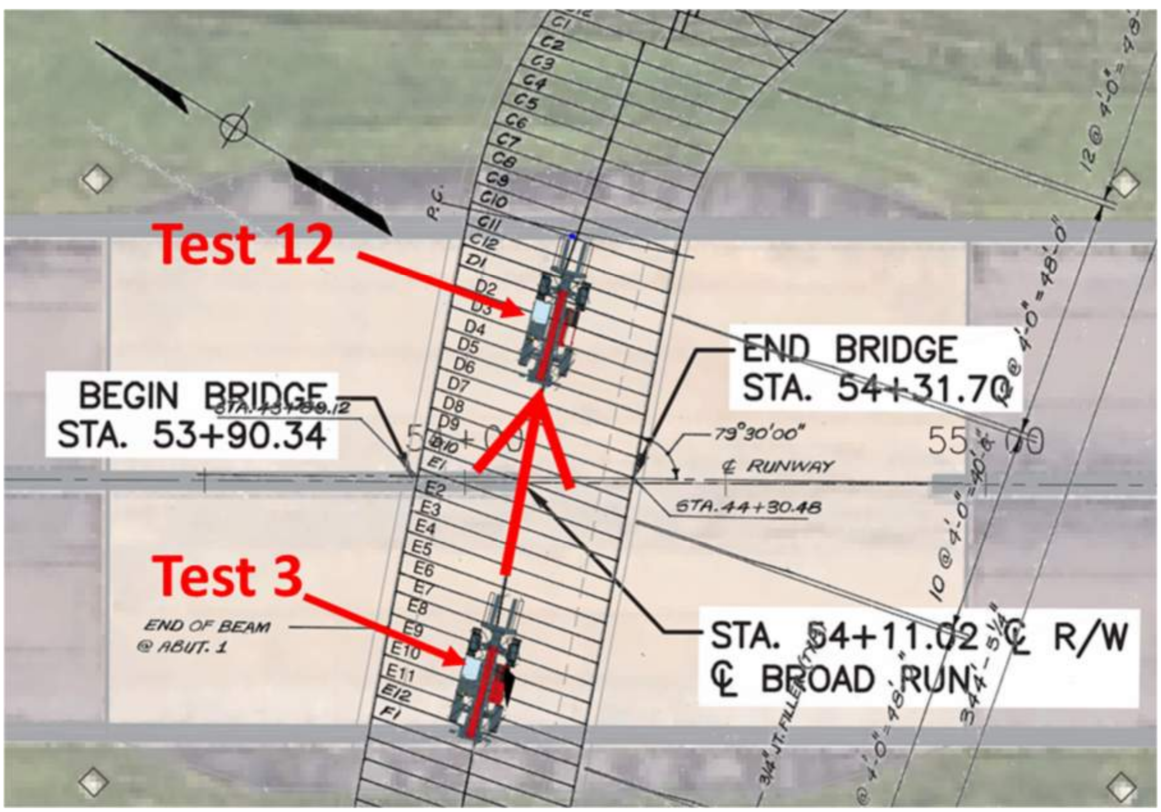


Figure 6. Load Test Pattern at Runway Bridge.

Following the completion of on-site logistics and baseline data collection, load testing on the Runway Bridge started at 12:15 AM on May 6, 2025, and concluded at 4:00 AM the same day. During testing, real-time data from the sensors were monitored by Walter P Moore engineers. In addition to real-time monitoring, vertical displacement data (collected via tilt rods and LVITs), and strain data (captured by strain gauges) were recorded after the forklift was removed from the bridge to evaluate potential residual deformations. Figure 7 shows the loaded forklift positioned on the Runway Bridge during one of the load tests.

Figure 7. Loaded Forklift on the Runway Bridge.

Taxiway Bridge Load Test

A series of seven (7) static load tests were conducted at the Taxiway Bridge using the same forklift used at the Runway Bridge in various positions. The loaded forklift was positioned perpendicular to the bridge beams and moved sequentially from Beam B3 to Beam A5. Figure 8 shows the load test pattern at the Taxiway Bridge.

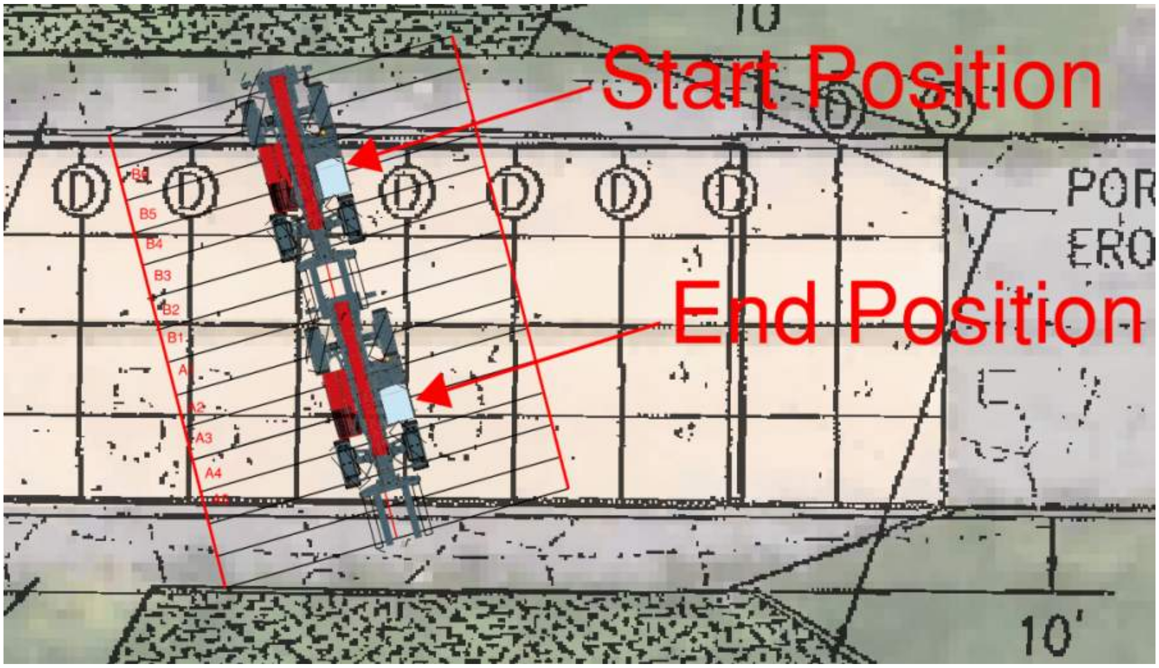


Figure 8. Load Test Pattern at Taxiway Bridge.

LOAD TEST RESULTS ANALYSIS

Runway Bridge Load Test Results

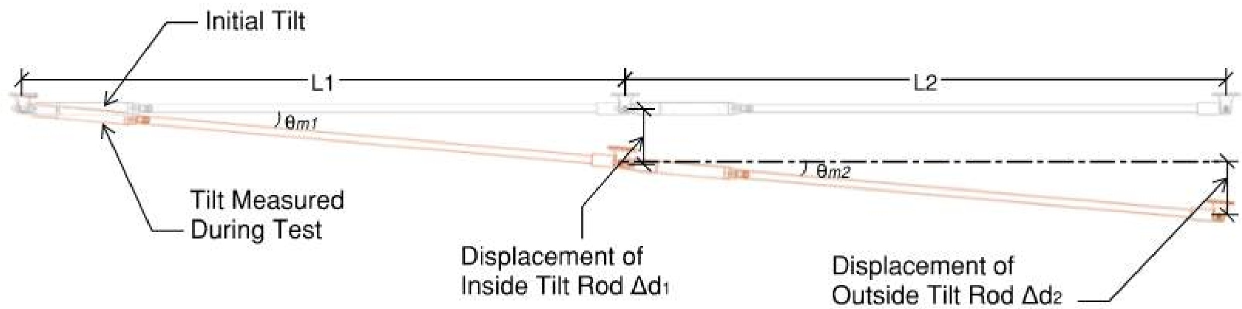
The measured data from the instrumented beams for each test were collected and processed. Baseline data collected from each sensor was utilized as an offset during the data processing. There were several assumptions as part of the data processing:

- The bridge beams behaved as simply supported spans.
- The forklift's wheel loads could be treated as concentrated forces.

Tilt Rod Results Analysis

The tilt rods measured the linear displacement along the length of the sensor. Figure 9 illustrates the method used to convert rotation angles into midspan displacements. These measured displacements were subsequently used to calculate the live load distribution factors for each instrumented beam. The distribution factor for a beam was determined by dividing its individual midspan displacement by the total displacement of all measured beams during a given load test. Figure 10 presents an example of this calculation for Beam D8 using data from the series of static load tests. In the figure, the X-axis represents the relative position (number of beams) of the forklift's front axle with respect to Beam D8. A value of 0 indicates that the front axle was directly above Beam D8, negative values correspond to positions west of Beam D8, while positive values correspond to positions east.

This approach enables the response of Beam D8 across multiple tests to be interpreted as the response of various beams when the load is applied directly over Beam D8. To ensure conservative estimates of the worst single load case, an envelope force effect method was utilized. The right side of the dataset was used to create a mirrored plot for distribution factor calculation (orange line in Figure 10). Based on this approach, the distribution factor for Beam D8 was calculated to be 0.17 when the load was applied on one side of the bridge. Using the same methodology, the distribution factors for other beams were calculated with a range from 0.13 to 0.17..



$\Delta d_{mid} := \Delta d_1 + \Delta d_2$ $\quad$ $\Delta d_1 := L_1 \cdot (sin\theta_{m1} - sin\theta_{i1})$ $\quad$ $\Delta d_2 := L_2 \cdot (sin\theta_{m2} - sin\theta_{i2})$

Figure 9. Displacement Calculation from Tilt Rod Data.

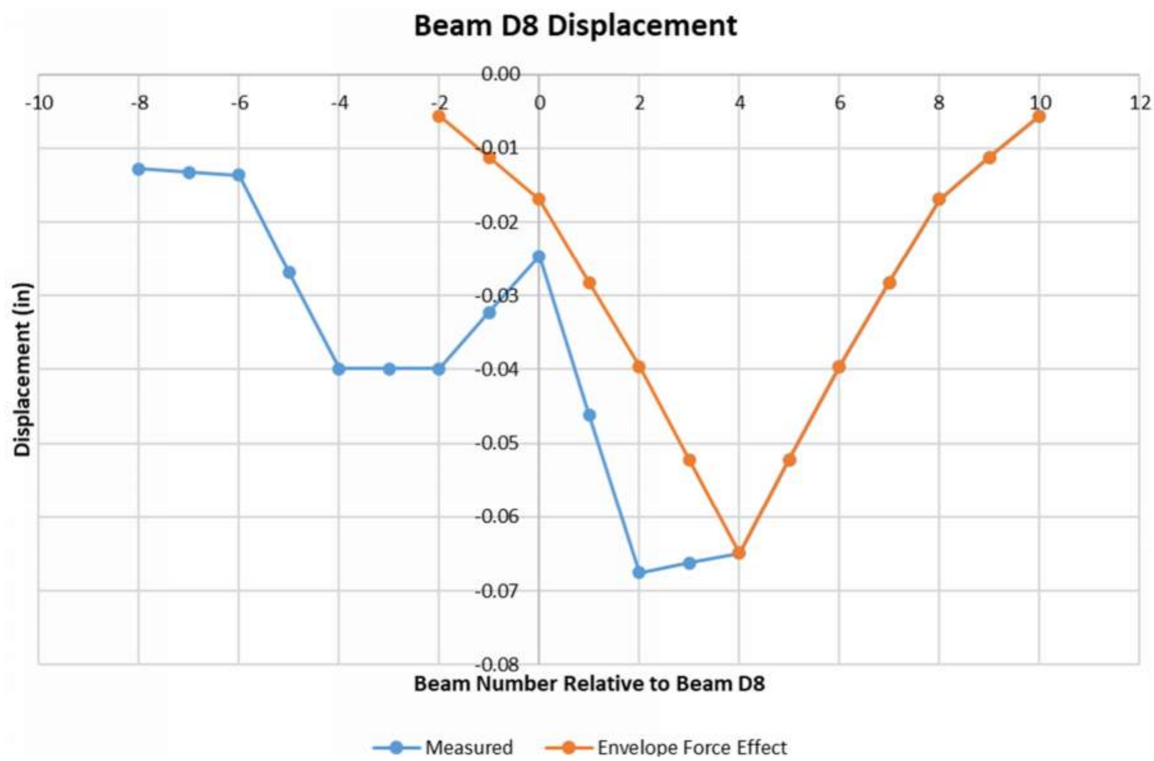


Figure 10. Example of Tilt Rod Plot for Distribution Factor Calculation with Envelope Force Effect.

<u>Strain Gauge Results Analysis</u>

The distribution factor based on strain gauge measurements was calculated by dividing the midspan strain of an individual beam by the total strain measured from all instrumented beams during a given load test. For the Runway Bridge, only two beams (D8 and E3) were instrumented with strain gauges. The strain data was processed using a similar approach to that applied to the tilt rod measurements. Specifically, the strain response of the instrumented beam (D8 or E3) across multiple load tests was interpreted as representing the response of various beams when the load is applied directly over the instrumented beam.

To ensure consistency and conservatism, only the right side of the dataset was used to create a mirrored plot for calculating the distribution factor using the envelope force effect method. Based on this method, the distribution factor was calculated to be 0.14 for Beam D8 and 0.13 for Beam E3.

<u>Distribution Factor Analysis for Target Aircraft</u>

The target aircraft is supported by two main struts with a minimum spacing of 16 feet. When the aircraft travels along the centerline of the Runway Bridge, each strut is positioned on either side of the bridge centerline, effectively distributing the load across both sides of the structure, as illustrated in Figure 11.To estimate the live load distribution factor for this loading scenario, results from the tilt rod measurements were used.

The previously reconstructed distribution plot was mirrored along the X-axis to simulate the presence of a second, symmetrically placed load. A superposition approach was then applied to combine the effects of both loads, as illustrated in Figure 11. Based on this analysis, the live load distribution factor for the dual-strut aircraft loading condition was determined to be 0.22.

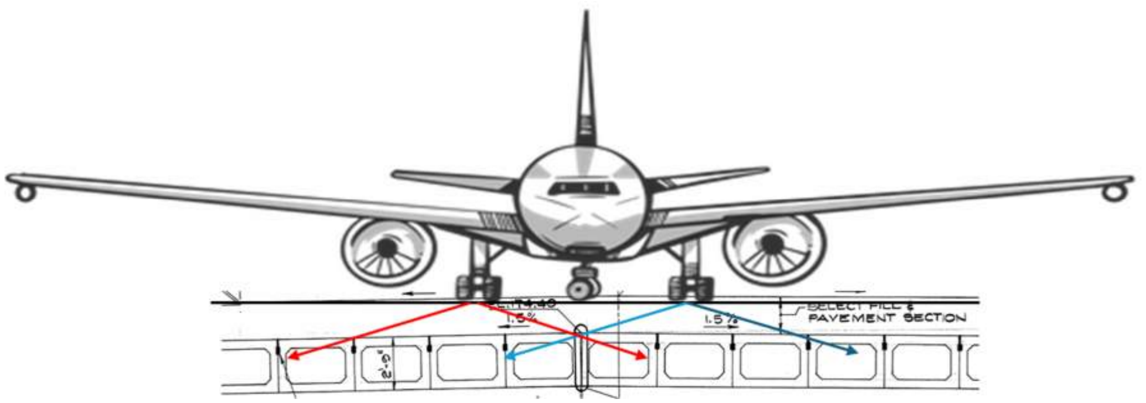


Figure 12. Strut Load Distributed to Box Beams Through Pavement and Soil Fill.

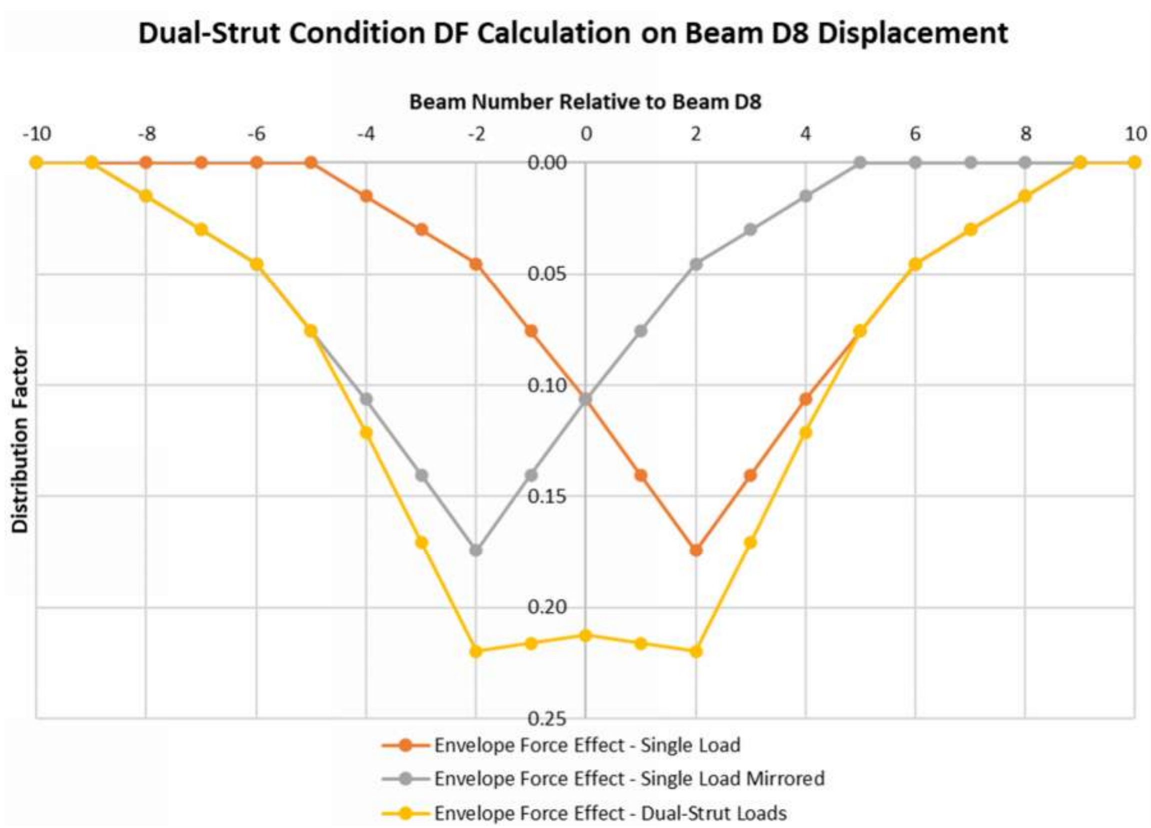


Figure 11. Example of Dual-Strut Aircraft Loading Condition Distribution Factor Calculation at Runway Bridge.

Taxiway Bridge Load Test Results

The measured data collected from the Taxiway Bridge load tests were processed and analyzed following the same methodology utilized in Runway Bridge load tests. The live load distribution factor for the dual-strut aircraft loading condition of the Taxiway Bridge was estimated to be 0.32 based on the tilt rod results from Beam B3, as shown in Figure 12.

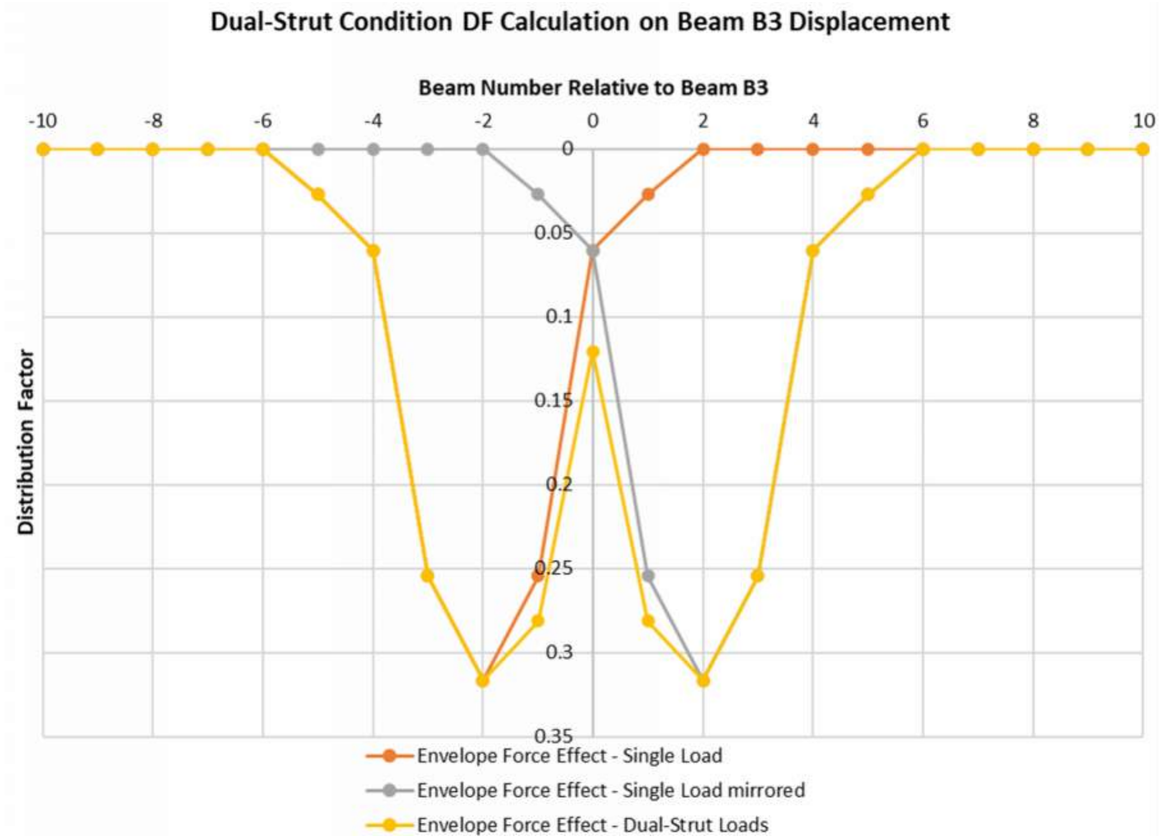


Figure 12. Example of Dual-Strut Aircraft Loading Condition Distribution Factor Calculation at Taxiway Bridge.

CONCLUSION

The Runway and Taxiway Bridges were instrumented with different types of sensors to measure the responses from load tests. A series of static load tests was performed with a heavy-duty forklift loaded with steel plates. The objective of the load tests was to determine the live load distribution factors of the bridges. Measured data from various sensors were collected and analyzed to determine the live load distribution characteristics. For the Runway Bridge analysis indicated that a conservative live load distribution factor for a worst case single concentrated load scenario was 0.17, based on tilt rod measurements. For the dual-strut aircraft loading condition, the calculated distribution factor was 0.22. However, due to the presence of overburden above the concrete beams and uncertainty regarding the performance of the shear keys between adjacent box beams, a more conservative approach was adopted. Specifically, the distribution factor for the dual-strut scenario was conservatively taken as twice that of the single-load case, resulting in a value of 0.35. In addition, results from the Taxiway Bridge indicated a distribution factor of 0.32. This implies that a concentrated load will be distributed across approximately three adjacent beams. The obtained distribution factor will be utilized to assess both bridges' load capacity to withstand dual gear Group III aircraft with up to a maximum gross takeoff weight of 210 kips.

The conclusions made from the load tests are applicable to the current state of both bridges. Load testing provides insights into the behavior of the referenced structure but does not provide a guarantee of future performance and does not capture the full magnitude and range of load applications that may occur in real world use under aircraft operations. Walter P Moore recommends implementing a long-term structural health monitoring program and conducting condition assessments of both bridges at six-month intervals.

LIMITATIONS

This report has been prepared to assist V-1 Consulting in understanding the live load distribution factors of the existing Runway and Taxiway Bridges located in Manassas Regional Airport, Virginia.

Walter P Moore has no direct knowledge of, and offers no warranty regarding the condition of concealed construction or subsurface conditions beyond what was revealed in our review. Any comments regarding concealed construction or subsurface conditions are our professional opinion, based on engineering experience and judgment, and derived in accordance with current standard of care and professional practice.

Comments in this report are not intended to be comprehensive but are representative of observed conditions and load test results. In this study, we did not include review of the design, review of concealed conditions, or detailed analysis to verify adequacy of the structure to carry the imposed loads and to check conformance to the applicable codes.

We have made every effort to reasonably present the various areas of concern as a result of the load tests performed on the existing Runway and Taxiway Bridges. If there are perceived omissions or misstatements in this report regarding the comments made, we ask that they be brought to our attention as soon as possible so that we have the opportunity to fully address them in a timely manner.

This report has been prepared on behalf of, and for the exclusive use of V-1 Consulting. This report and the findings contained herein shall not, in whole or in part, be disseminated or conveyed to any other party or used or relied upon by any other party, in whole or in part, without our prior written consent.

We very much appreciate this opportunity to provide these services to you. Please do not hesitate to contact us if we can further assist you.

APPENDIX A

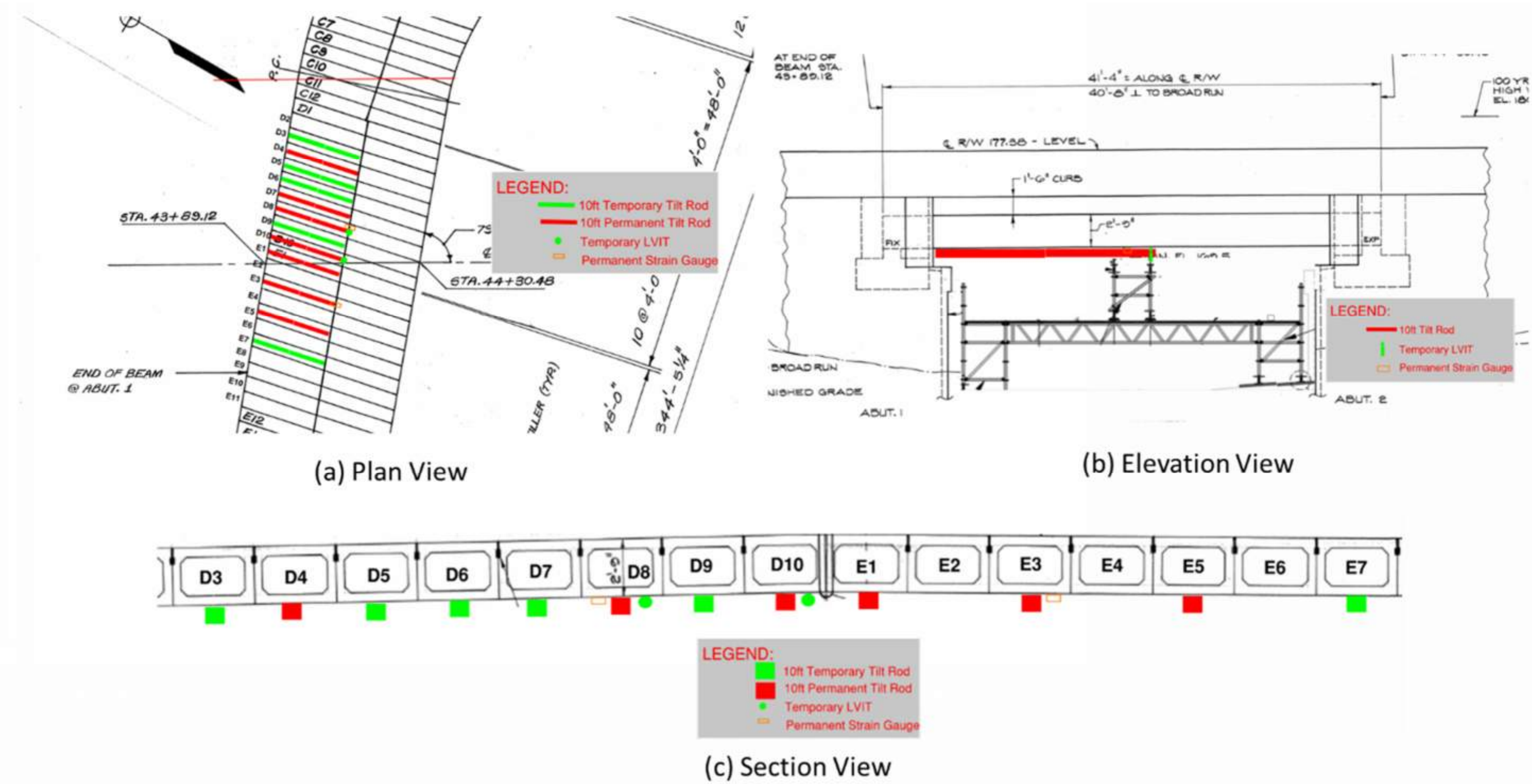
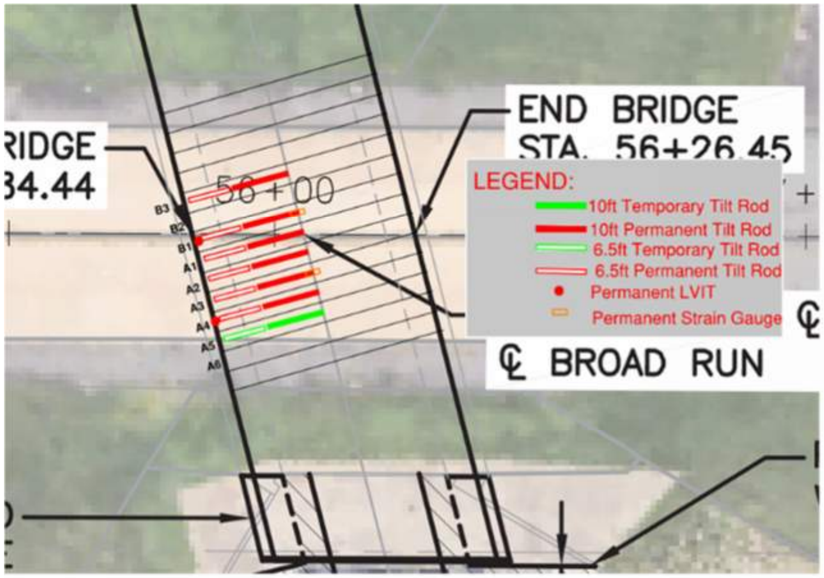
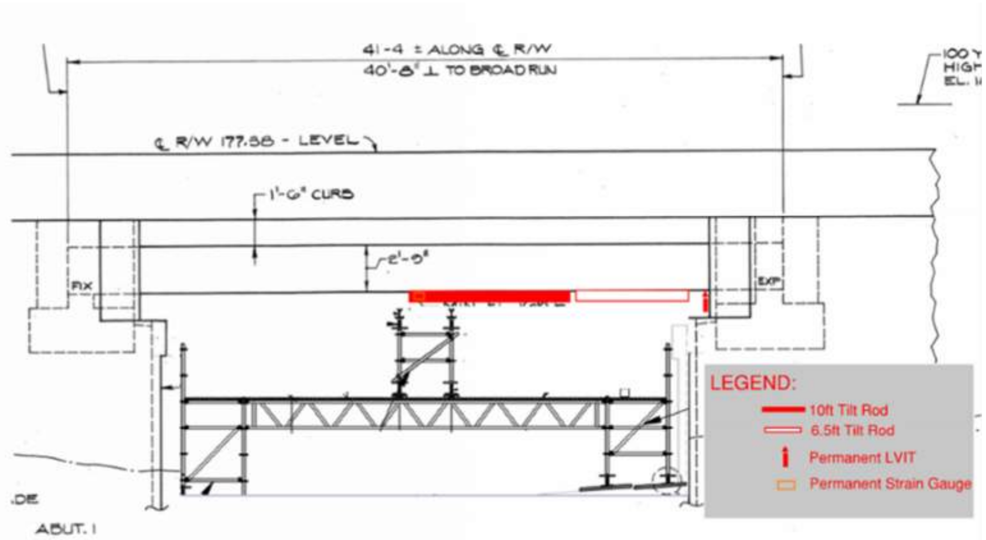


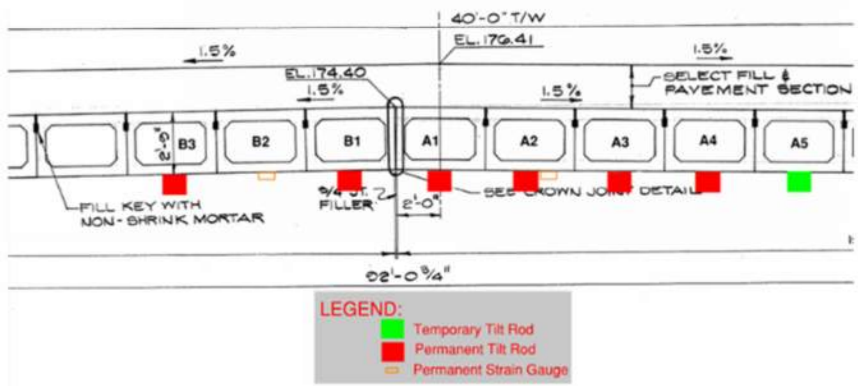
Figure A1. Runway Bridge Instrumentation Plan.



(a) Plan View



(b) Elevation View



(c) Section View At Midspan

Figure A2. Taxiway Bridge Instrumentation Plan.

Appendix B – Geotechnical Analysis Report of Soil Bearing Capacity by Walter P Moore's Subconsultant

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Manassas, VA 20110-2055

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wdpa.com

July 30, 2025

Walter P. Moore
1700 K Street, NW
Suite 1050
Washington, DC 20006

Attention: Mr. Steve Treser – Principal
Mr. Amir Manafpour – Project Manager

Subject: **Report of Geotechnical Engineering Services**
Manassas Regional Airport – Existing MSE Walls Evaluation
10600 Harry Parrish Boulevard
City of Manassas, Virginia 20110
WDP Project No. 25177

Reference: Proposal for Geotechnical Engineering Services
Manassas Regional Airport – Existing MSE Walls Evaluation
10600 Harry Parrish Boulevard
City of Manassas, Virginia 20110
WDP Proposal No: 25177, dated July 3, 2025

Gentlemen:

WDP & Associates Consulting Engineers, Inc. (WDP) is pleased to submit this Report of Geotechnical Engineering Services for the Manassas Regional Airport – Existing MSE Walls Evaluation project at the Manassas Regional Airport in the City of Manassas, Virginia. WDP's services were authorized by Walter P. Moore, referenced hereafter as the "Client," and performed in accordance with the referenced WDP proposal.

PROJECT BACKGROUND / PROJECT UNDERSTANDING

Based on scoping discussions with and preliminary information provided by Mr. Treser and Mr. Manafpour, both with the Client, WDP understands that the Client desires to have geotechnical information regarding the select backfill materials within the select backfill zone of the existing MSE walls adjacent to Runway 16L/34R and Taxiway B. More specifically, the Client desires to know if the select MSE wall backfill materials are adequate to support a 4,000 pound per square foot (PSF) bearing pressure as part of their structural analysis/evaluation of the existing MSE Walls and the runway/taxiway bridge beams and supporting stub abutment foundations. According to the Client, and based on available plan information, the existing bridge beams/stub abutment foundations were designed for a 3,500 psf allowable bearing pressure.

FIELD INVESTIGATION / TEST BORINGS

The test boring layout (Test Borings B-1 through B-8) was performed and coordinated with the Client and Manassas Airport Regional Operations personnel on July 9, 2025. Test boring locations were selected by WDP and the Client for purposes of investigating the existing MSE wall backfill materials in support of the purpose of this investigation and evaluation. Test borings were field located by WDP and the Client, with support from Manassas Regional Airport Operations personnel, using plans provided by the Client, existing site features, and pacing methods. Ground surface elevations at each test boring location were estimated from the elevations and contours indicated on the civil plans provided by the Client.

After the test boring layout, predrill coordination for utility damage prevention was performed and coordinated by the Client and Manassas Regional Airport Operations personnel. The approximate test boring locations are presented on the WDP Test Boring Location Plan attached to this report. **NOTE** – Test Boring B-5 was located slightly behind/beyond the existing MSE wall backfill zone, and subsurface materials encountered at Test Boring B-5 do not represent the existing MSE wall select backfill zone conditions. The remaining test borings were located and drilled within the existing MSE wall select backfill zone.

Field investigation was performed on the evenings of July 20 and 21, 2025 by WDP and Free State Drilling personnel, with field escort support from the Client. Prior to mobilizing the drill rig and crew each evening, the Client and Manassas Regional Airport Operations personnel provided and coordinated necessary closures of Taxiway B and Runway 16L/34R as required by FAA and Manassas Regional Airport safety procedures.

The field investigation was performed utilizing Standard Penetration Test Borings (ASTM D-6151) advanced through the upper 2 to 3 feet of overburden fill soils and then within the underlying select MSE wall backfill materials until auger refusal was experienced upon the underlying shale/siltstone bedrock at depths ranging from 21.5 to 23.5 feet below existing grade. Samples of the subsurface soils and MSE wall select backfill from the test borings were obtained using a split-spoon sampler and the Standard Penetration Test procedure (ASTM D1586), with SPT sampling generally conducted at 2.5± foot intervals within the upper 10 feet and then at 5± foot intervals to auger refusal on shale/siltstone bedrock. **NOTE** – no laboratory testing of collected samples was performed or requested/required.

The purpose of the test borings was to define the overburden soil and MSE wall backfill profile, define depth to rock and groundwater (if at shallow depths), provide representative samples where required, and identify conditions which will assist the Client with structural evaluation/analysis. The test borings were backfilled with the soil/select backfill that was augered at each test boring upon completion of each test boring, such that no test boring holes were left open.

Several pictures of the test boring/field investigation operations are presented below.

Test Boring B-1, east side of Taxiway B, north of Broad Run

Test Boring B-2, east side of Taxiway B, south of Broad Run

FIELD EXPLORATION RESULTS AND FINDINGS

Subsurface conditions encountered at the test boring locations are described below, noting that Test Boring B-5 was performed behind/beyond the MSE wall select backfill zone. These strata were encountered underlying approximately 4 to 6± inches of grass/topsoil in unpaved grass areas; no test borings were performed within Taxiway B or Runway 16L/34R.

Stratum I – Existing Fill – Overburden Soils

Stratum I existing fill overburden soil consist primarily of moist to very moist, brown, silty to fine sandy lean CLAY (USCS CL) to fat CLAY (USCS CH) with variable fine sand and gravel content in a firm to stiff consistency based upon the SPT results recorded during drilling. This stratum was encountered below the grass/topsoil and was approximately 2.5 to 3 feet in thickness, and above the underlying MSE wall select backfill materials of stratum II. The stratum I existing fill overburden soil are likely the result of original airport development and construction and any subsequent improvements. WDP understands and assumes the existing bridge stub abutment foundations do not bear within these existing fill overburden soils as they are not adequate in their present condition to warrant a 4,000 psf allowable bearing pressure.

NOTE - As indicated within this report, Test Boring B-5 was performed behind/beyond the MSE wall select backfill zone. The subsurface soils/materials encountered at Test Boring B-5 consisted of brown and red brown, sandy lean CLAY (USCS CL) and fat CLAY (USCS CH) within the upper 9.5 feet in a moist and firm to stiff consistency based upon the SPT results recorded during drilling. Gray, crushed angular aggregate with some brown clay soils mixed within the aggregate materials was encountered at 9.5 feet deep, extending to 18 feet deep whereupon native residuum soil was encountered. Native residuum soils at 18 feet deep to the underlying shale/siltstone bedrock at 23.5 feet deep consisted of brown, moist, silty fat CLAY (USCS CH) in a firm consistency based upon the SPT results recorded during drilling.

Stratum II – Fill - MSE Wall Select Backfill

Stratum II fill materials encountered beneath the upper existing fill overburden soils were comprised of gray, crushed angular aggregate with little to some fine sand content in a damp condition, and resembled an AASHTO No. 57 stone material to possibly VDOT 21A aggregate base material. These aggregate materials were loose to medium dense within the first sampled interval upon encountering them, and that is attributed to the absence of greater overburden pressure and lateral confinement. With depth, the aggregate materials generally increased in stiffness due to increasing overburden pressure and lateral confinement. The gray crushed angular aggregate materials were in a medium dense to dense relative density based upon the SPT results recorded during drilling. WDP understands and assumes the existing bridge stub abutment foundations bear within these higher modulus, aggregate materials, which are more suitable in their present condition to warrant a 4,000 psf allowable bearing pressure.

CONCLUSIONS AND RECOMMENDATIONS

Based on WDP's understanding of the scope and purpose of this field investigation and geotechnical evaluation and analysis, and the existing MSE wall select backfill materials comprised of medium dense to dense crushed angular aggregate, it is WDP's opinion that the use of 4,000 pounds per square foot (psf) is warranted for existing stub abutment foundation evaluation and supported by the findings and analysis of this study. WDP had performed a geotechnical investigation with recommendations in 2011 for four (4) new bridge extensions at Taxiway B and Runway 16L/34R crossings over Broad Run. WDP had recommended a 4,000 psf allowable bearing pressure for the new bridge stub abutment foundations that would bear within the recommended 21A/crushed angular aggregate materials used for MSE wall select backfill.

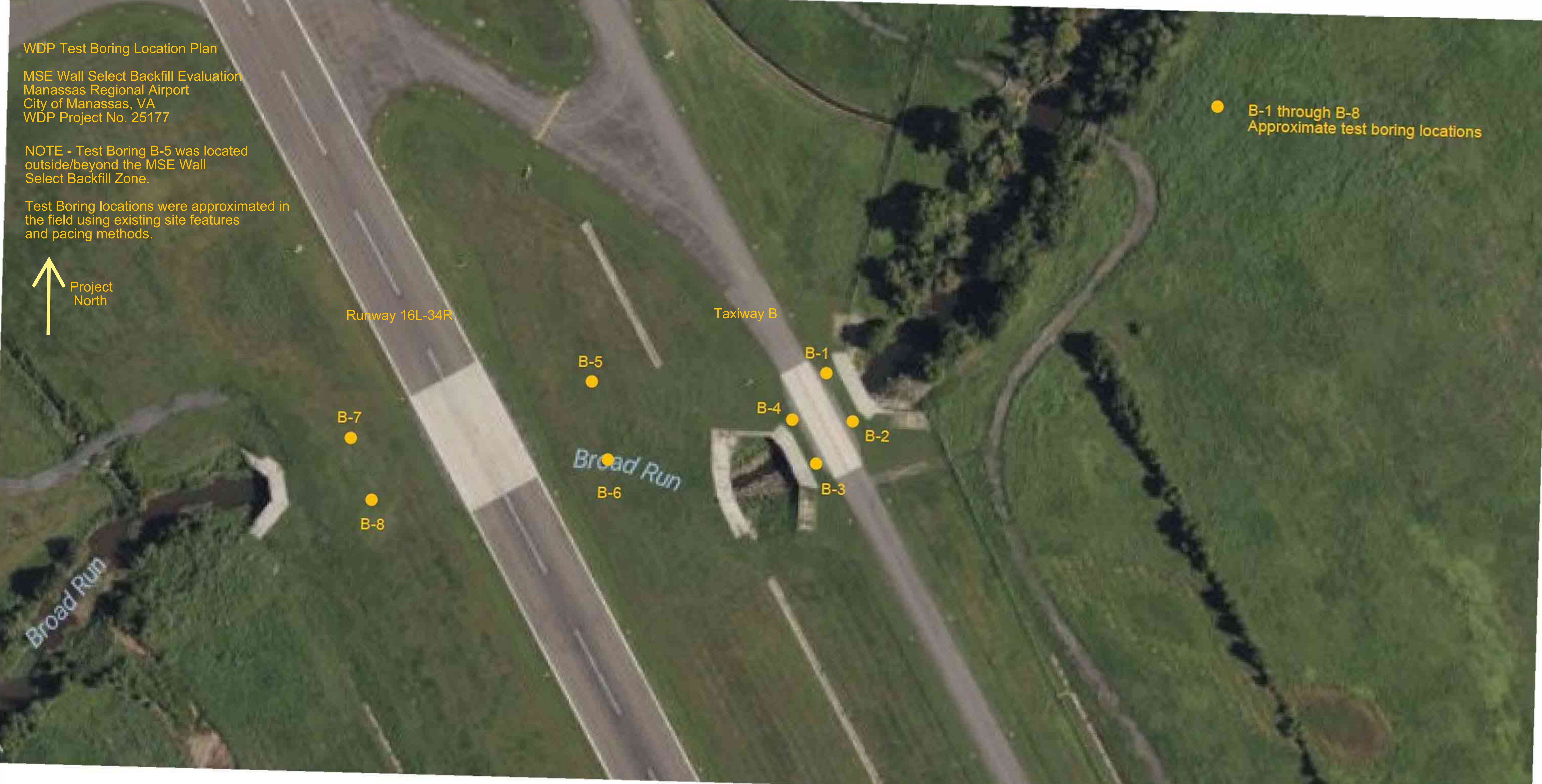
While the primary purpose of this study and analysis were to determine if the existing MSE wall select backfill materials condition would warrant and justify a 4,000 psf allowable bearing pressure, WDP recommends that this increased bearing value – from original 3,500 psf to recommended 4,000 psf – consider any increased lateral surcharge loading on the existing MSE wall system. This lateral pressure analysis of the existing MSE walls was not part of WDPs scope of services for this project.

WDP appreciates your confidence in our firm, and the opportunity to assist and support this project. Should you have any questions after review of this report, please contact me.

Respectfully submitted,
WDP & Associates Consulting Engineers, Inc.

Robert J. Niber, PE
Principal

COMMONWEALTH OF VIRGINIA
ROBERT J. NIBER
Lic. No. 24057
7/30/25
PROFESSIONAL ENGINEER


WDP Test Boring Location Plan
MSE Wall Select Backfill Evaluation
Manassas Regional Airport
City of Manassas, VA
WDP Project No. 25177
NOTE - Test Boring B-5 was located outside/beyond the MSE Wall Select Backfill Zone.
Test Boring locations were approximated in the field using existing site features and pacing methods.
Project North
B-1 through B-8
Approximate test boring locations
Runway 16L-34R
Taxiway B
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
Broad Run
Broad Run

SOIL BORING: B-1

WDP Architectural & Engineering Solutions

Depth: 22'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/20/2025
Date Completed: 07/20/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Location: Taxiway B-Eastside, North of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Sample Number	Blow Counts	Uncorrected N-Value	Moisture Content (%)	Atterberg Limits (LL-PL-PI)	Percent Passing #200 Sieve	Remarks
				FILL: brown, **Sandy Clay** (**CL**), trace fine black rock chips, firm, moist	1	4 2 9	11				Grass/topsoil 4-6"
				3.0							
170	5			**FILL**: gray, crushed angular aggregate, little to some fine sand, damp	2	7 5 6	11				MSE wall, select backfill. Loose-medium dense at top
				Medium Dense	3	6 9 11	20				
165	10			Dense	4	6 19 34	53				
160	15			Loose	5	14 5 5	10				
155	20			Dense-Very Dense	6	23 50/5"	50				
				22.0	7	50/0"					
150	25			Augur refusal at 22 feet on shale/siltstone. Backfilled upon completion. The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line							

Depth	Comment
-	-
-	-

WDP Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

SOIL BORING: B-2

WDP Architectural & Engineering Solutions

Depth: 22'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/20/2025
Date Completed: 07/20/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Location: Taxiway B-Eastside, South of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Samples: Sample Number	Samples: Blow Counts	Samples: Uncorrected N-Value	Lab: Moisture Content (%)	Lab: Atterberg Limits (LL-PL-PI)	Lab: Percent Passing #200 Sieve	Remarks
	0–3.0			**FILL**: brown, **Sandy Clay** (**CL**), trace fine Gravel, stiff, moist	1	50 15 6	21				Grass/topsoil 4-6"
	3.0			**FILL**: gray, crushed angular aggregate, little to some fine sand, damp	2	5 5 5	10				MSE Wall,select backfill. Loose-medium dense at top
170	5			Medium Dense	3	5 7 10	17				
165	10			Medium Dense	4	11 12 14	26				
160	15			Medium Dense - Dense	5	15 10 19	29				
155	20			Medium Dense	6	15 11 11	22				
	22.0			Augur refusal at 22 feet on shale/siltstone. Backfilled upon completion. The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line							
150	25										

Depth	Comment
-	-
-	-

WDP Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

SOIL BORING: B-3

WDP Architectural & Engineering Solutions

Depth: 22'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/20/2025
Date Completed: 07/20/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Location: Taxiway B - Westside, south of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Sample Number	Blow Counts	Uncorrected N-Value	Moisture Content (%)	Atterberg Limits (LL-PL-PI)	Percent Passing #200 Sieve	Remarks
				FILL: brown, **Fat Clay** (**CH**), litte fine Sand, firm, moist	1	3 3 3	6				Grass/topsoil 4-6"
				3.0							
170	5			**FILL**: gray, crushed angular aggregate, little to some fine sand, damp	2	6 3 4	7				MSE Wall,select backfill. Loose at top
				Medium Dense	3	8 10 12	22				
165	10			Medium Dense - Dense	4	13 16 13	29				
160	15			Medium Dense	5	13 11 12	23				
155	20			Medium Dense	6	12 9 11	20				
				22.0							
150	25			Augur refusal at 22 feet on shale/siltstone. Backfilled upon completion. The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line							

Depth	Comment
-	-
-	-

WDP Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

SOIL BORING: B-4

Architectural & Engineering Solutions

Depth: 21.5'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/21/2025
Date Completed: 07/21/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Taxiway B - Westside, north of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Samples: Sample Number	Samples: Blow Counts	Samples: Uncorrected N-Value	Lab: Moisture Content (%)	Lab: Atterberg Limits (LL-PL-PI)	Lab: Percent Passing #200 Sieve	Remarks
				FILL: brown, **Sandy Clay** (**CL**), stiff, moist	1	6 3 8	11				Grass/topsoil 4-6"
				3.0							
170	5			**FILL**: gray, crushed angular aggregate, little to some fine sand, damp	2	4 8 6	14				MSE Wall,select backfill. Medium dense at top
				Medium Dense	3	8 11 11	22				
165	10			Medium Dense - Dense	4	13 15 15	30				
160	15			Dense	5	15 26 28	54				
155	20			Dense	6	28 20 11	31				
				21.5							
150	25			Augur refusal at 21.5 feet on shale/siltstone. Backfilled upon completion.							

The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line

Depth	Comment
-	-
-	-

Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

SOIL BORING: B-5

WDP Architectural & Engineering Solutions

Depth: 23.5'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/21/2025
Date Completed: 07/21/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Location: Runway 16L-34R - East side, north of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Samples: Sample Number	Samples: Blow Counts	Samples: Uncorrected N-Value	Lab: Moisture Content (%)	Lab: Atterberg Limits (LL-PL-PI)	Lab: Percent Passing #200 Sieve	Remarks
				FILL: brown, **Sandy Clay** (**CL**), stiff, moist	1	6 8 10	18				Grass/topsoil 4-6"
				3.0							
170	5			**FILL**: reddish brown, **Fat Clay** (**CH**), firm to stiff, moist	2	4 4 6	10				
					3	1 2 3	5				
165	10			9.5	4	3 3 2	5				
				FILL: gray, crushed angular aggregate, little sand, trace geotextile, damp							Behind MSE wall, select backfill zone
160	15			Gray aggregate and brown clay, moist	5	2 1 2	3				
				18.0							
155	20			**NATIVE**: brown, **Silty Fat Clay** (**CH**), firm, moist	6	2 2 5	7				
				23.5	7	50/0"					
150	25			Augur refusal at 23.5 feet on shale/siltstone. Backfilled upon completion.							

The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line

Depth	Comment
-	-
-	-

WDP Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

SOIL BORING: B-6

WDP Architectural & Engineering Solutions

Depth: 22.5'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/21/2025
Date Completed: 07/21/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Location: Runway 16L - 34R- East side, south of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Sample Number	Blow Counts	Uncorrected N-Value	Moisture Content (%)	Atterberg Limits (LL-PL-PI)	Percent Passing #200 Sieve	Remarks
				FILL: brown, **Sandy to Silty Clay (CL/CH)**, moist to very moist, firm	1	3 3 4	7				Grass/topsoil 4-6"
				3.0							
170	5			**FILL**: gray, crushed angular aggregate, little to some fine sand, damp	2	3 9 7	16				MSE Wall,select backfill. Medium dense at top
				Medium Dense	3	5 5 6	11				
165	10			Medium Dense - Dense	4	11 11 19	30				
160	15			Dense	5	19 17 19	36				
155	20			Dense - Very Dense	6	43 47 33	80				
				22.5							
150	25			Augur refusal at 22.5 feet on shale/siltstone. Backfilled upon completion. The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line							

Depth	Comment
-	-
-	-

WDP Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

SOIL BORING: B-7

WDP Architectural & Engineering Solutions

Depth: 23'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/21/2025
Date Completed: 07/21/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Location: 16L-34R-West side, north of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Samples: Sample Number	Samples: Blow Counts	Samples: Uncorrected N-Value	Lab: Moisture Content (%)	Lab: Atterberg Limits (LL-PL-PI)	Lab: Percent Passing #200 Sieve	Remarks
	0–3.0			**FILL**: brown, reddish brown, **Silty Clay (CL/CH)**, moist	1	4 7 9	16				Grass/topsoil 4-6"
	3.0			**FILL**: gray, crushed angular aggregate, little to some fine sand, damp	2	8 7 6	13				MSE Wall,select backfill. Loose-meduim dense at top
170	5			Loose	3	4 3 4	7				
165	10			Loose - Medium Dense	4	5 4 6	10				
160	15			Medium Dense	5	8 4 8	12				
155	20			Medium Dense	6	5 7 10	17				
	23.0			Augur refusal at 23 feet on shale/siltstone. Backfilled upon completion.							
150	25										

The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line

Depth	Comment
-	-
-	-

WDP Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

SOIL BORING: B-8

WDP Architectural & Engineering Solutions

Depth: 23.5'
Drilling Firm: Free State Drilling
Rig Type: -
Tooling: -

Logged By: R. Niber
Date Started: 07/21/2025
Date Completed: 07/21/2025

Surface Elevation: 175'
Station/Offset: - /
Coords Sys: Lat/Lon
Coordinates: ,

Comments Location: Runway 16L-34R - West side, south of Broad Run

Elevation (ft)	Depth (ft)	Water Levels	Graphic Log	Materials Description	Samples: Sample Number	Samples: Blow Counts	Samples: Uncorrected N-Value	Lab: Moisture Content (%)	Lab: Atterberg Limits (LL-PL-PI)	Lab: Percent Passing #200 Sieve	Remarks
				FILL: brown, reddish brown, **Sandy to Silty Clay (CL/CH)**, stiff, moist	1	4 7 6	13				Grass/topsoil 4-6"
	3.0			**FILL**: gray, crushed angular aggregate, little to some fine sand, damp	2	9 10 11	21				MSE Wall,select backfill. Medium dense at top
170	5			Medium Dense	3	4 9 15	24				
165	10			Medium Dense	4	8 13 15	28				
160	15			Medium Dense	5	7 11 11	22				
155	20			Very Dense	6	50/6"					
	23.0			Augur refusal at 23 feet on shale/siltstone. Backfilled upon completion.	7	50/0"					
150	25										

The lines between various strata on the test boring logs represent the approximate strata boundary; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line

Depth	Comment
-	-
-	-

WDP Architectural & Engineering Solutions

Project: MSE Walls Evaluation
Location: Manassas Regional Airport, 10600 Harry J Parrish Blvd, Manassas, VA
Project Number: 25177

Appendix C – Bridge Superstructure Structural Analysis Calculations (Part 1 Runway and Part 2 Taxiway) by Walter P Moore

D01.24004.00 HEF Manassas Bridge Assessments
Bridge Superstructure Analysis PART 1: RUNWAY BRIDGE

Site: Manassas Airport

Client is using bridges with airplanes that are heavier than what the bridges were originally designed for. We want to determine the maximum airplane weight these bridges can handle.

Please note, throughout the following calculation package, blue highlighted values are maximum allowable demands that max out the yellow highlighted demand to capacity ratios.

Testing has been conducted by WPM to determine actual distribution factors for the bridges.

Runway - DF = 0.35 - unless noted otherwise (conservative since 0.22 was determined by Load Test)

Codes/Guidelines Utilized:
AASHTO LRFD 8th Edition

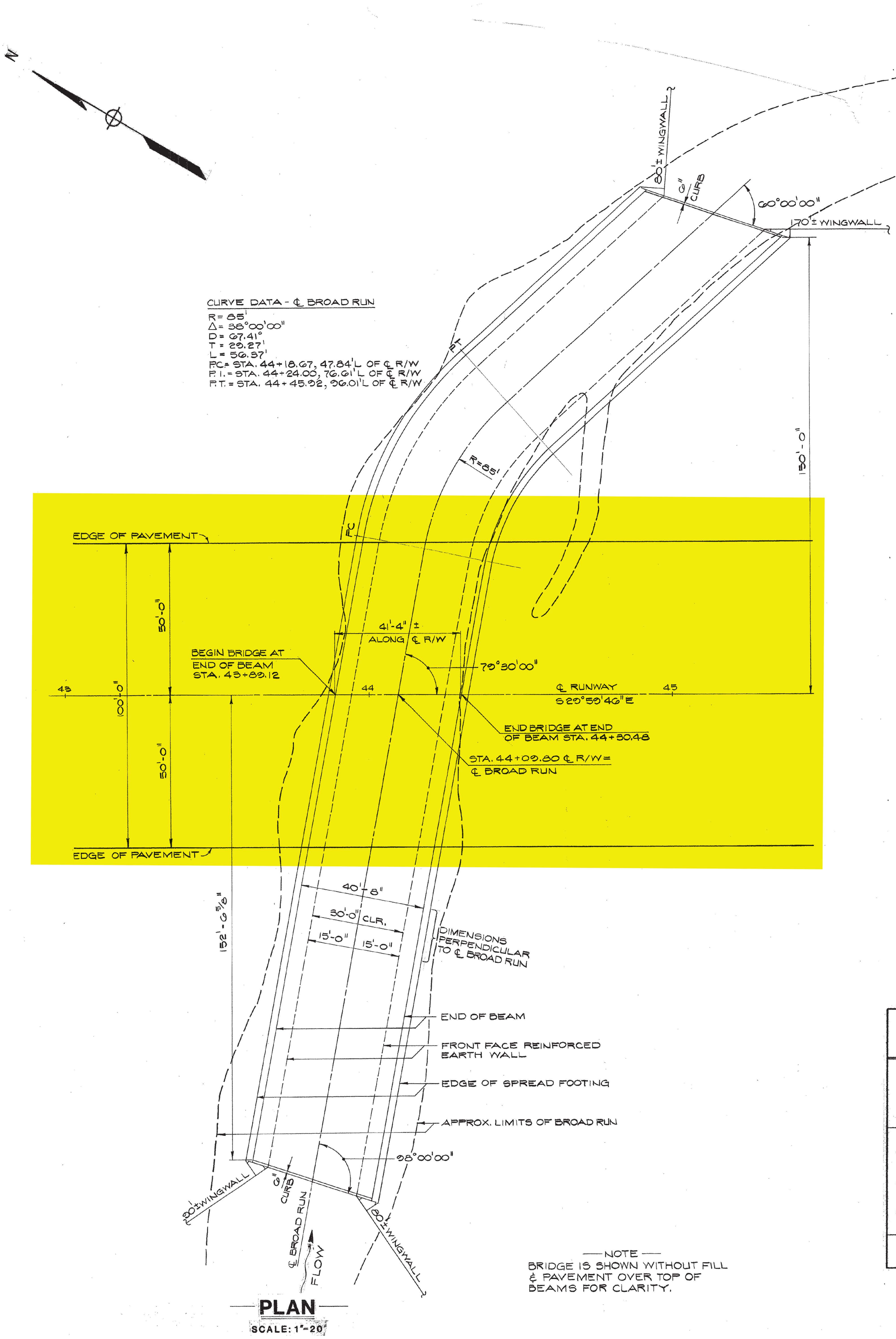
RUNWAY BRIDGE - MAX AIRPLANE WEIGHT BASED ON FOLLOWING CRITERIA:

- Box Lid Failure
- - Moment: 747 kip
- - Shear: 452 kip

- Box Girder - PGSuper Analysis
- - Design Limit State: 223 kip
- - The analysis does indicate that the existing structure does not meet current AASHTO shear reinforcing steel detailing requirements at the ends of the girder. The stirrups in the box girder are #6 @ 12" for almost the entire length of the member. 10" on each end, the two sets of stirrups are spaced 4" apart. The box girder has adequate shear capacity, regardless.

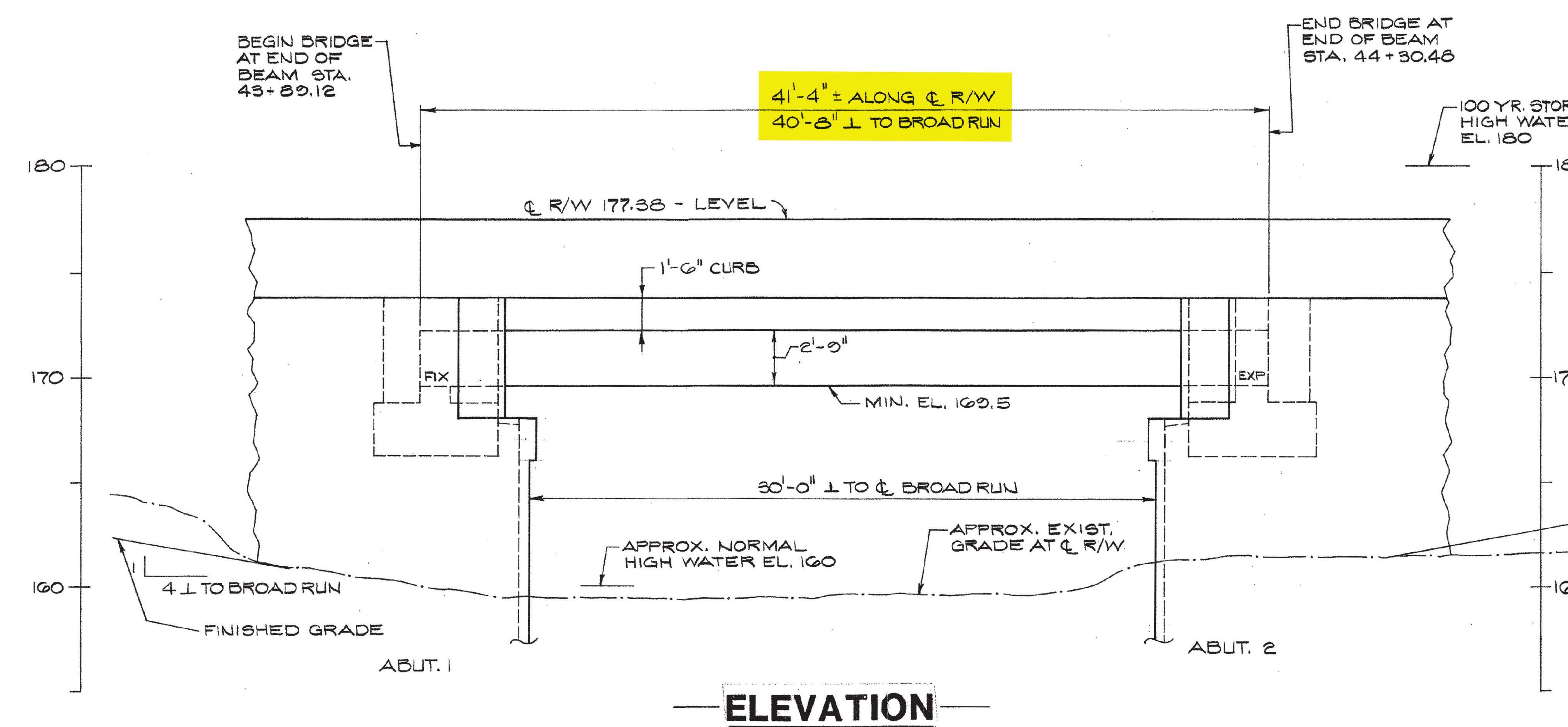
- Foundation (Bearing Controlling): 384 kip

- Bearing Pads
- - Bearing pad thickness are inadequate per code however over 40 years of adequate in situ performance indicates in situ performance is adequate.
- - Checks done:
- - - Service Level Compression Stress: 151 kip
- - - Deflection Check: PASS
- - - Shear Deformation Check: FAIL. Bearing pads fail due to shear deformation even WITHOUT any plane load.
- - - Stability Check: PASS



PLAN

SCALE: 1"=20'



ELEVATION

SCALE: 1"=5'

(SHOWN ⊥ TO BROAD RUN)

ESTIMATED QUANTITIES

	CAST-IN-PLACE CONCRETE C.Y.	PRESTRESSED CONCRETE BOX BEAMS (4'-0"x2'-9") EACH
ABUTMENT 1	251.8	———
ABUTMENT 2	254.7	———
SUPERSTRUCTURE	2.4	86
TOTAL	508.9	86

GENERAL NOTES

DESIGN SPECIFICATIONS:	AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1977, INCLUDING INTERIM SPECIFICATIONS, 1978, 1979, 1980, 1981 & 1982.
LOADING:	737-200 AIRCRAFT LOADING. TOTAL GROSS LOAD 108,000 POUNDS - DUAL GEAR, 48,500 POUNDS EACH.
CONCRETE:	CONCRETE IN PRESTRESSED MEMBERS - f'c = 5000 PSI. CONCRETE IN SUBSTRUCTURE - f'c = 3000 PSI.
REINFORCING STEEL:	REINFORCING STEEL SHALL CONFORM TO ASTM A-615, GRADE 60. ALL REINFORCING BAR DIMENSIONS ON THE DETAILED DRAWINGS ARE TO CENTERS OF BARS EXCEPT WHERE OTHERWISE NOTED.
FOUNDATIONS:	FOOTINGS FOR ABUTMENTS SHALL REST ON FIRM MATERIAL. BEARING CAPACITY OF FOUNDATION SHALL BE 3500 PSF.
CHAMFER:	ALL EXPOSED CORNERS OF CONCRETE SHALL BE CHAMFERED WITH 3/4" x 3/4" MILLED CHAMFER STRIPS.

COMMONWEALTH OF VIRGINIA
LARRY L. ICHTER
LICENSE No. 11122
PROFESSIONAL ENGINEER
Larry L. Ichter 6/27/83

HDR
Henningson, Durham & Richardson, Inc.
Engineers • Architects • Planners
Alexandria, Virginia

DESIGNED BY:	DRAWN BY:	CHECKED BY:	APPROVED BY:
LLI	JCL	LLI	HFR

DATE: JULY 1983 | SCALE: AS NOTED | CMP PROJ. NO. CMP 8305

NO.	DATE	REVISIONS	BY	APPR.

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

RUNWAY BRIDGE OVER BROAD RUN
PLAN & ELEVATION

PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
18 OF 41

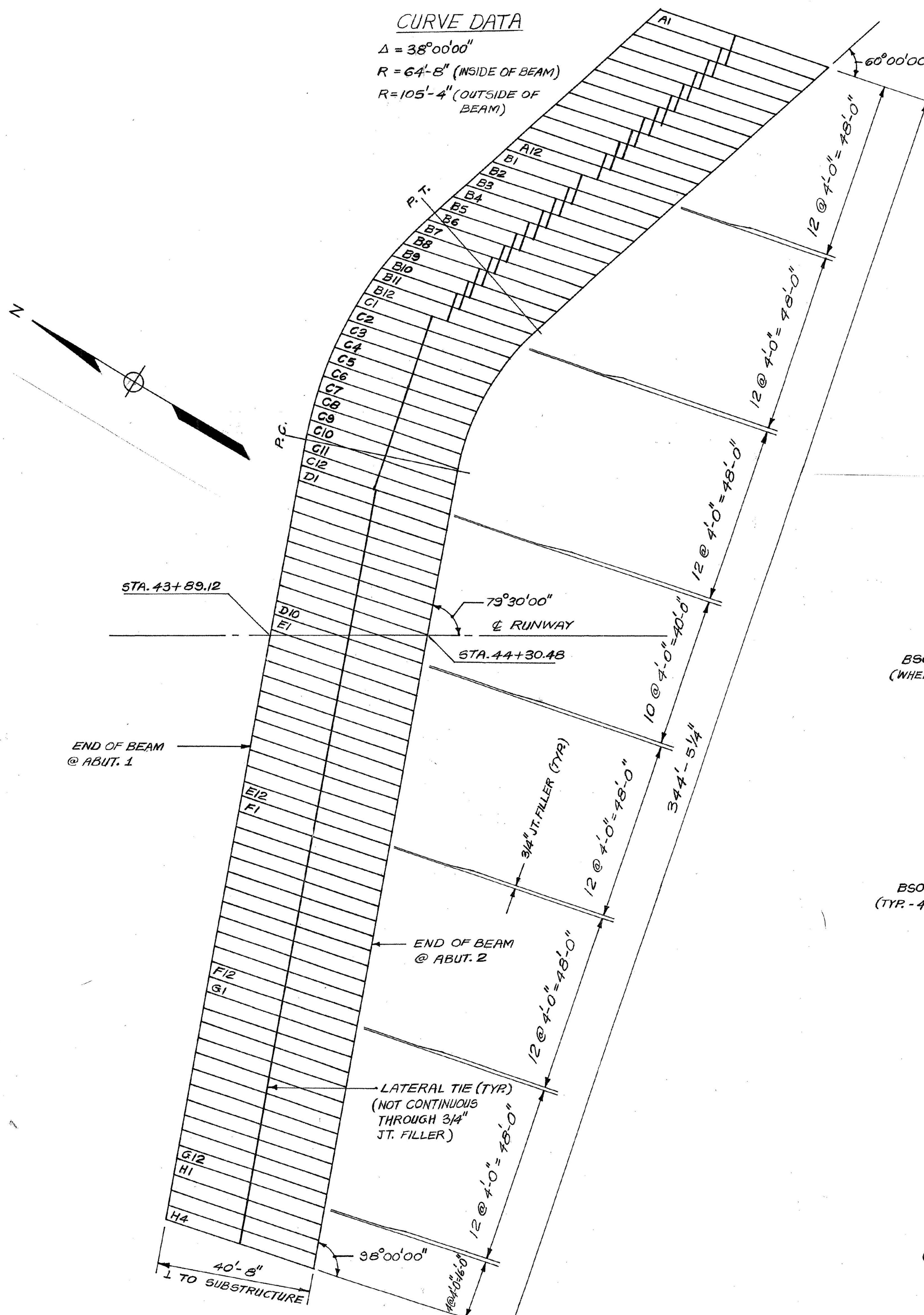
K&E 19 1233 4-83 A1318*

⚠ BEAM SCHEDULE

BEAM NO.	LENGTH		ABUTMENT 1 END						ABUTMENT 2 END						REINF. SPACING			LONG. REINF.	BS0656
	D1	D2	A1	B1	C1	E1	F1	Θ1	A2	B2 *	C2	E2 *	F2 *	Θ2 *	A	N1	N2		
A1 TO B5	46'-11 1/2"	46'-11 1/2"	1'-6"	(2'-3 11/16")	23'-5 3/4	0'-10 3/8"	3'-9 11/16	120°-00'-00"	1'-6"	(2'-3 11/16")	23'-5 3/4	0'-10 3/8	3'-9 11/16	120°-00'-00"	2'-8"	44	43	BL0454	EACH END
B6	46'-11 5/16"	46'-11 1/2"	1'-6"	(2'-3 9/16")	23'-5 11/16"	0'-10 3/8"	3'-9 9/16"	119°-50'-29"	1'-6"	(2'-3 11/16")	23'-5 11/16"	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-8"	44	43	"	"
B7	46'-9"	46'-11 5/16"	1'-6"	(2'-1 3/8")	23'-4 1/2"	0'-10 3/16"	3'-7 3/8"	117°-52'-33"	1'-6"	(2'-3 11/16")	23'-4 1/2"	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-5"	44	43	BL0452,453	"
B8	46'-4 1/8"	46'-9"	1'-6"	(1'-10 13/16")	23'-2 1/16"	0'-9 15/16"	3'-4 13/16"	115°-26'-25"	1'-6"	(2'-3 11/16")	23'-2 1/16"	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-2"	44	43	"	"
B9	45'-8 13/16"	46'-4 1/8"	1'-6"	(1'-8 7/16")	22'-10 7/16"	0'-9 3/4"	3'-2 7/16"	113°-03'-11"	1'-6"	(2'-3 11/16")	22'-10 7/16"	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-0"	43	43	"	"
B10 *	44'-11 1/4"	45'-8 13/16"	1'-6"	(1'-6 1/8")	22'-5 5/8"	0'-9 5/8"	3'-0 1/8"	110°-42'-27"	1'-6"	(2'-3 11/16")*	22'-5 5/8"	0'-10 3/8"	3'-9 11/16"*	120° 00'00" *	2'-1"	42	42	"	"
B11 *	43'-11 11/16"	44'-11 1/4"	1'-6"	(1'-3 15/16")	21'-11 13/16	0'-9 1/2"	2'-9 15/16"	108°-23'-52"	1'-6"	(2'-3 9/16")*	21'-11 13/16"	0'-10 3/8"*	3'-9 9/16"*	119°50'54" *	1'-9"	42	41	"	"
B12 *	43'-1 3/16"	43'-11 11/16"	1'-6"	(1'-1 7/8")	21'-6 5/8"	0'-9 3/8"	2'-7 7/8"	106°-07'-08"	1'-6"	(2'-0 3/8")*	21'-6 5/8"	0'-10 1/16"*	3'-6 3/8"*	116°53'53" *	1'-9"	41	40	"	"
C1	42'-4 9/16"	43'-1 1/16"	1'-6"	(0'-11 13/16")	19'-3 3/8"	0'-9 1/4"	2'-5 13/16"	103°-49'-50"	1'-6"	(1'-8 5/8")	23'-1 3/16"	0'-9 3/4"	3'-2 5/8"	112°-55'-31"	1'-7"	40	40	"	"
C2	41'-9 3/4"	42'-4 9/16"	1'-6"	(0'-9 7/8")	20'-1 1/4"	0'-9 3/16"	2'-3 7/8"	101°-35'-59"	1'-6"	(1'-4 5/8")	21'-8 9/16"	0'-9 1/2"	2'-10 5/8"	109°-07'-30"	1'-6"	40	40	"	"
C3	41'-4 1/2"	43'-9 3/4"	1'-6"	(0'-7 15/16")	20'-9 3/16"	0'-9 1/8"	2'-1 15/16"	99°-23'-10"	1'-6"	(1'-1 1/4")	20'-7 5/16"	0'-9 5/16"	2'-7 1/4"	105°-24'-39"	1'-7"	39	38	"	"
C4	41'-0 9/16"	41'-4 1/2"	1'-6"	(0'-6 1/16")	21'-3 3/16"	0'-9 1/16"	2'-0 1/16"	97°-11'-13"	1'-6"	(0'-10")	19'-9 5/16"	0'-9 3/16"	2'-4"	101°-45'-43"	1'-4"	39	38	"	"
C5	40'-9 7/8"	41'-0 9/16"	1'-6"	(0'-4 3/16")	21'-7 7/16"	0'-9 1/16"	1'-10 3/16"	94°-59'-54"	1'-6"	(0'-6 7/8")	19'-2 7/16"	0'-9 1/16"	2'-0 7/8"	98°-09'-40"	1'-4"	38	38	"	ABUT.2 END
C6	40'-8 3/8"	40'-9 7/8"	1'-6"	(0'-2 3/8")	21'-9 3/4"	0'-9"	1'-8 3/8"	92°-49'-01"	1'-6"	(0'-3 7/8")	18'-10 9/16"	0'-9"	1'-9 7/8"	94°-35'-34"	1'-2"	38	38	BL0452	—
C7	40'-8"	40'-8 3/8"	1'-6"	(0'-0 9/16")	21'-10 9/16"	0'-9"	1'-6 9/16"	90°-38'-22"	1'-6"	(0'-0 7/8")	18'-9 11/16"	0'-9"	1'-6 7/8"	91°-02'-31"	1'-1"	38	38	"	—
C8	40'-8 13/16"	40'-8"	1'-7 5/16"	0'-1 5/16"	21'-9"	0'-9"	1'-6"	88°-27'-47"	1'-8 1/8"	0'-2 1/8"	18'-11 13/16"	0'-9"	1'-6"	87°-29'-44"	1'-1"	38	38	"	—
C9	40'-10 13/16"	40'-8 13/16"	1'-9 1/8"	0'-3 1/8"	21'-5 7/8"	0'-9"	1'-6"	86°-17'-04"	1'-11 1/8"	0'-5 1/8"	19'-4 7/8"	0'-9 1/16"	1'-6"	83°-56'-21"	1'-0"	38	38	"	—
C10	41'-0 5/8"	40'-10 13/16"	1'-10 15/16"	0'-4 15/16"	21'-1 1/8"	0'-9 1/16"	1'-6"	84°-06'-02"	2'-0 3/4"	0'-6 3/4"	19'-11 5/8"	0'-9 1/16"	1'-6"	82°-00'-13"	0'-10"	38	38	BL0451	—
C11	41'-0 13/16"	41'-0 5/8"	2'-0 9/16"	0'-6 9/16"	20'-6 3/8"	0'-9 1/16"	1'-6"	82°-13'-51"	2'-0 3/4"	0'-6 3/4"	20'-6 7/8"	0'-9 1/16"	1'-6"	82°-00'-00"	0'-9"	39	38	"	—
C12	41'-0 13/16"	41'-0 13/16"	2'-0 9/16"	0'-6 9/16"	19'-11 5/8"	0'-9 1/16"	1'-6"	82°-00'-00"	2'-0 3/4"	0'-6 3/4"	21'-1 1/8"	0'-9 1/16"	1'-6"	82°-00'-00"	0'-9"	39	38	"	—
D1 TO H4	41'-0 13/16"	41'-0 13/16"	2'-0 9/16"	0'-6 9/16"	20'-6 3/8"	0'-9 1/16"	1'-6"	82°-00'-00"	2'-0 3/4"	0'-6 3/4"	20'-6 3/8"	0'-9 1/16"	1'-6"	82°-00'-00"	0'-9"	39	38	"	—

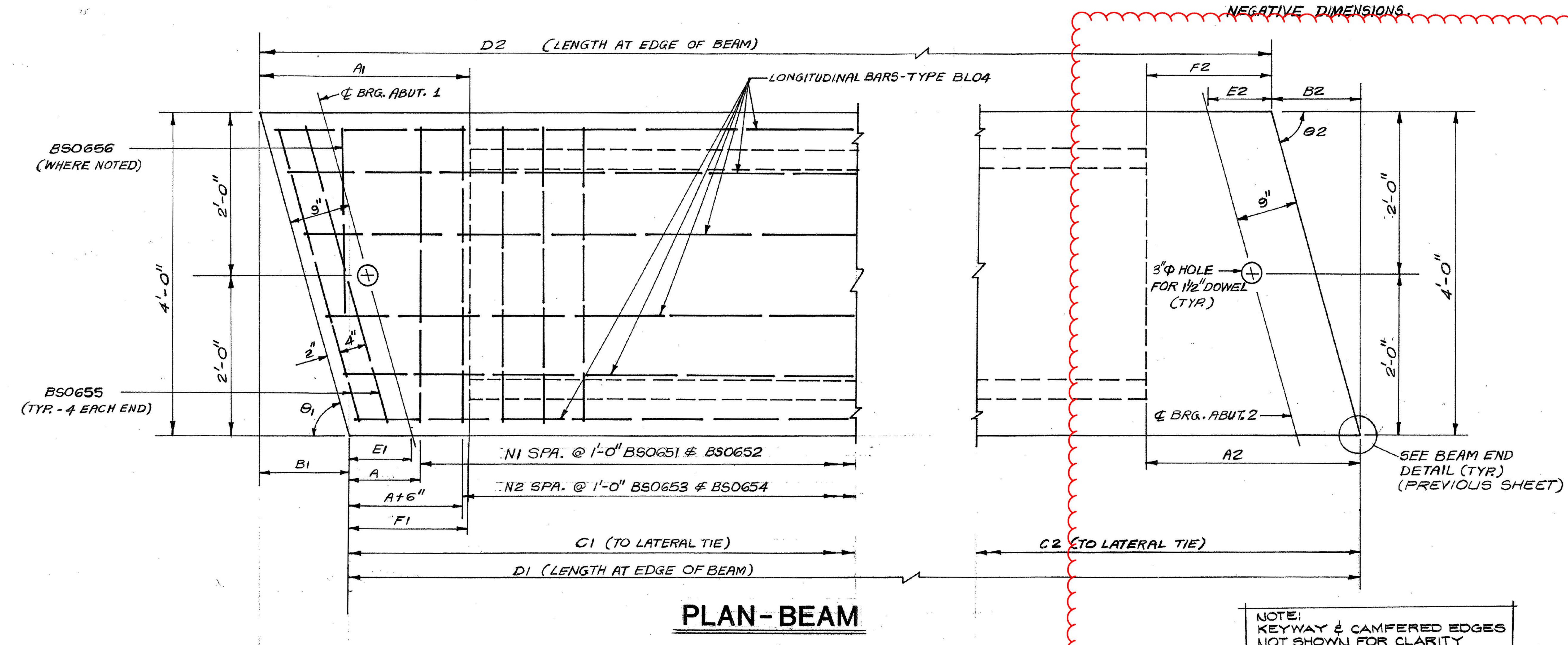
*INDICATES DIMENSION CHANGES

NOTE: DIMENSIONS IN ( ) DENOTE NEGATIVE DIMENSIONS.



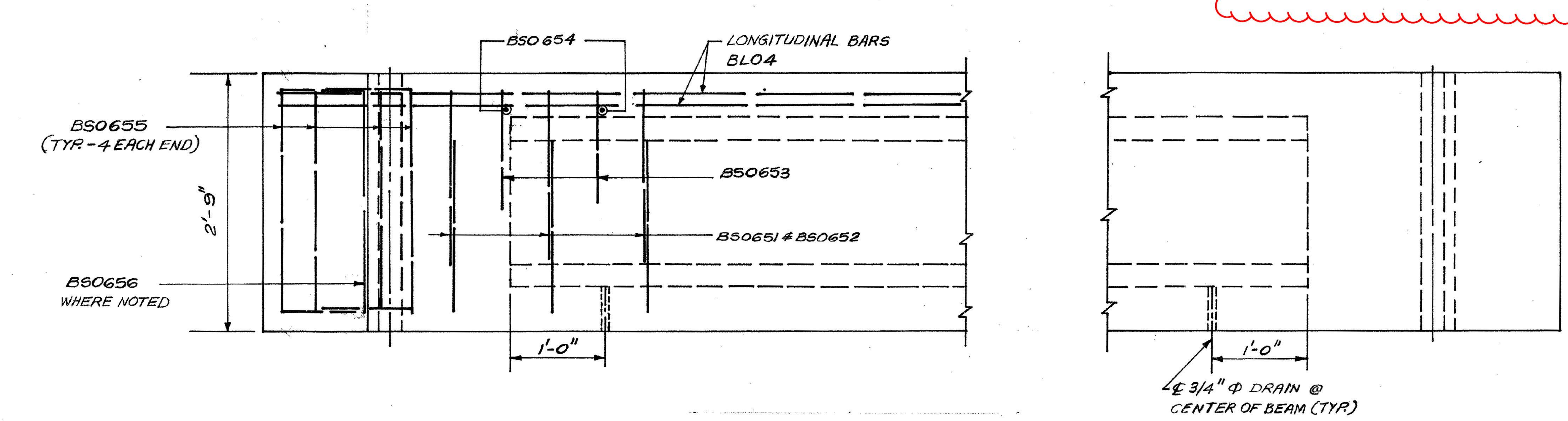
FRAMING PLAN

SCALE: 1"=20'



PLAN-BEAM

SCALE: 1"=1'-0"



ELEVATION-BEAM

SCALE: 1"=1'-0"

COMMONWEALTH OF VIRGINIA
LARRY L. ICHTER
LICENSE No. 11122
PROFESSIONAL ENGINEER

Larry L. Ichter 6/27/83

HDR Henningson, Durham & Richardson, Inc.
Engineers • Architects • Planners
Alexandria, Virginia

DESIGNED BY: LLI	DRAWN BY: UJS	CHECKED BY: LLI	APPROVED BY: HFR
DATE: JULY 1983	SCALE: AS NOTED		CMP PROJ. NO. CMP 8305

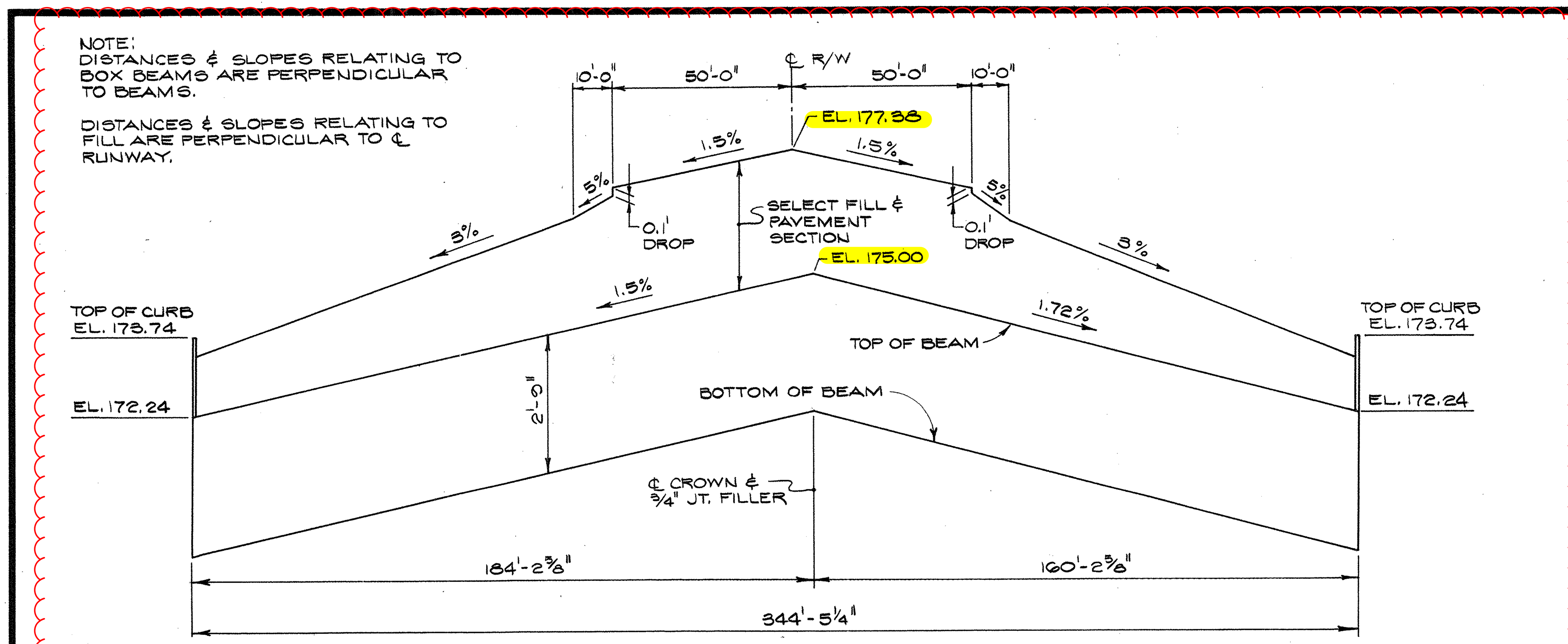
NO.	DATE	REVISIONS	BY	APPR.
⚠	5/87	Corr. To Bm. Sch.	REM	KHG

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

RUNWAY BRIDGE OVER BROAD RUN
FRAMING PLAN

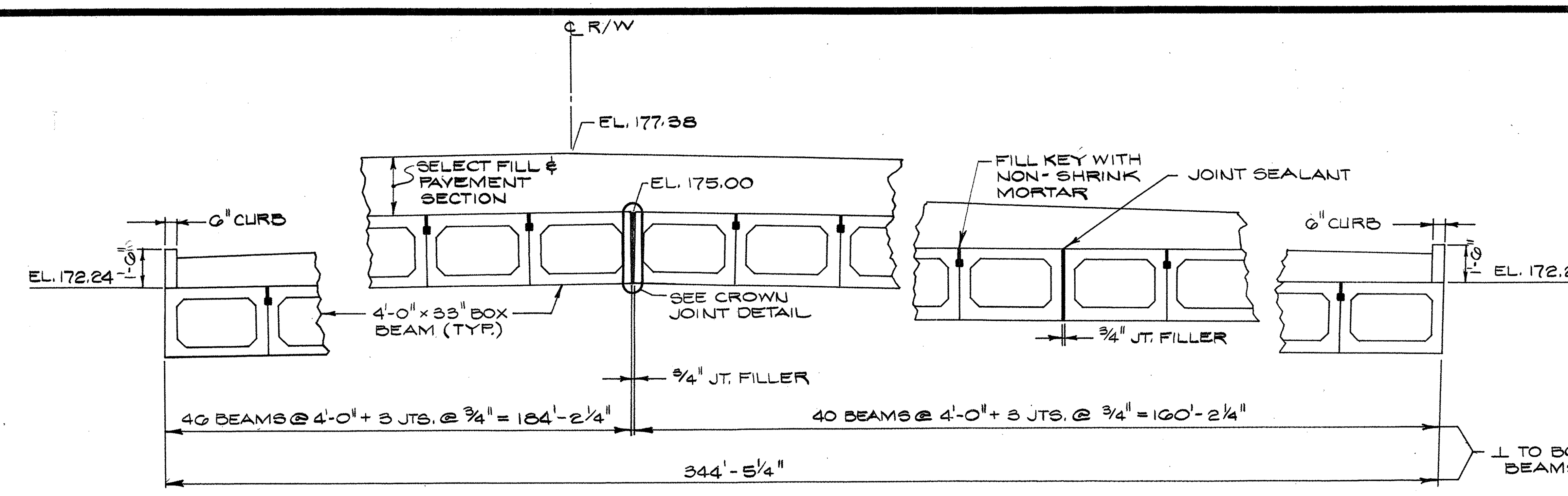
PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
25 OF 41



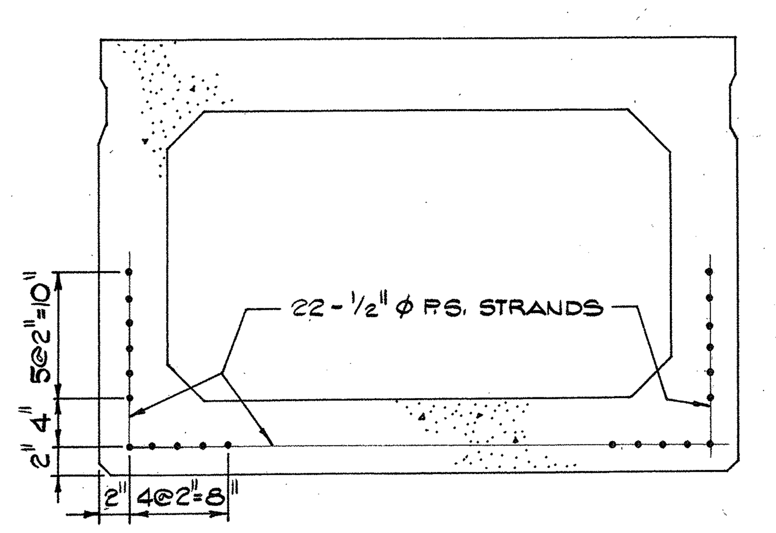
CROSS SLOPE OF TYPICAL SECTION

SCALE: 1"=30' HORIZ.
1"=2' VERT.



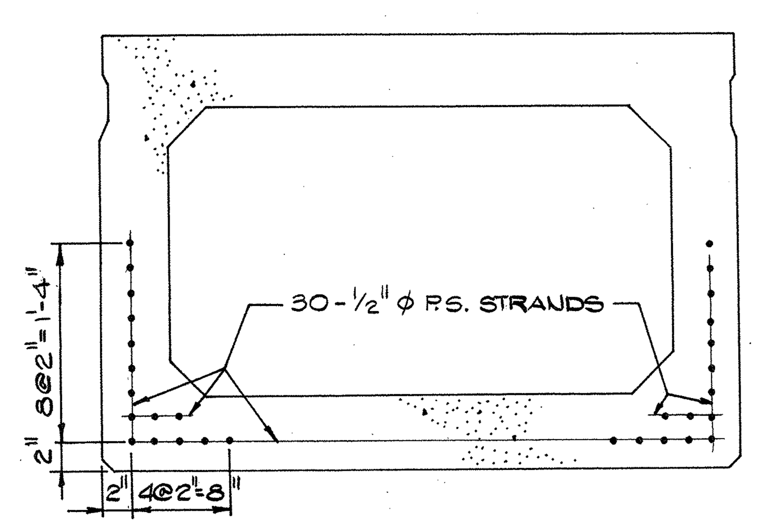
PART TYPICAL SECTION

SCALE: 1/4"=1'-0"



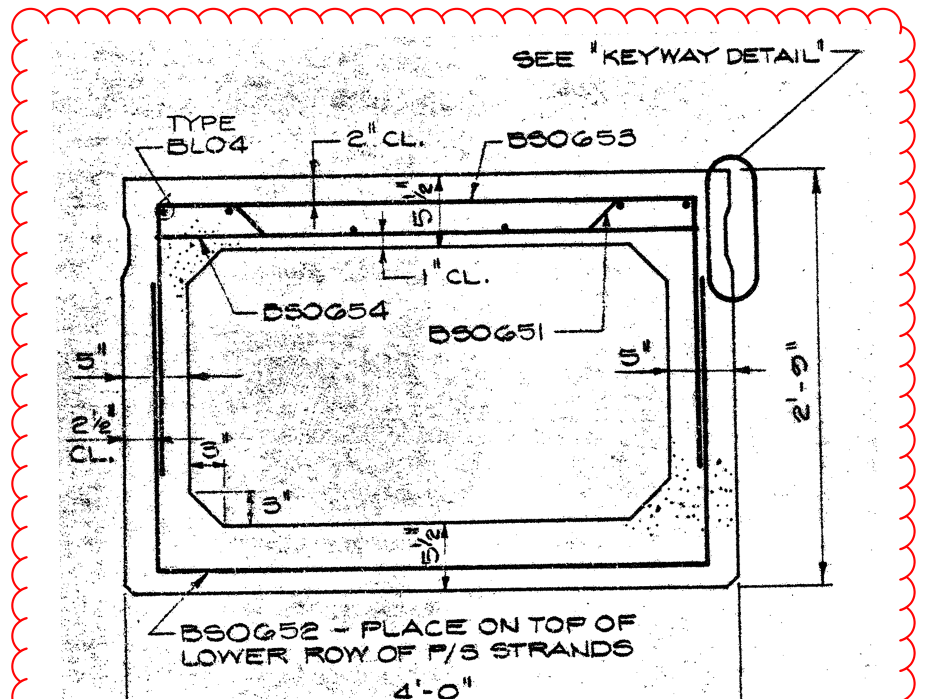
STRAND PATTERN

SCALE: 1"=1'-0"
BEAMS C1 THROUGH H4
RUNWAY LANDS ON BEAMS WITH THIS STRAND PATTERN



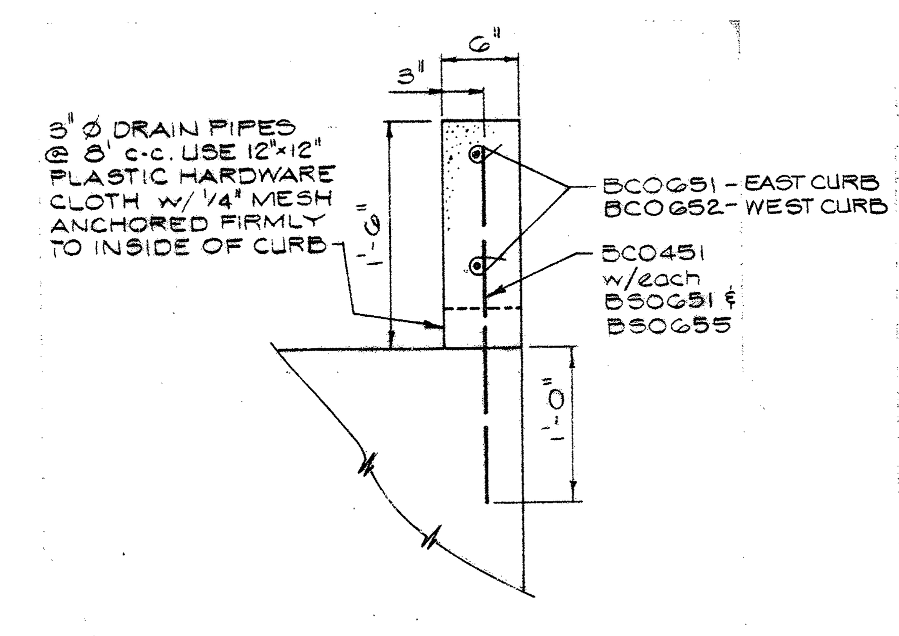
STRAND PATTERN

SCALE: 1"=1'-0"
BEAMS A1 THROUGH B12



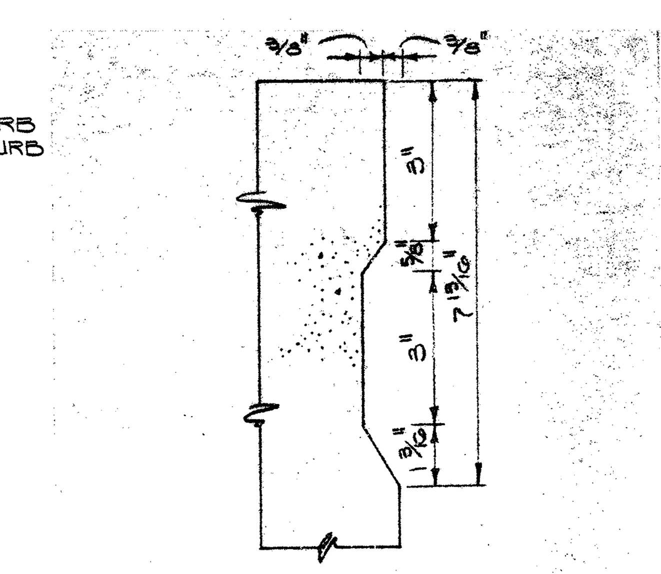
TYPICAL SECTION

SCALE: 1"=1'-0"
(INTERIOR BEAM SHOWN)



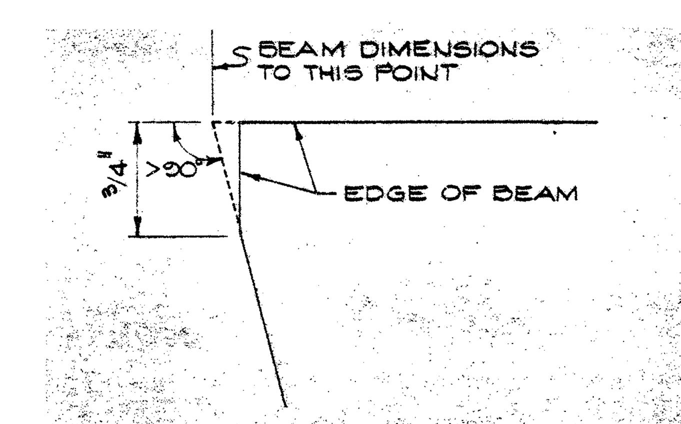
CURB DETAIL

(EXTERIOR BEAMS)
SCALE: 1"=1'-0"



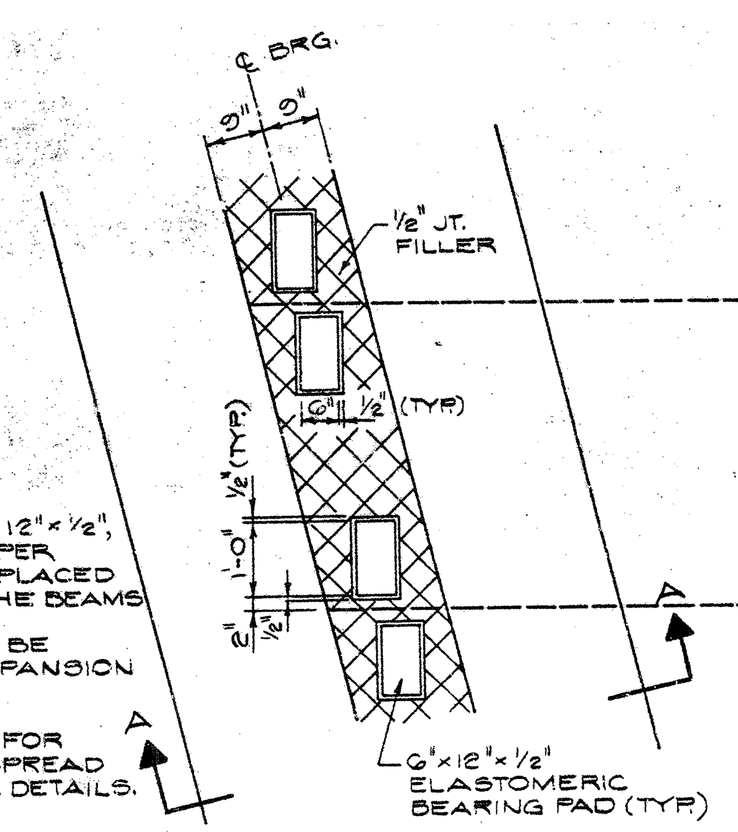
KEYWAY DETAIL

N. T. S.



BEAM END DETAIL

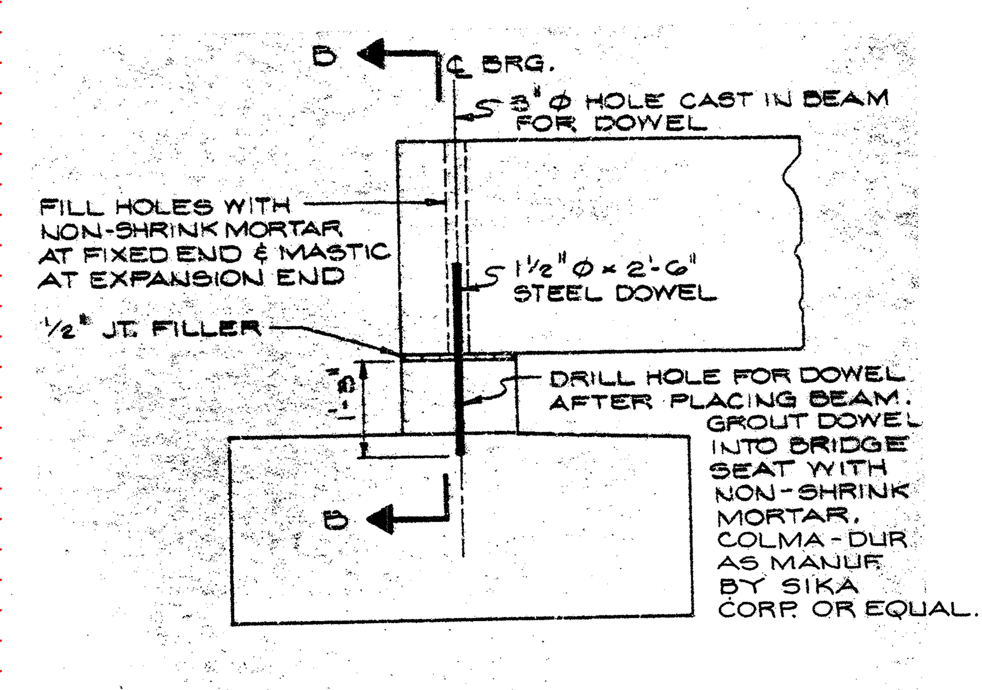
N. T. S.



NOTES:
1. FOUR BEARING PADS 6"x12"x1/2", 70 DUROMETER REQ'D. PER BEAM. BEARINGS ARE PLACED PERPENDICULAR TO THE BEAMS.
2. ALL JT. FILLER SHALL BE PREFORMED CORK EXPANSION JOINT MATERIAL.
3. SEE ABUTMENT SHEET FOR TYPICAL SECTION OF SPREAD FOOTING & WINGWALL DETAILS.

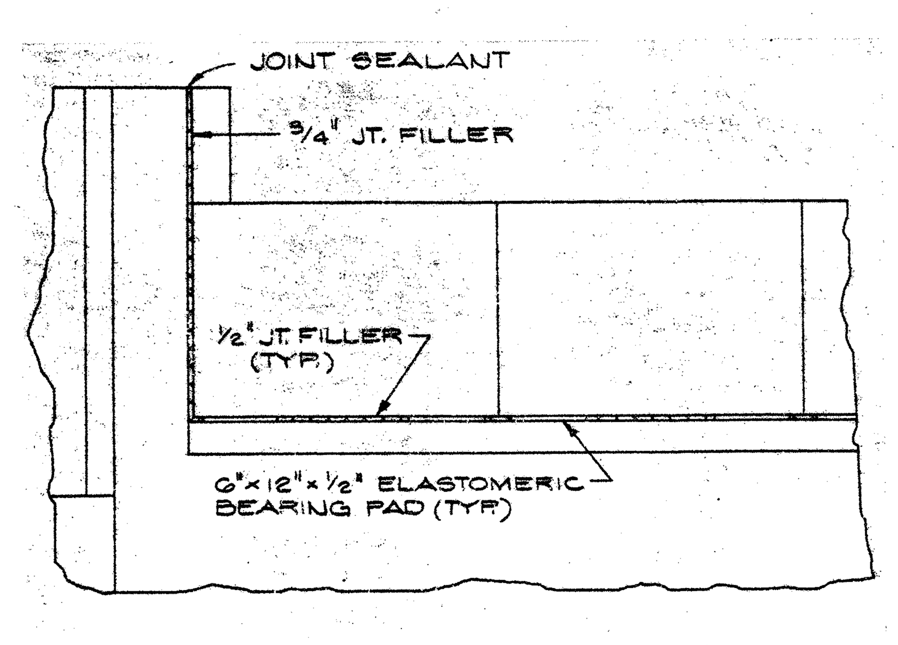
BEARING LAYOUT

SCALE: 1/2"=1'-0"



SECTION A-A

SCALE: 1/2"=1'-0"



SECTION B-B

SCALE: 1/2"=1'-0"

NOTES:

1. PRESTRESSING STRANDS SHALL BE 1/2" ∅ STRESS-RELIEVED STRANDS, GRADE 270. THE INITIAL PRESTRESSING FORCE SHALL BE 28,900 POUNDS PER STRAND.
2. PRESTRESSING STRANDS SHALL NOT BE RELEASED UNTIL THE CONCRETE HAS REACHED A COMPRESSIVE STRENGTH OF 4000 PSI.
3. LATERAL TIES SHALL BE TENSIONED TO 30,000 POUNDS AND MAY BE EITHER 1-1/4" ∅ PLAIN STRUCTURAL RODS OR 1/2" ∅ PRESTRESSING STRANDS AS DESCRIBED IN NOTE 1. ALL LATERIAL TIE POCKET RECESSES SHALL BE FILLED WITH NON-SHRINK MORTAR AFTER LATERAL TIES ARE TENSIONED. LATERIAL TIES SHALL BE TENSIONED PRIOR TO FILLING KEYS WITH NON-SHRINK MORTAR AND PLACING DOWELS.
4. CURBS SHALL BE CAST-IN-PLACE AFTER BEAMS ARE ERECTED.
5. THE VERTICAL 3/4" JOINT FILLER MATERIAL SHALL BE RIGIDLY ATTACHED TO THE SIDE OF ONE OF THE BOX BEAMS USING MECHANICAL FASTENERS OR NAILS.
6. SEE ABUTMENT SHEETS FOR SILICONE JOINT SEALANT DETAILS.
7. SEE LATERAL TIE DETAILS SHEET FOR CROWN JOINT DETAIL.

COMMONWEALTH OF VIRGINIA
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DESIGNED BY: LLI	DRAWN BY: JCL	CHECKED BY: LLI	APPROVED BY: HFR
DATE: JULY 1983	SCALE: AS NOTED	CMP PROJ. NO. CMP 8305	

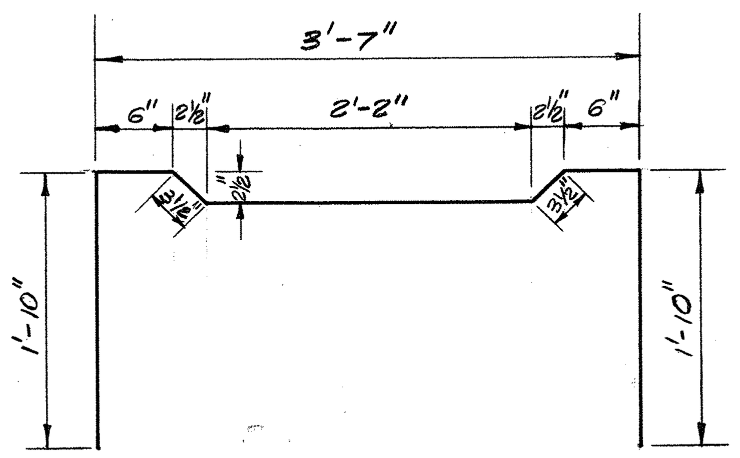
NO.	DATE	REVISIONS	BY	APPR.

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

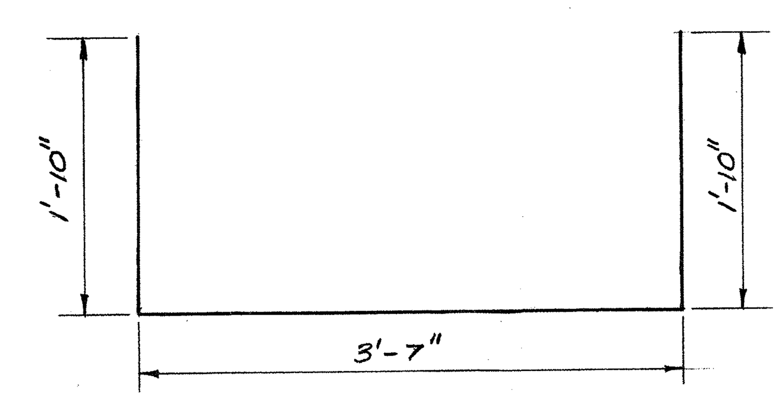
RUNWAY BRIDGE OVER BROAD RUN
TYPICAL SECTION & DETAILS

PROJECT NO. AIP 5-51-0030-01
SHEET NO. 24 OF 41

RUNWAY BRIDGE



BSO601



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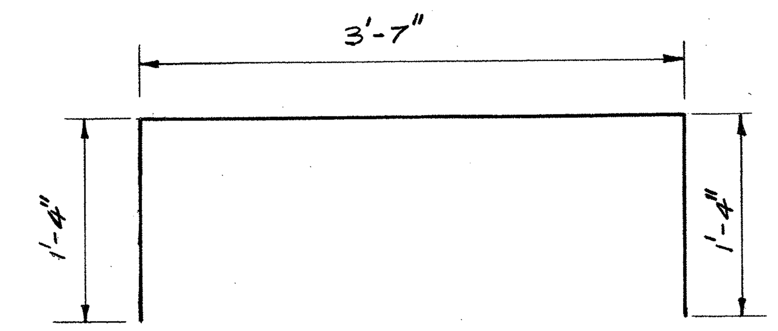
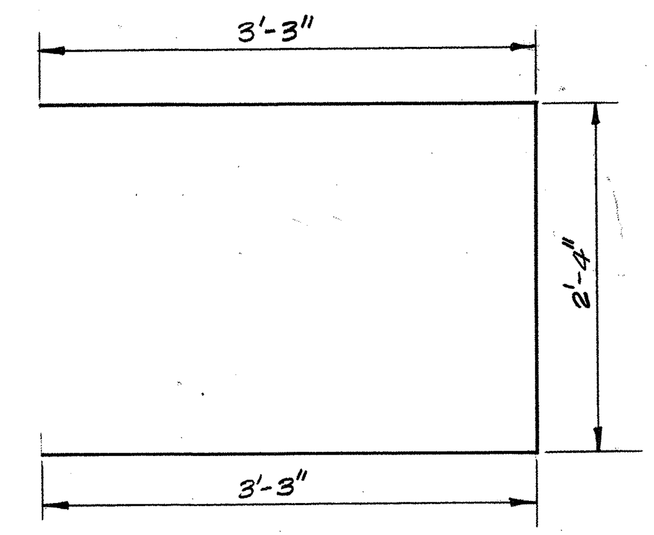
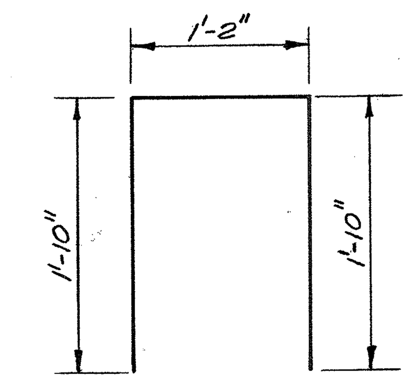


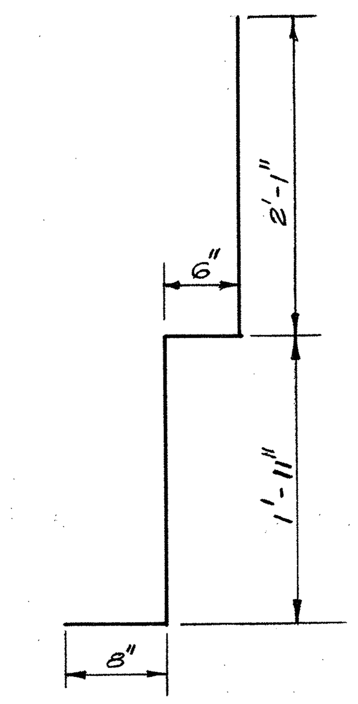
BSO603



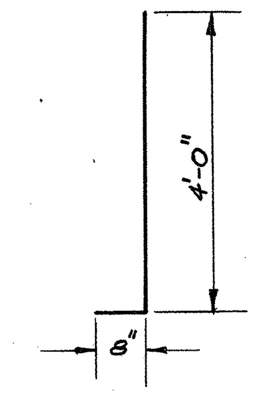
BSO605



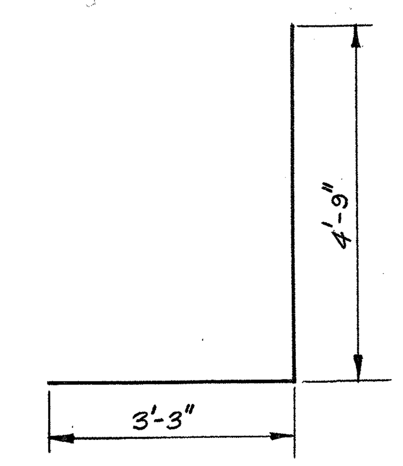
A SO401



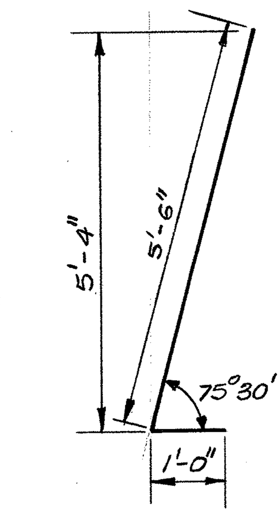
AWO402



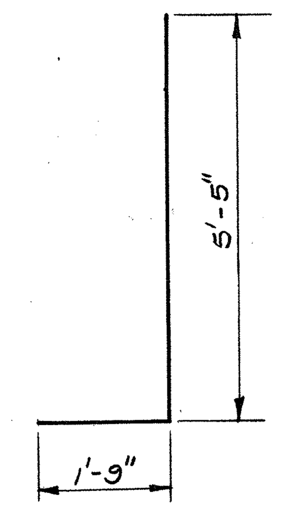
AWO404



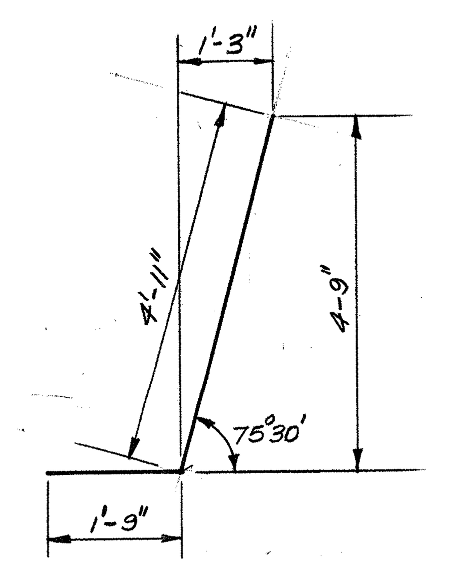
AWO405 & AWO601



AWO406 & AWO602



AWO407 & AWO605



AWO408 & AWO606

REINFORCING STEEL SCHEDULE

MARK	NO.	SIZE	LENGTH FT-IN	WEIGHT LBS.	TYPE STRAIGHT	TYPE BENT	PIN DIA. IN.	REMARKS
SUPERSTRUCTURE - TAXIWAY								
BC0401	88	4	2-4	137	X			CURB
BC0601	4	6	41-6	249	X			CURB
BL0401	138	4	41-8	3,841	X			
BS0601	920	6	7-0	9,673		X	3-3/4	
BS0602	920	6	6-11	9,558		X	4-1/2	
BS0603	920	6	5-11	8,176		X	4-1/2	
BS0604	920	6	3-7	4,952	X			
BS0605	184	6	8-6	2,349		X	4-1/2	END OF BEAMS
SUPERSTRUCTURE TOTAL				38,935				
SUBSTRUCTURE - ABUTMENTS 1 & 2 - TAXIWAY								
AF0401	78	4	35-6	1,850	X			
AF0402	184	4	5-6	676	X			
AF0601	192	6	5-6	1,586	X			
AS0401	184	4	4-8	574		X	2	BEARING SEAT
AS0601	8	6	48-5	582	X			BEARING SEAT
AW0401	58	4	4-3	165	X			WING
AW0402	16	4	4-11	53		X	3	WING
AW0404	42	4	4-7	129		X	3	WING
AW0405	10	4	7-11	53		X	3	WING
AW0406	10	4	6-5	43		X	3	WING
AW0407	10	4	7-1	47		X	3	WING
AW0408	10	4	6-7	44		X	3	WING
AW0601	2	6	7-10	24		X	4-1/2	WING
AW0602	2	6	6-4	19		X	4-1/2	WING
AW0603	2	6	4-6	14	X			WING
AW0604	2	6	2-6	8	X			WING
AW0605	2	6	7-0	21		X	4-1/2	WING
AS0606	2	6	6-7	20		X	4-1/2	WING
AW0607	2	6	3-6	11	X			WING
SUBSTRUCTURE TOTAL				5,919				

NOTES:
1. ALL BAR DIMENSIONS ARE OUT TO OUT.
2. LENGTH OF BARS INCLUDE REDUCTIONS FOR BENDS.

COMMONWEALTH OF VIRGINIA
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Alexandria, Virginia

APPROVED BY: HFR
CMP PROJ. NO. CMP 8305
CHECKED BY: LLI
SCALE: NONE
DRAWN BY: UJS
DESIGNED BY: LLI
DATE: JULY 1983

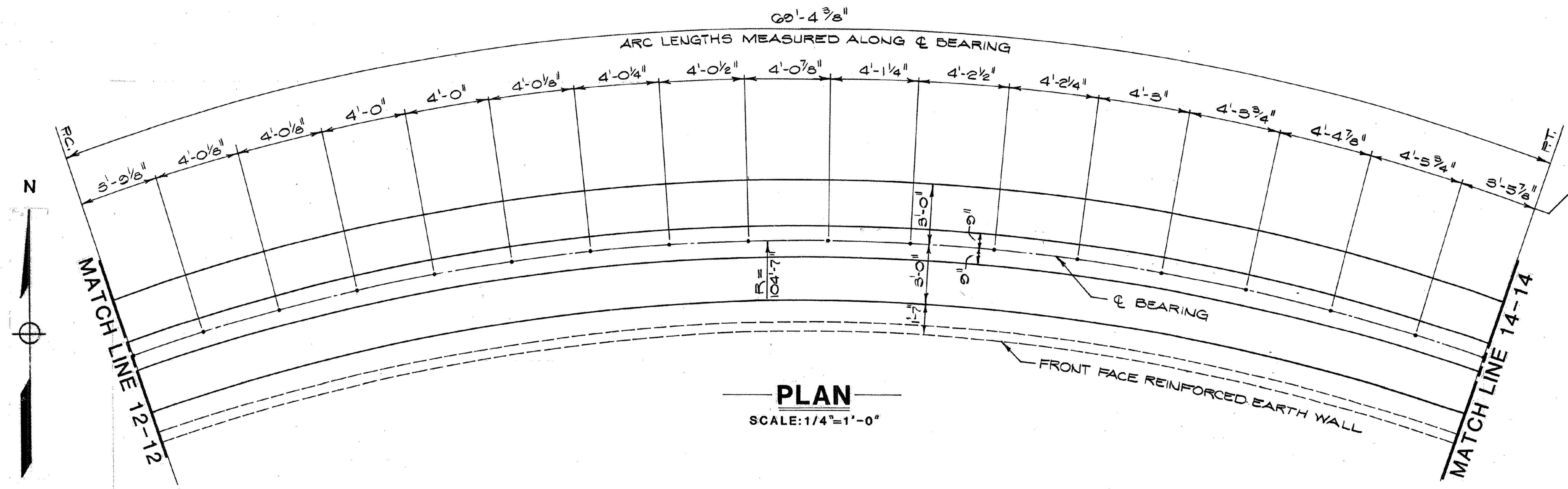
NO.	DATE	REVISIONS	BY	APPR.

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

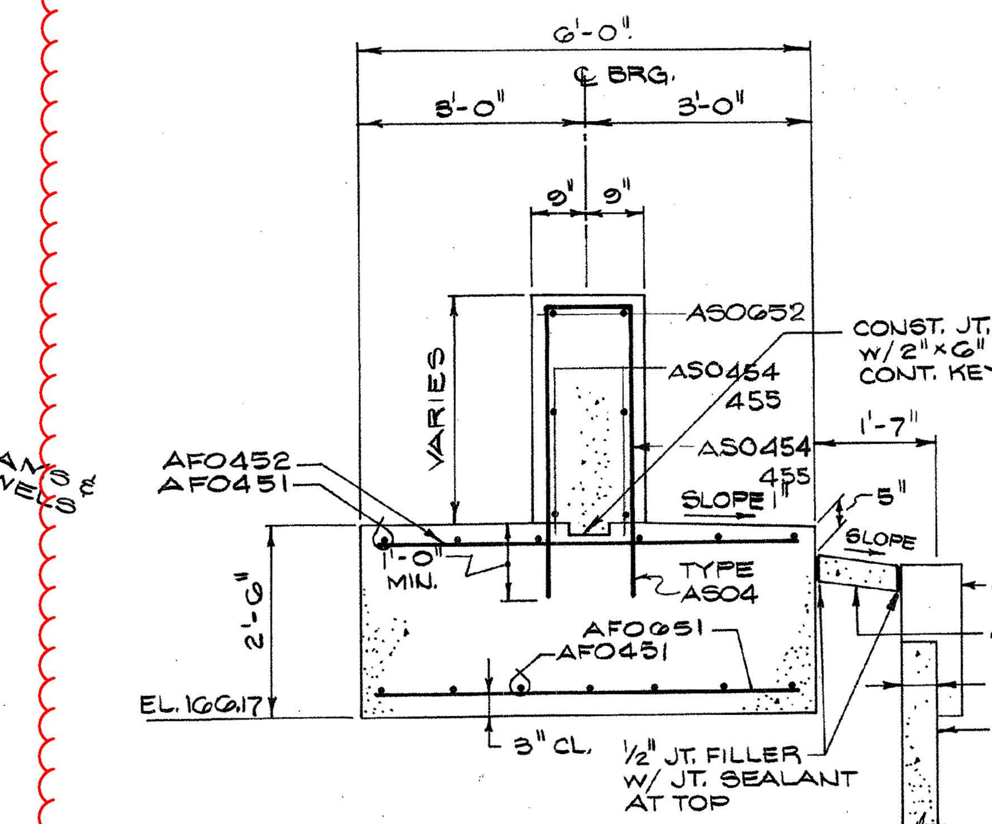
TAXIWAY BRIDGE OVER BROAD RUN
REINFORCING SCHEDULE

PROJECT NO.
AIP 5-51-0030-01
SHEET NO.
33 OF 41

K&E 19 1233 4-83 A1318*



PLAN
SCALE: 1/4"=1'-0"



TYPICAL SECTION
SCALE: 1/2"=1'-0"

NOTES:
1. MAINTAIN 2" CLR. UNLESS NOTED OTHERWISE.
2. PLACE 4" CONC. SLAB AS SOON AS POSSIBLE AFTER PLACING SPREAD FOOTING.
3. PLACE 3/4" JT. FILLER BETWEEN ALL INTERSECTING FACES OF CAST-IN-PLACE CONCRETE & REINFORCED EARTH WALL UNLESS NOTED OTHERWISE.
4. SEE ABUTMENT 2 SHEETS FOR SILICONE JOINT SEALANT DETAILS.
5. SEE TYPICAL SECTION SHEET FOR BEARING DETAILS.

COMMONWEALTH OF VIRGINIA
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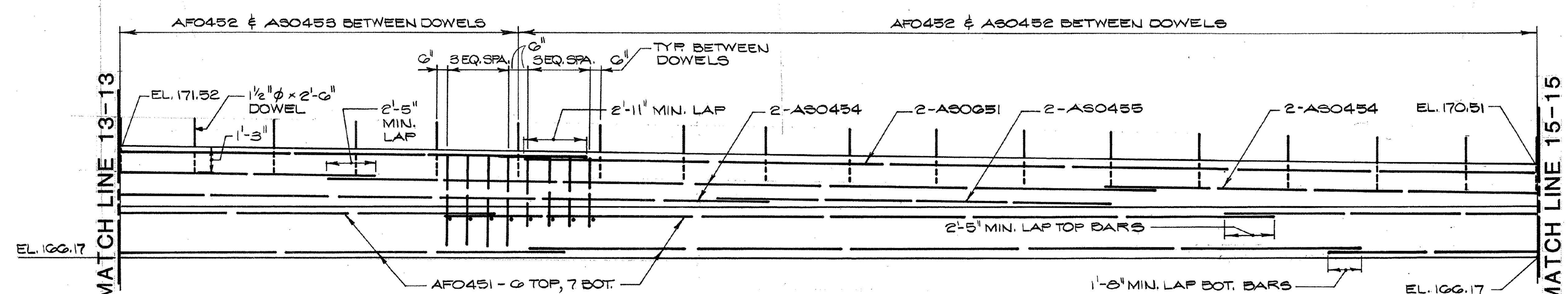
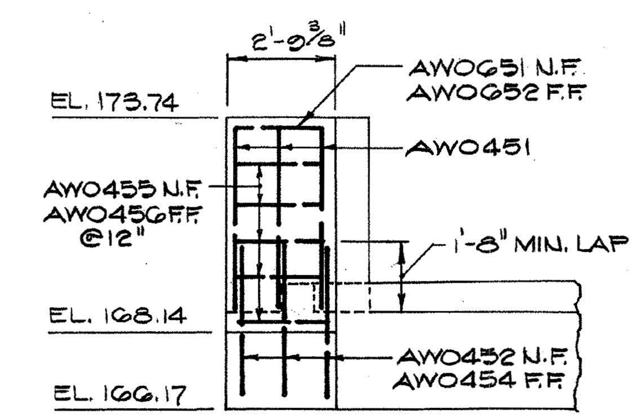
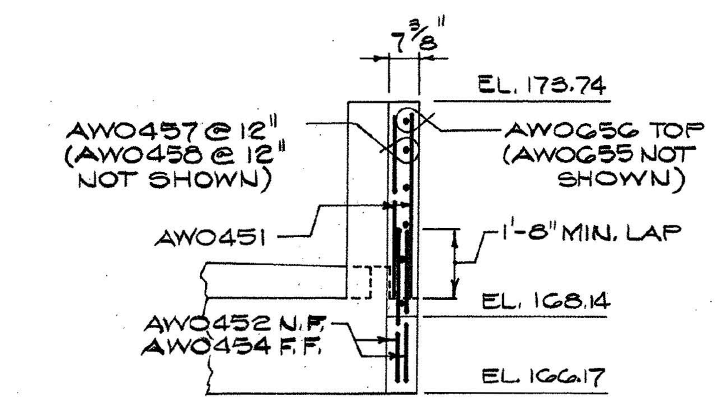


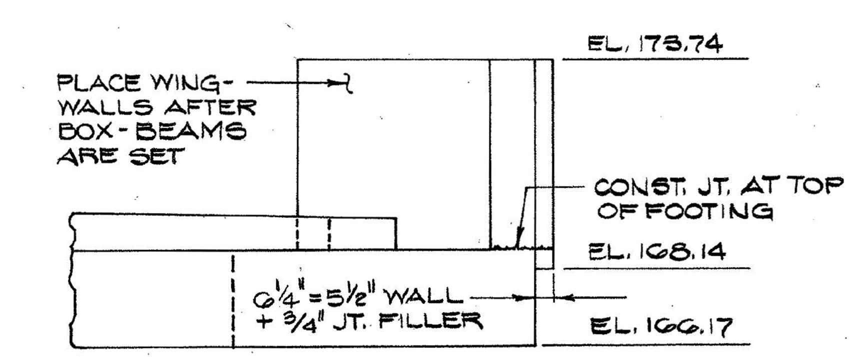
ELEVATION
SCALE: 1/4"=1'-0"



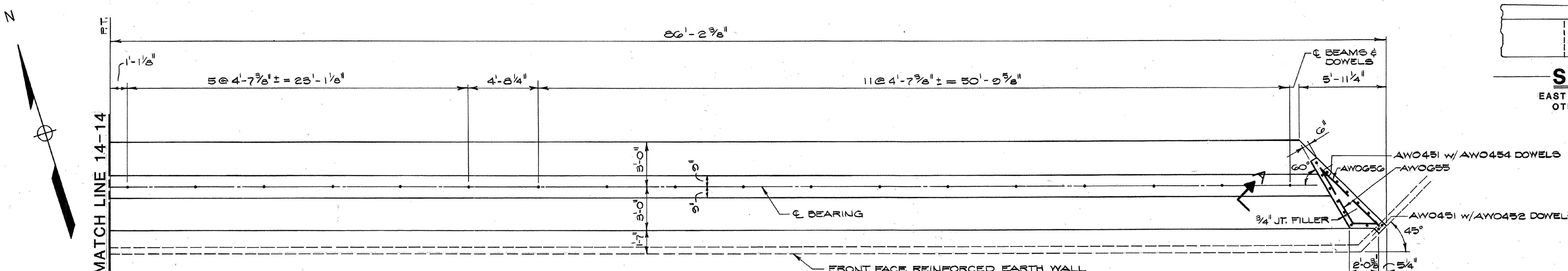
ELEVATION
WEST WINGWALL-ABUT. 1
SCALE: 1/4"=1'-0"



ELEVATION
EAST WINGWALL-ABUT. 1
SCALE: 1/4"=1'-0"

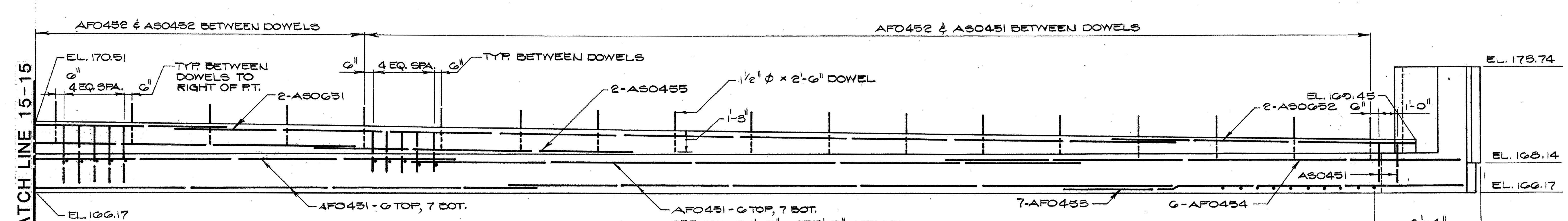


SECTION A-A
EAST WINGWALL ABUT. 1 SHOWN
OTHER WINGWALLS SIMILAR
SCALE: 1/4"=1'-0"



PLAN
SCALE: 1/4"=1'-0"

⚠ LTR. NO 03
RECOMEND EXTEND BM. SEAT 0.8' EAST END



ELEVATION
SCALE: 1/4"=1'-0"

ABUTMENT CONSTRUCTION NOTES
1. CONSTRUCTION JOINTS SHALL BE SPACED AT INTERVALS NOT EXCEEDING 60 FEET. AT LEAST 48 HOURS SHALL ELAPSE BETWEEN PLACING ADJACENT SECTIONS.
2. THE 4" CONCRETE SLAB SHALL BE REINFORCED WITH WELDED WIRE FABRIC — WWF 4×4 — W2.9 × W2.9. THE SLAB SHALL BE PLACED IN SECTIONS NOT EXCEEDING 50 FEET IN LENGTH. AT LEAST 48 HOURS SHALL ELAPSE BETWEEN PLACING ADJACENT SECTIONS. A 1/2" JOINT WITH JOINT SEALANT SHALL SEPARATE ADJACENT SECTIONS. REINFORCING SHALL NOT BE CONTINUOUS THROUGH THESE JOINTS.

HDR Henningson, Durham & Richardson, Inc.
Engineers • Architects • Planners
Alexandria, Virginia

DESIGNED BY:	DRAWN BY:	CHECKED BY:	APPROVED BY:
LLI	JCL	LLI	HFR

DATE:	SCALE:	CMP PROJ. NO.
JULY 1983	AS NOTED	CMP 8305

REVISIONS

NO.	DATE	REVISIONS	BY	APPR.
⚠	5/87	Corr. Bar Mks., Add. Note	REM	KHG

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

RUNWAY BRIDGE OVER BROAD RUN
ABUTMENT 1

PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
21 OF 41

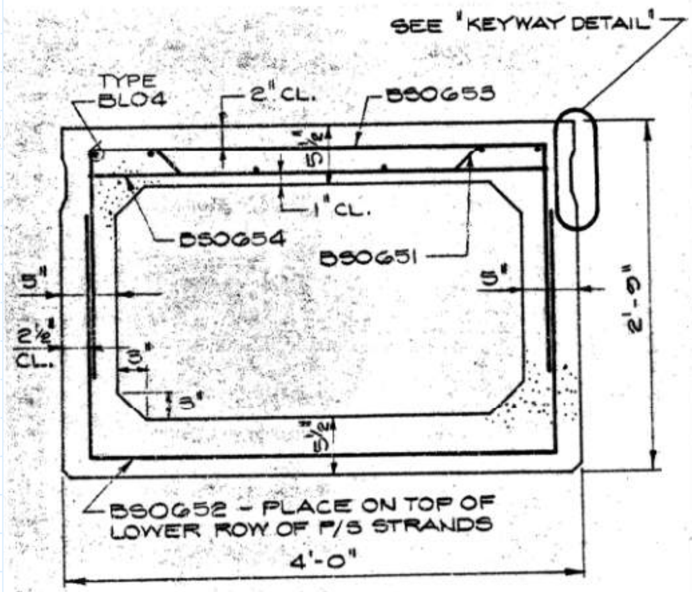
Abutment Check

This section will cover 3 different design checks:

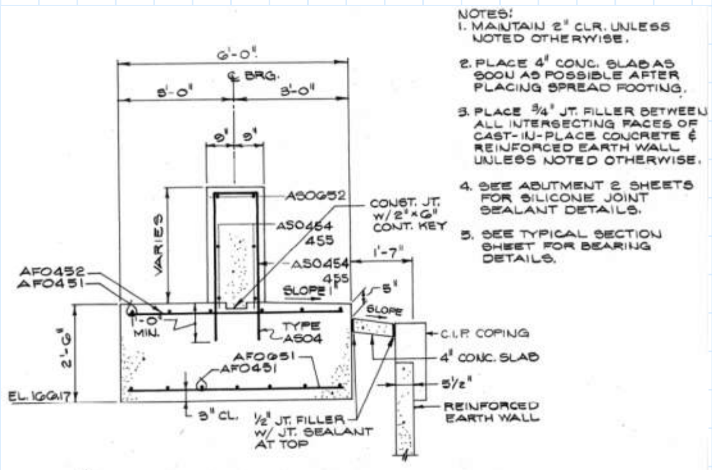
Strip Footing Bearing Load Check
Dowel Connection Check
Bearing Pad Check

GENERAL NOTES

DESIGN SPECIFICATIONS:	AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1977, INCLUDING INTERIM SPECIFICATIONS, 1978, 1979, 1980, 1981 & 1982.
LOADING:	737-200 AIRCRAFT LOADING. TOTAL GROSS LOAD 108,000 POUNDS - DUAL GEAR, 48,500 POUNDS EACH.
CONCRETE:	CONCRETE IN PRESTRESSED MEMBERS - f'c = 5000 PSI. CONCRETE IN SUBSTRUCTURE - f'c = 3000 PSI.
REINFORCING STEEL:	REINFORCING STEEL SHALL CONFORM TO ASTM A-615, GRADE 60. ALL REINFORCING BAR DIMENSIONS ON THE DETAILED DRAWINGS ARE TO CENTERS OF BARS EXCEPT WHERE OTHERWISE NOTED.
FOUNDATIONS:	FOOTINGS FOR ABUTMENTS SHALL REST ON FIRM MATERIAL. BEARING CAPACITY OF FOUNDATION SHALL BE 3500 PSF.
CHAMFER:	ALL EXPOSED CORNERS OF CONCRETE SHALL BE CHAMFERED WITH 3/4" x 3/4" MILLED CHAMFER STRIPS.



TYPICAL SECTION
SCALE: 1"= 1'-0"
(INTERIOR BEAM SHOWN)



TYPICAL SECTION
SCALE: 1/2"=1'-0"

Site Geometry

Bridge Span (box beam total length)	$L_{span} := 40\ ft + 8\ in$
Fill and Pavement Height	$h_{pf} := 3.13\ ft$
Box Beam Width	$b_{box} := 4\ ft$
Box Beam Height	$h_{box} := 2\ ft + 9\ in$
Box Beam Area (cross sectional area)	$A_{box} := 766\ in^2$
Footing Thickness	$h_{footing} := 2\ ft + 6\ in$
Footing Width	$b_{footing} := 6\ ft$
Max Stem Height	$h_{stemmax} := 172\ ft + 3\ in - (166\ ft + 2\ in) - h_{footing} = 43\ in$
Stem Height at Edge of Runway	$h_{stemmin} := h_{stemmax} - 0.015 \cdot 50\ ft = 34\ in$
Average Stem Height	$h_{stemavg} := \text{mean}\left(h_{stemmax}, h_{stemmin}\right) = 38.5\ in$

Stem Width $b_{stem} := 18\ in$

Clear Cover of Footing $cc_{bot} := 3\ in$ $d_{bot} := h_{footing} - \left(cc_{bot} + \frac{d_{b6}}{2}\right) = 26.625\ in$

$cc_{top} := 2\ in$ $d_{top} := h_{footing} - \left(cc_{top} + 1.5\ d_{b6}\right) = 26.875\ in$

Soil Height on Footing $h_{soil} := 178\ ft + 1.5\ in - \left(166\ ft + 2\ in\right) - h_{footing} = 9.458\ ft$

Dead Load

Specific Weight of Concrete (and Fill) $\gamma_c := 150\ pcf$ Conservatively assumed heaviest material (i.e., concrete) for overburden dead weight

Live Load

Snippet below is from B737 aircraft specification sheets.

7.3 MAXIMUM PAVEMENT LOADS

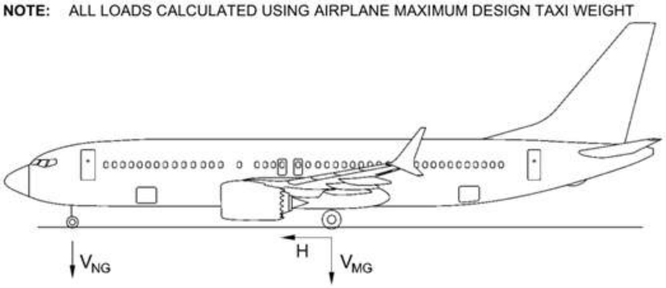
7.3.1 Maximum Pavement Loads: All Models

V_{NG} = MAXIMUM VERTICAL NOSE GEAR GROUND LOAD AT MOST FORWARD CENTER OF GRAVITY

V_{MG} = MAXIMUM VERTICAL MAIN GEAR GROUND LOAD AT MOST AFT CENTER OF GRAVITY

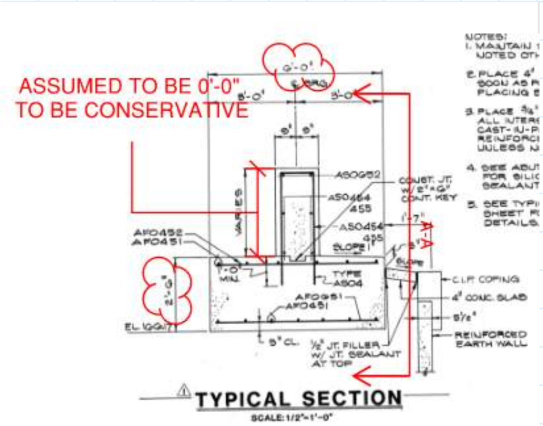
H = MAXIMUM HORIZONTAL GROUND LOAD FROM BRAKING

NOTE: ALL LOADS CALCULATED USING AIRPLANE MAXIMUM DESIGN TAXI WEIGHT



AIRPLANE MODEL	UNITS	MAX DESIGN TAXI WEIGHT	V_{NG}		V_{MG} PER STRUT AT MAX LOAD AT STATIC AFT C.G.	H PER STRUT	
			STATIC AT MOST FWD C.G.	STATIC + BRAKING 10 FT/SEC² DECEL		STEADY BRAKING 10 FT/SEC² DECEL	AT INSTANTANEOUS BRAKING (μ = 0.8)
737-7	LB	177,500	18,918	30,637	82,866	27,566	66,293
	KG	80,512	8,581	13,897	37,587	12,504	30,070
737-8 / -8-200 / BBJ MAX 8	LB	182,700	15,894	26,282	85,258	28,373	68,206
	KG	82,871	7,209	11,921	38,672	12,870	30,938
737-9 / BBJ MAX 9	LB	195,200	15,514	25,639	91,868	30,315	73,494
	KG	88,541	7,037	11,630	41,671	13,751	33,336
737-10	LB	198,400	13,613	23,251	93,679	30,812	74,944
	KG	89,992	6,175	10,546	42,492	13,976	33,994

All the following calculations are relative to the sketch below for dimensions and distance



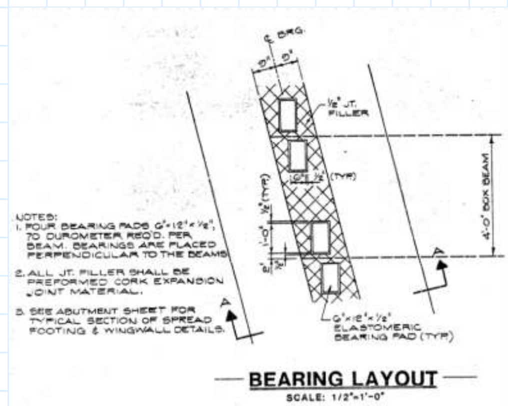
Plane Weight $W_{max\_DF} := 384.07\ kip$

Distribution Factor $DF := 0.35$

Gear Loading Per Strut $P_{strut\_DF} := W_{max\_DF} \cdot 0.475 \cdot DF = 63.852\ kip$

Bearing Check

Bearing Capacity Per General Notes $q_{all} := 4000\ psf$



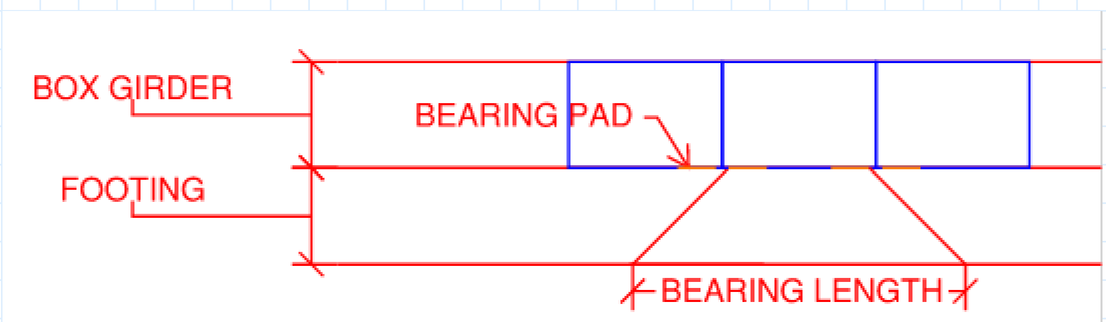
Bearing Pad Geometry

Bearing Pad Width $b_{bp} := 6\ in$

Bearing Pad Length $l_{bp} := 12\ in$

Bearing Area

Distribution Length - We've conservatively assumed 45 deg distribution within the concrete box beam down to footing elevation



$$l_{dw} := b_{box} - 2 \cdot 2\ in + 2\ \frac{h_{stemavg} + h_{footing}}{\tan(45\ °)} = 15.083\ ft$$

$$A_{bearing} := b_{footing} \cdot l_{dw} = 90.5\ ft^2$$

Length for Single Bearing Pad $$l_{dwonepad} := 12\ in + 2\ \frac{h_{stemavg} + h_{footing}}{\tan(45\ °)} = 12.417\ ft$$

$$A_{bearingonepad} := b_{footing} \cdot l_{dwonepad} = 74.5\ ft^2$$

Loads below are divided by two to distribute total load between two abutments on either end

Weight of Pavement and Fill Above per foot $w_{pf} := \gamma_c \cdot h_{pf} \cdot \frac{L_{span}}{2} = 9.547 \; \mathbf{klf}$

Box Beam Weight per foot $w_{box} := \gamma_c \cdot A_{box} \cdot \frac{L_{span}}{2\ b_{box}} = 4.056 \; \mathbf{klf}$

Footing Weight $w_{SW} := \gamma_c \cdot \left(b_{footing} \cdot h_{footing} + b_{stem} \cdot h_{stemavg}\right) = 2.972 \; \mathbf{klf}$

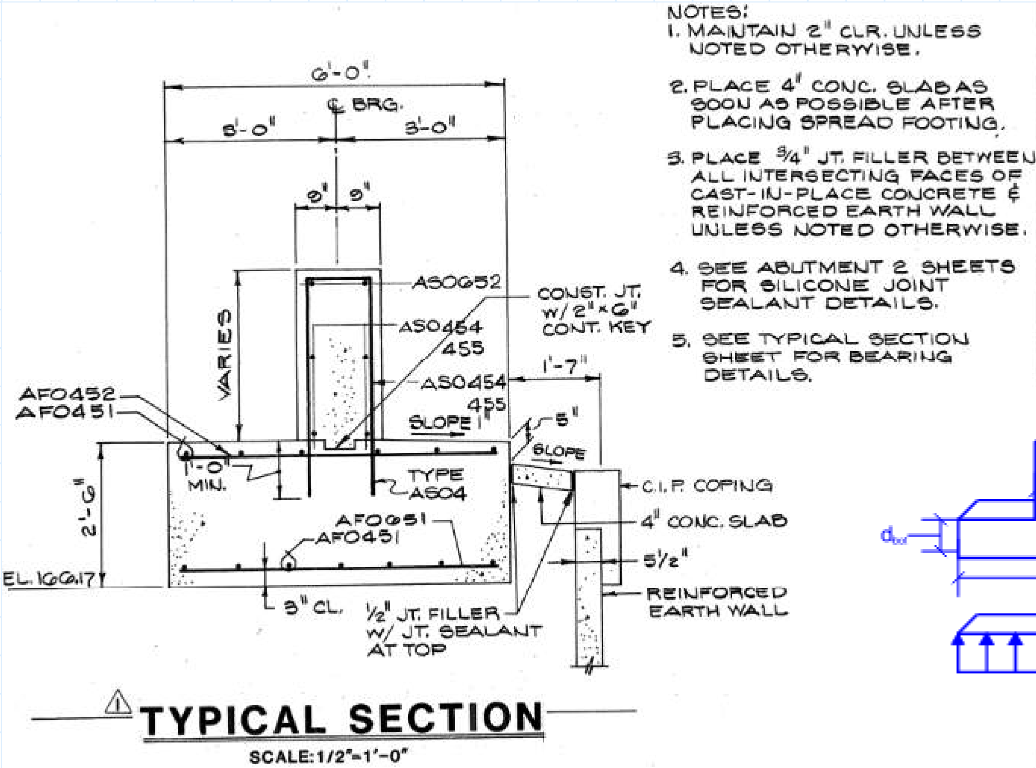
Soil on Footing Weight $w_{soil} := \gamma_c \cdot \frac{\left(b_{footing} - b_{stem}\right)}{2} \cdot h_{soil} = 3.192 \; \mathbf{klf}$

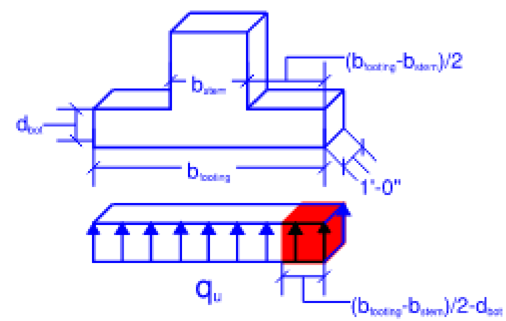
Bearing Pressure $q_{strut} := \frac{w_{pf} + w_{box} + w_{SW} + w_{soil}}{b_{footing}} + \frac{P_{strut\_DF}}{A_{bearing}} = 3999.983 \; \mathbf{psf}$

$\frac{q_{strut}}{q_{all}} = 1$ <1.0 **OK**

Strength Check

Strength of Foundation not Controlling





Distribution Factor $DF = 0.35$

Max Strut Loading (based on moment and shear capacity)

$$P_{strut\_DFM} := 8000\ \mathbf{kip} \cdot 0.475 \cdot DF = 1330\ \mathbf{kip}$$

$$P_{strut\_DFV} := 9000\ \mathbf{kip} \cdot 0.475 \cdot DF = 1496.25\ \mathbf{kip}$$

Ultimate Soil Pressure
Strength II

$$q_{uM} := 1.25\ \frac{w_{pf} + w_{box} + w_{SW} + w_{soil}}{b_{footing}} + 1.35\ \frac{P_{strut\_DFM}}{A_{bearing}} = 23.958\ \mathbf{ksf}$$

$$q_{uV} := 1.25\ \frac{w_{pf} + w_{box} + w_{SW} + w_{soil}}{b_{footing}} + 1.35\ \frac{P_{strut\_DFV}}{A_{bearing}} = 26.438\ \mathbf{ksf}$$

Moment Demand
for 1 ft section

$$M_u := q_{uM} \cdot 1\ \mathbf{ft} \cdot \frac{b_{footing} - 18\ \mathbf{in}}{2} \cdot \frac{b_{footing} - 18\ \mathbf{in}}{4} = 60.643\ \mathbf{kip} \cdot \mathbf{ft}$$

Shear Demand
for 1 ft section

$$V_u := q_{uV} \cdot 1\ \mathbf{ft} \cdot \left(\frac{b_{footing} - 18\ \mathbf{in}}{2} - d_{bot} \right) = 0.826\ \mathbf{kip}$$

$$M_{uV} := q_{uV} \cdot 1\ \mathbf{ft} \cdot \frac{b_{footing} - 18\ \mathbf{in}}{2} \cdot \frac{b_{footing} - 18\ \mathbf{in}}{4} = 66.921\ \mathbf{kip} \cdot \mathbf{ft}$$

Concrete Properties

Initial Concrete Strength $f'_{ci} := 4000\ psi$

28-Day Concrete Strength $f'_c := 5000\ psi$

Concrete Specific Weight (Table 3.5.1-1) $w_c := 145\ pcf$

$w_{cDL} := 150\ pcf$

Correction Factor $K_1 := 1$

Modulus of Elasticity (Eq. 5.4.2.4-1)

$$E_{ci} := 120000\ K_1 \cdot \left(\frac{w_c}{1000\ pcf}\right)^2 \left(\frac{f'_{ci}}{ksi}\right)^{0.33} ksi = 3986.548\ ksi$$

$$E_c := 120000\ K_1 \cdot \left(\frac{w_c}{1000\ pcf}\right)^2 \left(\frac{f'_c}{ksi}\right)^{0.33} ksi = 4291.186\ ksi$$

Stress Block Factors (5.6.2.2)

$$\alpha_1(x) := \begin{cases} 0.85 & \text{if } x \le 10\ ksi \\ \max\left(0.85 - \frac{0.02\ (x - 10\ ksi)}{1\ ksi}, 0.75\right) & \text{else} \end{cases}$$

$$\alpha_1 := \alpha_1(f'_c) = 0.85$$

$$\beta_1(x) := \begin{cases} 0.85 & \text{if } x \le 4\ ksi \\ \max\left(0.85 - \frac{0.05\ (x - 4\ ksi)}{1\ ksi}, 0.65\right) & \text{else} \end{cases}$$

$$\beta_1 := \beta_1(f'_c) = 0.8$$

Ultimate Compressive Strain $\varepsilon_{cu} := 0.003$

Steel Reinforcement, Grade 60

Modulus of Elasticity $f_y := 60\ ksi$

Yield Strength $E_y := 29000\ ksi$

Yield Strain $\varepsilon_y := min\left(\frac{f_y}{E_y}, 0.002\right) = 0.002$

Reinforcement Detailing Check

AASHTO 12.14 is related to Precast Reinforced Concrete Three-Sided Structures. This section may not be the most applicable to the bridge we are analyzing, but let's use it as a baseline,

Justifications:
- Bridge is 3-sided
- precast box girders

Per 12.14.5.8:

Minimum Reinforcement Short Direction, per ft $A_{sminshort} := 0.002 \cdot 1\ \boldsymbol{ft} \cdot h_{footing} = 0.72\ \boldsymbol{in}^2$

Provided in Drawings $A_{sprovshort} := A_{b6} + A_{b4} = 0.64\ \boldsymbol{in}^2$ $A_{sminshort} \le A_{sprovshort} = 0$

Minimum Reinforcement Long Direction, per ft $A_{sminlong} := 0.002 \cdot b_{footing} \cdot h_{footing} = 4.32\ \boldsymbol{in}^2$

Provided in Drawings $A_{sprovlong} := 12\ A_{b4} = 2.4\ \boldsymbol{in}^2$ $A_{sminlong} \le A_{sprovlong} = 0$

Provided Reinforcement not meeting the minimum.

Moment Capacity

Reinforcement Geometry, Assuming spacing for top and bottom reinforcement is @1'-0" C.C.

Top Reinforcement $A_{top} := A_{b4}$

Bottom Reinforcement $A_{bot} := A_{b6}$

Assuming:

Bottom Reinforcement Yielding
Top Reinforcement Yielding, on Tension Side
No PT

$A_s \cdot f_s - A'_s \cdot f'_s = \alpha_1 \cdot f'_c \cdot \beta_1 \cdot b \cdot c$ Eqn. 5.6.3.1.1-4

$A_s := A_{bot}$ $A'_s := A_{top}$ $b := 1\ ft$

$f_s := f_y$ $f'_s := f_y$

$$c := \frac{A_s \cdot f_s - A'_s \cdot f'_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} = 0.353\ in \qquad \frac{c}{d_{bot}} \le \frac{0.003}{0.003 + \varepsilon_{cu}} = 1 \qquad (5.6.2.1-1)$$

Checks Reinforcement Assumption
- Correct Assumptions

$c > d_{bot} = 0$

$c > h_{footing} - d_{top} = 0$

Check steel yielding assumptions
- Correct Assumptions

$$\varepsilon_{sbot} := \frac{d_{bot} - c}{c} \cdot \varepsilon_{cu} = 0.223 \qquad \varepsilon'_{stop} := \frac{\left(h_{footing} - d_{top}\right) - c}{c} \cdot \varepsilon_{cu} = 0.024$$

$\varepsilon_{sbot} > \varepsilon_y = 1$ $\varepsilon'_{stop} > \varepsilon_y = 1$

$$M_n := A_s \cdot f_s \cdot \left(d_{bot} - \frac{\beta_1 \cdot c}{2}\right) + A'_s \cdot f'_s \cdot \left(\left(h_{footing} - d_{top}\right) - \frac{\beta_1 \cdot c}{2}\right) = 61.248\ kip \cdot ft$$

$$\phi(\varepsilon) := \begin{cases} 0.75 & \text{if } \varepsilon \le \varepsilon_y \\ 0.75 + 0.25 \dfrac{\left(\varepsilon - \varepsilon_y\right)}{0.005 - \varepsilon_y} & \text{else if } \varepsilon < 0.005 \\ 1 & \text{else} \end{cases} \qquad \phi := \phi\left(\varepsilon_{sbot}\right) = 1$$

$\phi M_n := \phi \cdot M_n = 61.248\ kip \cdot ft$

$$\frac{M_u}{\phi M_n} = 0.99$$

Shear Capacity

$d_v := \max\left(0.9 \cdot d_{bot}, 0.72\ h_{box}\right) = 23.963\ \textbf{in}$

$b_v := 1\ \textbf{ft}$

Area of Concrete on Flexural Tension Side

$A_{ct} := \dfrac{A_{box}}{2}$

Figure B5.2-1 shows half of cross-section to be considered as concrete on the flexural tension side

No Transverse Reinforcement, none required per 5.7.2.3

$\phi_v := 0.9$

$A_s := A_{sprovshort}$

β Determination

$M_u := \max\left(M_{uV}, V_u \cdot d_v\right) = 66.921\ \textbf{kip} \cdot \textbf{ft}$

$V_u = 0.826\ \textbf{kip}$

$v_u := \dfrac{V_u}{\phi_v \cdot b_v \cdot d_v} = 0.003\ \textbf{ksi}$

$\dfrac{v_u}{f'_c} = 6.385 \cdot 10^{-4}$

Iteration 1

$\varepsilon_s := \dfrac{\left(\dfrac{M_u}{d_v} + 0.5\ V_u\right)}{2\ E_y \cdot A_s} = 9.139 \cdot 10^{-4}$

$\theta := \left(29 + 3500\ \varepsilon_s\right)\ \circ = 32.199\ \circ$

Eq. 5.7.3.4.2-3

$\theta_{before} := \theta$

Iteration 2

$\varepsilon_s := \dfrac{\left(\dfrac{M_u}{d_v} + 0.5\ V_u \cdot \cot\left(\theta\right)\right)}{2\ E_y \cdot A_s} = 9.205 \cdot 10^{-4}$

$\theta := \left(29 + 3500\ \varepsilon_s\right)\ \circ = 32.222\ \circ$

$\dfrac{\theta - \theta_{before}}{\theta} = 7.108 \cdot 10^{-4}$

$\theta_{before} := \theta$

Iteration 3

$$\varepsilon_s := \frac{\left(\frac{M_u}{d_v} + 0.5\ V_u \cdot \cot(\theta)\right)}{2\ E_y \cdot A_s} = 9.205 \cdot 10^{-4}$$

$$\theta := (29 + 3500\ \varepsilon_s)\ ° = 32.222\ °$$

$$\frac{\theta - \theta_{before}}{\theta} = -1.701 \cdot 10^{-6}$$

End of Iterations

$$\beta := \frac{4.8}{1 + 750\ \varepsilon_s} = 2.84$$

Eq. 5.7.3.4.2-1,
for meeting minimum shear reinforcement

Concrete Shear Capacity

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{f'_c}{ksi}}\ ksi \cdot b_v \cdot d_v = 57.696\ kip$$

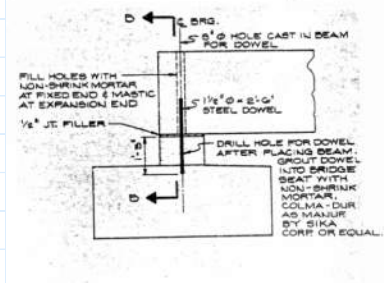
Total Capacity

$$\phi V_{nend} := \phi_v \cdot V_c = 51.927\ kip$$

$$\frac{V_u}{\phi V_{nend}} = 0.016$$

Dowel Check

At the one end the box beam is free but on the other end it is fixed to the abutment with a 1-1/2" steel down shown in the sketch here. We need to check the capacity of this dowel to withstand braking forces from the aircraft



SECTION A-A
SCALE: 1/2"=1'-0"

Dowel Properties

Dowel Diameter $d_{dowel} := 1.5\ \mathbf{in}$

Yield Stress (Assumed) $f_y := 60\ \mathbf{ksi}$

Plastic Section $Z := \dfrac{d_{dowel}^{\ 3}}{6} = 0.563\ \mathbf{in}^3$

Elastic Section $S := \dfrac{\pi \cdot d_{dowel}^{\ 3}}{32} = 0.331\ \mathbf{in}^3$

$\phi_v := 1 \qquad \phi_m := 1$

Shear Capacity per AISC $\phi V_n := \phi_v \cdot 0.6\ f_y\ \dfrac{\pi}{4} \cdot d_{dowel}^{\ 2} = 63.617\ \mathbf{kip}$

Moment Capacity per AISC $\phi M_n := \phi_m \cdot min\left(f_y \cdot Z, 1.6\ f_y \cdot S\right) = 2.651\ \mathbf{kip \cdot ft}$

Passive Pressure

Consideration of the passive pressure from soil will allow for increase in dowel capacity.

Passive Pressure
From Geotechnical Report $q_{pass} := 300\ \dfrac{\mathbf{psf}}{\mathbf{ft}}$

Passive Force $V_{passive} := 0.5\ q_{pass} \cdot \left(h_{pf} + h_{box}\right)^2 \cdot b_{box} = 20.745\ \mathbf{kip}$

Factored Passive Force $'0.9V_{passive} := V_{passive} \cdot 0.9 = 18.67\ \mathbf{kip}$

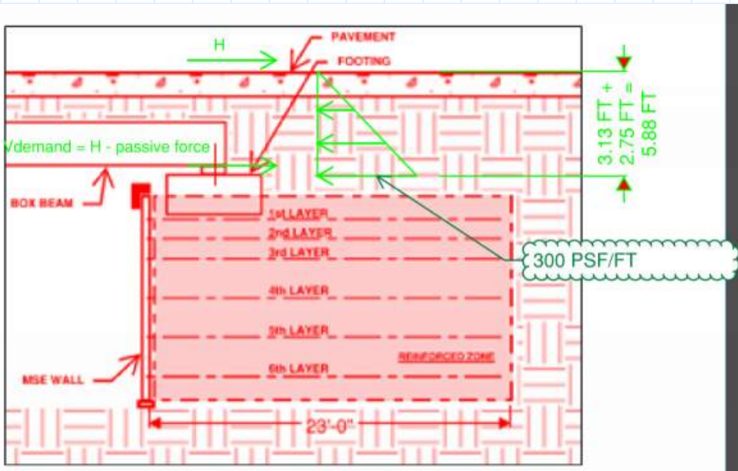


Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
○ Thermoplastic Culverts		1.30	0.90
○ All others		1.95	0.90
ES: Earth Surcharge		1.50	0.75

AASHTO LOAD FACTOR TABLE

Determining the shear demand acting on the existing dowels.

Coe icient of friction $\mu := 0.3$

Conservatively, using 16 inches as the pavement thickness $TW\&RW.H_{pavement} := 16\ \mathbf{in}$

Pavement selftweight $PavementWeight := TW\&RW.H_{pavement} \cdot \gamma_c = 200\ \mathbf{psf}$

Friction alculations between pavement weight and soil $'\mu := \mathbf{F} \cdot \mathbf{N} \ \rightarrow \ N := '\mu \cdot \mathbf{F}$

Horizontal resisting force acting at bottom of pavement $N1 := \mu \cdot PavementWeight = 60\ \mathbf{psf}$

Shear demand $V_{demand} := 60\ \mathbf{psf} \cdot b_{box} \cdot L_{span} = 9.76\ \mathbf{kip}$

Final Shear demand $V_U := \left(V_{demand} \cdot 1.75\right) - \left('0.9V_{passive}\right) = -1.59\ \mathbf{kip}$

Based on Concrete Breakout Capacity

Breakout capacity was determined using Hilti Profis. See attached Hilti Profis calculation package.

Breakout Capacity $\phi V_{nbreakout} := 16839\ \mathbf{lbf}$

$V_U = -1.59\ \mathbf{kip} \quad < \quad \phi V_{nbreakout} = 16.839\ \mathbf{kip}$

Bearing Pad Check

Demands

Values below divided by two because there are two bearing pads per box beam at each end

Airplane Load on Bearing Pad — To Be Determined Based on Compressive Service Stress Capacity

Dead Load on Bearing Pad — $P_{DLbp} := \frac{\left(w_{pf} + w_{box}\right) \cdot b_{box}}{2} = 27.205 \; kip$

Geometry

Width (perpendicular to tra ic direction) — $pad_{width} := l_{bp} = 12 \; in$

Length (parallel to tra ic direction) — $pad_{length} := b_{bp} = 6 \; in$

Number of Steel Layers — $n_{steel\_layers} := 0$

Thickness of Exterior Elastomeric Layers — $h_{elast\_ext} := 0.5 \cdot in$

Thickness of Interior Elastomeric Layers — $h_{elast\_int} := 0.5 \cdot in$

Thickness of Steel Plates — $h_s := 0 \cdot in$

Minimum Shear Modulus of Elastomerer
AASHTO Table 14.7.6.2.1 — $G_{min} := 0.200 \cdot ksi$

Maximum Shear Modulus of Elastomerer
AASHTO Table 14.7.6.2.1 — $G_{max} := 0.300 \cdot ksi$

Total Elastomeric Material Thickness — $h_{rt} := 2 \cdot h_{elast\_ext} + \left(n_{steel\_layers} - 1\right) \cdot h_{elast\_int} = 0.5 \; in$

Total Pad Thickness — $h_{pad} := h_{rt} + n_{steel\_layers} \cdot h_s = 0.5 \; in$

Bearing Area of Pad — $A_{pad} := pad_{width} \cdot pad_{length} = 72 \; in^2$

Compressive Stress Check

Shape factor of the ith elastomeric layer $S_{i\_ext} := \dfrac{pad_{width} \cdot pad_{length}}{2 \cdot h_{elast\_ext} \cdot \left(pad_{width} + pad_{length}\right)} = 4$

Shape factor of the ith elastomeric layer $S_{i\_int} := \dfrac{pad_{width} \cdot pad_{length}}{2 \cdot h_{elast\_int} \cdot \left(pad_{width} + pad_{length}\right)} = 4$

Max Compressive Stress Limit for
PEP (Plain Elastomeric Pad)
AASHTO 14.7.6.3.2

$\sigma_s := min\left(1.0\ G_{min} \cdot S_{i\_ext}, 800\ psi\right) = 800\ psi$

ACI 343R-95 10.5
Impact-Live Load

$i_{highspeed} := 0.40$

$i_{touchdown} := 1.00$ $\qquad i_{use} := i_{touchdown}$

Maximum Airplane Load on Bearing Pad Based on Max Compressive Stress Limit for PEP
ACI 343R-95 10.5.2

$$P_{planebp} := \frac{\sigma_s \cdot A_{pad} - P_{DLbp}}{\left(1 + i_{use} \cdot \left(1 - \dfrac{h_{pf}}{10\ ft}\right)\right) \cdot 0.475 \cdot 0.25} = 151.723\ kip$$

Airplane Braking Force $V_{uInstantmax} := P_{planebp} \cdot 0.375$

Compressive Deflection Check

No graph for 70 Durometer, Use 60 Durometer graph to be conservative.

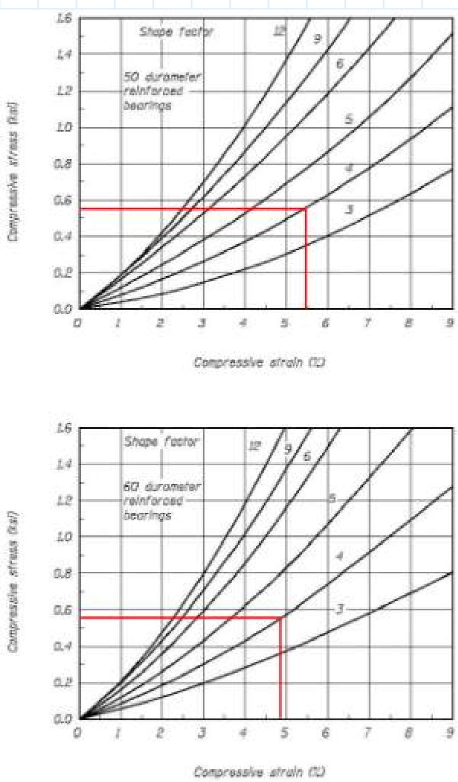


Figure C14.7.6.3.3-1—Stress-strain Curves

$S_{i\_ext} = 4$ $\qquad S_{i\_int} = 4$

$\sigma_L := \sigma_s = 0.8\ ksi$

At 60 Durometer, Approximately: $\varepsilon_{cPEP} := 7.1\%$

AASHTO 8th Edition 14.7.6.3.3 $\varepsilon_{cPEPLimit} := 0.09 = 0.09$

$\dfrac{\varepsilon_{cPEP}}{\varepsilon_{cPEPLimit}} = 0.79$

<1, OK

Estimate Bearing Pad Horizontal Deformations

Estimate the axial shortening of girders to check the bearing pads. Exclude elastic shortening from calculations because this occurred in the precast yard.

$$\Delta_s = \Delta_{CR} + \Delta_{SH} + \Delta_{TU} + \Delta_{BR}$$

$$\Delta_s = \left(\frac{P \cdot L}{A_g \cdot E_c} \cdot \psi_{cr} + \varepsilon_{sh} \cdot L_{span} + \varepsilon_{temp} \cdot L_{span} + \frac{F_{BR} \cdot h_{rt}}{G_{min} \cdot A_{pad}} \right)$$

Loads:
PS: Post tensioning
CR: Creep
SH: Shrinkage
TU: Uniform Temperature
BR: Braking forces

Uniform Temperature Deformation

Uniform Temperature Change
Table 3.12.3-1, Let's have T1-T3

$\Delta_{temp} := (41 - 0)\ \Delta°F$

Temperature Strain $\varepsilon_{temp} := \frac{6.0 \cdot 10^{-6}}{\Delta°F} \cdot \Delta_{temp} = 2.46 \cdot 10^{-4}$

Deformation $\Delta_{TU} := \varepsilon_{temp} \cdot L_{span} = 0.12\ in$

Braking Forces Deformation

Longitudinal Braking Force $F_{BR} := \frac{V_{uInstantmax}}{4} = 14.224\ kip$

Breaking Force Deformation $\Delta_{BR} := \frac{F_{BR} \cdot h_{rt}}{G_{min} \cdot A_{pad}} = 0.494\ in$

Creep Deformation (AASHTO 8th Edition 5.4.2.3.2)

Initial Strength $f'_{ci} := 4.0 \cdot ksi$

28-Day Strength $f'_c := 5.0 \cdot ksi$

Correction Factor $K_1 := 1$

Initial Modulus of Elasticity (AASHTO Eq. 5.4.2.4-1)

$$E_{ciAASHTO} := 120000\ K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^2 \left(\frac{f'_{ci}}{ksi}\right)^{0.33} ksi = 4266.223\ ksi$$

Initial Modulus of Elasticity (ACI 318)

$$E_{ciACI} := 33000 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi = 3834.3\ ksi$$

$$E_{ciuse} := min\left(E_{ciAASHTO}, E_{ciACI}\right) = 3834.254\ ksi$$

28-Day Modulus of Elasticity (AASHTO Eq. 5.4.2.4-1)

$$E_{cAASHTO} := 120000\ K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^2 \left(\frac{f'_c}{ksi}\right)^{0.33} ksi = 4592.232\ ksi$$

28-Day Modulus of Elasticity (ACI 318)

$$E_{cACI} := 33000 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi = 4286.8\ ksi$$

$$E_{cuse} := min\left(E_{cAASHTO}, E_{cACI}\right) = 4286.826\ ksi$$

Girder Area $A_{girder} := 766\ in^2$

Girder Width $b_{width} := 48\ in$

Volume to Surface Ratio $V\_S := \frac{A_{girder}}{b_{width}} = 15.958\ in$

Assume for Release Age $t_i := 1$ days Critical to Assume Earlier Age

Assume for Service $t_s := 41 \cdot 365$ days Current Age of Bridge ~ 41 years

$$k_s := \max\left(1, 1.45 - 0.13 \cdot \left(\frac{V\_S}{in}\right)\right) = 1$$

Relative humidity for Manassas,
Figure 5.4.2.3.3-1

$H := 75\%$

$k_{hc} := 1.56 - 0.008 \cdot \frac{H}{1\%} = 0.96$

$k_f := \frac{5}{1 + \frac{f'_{ci}}{ksi}} = 1$

$k_{td}(t) := \frac{t}{61 - 4 \cdot \frac{f'_{ci}}{ksi} + t}$

$\psi(t, t_i) := 1.9 \cdot k_s \cdot k_{hc} \cdot k_f \cdot k_{td}(t) \cdot t_i^{-0.118}$

Creep Coe icient at Service

$\psi_{cr} := \psi(t_s, t_i) = 1.819$

$A_{0.5pt} := 0.153 \cdot in^2$

$f_{pu} := 270\ ksi$

E ective Forces
Assume 25 ksi Loss per
PCI Handbook 7th edition 5.7.2

$F_e := 30 \cdot A_{0.5pt} \cdot (0.75 \cdot f_{pu} - 25 \cdot ksi) = 814.7\ kip$

Creep Deformation

$\Delta_{CR} := \frac{F_e \cdot L_{span}}{A_{girder} \cdot E_{cuse}} \cdot \psi_{cr} = 0.22\ in$

Shrinkage Deformation (AASHTO 8th Edition 5.4.2.3.3)

$k_{hs} := 2 - 0.014 \cdot \frac{H}{1\%} = 0.95$ (Eq. 5.4.2.3.3-1)

Shrinkage Strain at Service

$\varepsilon_{sh} := k_s \cdot k_{hs} \cdot k_f \cdot k_{td}(t_s) \cdot 0.48 \cdot 10^{-3} = 4.546 \cdot 10^{-4}$

Shrinkage deformation

$\Delta_{SH} := \varepsilon_{sh} \cdot L_{span} = 0.222\ in$

Total Horizontal Deformation (per pad)

$\Delta_s := 1.0 \cdot \Delta_{CR} + 1.0 \cdot \Delta_{SH} + 1.0 \cdot \Delta_{TU} + 1.0 \cdot \Delta_{BR} = 1.056\ in$

Shear Deformation Check

For PEP (14.7.6.3.4-1)

$$\frac{\Delta_s}{0.5 \cdot h_{rt}} = 4.224$$

>1, Not OK

Without Braking Force (no plane load), the deformation check fails.

$$\frac{1.0 \cdot \Delta_{CR} + 1.0 \cdot \Delta_{SH} + 1.0 \cdot \Delta_{TU}}{0.5 \cdot h_{rt}} = 2.248$$

Rotation Check (14.7.6.3.5)

No Specific Limit for PEP

Stability Check (14.7.6.3.6)

$$\frac{h_{rt}}{\frac{1}{3} \, min\left(pad_{width}, pad_{length}\right)} = 0.25$$

<1, OK

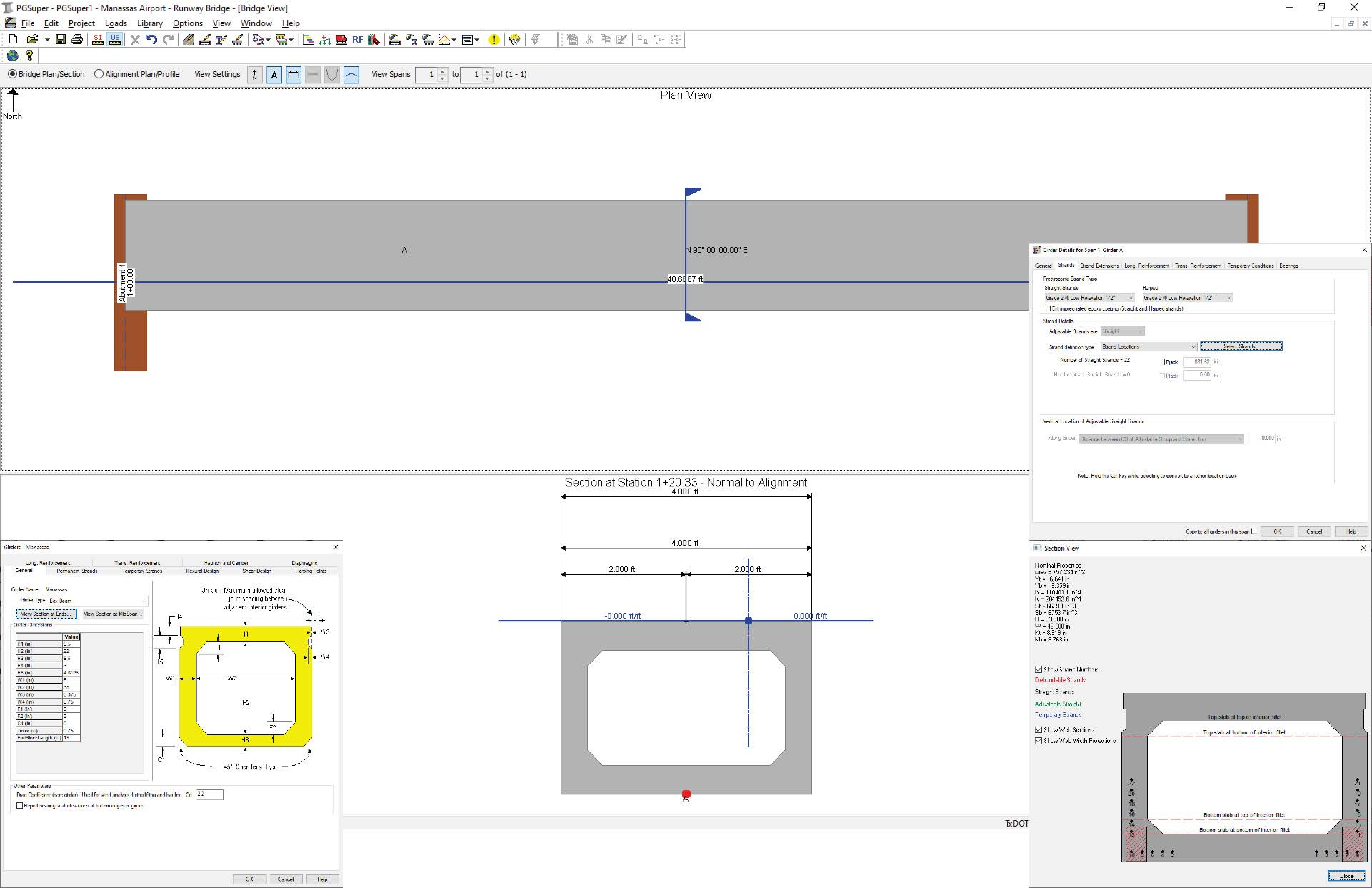
Reinforcement Check (14.7.6.3.7)

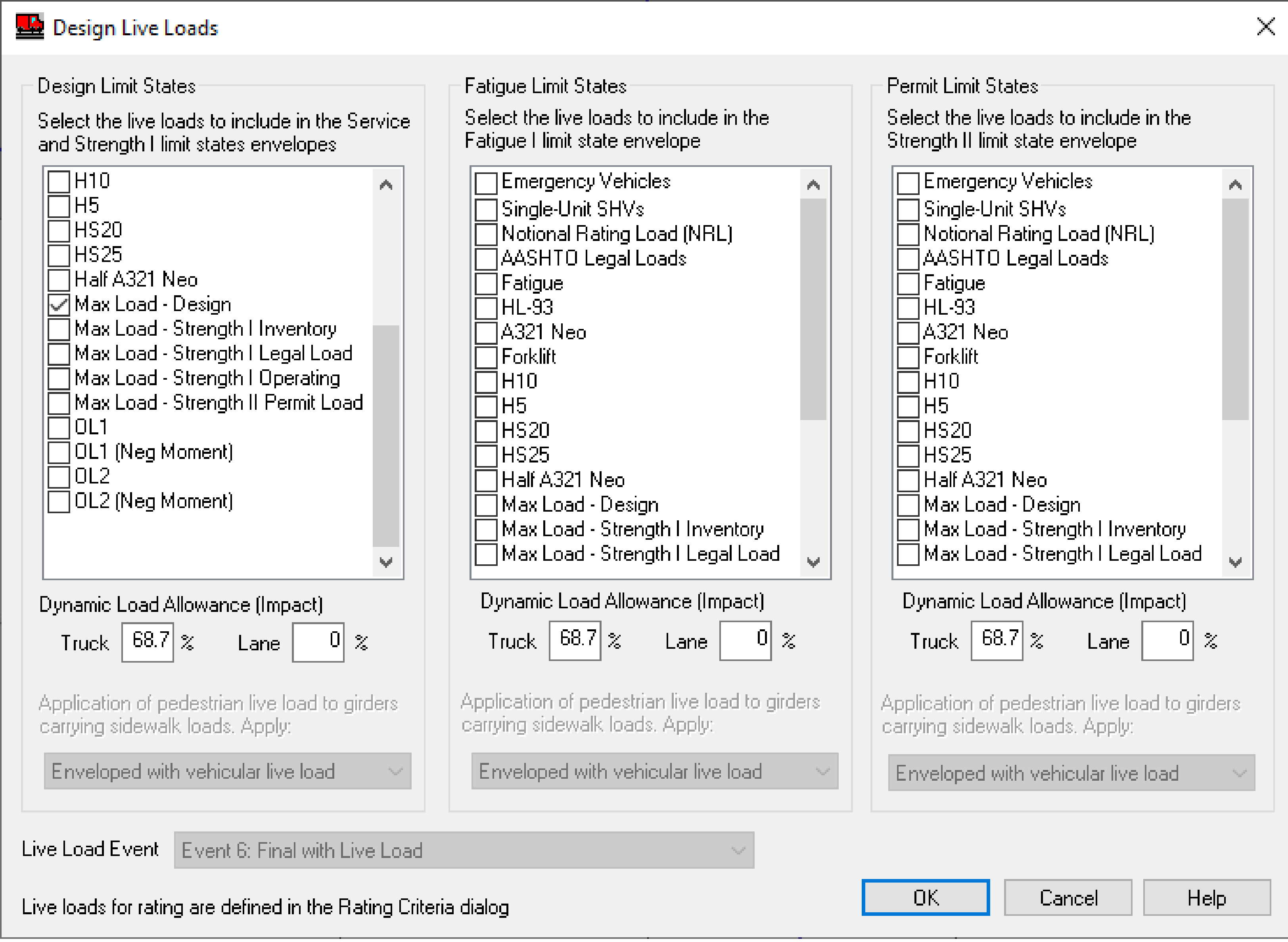
Not Applicable to PEP

Bearing pad analysis summary: Max service load for compression determined. Per calculations, bearing pads fail in shear deformation. On site inspection, the bearing pads are ok. We will need to closely monitor performance of the bearing pads.

PGSuper is a bridge software analysis. With the software we were able to determine the capacity of the bridge's single box girder and the maximum single axle point load that can be on the box girder. We have defined two moving vehicle loads, which represent an airplane's strut (Strength Limit State, Fatigue Limit State). The airplanes we are concerned about have struts that are further apart than the span of the bridge. Therefore, one "axle" is enough. Other static uniform or point loads can be placed along the span of the bridge. The cross section of the box was inputed into the program.

The following pages show the geometry and assumptions we made in PGSuper and the results from it.


PGSuper - PGSuper1 - Manassas Airport - Runway Bridge - [Bridge View]
File Edit Project Loads Library Options View Window Help
Bridge Plan/Section
Alignment Plan/Profile
View Settings
View Spans 1 to 1 of (1 - 1)
Plan View
North
A
N 90° 00' 00.00" E
40.6667 ft
Abutment 1
1+00.00
Section at Station 1+20.33 - Normal to Alignment
4.000 ft
4.000 ft
2.000 ft
2.000 ft
-0.000 ft/ft
0.000 ft/ft
TxDOT
Section View



Type	Event	Load Case	Location	Magnitude	Description
Uniform	Event 4: Install composite overlay	DC	All Spans, All Girders, Entire Span	1.878 kip/ft	Fill + Pavement

DESIGN LIMIT STATE
STRUT LOAD: 37.09 KIP
PLANE LOAD: 37.09 KIP/0.475/0.35 = 223.09 KIP

Spec Check Report

For Span 1 Girder A

May 15, 2025 2:51:26 pm

PGSuper™ (x64)

Version 8.0.6 - Built on Jul 9 2024

Project Properties

Bridge Name	Manassas Airport Bridge
Bridge ID	
Company	WPM
Engineer	Jeffrey Liu
Job Number	D012400400
Comments	
File	C:\Users\JeffreyL\Walter P. Moore and Associates\D01-24004-00 HEF Manassas Bridge Assessments - Documents\General\Diagnostics\2-Design\Analysis\Superstructure\JL Working\DF = 0.35\PGSuper1 - Manassas Airport - Runway Bridge.pgs

Configuration

Configuration Server: Default libraries installed with PGSuper
Configuration Name: PGSuper.com Community
Configuration Source: C:\PROGRAM FILES\WSDOT\BRIDGELINK\Configurations\WSDOT.lbr
Configuration Date Stamp: March 19, 2024 3:42:28 pm

Library	Entry	Source
Girders	Manassas	Project Library
Traffic Barriers	none	Project Library
Project Criteria	TxDOT 2020 based on AASHTO LRFD Bridge Design Specifications, 9th Edition 2020	Project Library
Vehicular Live Load	Max Load - Design	Project Library
Load Rating Criteria	MBE 2020 based on The Manual for Bridge Evaluation, Third Edition 2018, with 2020 interim provisions	Project Library
Haul Trucks	Old Haul Truck -0	Project Library

Trans. Reinforcement data for Girder A does not match Girder Library entry Manassas
Long. Reinforcement data for Girder A does not match Girder Library entry Manassas

Analysis Controls

Structural Analysis Method: Simple Span
Section Properties: Gross
Losses: Refined estimate per TxDOT Research Report 0-6374-2

Notes

Symbol	Definition
L_r	Span Length of Girder at Release
L_l	Span Length of Girder during Lifting
L_{st}	Span Length of Girder during Storage
L_h	Span Length of Girder during Hauling
L_e	Span Length of Girder after Erection
L_s	Length of Span
Debond	Point where bond begins for a debonded strand
PSXFR	Point of prestress transfer
FoS	Face of Support in final bridge configuration
ST	Section Transitions
STLF	Section Transitions, Left Face
STRF	Section Transitions, Right Face
SDCR	Start of Deck Casting Region
EDCR	End of Deck Casting Region
Diaphragm	Location of a precast or cast in place diaphragm
Bar Cutoff	End of a reinforcing bar in the girder
Deck Bar Cutoff	End of a reinforcing bar in the deck
CS	Critical Section for Shear
SZB	Stirrup Zone Boundary
H	H from end of girder or face of support
1.5H	1.5H from end of girder or face of support
HP	Harp Point
Pick Point	Support point where girder is lifted from form
Bunk Point	Point where girder is supported during transportation

Status Items

Level	Description
Information	Live Load Distribution Factors were User-Input.

Level	Description
Warning	Span 1, Girder A: Either the Jacking stress is not equal to 0.75Fpu, or Debonded strands are present, or Temporary strands are present, or the girder is Not Prismatic. Therefore, for the calculation of elastic shortening; an iterative solution was used to find Fcgp after release rather than assuming 0.7*Fpu per the TxDOT design manual.

Specification Checks

Specification: TxDOT 2020

Stress Limitations on Prestressing Tendons [5.9.2.2]

Strand Stresses

Loss Stage	Stress Limit (KSI)	Straight		Adjustable Straight	
		Strand Stress (KSI)	Status (C/D)	Strand Stress (KSI)	Status (C/D)
At Jacking (f_{pj})	202.500	202.500	Pass (1.00)	0.000	Pass (∞)
After All Losses and Elastic Gains including Live Load (f_{pe})	194.400	171.925	Pass (1.13)	0.000	Pass (∞)

Required Minimum Concrete Strengths

Required f'_{ci} = 2.611 KSI ⇒ 2.700 KSI

Provided f'_{ci} = 4.000 KSI

Required f'_{c} = 3.322 KSI ⇒ 3.400 KSI

Provided f'_{c} = 5.000 KSI

Interval 2: Prestress Release : Service I Compression

For Temporary Stresses before Losses (LRFD 5.9.2.3.1)
Compression Stresses (LRFD 5.9.2.3.1a)

f'_{ci} = 4.000 KSI

Compression stress limit = -0.65f'_{ci} = -2.600 KSI

Concrete strength required to satisfy this requirement = 2.611 KSI

Location from Left Support (ft)	Location from End of Girder (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
		f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(STRF) -0.750	(STRF, 0.0L_r) 0.000	0.000	0.000	0.000	0.000	0.000	0.000	No	Yes	Pass (∞)
(STLF) 0.750	(STLF) 1.500	0.186	-0.688	-0.034	0.034	0.153	-0.654	No	Yes	Pass (3.97)
(STRF) 0.750	(STRF) 1.500	0.043	-1.059	-0.043	0.043	-0.001	-1.016	No	Yes	Pass (2.56)
(PSXFR) 1.750	(PSXFR) 2.500	0.071	-1.766	-0.069	0.069	0.002	-1.697	No	Yes	Pass (1.53)
3.317	(0.1L_r) 4.067	0.071	-1.768	-0.107	0.107	-0.036	-1.661	No	Yes	Pass (1.57)
7.383	(0.2L_r) 8.133	0.071	-1.771	-0.189	0.189	-0.117	-1.582	No	Yes	Pass (1.64)
11.450	(0.3L_r) 12.200	0.071	-1.773	-0.247	0.247	-0.176	-1.526	No	Yes	Pass (1.70)
15.517	(0.4L_r) 16.267	0.071	-1.774	-0.282	0.282	-0.211	-1.492	No	Yes	Pass (1.74)
(0.5L_s) 19.583	(0.5L_r) 20.333	0.071	-1.775	-0.294	0.294	-0.222	-1.481	No	Yes	Pass (1.76)
23.650	(0.6L_r) 24.400	0.071	-1.774	-0.282	0.282	-0.211	-1.492	No	Yes	Pass (1.74)
27.717	(0.7L_r) 28.467	0.071	-1.773	-0.247	0.247	-0.176	-1.526	No	Yes	Pass (1.70)
31.783	(0.8L_r) 32.533	0.071	-1.771	-0.189	0.189	-0.117	-1.582	No	Yes	Pass (1.64)
35.850	(0.9L_r) 36.600	0.071	-1.768	-0.107	0.107	-0.036	-1.661	No	Yes	Pass (1.57)
(PSXFR) 37.417	(PSXFR) 38.167	0.071	-1.766	-0.069	0.069	0.002	-1.697	No	Yes	Pass (1.53)

Location from Left Support (ft)	Location from End of Girder (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
		f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(STLF) 38.417	(STLF) 39.167	0.043	-1.059	-0.043	0.043	-0.001	-1.016	No	Yes	Pass (2.56)
(STRF) 38.417	(STRF) 39.167	0.186	-0.688	-0.034	0.034	0.153	-0.654	No	Yes	Pass (3.97)
(STLF) 39.917	(STLF, 1.0L_r) 40.667	0.000	0.000	0.000	0.000	0.000	0.000	No	Yes	Pass (∞)

Interval 2: Prestress Release : Service I Tension

For Temporary Stresses before Losses (LRFD 5.9.2.3.1)
Tension Stresses (LRFD 5.9.2.3.1b)

f'_{ci} = 4.000 KSI
Tension stress limit in precompressed tensile zone without bounded reinforcement = N/A
Tension stress limit in areas other than the precompressed tensile zone and without bonded reinforcement = $0.2400\lambda\sqrt{f'_{ci}}$ = 0.480 KSI
Tension stress limit in areas with sufficient bonded reinforcement = $0.2400\lambda\sqrt{f'_{ci}}$ = 0.480 KSI
Concrete strength required to satisfy this requirement = 0.406 KSI

Location from Left Support (ft)	Location from End of Girder (ft)	Pre-tension		Service I		Demand		Tension Limit		Precompressed Tensile Zone		Status (C/D)
		f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top (KSI)	Bottom (KSI)	Top	Bottom	
(STRF) -0.750	(STRF, 0.0L_r) 0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.480	-	No	Yes	Pass (∞)
(STLF) 0.750	(STLF) 1.500	0.186	-0.688	-0.034	0.034	0.153	-0.654	0.480	-	No	Yes	Pass (3.14)
(STRF) 0.750	(STRF) 1.500	0.043	-1.059	-0.043	0.043	-0.001	-1.016	0.480	-	No	Yes	Pass (-)
(PSXFR) 1.750	(PSXFR) 2.500	0.071	-1.766	-0.069	0.069	0.002	-1.697	0.480	-	No	Yes	Pass (10+)
3.317	(0.1L_r) 4.067	0.071	-1.768	-0.107	0.107	-0.036	-1.661	0.480	-	No	Yes	Pass (-)
7.383	(0.2L_r) 8.133	0.071	-1.771	-0.189	0.189	-0.117	-1.582	0.480	-	No	Yes	Pass (-)
11.450	(0.3L_r) 12.200	0.071	-1.773	-0.247	0.247	-0.176	-1.526	0.480	-	No	Yes	Pass (-)
15.517	(0.4L_r) 16.267	0.071	-1.774	-0.282	0.282	-0.211	-1.492	0.480	-	No	Yes	Pass (-)
(0.5L_s) 19.583	(0.5L_r) 20.333	0.071	-1.775	-0.294	0.294	-0.222	-1.481	0.480	-	No	Yes	Pass (-)
23.650	(0.6L_r) 24.400	0.071	-1.774	-0.282	0.282	-0.211	-1.492	0.480	-	No	Yes	Pass (-)
27.717	(0.7L_r) 28.467	0.071	-1.773	-0.247	0.247	-0.176	-1.526	0.480	-	No	Yes	Pass (-)
31.783	(0.8L_r) 32.533	0.071	-1.771	-0.189	0.189	-0.117	-1.582	0.480	-	No	Yes	Pass (-)
35.850	(0.9L_r) 36.600	0.071	-1.768	-0.107	0.107	-0.036	-1.661	0.480	-	No	Yes	Pass (-)
(PSXFR) 37.417	(PSXFR) 38.167	0.071	-1.766	-0.069	0.069	0.002	-1.697	0.480	-	No	Yes	Pass (10+)
(STLF) 38.417	(STLF) 39.167	0.043	-1.059	-0.043	0.043	-0.001	-1.016	0.480	-	No	Yes	Pass (-)
(STRF) 38.417	(STRF) 39.167	0.186	-0.688	-0.034	0.034	0.153	-0.654	0.480	-	No	Yes	Pass (3.14)
(STLF) 39.917	(STLF, 1.0L_r) 40.667	0.000	0.000	0.000	0.000	0.000	0.000	0.480	-	No	Yes	Pass (∞)

Interval 10: Install composite overlay, Apply User Defined Loads : Service I Compression

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = -0.6f'_c = -3.000 KSI

Concrete strength required to satisfy this requirement = 2.278 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(0.0L_s) 0.000	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (9.76)
(STLF) 0.750	0.167	-0.616	-0.058	0.058	0.109	-0.558	No	Yes	Pass (5.38)
(STRF) 0.750	0.037	-0.920	-0.075	0.075	-0.038	-0.845	No	Yes	Pass (3.55)
(PSXFR) 1.750	0.062	-1.537	-0.170	0.170	-0.108	-1.367	No	Yes	Pass (2.19)
(0.1L_s) 3.917	0.062	-1.546	-0.358	0.358	-0.296	-1.188	No	Yes	Pass (2.53)
(0.2L_s) 7.833	0.063	-1.558	-0.636	0.636	-0.573	-0.922	No	Yes	Pass (3.25)
(0.3L_s) 11.750	0.063	-1.567	-0.835	0.835	-0.772	-0.732	No	Yes	Pass (3.89)
(0.4L_s) 15.667	0.063	-1.573	-0.954	0.954	-0.891	-0.619	No	Yes	Pass (3.37)
(0.5L_s) 19.583	0.063	-1.574	-0.994	0.994	-0.930	-0.581	No	Yes	Pass (3.22)
(0.6L_s) 23.500	0.063	-1.573	-0.954	0.954	-0.891	-0.619	No	Yes	Pass (3.37)
(0.7L_s) 27.417	0.063	-1.567	-0.835	0.835	-0.772	-0.732	No	Yes	Pass (3.89)
(0.8L_s) 31.333	0.063	-1.558	-0.636	0.636	-0.573	-0.922	No	Yes	Pass (3.25)
(0.9L_s) 35.250	0.062	-1.546	-0.358	0.358	-0.296	-1.188	No	Yes	Pass (2.53)
(PSXFR) 37.417	0.062	-1.537	-0.170	0.170	-0.108	-1.367	No	Yes	Pass (2.19)
(STLF) 38.417	0.037	-0.920	-0.075	0.075	-0.038	-0.845	No	Yes	Pass (3.55)
(STRF) 38.417	0.167	-0.616	-0.058	0.058	0.109	-0.558	No	Yes	Pass (5.38)
(1.0L_s) 39.167	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (9.76)

Interval 10: Install composite overlay, Apply User Defined Loads : Service I Tension

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Tension Stresses (LRFD 5.9.2.3.2b)

f'_c = 5.000 KSI

Tension stress limit for components with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions = 0.2400$\lambda\sqrt{f'_c}$ = 0.537 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Tension Limit		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top (KSI)	Bottom (KSI)	Top	Bottom	
(0.0L_s) 0.000	0.083	-0.308	0.000	0.000	0.083	-0.308	-	0.537	No	Yes	Pass (-)

Location from Left Support (ft)	Pre-tension		Service I		Demand		Tension Limit		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top (KSI)	Bottom (KSI)	Top	Bottom	
(STLF) 0.750	0.167	-0.616	-0.058	0.058	0.109	-0.558	-	0.537	No	Yes	Pass (-)
(STRF) 0.750	0.037	-0.920	-0.075	0.075	-0.038	-0.845	-	0.537	No	Yes	Pass (-)
(PSXFR) 1.750	0.062	-1.537	-0.170	0.170	-0.108	-1.367	-	0.537	No	Yes	Pass (-)
($0.1L_s$) 3.917	0.062	-1.546	-0.358	0.358	-0.296	-1.188	-	0.537	No	Yes	Pass (-)
($0.2L_s$) 7.833	0.063	-1.558	-0.636	0.636	-0.573	-0.922	-	0.537	No	Yes	Pass (-)
($0.3L_s$) 11.750	0.063	-1.567	-0.835	0.835	-0.772	-0.732	-	0.537	No	Yes	Pass (-)
($0.4L_s$) 15.667	0.063	-1.573	-0.954	0.954	-0.891	-0.619	-	0.537	No	Yes	Pass (-)
($0.5L_s$) 19.583	0.063	-1.574	-0.994	0.994	-0.930	-0.581	-	0.537	No	Yes	Pass (-)
($0.6L_s$) 23.500	0.063	-1.573	-0.954	0.954	-0.891	-0.619	-	0.537	No	Yes	Pass (-)
($0.7L_s$) 27.417	0.063	-1.567	-0.835	0.835	-0.772	-0.732	-	0.537	No	Yes	Pass (-)
($0.8L_s$) 31.333	0.063	-1.558	-0.636	0.636	-0.573	-0.922	-	0.537	No	Yes	Pass (-)
($0.9L_s$) 35.250	0.062	-1.546	-0.358	0.358	-0.296	-1.188	-	0.537	No	Yes	Pass (-)
(PSXFR) 37.417	0.062	-1.537	-0.170	0.170	-0.108	-1.367	-	0.537	No	Yes	Pass (-)
(STLF) 38.417	0.037	-0.920	-0.075	0.075	-0.038	-0.845	-	0.537	No	Yes	Pass (-)
(STRF) 38.417	0.167	-0.616	-0.058	0.058	0.109	-0.558	-	0.537	No	Yes	Pass (-)
($1.0L_s$) 39.167	0.083	-0.308	0.000	0.000	0.083	-0.308	-	0.537	No	Yes	Pass (-)

Interval 15: Open to Traffic, Roadway Geometry Control : Service I Compression without live load

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = $-0.45f'_c$ = -2.250 KSI

Concrete strength required to satisfy this requirement = 3.038 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (7.32)
(STLF) 0.750	0.167	-0.616	-0.058	0.058	0.109	-0.558	No	Yes	Pass (4.03)
(STRF) 0.750	0.037	-0.920	-0.075	0.075	-0.038	-0.845	No	Yes	Pass (2.66)
(PSXFR) 1.750	0.062	-1.537	-0.170	0.170	-0.108	-1.367	No	Yes	Pass (1.65)
($0.1L_s$) 3.917	0.062	-1.546	-0.358	0.358	-0.296	-1.188	No	Yes	Pass (1.89)
($0.2L_s$) 7.833	0.063	-1.558	-0.636	0.636	-0.573	-0.922	No	Yes	Pass

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
									(2.44)
($0.3L_s$) 11.750	0.063	-1.567	-0.835	0.835	-0.772	-0.732	No	Yes	Pass (2.92)
($0.4L_s$) 15.667	0.063	-1.573	-0.954	0.954	-0.891	-0.619	No	Yes	Pass (2.53)
($0.5L_s$) 19.583	0.063	-1.574	-0.994	0.994	-0.930	-0.581	No	Yes	Pass (2.42)
($0.6L_s$) 23.500	0.063	-1.573	-0.954	0.954	-0.891	-0.619	No	Yes	Pass (2.53)
($0.7L_s$) 27.417	0.063	-1.567	-0.835	0.835	-0.772	-0.732	No	Yes	Pass (2.92)
($0.8L_s$) 31.333	0.063	-1.558	-0.636	0.636	-0.573	-0.922	No	Yes	Pass (2.44)
($0.9L_s$) 35.250	0.062	-1.546	-0.358	0.358	-0.296	-1.188	No	Yes	Pass (1.89)
(PSXFR) 37.417	0.062	-1.537	-0.170	0.170	-0.108	-1.367	No	Yes	Pass (1.65)
(STLF) 38.417	0.037	-0.920	-0.075	0.075	-0.038	-0.845	No	Yes	Pass (2.66)
(STRF) 38.417	0.167	-0.616	-0.058	0.058	0.109	-0.558	No	Yes	Pass (4.03)
($1.0L_s$) 39.167	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (7.32)

Interval 15: Open to Traffic, Roadway Geometry Control : Service I Compression

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = $-0.6f'_c$ = -3.000 KSI

Concrete strength required to satisfy this requirement = 3.322 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (9.76)
(STLF) 0.750	0.167	-0.616	-0.121	0.058	0.046	-0.558	No	Yes	Pass (5.38)
(STRF) 0.750	0.037	-0.920	-0.155	0.075	-0.118	-0.845	No	Yes	Pass (3.55)
(PSXFR) 1.750	0.062	-1.537	-0.351	0.170	-0.290	-1.367	No	Yes	Pass (2.19)
($0.1L_s$) 3.917	0.062	-1.546	-0.741	0.358	-0.678	-1.188	No	Yes	Pass (2.53)
($0.2L_s$) 7.833	0.063	-1.558	-1.316	0.636	-1.254	-0.922	No	Yes	Pass (2.39)
($0.3L_s$) 11.750	0.063	-1.567	-1.727	0.835	-1.664	-0.732	No	Yes	Pass (1.80)
($0.4L_s$) 15.667	0.063	-1.573	-1.974	0.954	-1.911	-0.619	No	Yes	Pass (1.57)
($0.5L_s$) 19.583	0.063	-1.574	-2.056	0.994	-1.993	-0.581	No	Yes	Pass (1.51)
($0.6L_s$) 23.500	0.063	-1.573	-1.974	0.954	-1.911	-0.619	No	Yes	Pass (1.57)

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.7L_s$) 27.417	0.063	-1.567	-1.727	0.835	-1.664	-0.732	No	Yes	Pass (1.80)
($0.8L_s$) 31.333	0.063	-1.558	-1.316	0.636	-1.254	-0.922	No	Yes	Pass (2.39)
($0.9L_s$) 35.250	0.062	-1.546	-0.741	0.358	-0.678	-1.188	No	Yes	Pass (2.53)
(PSXFR) 37.417	0.062	-1.537	-0.351	0.170	-0.290	-1.367	No	Yes	Pass (2.19)
(STLF) 38.417	0.037	-0.920	-0.155	0.075	-0.118	-0.845	No	Yes	Pass (3.55)
(STRF) 38.417	0.167	-0.616	-0.121	0.058	0.046	-0.558	No	Yes	Pass (5.38)
($1.0L_s$) 39.167	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (9.76)

Interval 15: Open to Traffic, Roadway Geometry Control : Service III Tension

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Tension Stresses (LRFD 5.9.2.3.2b)

f'_c = 5.000 KSI

Tension stress limit for components with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions = $0.1900\lambda\sqrt{f'_c} \leq 0.600$ KSI = 0.425 KSI

Concrete strength required to satisfy this requirement = 2.203 KSI

Location from Left Support (ft)	Pre-tension		Service III		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (-)
(STLF) 0.750	0.167	-0.616	-0.058	0.109	0.109	-0.507	No	Yes	Pass (-)
(STRF) 0.750	0.037	-0.920	-0.075	0.140	-0.038	-0.780	No	Yes	Pass (-)
(PSXFR) 1.750	0.062	-1.537	-0.170	0.317	-0.108	-1.220	No	Yes	Pass (-)
($0.1L_s$) 3.917	0.062	-1.546	-0.358	0.669	-0.296	-0.877	No	Yes	Pass (-)
($0.2L_s$) 7.833	0.063	-1.558	-0.636	1.188	-0.573	-0.370	No	Yes	Pass (-)
($0.3L_s$) 11.750	0.063	-1.567	-0.835	1.559	-0.772	-0.008	No	Yes	Pass (-)
($0.4L_s$) 15.667	0.063	-1.573	-0.954	1.782	-0.891	0.210	No	Yes	Pass (2.03)
($0.5L_s$) 19.583	0.063	-1.574	-0.994	1.856	-0.930	0.282	No	Yes	Pass (1.51)
($0.6L_s$) 23.500	0.063	-1.573	-0.954	1.782	-0.891	0.210	No	Yes	Pass (2.03)
($0.7L_s$) 27.417	0.063	-1.567	-0.835	1.559	-0.772	-0.008	No	Yes	Pass (-)
($0.8L_s$) 31.333	0.063	-1.558	-0.636	1.188	-0.573	-0.370	No	Yes	Pass (-)
($0.9L_s$) 35.250	0.062	-1.546	-0.358	0.669	-0.296	-0.877	No	Yes	Pass (-)
(PSXFR) 37.417	0.062	-1.537	-0.170	0.317	-0.108	-1.220	No	Yes	Pass (-)

Location from Left Support (ft)	Pre-tension		Service III		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(STLF) 38.417	0.037	-0.920	-0.075	0.140	-0.038	-0.780	No	Yes	Pass (-)
(STRF) 38.417	0.167	-0.616	-0.058	0.109	0.109	-0.507	No	Yes	Pass (-)
($1.0L_s$) 39.167	0.083	-0.308	0.000	0.000	0.083	-0.308	No	Yes	Pass (-)

Interval 15: Open to Traffic, Roadway Geometry Control : Fatigue I Compression

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = $-0.4f'_c$ = -2.000 KSI

Concrete strength required to satisfy this requirement = 1.709 KSI

Location from Left Support (ft)	Pre-tension		Fatigue I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.083	-0.308	0.000	0.000	0.042	-0.154	No	Yes	Pass (10+)
(STLF) 0.750	0.167	-0.616	-0.029	0.029	0.054	-0.279	No	Yes	Pass (7.17)
(STRF) 0.750	0.037	-0.920	-0.038	0.038	-0.019	-0.422	No	Yes	Pass (4.74)
(PSXFR) 1.750	0.062	-1.537	-0.085	0.085	-0.054	-0.684	No	Yes	Pass (2.93)
($0.1L_s$) 3.917	0.062	-1.546	-0.179	0.179	-0.148	-0.594	No	Yes	Pass (3.37)
($0.2L_s$) 7.833	0.063	-1.558	-0.318	0.318	-0.287	-0.461	No	Yes	Pass (4.34)
($0.3L_s$) 11.750	0.063	-1.567	-0.417	0.417	-0.386	-0.366	No	Yes	Pass (5.18)
($0.4L_s$) 15.667	0.063	-1.573	-0.477	0.477	-0.445	-0.309	No	Yes	Pass (4.49)
($0.5L_s$) 19.583	0.063	-1.574	-0.497	0.497	-0.465	-0.290	No	Yes	Pass (4.30)
($0.6L_s$) 23.500	0.063	-1.573	-0.477	0.477	-0.445	-0.309	No	Yes	Pass (4.49)
($0.7L_s$) 27.417	0.063	-1.567	-0.417	0.417	-0.386	-0.366	No	Yes	Pass (5.18)
($0.8L_s$) 31.333	0.063	-1.558	-0.318	0.318	-0.287	-0.461	No	Yes	Pass (4.34)
($0.9L_s$) 35.250	0.062	-1.546	-0.179	0.179	-0.148	-0.594	No	Yes	Pass (3.37)
(PSXFR) 37.417	0.062	-1.537	-0.085	0.085	-0.054	-0.684	No	Yes	Pass (2.93)
(STLF) 38.417	0.037	-0.920	-0.038	0.038	-0.019	-0.422	No	Yes	Pass (4.74)
(STRF) 38.417	0.167	-0.616	-0.029	0.029	0.054	-0.279	No	Yes	Pass (7.17)
($1.0L_s$) 39.167	0.083	-0.308	0.000	0.000	0.042	-0.154	No	Yes	Pass (10+)

Ultimate Moment Capacity

Positive Moment Capacity for Strength I Limit State [5.6]

Location from Left Support (ft)	M_u (kip-ft)	ϕM_n (kip-ft)	ϕM_n Min (kip-ft)	Status	
				ϕM_n Min ≤ ϕM_n ($\phi M_n/\phi M_n$ Min)	$M_u \le \phi M_n$ ($\phi M_n/M_u$)
($0.0L_s$) 0.000	0.00	383.80	0.00	Pass (∞)	Pass (∞)
($0.1L_s$) 3.917	638.88	1393.57	849.71	Pass (1.64)	Pass (2.18)
($0.2L_s$) 7.833	1135.55	1684.80	1459.98	Pass (1.15)	Pass (1.48)
($0.3L_s$) 11.750	1490.32	1773.99	1464.90	Pass (1.21)	Pass (1.19)
($0.4L_s$) 15.667	1703.18	1774.11	1467.85	Pass (1.21)	Pass (1.04)
($0.5L_s$) 19.583	1774.13	1774.15	1468.84	Pass (1.21)	Pass (1.00)
($0.6L_s$) 23.500	1703.18	1774.11	1467.85	Pass (1.21)	Pass (1.04)
($0.7L_s$) 27.417	1490.32	1773.99	1464.90	Pass (1.21)	Pass (1.19)
($0.8L_s$) 31.333	1135.55	1684.80	1459.98	Pass (1.15)	Pass (1.48)
($0.9L_s$) 35.250	638.88	1393.57	849.71	Pass (1.64)	Pass (2.18)
($1.0L_s$) 39.167	0.00	383.80	0.00	Pass (∞)	Pass (∞)

Ultimate Shear Capacity

Ultimate Shears for Strength I Limit State [5.8]

Location from Left Support (ft)	Stirrups Required	Stirrups Provided	$\|V_u\|$ (kip)	ϕV_n (kip)	Status ($\phi V_n/V_u$)
(CS) 2.313	Yes	Yes	166.23	264.78	Pass (1.59)
(H) 3.000	Yes	Yes	161.79	261.11	Pass (1.61)
($0.1L_s$) 3.917	Yes	Yes	155.88	257.04	Pass (1.65)
(1.5H) 4.375	Yes	Yes	152.92	255.32	Pass (1.67)
($0.2L_s$) 7.833	Yes	Yes	130.60	205.05	Pass (1.57)
($0.3L_s$) 11.750	Yes	Yes	105.32	156.36	Pass (1.48)
($0.4L_s$) 15.667	Yes	Yes	80.04	138.11	Pass (1.73)
($0.5L_s$) 19.583	Yes	Yes	54.76	136.10	Pass (2.49)
($0.6L_s$) 23.500	Yes	Yes	80.04	138.11	Pass (1.73)
($0.7L_s$) 27.417	Yes	Yes	105.32	156.36	Pass (1.48)
($0.8L_s$) 31.333	Yes	Yes	130.60	205.05	Pass (1.57)
(1.5H) 34.792	Yes	Yes	152.92	255.32	Pass (1.67)
($0.9L_s$) 35.250	Yes	Yes	155.88	257.04	Pass (1.65)

Location from Left Support (ft)	Stirrups Required	Stirrups Provided	$\lvert V_u \rvert$ (kip)	ϕV_n (kip)	Status ($\phi V_n/V_u$)
(H) 36.167	Yes	Yes	161.79	261.11	Pass (1.61)
(CS) 36.854	Yes	Yes	166.23	264.78	Pass (1.59)

[LRFD 5.7.3.2] The reaction introduces compression into the end of the girder. Load between the CSS and the support is transferred directly to the support by compressive arching action without causing additional stresses in the stirrups. Hence, A_V/S in this region must be equal or greater than A_V/S at the critical section.

Longitudinal Reinforcement for Shear Check - Strength I [5.7.3.5]

$$A_s f_y + A_{ps} f_{ps} \geq \left[\frac{M_u}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_a} + \left(\left|\frac{V_u}{\varphi_v} - V_p\right| - 0.5V_s\right)\cot\theta\right] \quad 5.7.3.5\text{-}1$$

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\varphi_v} - V_p - 0.5V_s\right)\cot\theta \quad 5.7.3.5\text{-}2$$

$$A_{ps} f_{ps} > A_s f_y$$

Location from Left Support (ft)	Capacity (kip)	Demand (kip)	Equation	Status (C/D)	$A_{ps}f_{ps}$ (kip)	$A_s f_y$ (kip)	Status
(FoS) 0.250	238.56	169.99	5.7.3.5-2	Pass (1.40)	238.56	0.00	Pass
(Bar Develop.) 1.933	599.26	169.99	5.7.3.5-2	Pass (3.53)	599.26	0.00	Pass
(CS) 2.313	614.74	361.25	5.7.3.5-1	Pass (1.70)	614.74	0.00	Pass
(H) 3.000	642.77	408.94	5.7.3.5-1	Pass (1.57)	642.77	0.00	Pass
($0.1L_s$) 3.917	680.13	469.72	5.7.3.5-1	Pass (1.45)	680.13	0.00	Pass
(1.5H) 4.375	698.34	498.78	5.7.3.5-1	Pass (1.40)	698.34	0.00	Pass
($0.2L_s$) 7.833	833.36	680.10	5.7.3.5-1	Pass (1.23)	833.36	0.00	Pass
($0.3L_s$) 11.750	881.77	819.38	5.7.3.5-1	Pass (1.08)	881.77	0.00	Pass
($0.4L_s$) 15.667	881.84	900.34	5.7.3.5-1	Pass* (0.98)	881.84	0.00	Pass
($0.5L_s$) 19.583	881.86	918.16	5.7.3.5-1	Pass* (0.96)	881.86	0.00	Pass
($0.6L_s$) 23.500	881.84	900.34	5.7.3.5-1	Pass* (0.98)	881.84	0.00	Pass
($0.7L_s$) 27.417	881.77	819.38	5.7.3.5-1	Pass (1.08)	881.77	0.00	Pass
($0.8L_s$) 31.333	833.36	680.10	5.7.3.5-1	Pass (1.23)	833.36	0.00	Pass
(1.5H) 34.792	698.34	498.78	5.7.3.5-1	Pass (1.40)	698.34	0.00	Pass
($0.9L_s$) 35.250	680.13	469.72	5.7.3.5-1	Pass (1.45)	680.13	0.00	Pass
(H) 36.167	642.77	408.94	5.7.3.5-1	Pass (1.57)	642.77	0.00	Pass
(CS) 36.854	614.74	361.25	5.7.3.5-1	Pass (1.70)	614.74	0.00	Pass
(Bar Develop.) 37.233	599.26	169.99	5.7.3.5-2	Pass (3.53)	599.26	0.00	Pass

Location from Left Support (ft)	Capacity (kip)	Demand (kip)	Equation	Status (C/D)	$A_{ps}f_{ps}$ (kip)	A_sf_y (kip)	Status
(FoS) 38.917	238.56	169.99	5.7.3.5-2	Pass (1.40)	238.56	0.00	Pass

* The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone

Horizontal Interface Shear for Strength I Limit State

AASHTO LRFD BDS 5.7.4

Location from Left Support (ft)	5.7.4.5			5.7.4.2			5.7.4.1		
	s (in)	s_{max} (in)	Status	a_{vf} (in^2/ft)	$a_{vf\ min}$ (in^2/ft)	Status	$\|v_{ui}\|$ (kip/ft)	ϕv_{ni} (kip/ft)	Status ($\phi v_{ni}/\|v_{ui}\|$)
($0.0L_s$) 0.000	12.000	33.100	Pass	0.800	0.000	Pass	1.266	64.801	Pass (10+)
(FoS) 0.250	12.000	33.100	Pass	0.800	0.000	Pass	1.266	64.801	Pass (10+)
(Bar Develop.) 1.933	12.000	33.100	Pass	0.800	0.000	Pass	1.617	64.801	Pass (10+)
(CS) 2.313	12.000	33.100	Pass	0.800	0.000	Pass	1.617	64.801	Pass (10+)
(H) 3.000	12.000	33.100	Pass	0.800	0.000	Pass	1.574	64.801	Pass (10+)
($0.1L_s$) 3.917	12.000	33.100	Pass	0.800	0.000	Pass	1.516	64.801	Pass (10+)
(1.5H) 4.375	12.000	33.100	Pass	0.800	0.000	Pass	1.487	64.801	Pass (10+)
($0.2L_s$) 7.833	12.000	33.100	Pass	0.800	0.000	Pass	1.270	64.801	Pass (10+)
($0.3L_s$) 11.750	12.000	33.100	Pass	0.800	0.000	Pass	1.024	64.801	Pass (10+)
($0.4L_s$) 15.667	12.000	33.100	Pass	0.800	0.000	Pass	0.779	64.801	Pass (10+)
($0.5L_s$) 19.583	12.000	33.100	Pass	0.800	0.000	Pass	0.533	64.801	Pass (10+)
($0.6L_s$) 23.500	12.000	33.100	Pass	0.800	0.000	Pass	0.779	64.801	Pass (10+)
($0.7L_s$) 27.417	12.000	33.100	Pass	0.800	0.000	Pass	1.024	64.801	Pass (10+)
($0.8L_s$) 31.333	12.000	33.100	Pass	0.800	0.000	Pass	1.270	64.801	Pass (10+)
(1.5H) 34.792	12.000	33.100	Pass	0.800	0.000	Pass	1.487	64.801	Pass (10+)
($0.9L_s$) 35.250	12.000	33.100	Pass	0.800	0.000	Pass	1.516	64.801	Pass (10+)
(H) 36.167	12.000	33.100	Pass	0.800	0.000	Pass	1.574	64.801	Pass (10+)
(CS) 36.854	12.000	33.100	Pass	0.800	0.000	Pass	1.617	64.801	Pass (10+)
(Bar Develop.) 37.233	12.000	33.100	Pass	0.800	0.000	Pass	1.617	64.801	Pass (10+)
(FoS) 38.917	12.000	33.100	Pass	0.800	0.000	Pass	1.266	64.801	Pass (10+)
($1.0L_s$) 39.167	12.000	33.100	Pass	0.800	0.000	Pass	1.266	64.801	Pass (10+)

Principal Tensile Stresses in Webs

Principal Tensile Stresses in Webs limitations are not applicable.
Concrete strength does not exceed the 10.000 KSI threshold

Live Load Deflection Check [2.5.2.6.2]

Allowable deflection span ratio = L/800

Allowable maximum deflection = ± 0.587 in
Minimum live load deflection along girder = -0.476 in
Maximum live load deflection along girder = 0.000 in
Status = Pass

Check for Lifting in Casting Yard

Lifting analysis disabled in Project Criteria. No analysis performed.
Per LRFD 5.5.4.3, "Buckling and stability of precast members during handling, transportation, and erection shall be investigated." Also see C5.5.4.3 and C5.12.3.2.1.

Check for Hauling to Bridge Site

Hauling analysis disabled in Project Criteria. No analysis performed.
Per LRFD 5.5.4.3, "Buckling and stability of precast members during handling, transportation, and erection shall be investigated." Also see C5.5.4.3 and C5.12.3.2.1.

Constructability Checks

Girder Dimensions Detailing Check [5.12.3.2.2]

Dimension	Minimum (in)	Actual (in)	Status
Top Flange Thickness	2.000	5.500	Pass
Web Thickness	5.000	5.000	Pass
Bottom Flange Thickness	5.000	5.500	Pass

Stirrup Detailing Check: Strength I [5.7.2.5, 5.7.2.6, 5.10.3.1.2]

Location from Left Support (ft)	Bar Size	S (in)	S_{max} (in)	S_{min} (in)	A_v/S (in^2/ft)	A_v/S_{min} (in^2/ft)	Status
($0.0L_s$) 0.000	#6	12.000	10.297	2.745	0.880	0.141	Fail
(FoS) 0.250	#6	12.000	10.242	2.745	0.880	0.141	Fail
(Bar Develop.) 1.933	#6	12.000	9.919	2.745	0.880	0.141	Fail
(CS) 2.313	#6	12.000	9.902	2.745	0.880	0.141	Fail
(H) 3.000	#6	12.000	9.870	2.745	0.880	0.141	Fail
($0.1L_s$) 3.917	#6	12.000	9.835	2.745	0.880	0.141	Fail
(1.5H) 4.375	#6	12.000	9.822	2.745	0.880	0.141	Fail
($0.2L_s$) 7.833	#6	12.000	19.408	2.745	0.880	0.141	Pass
($0.3L_s$) 11.750	#6	12.000	19.314	2.745	0.880	0.141	Pass
($0.4L_s$) 15.667	#6	12.000	19.314	2.745	0.880	0.141	Pass
($0.5L_s$) 19.583	#6	12.000	19.313	2.745	0.880	0.141	Pass
($0.6L_s$) 23.500	#6	12.000	19.314	2.745	0.880	0.141	Pass
($0.7L_s$) 27.417	#6	12.000	19.314	2.745	0.880	0.141	Pass
($0.8L_s$) 31.333	#6	12.000	19.408	2.745	0.880	0.141	Pass
(1.5H) 34.792	#6	12.000	9.822	2.745	0.880	0.141	Fail
($0.9L_s$) 35.250	#6	12.000	9.835	2.745	0.880	0.141	Fail
(H) 36.167	#6	12.000	9.870	2.745	0.880	0.141	Fail
(CS) 36.854	#6	12.000	9.902	2.745	0.880	0.141	Fail
(Bar Develop.) 37.233	#6	12.000	9.919	2.745	0.880	0.141	Fail
(FoS) 38.917	#6	12.000	10.242	2.745	0.880	0.141	Fail
($1.0L_s$) 39.167	#6	12.000	10.297	2.745	0.880	0.141	Fail

SEE STRUCTURAL DRAWINGS PAGE 25 OUT OF 41 (NEXT PAGE) FOR REINFORCING STEEL DETAILING. EXCERPTS OF RELEVANT CODE IS ON THE PAGE AFTER. THE DRAWINGS SHOW THAT #6 @ 1'-0" STIRRUPS ARE USED. THE ENDS SHOW (2)#4 @ 0'-4" FOR ~0'-10". THE BOX GIRDER DOES NOT MEET AASHTO DETAILING REQUIREMENTS.

Haunch Geometry Checks

Slab Offset ("A" Dimension)

This table compares the input slab offset to the rounded computed slab offset required to have the least haunch depth be equal to the Fillet dimension. A failed status indicates that the top of the girder will encroach into the deck slab and the Slab Offset dimension should be refined.

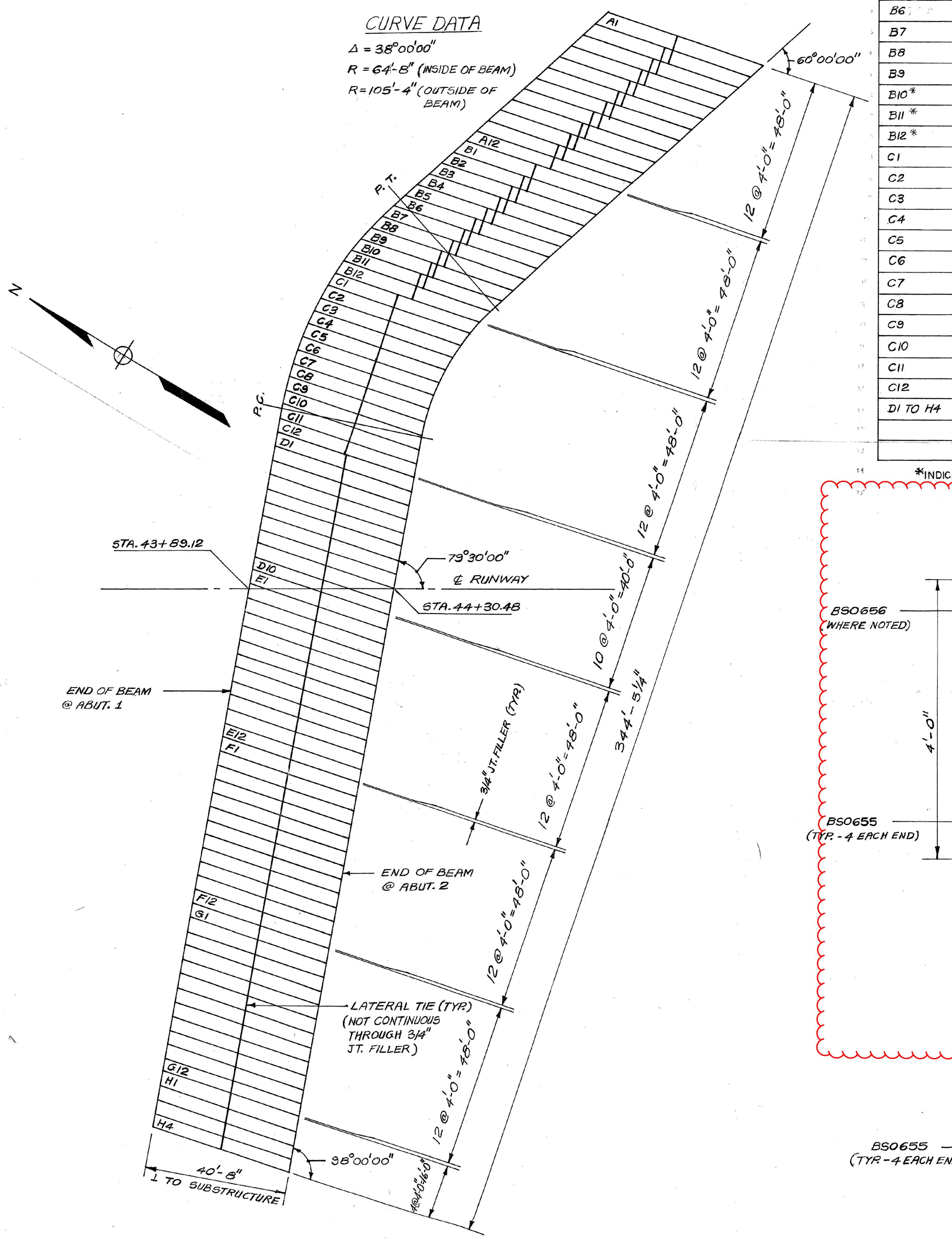
K&E 19 1253 4-83 A1318*

⚠ BEAM SCHEDULE

BEAM NO.	LENGTH D1	LENGTH D2	ABUTMENT 1 END A1	B1	C1	E1	F1	θ1	ABUTMENT 2 END A2	B2 *	C2	E2 *	F2 *	θ2 *	REINF. SPACING A	N1	N2	LONG. REINF.	BS0656
A1 TO B5	46'-11 1/2"	46'-11 1/2"	1'-6"	(2'-3 11/16")	23'-5 3/4	0'-10 3/8"	3'-9 11/16	120°-00'-00"	1'-6"	(2'-3 11/16")	23'-5 3/4"	0'-10 3/8	3'-9 11/16	120°-00'-00"	2'-8"	44	43	BL0454	EACH END
B6	46'-11 5/16	46'-11 1/2"	1'-6"	(2'-3 9/16")	23'-5 11/16"	0'-10 3/8"	3'-9 9/16"	119°-50'-29"	1'-6"	(2'-3 11/16")	23'-5 11/16	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-8"	44	43	"	"
B7	46'-9"	46'-11 5/16	1'-6"	(2'-1 3/8")	23'-4 1/2"	0'-10 3/16"	3'-7 3/8"	117°-52'-33"	1'-6"	(2'-3 11/16")	23'-4 1/2"	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-5"	44	43	BL0452,453	"
B8	46'-4 1/8	46'-9"	1'-6"	(1'-10 13/16")	23'-2 1/16	0'-9 15/16"	3'-4 13/16"	115°-26'-25"	1'-6"	(2'-3 11/16")	23'-2 1/16"	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-2"	44	43	"	"
B9	45'-8 13/16"	46'-4 1/8"	1'-6"	(1'-8 7/16")	22'-10 7/16"	0'-9 3/4"	3'-2 7/16"	113°-03'-11"	1'-6"	(2'-3 11/16")	22'-10 7/16"	0'-10 3/8"	3'-9 11/16"	120°-00'-00"	2'-0"	43	43	"	"
B10*	44'-11 1/4"	45'-8 13/16"	1'-6"	(1'-6 1/8")	22'-5 5/8"	0'-9 5/8"	3'-0 1/8"	110°-42'-27"	1'-6"	(2'-3 11/16")*	22'-5 5/8"	0'-10 3/8"	3'-9 11/16"*	120° 00'00" *	2'-1"	42	42	"	"
B11 *	43'-11 11/16"	44'-11 1/4"	1'-6"	(1'-3 15/16")	21'-11 13/16	0'-9 1/2"	2'-9 15/16"	108°-23'-52"	1'-6"	(2'-3 9/16")*	21'-11 13/16"	0'-10 3/8"*	3'-9 9/16"*	119°50'54" *	1'-9"	42	41	"	"
B12 *	43'-1 3/16"	43'-11 11/16"	1'-6"	(1'-1 7/8")	21'-6 5/8"	0'-9 3/8"	2'-7 7/8"	106°-07'-08"	1'-6"	(2'-0 3/8")*	21'-6 5/8"	0'-10 1/16"*	3'-6 3/8"*	116°53'53" *	1'-9"	41	40	"	"
C1	42'-4 9/16	43'-1 1/16"	1'-6"	(0'-11 13/16")	19'-3 3/8"	0'-9 1/4"	2'-5 13/16"	103°-49'-50"	1'-6"	(1'-8 5/8")	23'-1 3/16"	0'-9 3/4"	3'-2 5/8"	112°-55'-31"	1'-7"	40	40	"	"
C2	41'-9 3/4"	42'-4 9/16"	1'-6"	(0'-9 7/8")	20'-1 1/4"	0'-9 3/16"	2'-3 7/8"	101°-35'-59"	1'-6"	(1'-4 5/8")	21'-8 9/16	0'-9 1/2"	2'-10 5/8"	109°-07'-30"	1'-6"	40	40	"	"
C3	41'-4 1/2"	43'-9 3/4"	1'-6"	(0'-7 15/16")	20'-9 3/16	0'-9 1/8"	2'-1 15/16"	99°-23'-10"	1'-6"	(1'-1 1/4")	20'-7 5/16"	0'-9 5/16"	2'-7 1/4"	105°-24'-39"	1'-7"	39	38	"	"
C4	41'-0 9/16"	41'-4 1/2"	1'-6"	(0'-6 1/16")	21'-3 3/16"	0'-9 1/16"	2'-0 1/16"	97°-11'-13"	1'-6"	(0'-10")	19'-9 5/16"	0'-9 3/16"	2'-4"	101°-45'-43"	1'-4"	39	38	"	"
C5	40'-9 7/8"	41'-0 9/16"	1'-6"	(0'-4 3/16")	21'-7 7/16"	0'-9 1/16"	1'-10 3/16"	94°-59'-54"	1'-6"	(0'-6 7/8")	19'-2 7/16"	0'-9 1/16"	2'-0 7/8"	98°-09'-40"	1'-4"	38	38	"	ABUT.2 END
C6	40'-8 3/8"	40'-9 7/8"	1'-6"	(0'-2 3/8")	21'-9 3/4"	0'-9"	1'-8 3/8"	92°-49'-01"	1'-6"	(0'-3 7/8")	18'-10 9/16"	0'-9"	1'-9 7/8"	94°-35'-34"	1'-2"	38	38	BL0452	—
C7	40'-8"	40'-8 3/8"	1'-6"	(0'-0 9/16")	21'-10 5/16"	0'-9"	1'-6 9/16"	90°-38'-22"	1'-6"	(0'-0 7/8")	18'-9 11/16"	0'-9"	1'-6 7/8"	91°-02'-31"	1'-1"	38	38	"	—
C8	40'-8 13/16	40'-8"	1'-7 5/16"	0'-1 5/16"	21'-9"	0'-9"	1'-6"	88°-27'-47"	1'-8 1/8"	0'-2 1/8"	18'-11 13/16"	0'-9"	1'-6"	87°-29'-44"	1'-1"	38	38	"	—
C9	40'-10 13/16"	40'-8 13/16	1'-9 1/8"	0'-3 1/8"	21'-5 7/8"	0'-9"	1'-6"	86°-17'-04"	1'-11 1/8"	0'-5 1/8"	19'-4 7/8"	0'-9 1/16"	1'-6"	83°-56'-21"	1'-0"	38	38	"	—
C10	41'-0 5/8"	40'-10 13/16"	1'-10 15/16	0'-4 15/16"	21'-1 1/8"	0'-9 1/16"	1'-6"	84°-06'-02"	2'-0 3/4"	0'-6 3/4"	19'-11 5/8"	0'-9 1/16"	1'-6"	82°-00'-13"	0'-10"	38	38	BL0451	—
C11	41'-0 13/16	41'-0 5/8"	2'-0 9/16"	0'-6 9/16"	20'-6 3/8"	0'-9 1/16"	1'-6"	82°-13'-51"	2'-0 3/4"	0'-6 3/4"	20'-6 7/8"	0'-9 1/16"	1'-6"	82°-00'-00"	0'-9"	39	38	"	—
C12	41'-0 13/16	41'-0 13/16"	2'-0 9/16"	0'-6 9/16"	19'-11 5/8"	0'-9 1/16"	1'-6"	82°-00'-00"	2'-0 3/4"	0'-6 3/4"	21'-1 1/8"	0'-9 1/16"	1'-6"	82°-00'-00"	0'-9"	39	38	"	—
D1 TO H4	41'-0 13/16"	41'-0 13/16"	2'-0 9/16"	0'-6 9/16"	20'-6 3/8"	0'-9 1/16"	1'-6"	82°-00'-00"	2'-0 3/4"	0'-6 3/4"	20'-6 3/8"	0'-9 1/16"	1'-6"	82°-00'-00"	0'-9"	39	38	"	—

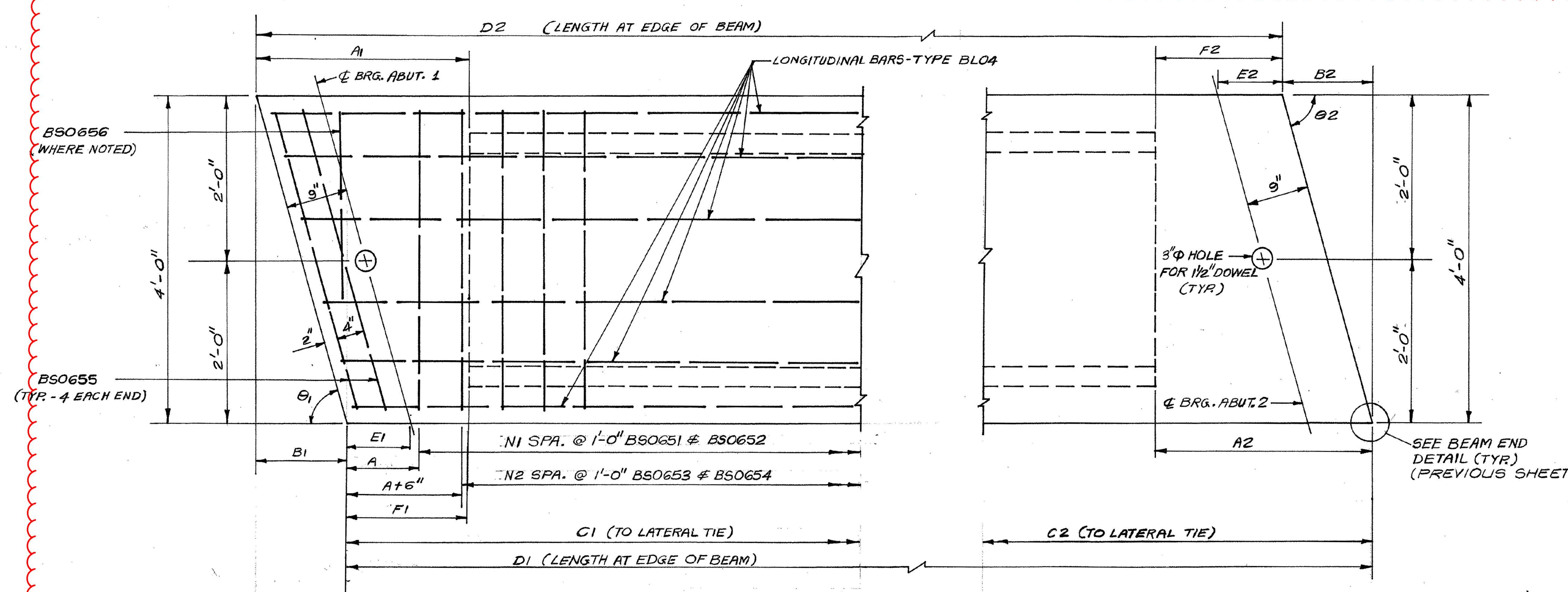
*INDICATES DIMENSION CHANGES

NOTE: DIMENSIONS IN ( ) DENOTE NEGATIVE DIMENSIONS



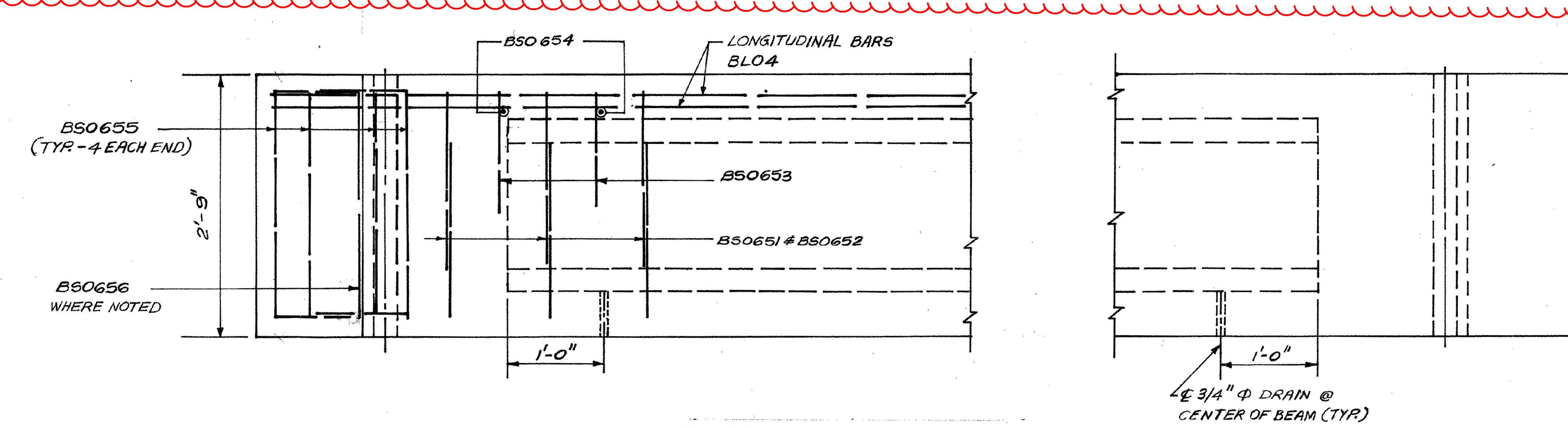
FRAMING PLAN

SCALE: 1"=20'



PLAN-BEAM

SCALE: 1"=1'-0"



ELEVATION-BEAM

SCALE: 1"=1'-0"

COMMONWEALTH OF VIRGINIA
LARRY L. ICHTER
LICENSE No. 11122
PROFESSIONAL ENGINEER

Larry L. Ichter 6/23/83

HDR Henningson, Durham & Richardson, Inc.
Engineers • Architects • Planners
Alexandria, Virginia

DESIGNED BY:	DRAWN BY:	CHECKED BY:	APPROVED BY:
LLI	UJS	LLI	HFR
DATE: JULY 1983	SCALE: AS NOTED		CMP PROJ. NO. CMP 8305

NO.	DATE	REVISIONS	BY	APPR.
⚠	5/87	Corr. To Bm. Sch.	REM	KHG

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

RUNWAY BRIDGE OVER BROAD RUN
FRAMING PLAN

PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
25 OF 41

5.7.2.6—Maximum Spacing of Transverse Reinforcement

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.125 f'_c$, then:

$$s_{max} = 0.8 d_v \leq 24.0 \text{ in.} \qquad (5.7.2.6\text{-}1)$$

- If $v_u \geq 0.125 f'_c$, then:

$$s_{max} = 0.4 d_v \leq 12.0 \text{ in.} \qquad (5.7.2.6\text{-}2)$$

where:

v_u = shear stress calculated in accordance with Article 5.7.2.8 (ksi)
d_v = effective shear depth as defined in Article 5.7.2.8 (in.)

For segmental post-tensioned concrete box girder bridges, spacing of closed stirrups or closed ties required to resist shear effects due to torsional moments shall not exceed one-half of the shortest dimension of the cross-section, nor 12.0 in.

5.7.2.5—Minimum Transverse Reinforcement

Where transverse reinforcement is required as specified in either Article 5.7.2.3 or Article 5.12.5.3.8c, and nonprestressed reinforcement is used to satisfy that requirement, the area of steel shall satisfy:

$$A_v \geq 0.0316\, \lambda \sqrt{f'_c}\, \frac{b_v s}{f_y} \qquad (5.7.2.5\text{-}1)$$

where:

A_v = area of transverse reinforcement within distance s (in.^{2})

b_v = width of web adjusted for the presence of ducts as specified in Article 5.7.2.8 (in.)
s = spacing of transverse reinforcement (in.)
f_y = yield strength of transverse reinforcement (ksi) $\leq$ 100 ksi
λ = concrete density modification factor as specified in Article 5.4.2.8

The design yield strength of prestressed transverse reinforcement in Eq. 5.7.2.5-1 shall be taken as the effective stress, after allowance for all prestress losses, plus 60.0 ksi, but not greater than f_{py}.

For segmental post-tensioned concrete box girder bridges, where transverse reinforcement is not required, as specified in Article 5.12.5.3.8c, the minimum area of transverse shear reinforcement per web shall not be less than the equivalent of two No. 4 Grade 60 reinforcement bars per foot of length.

5.10.3—Spacing of Reinforcement

5.10.3.1 Minimum Spacing of Reinforcing Bars

5.10.3.1.1—Cast-in-Place Concrete

For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- 1.5 times the nominal diameter of the bars;
- 1.5 times the maximum size of the coarse aggregate; or
- 1.5 in.

5.10.3.1.2—Precast Concrete

For precast concrete manufactured under plant control conditions, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- the nominal diameter of the bars;
- 1.33 times the maximum size of the coarse aggregate; or
- 1.0 in.

Span	Girder	Provided (in)	Required (in)	Status	Notes
1	A	5.000	0.500	Excessive	The difference between the minimum and maximum CL haunch depths along the girder is 0.298 in. This exceeds one half of the slab depth. Check stirrup lengths to ensure they engage the deck in all locations. Refer to the Haunch Details chapter in the Details report for more information. Provided Slab Offset exceeded Required by allowable tolerance of 0.500 in

Excess Camber Check

Haunch dead load is affected by variable haunch depth along the girder. Haunch depth along a girder is defined by the roadway geometry, slab offset ("A"), and the parabolic girder camber defined by the user input Assumed Excess Camber at mid-span. The table below compares the Assumed Excess Camber with the Computed Excess Camber. A failed status indicates the assumed value is not within tolerance of the computed value - meaning that results dependent on haunch dead load may be inaccurate. See the Haunch Details and Loading Details chapters in Details Report for more information.

Span	Girder	Computed Excess Camber (in)	Assumed Excess Camber (in)	Difference (in)	Allowable Difference (in)	Status	Notes
1	A	0.298	0.000	0.298	± 0.500	Pass	Assumed Excess Camber is within tolerance

Camber

	Camber (in)
Screed Camber, C at mid-span	0.166
Camber at 21 days, D_{21}	0.465

D01.24004.00 HEF Manassas Bridge Assessments
Bridge Superstructure Analysis PART 2: TAXIWAY BRIDGE

Site: Manassas Airport

Client is using bridges with airplanes that are heavier than what the bridges were originally designed for. We want to determine the maximum airplane weight these bridges can handle.

Please note, throughout the following calculation package, blue highlighted values are maximum allowable demands that max out the yellow highlighted demand to capacity ratios.

Testing has been conducted by WPM to determine actual distribution factors for the bridges.

Taxiway - DF = 0.35 unless noted otherwiwse (conservative since 0.32 was determined by Load Test)

Codes/Guidelines Utilized:
AASHTO LRFD 8th Edition

TAXIWAY BRIDGE - MAX AIRPLANE WEIGHT BASED ON FOLLOWING CRITERIA:

- Box Lid Failure
- - Moment: 775 kip
- - Shear: 475 kips

- Box Girder - PGSuper Analysis
- - Design Limit State: 223 kip
- - The analysis does indicate that the existing structure does not meet current AASHTO shear reinforcing steel detailing requirements at the ends of the girder. The stirrups in the box girder are #6 @ 12" for almost the entire length of the member. 10" on each end, the two sets of stirrups are spaced 4" apart. The box girder has adequate shear capacity, regardless.

- Foundation (Bearing Controlling): 564 kip

- Bearing Pads
- - Bearing pad thickness are inadequate per code however over 40 years of adequate in situ performance indicates in situ performance is adequate.
- - Checks done:
- - - Service Level Compression Stress: 174 kip
- - - Deflection Check: PASS
- - - Shear Deformation Check: FAIL. Bearing pads fail due to shear deformation even WITHOUT any plane load.
- - - Stability Check: PASS

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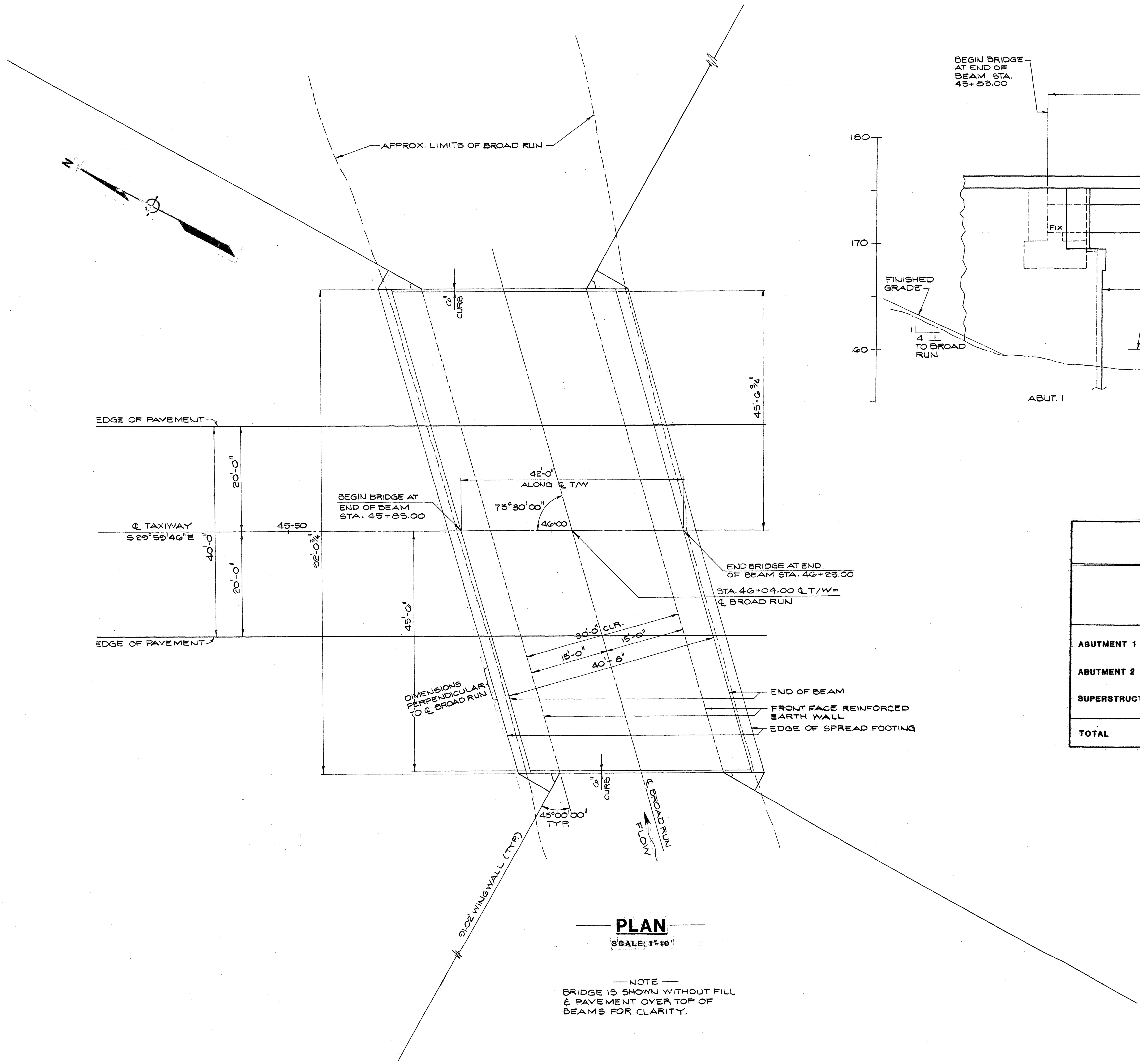
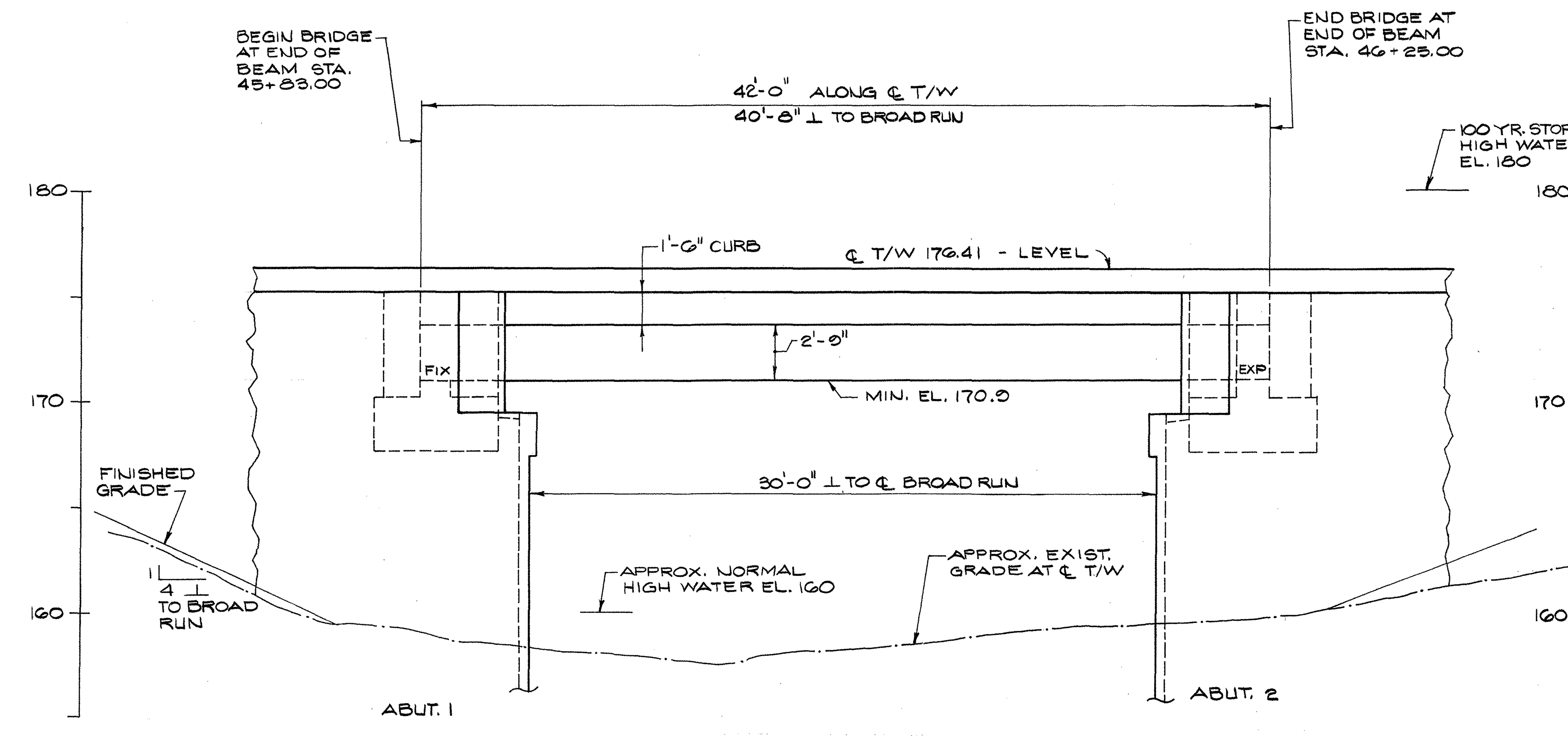


PLAN

SCALE: 1"=10'

— NOTE —
BRIDGE IS SHOWN WITHOUT FILL & PAVEMENT OVER TOP OF BEAMS FOR CLARITY.



ELEVATION

SCALE: 1"=5'
(SHOWN ⊥ TO BROAD RUN)

ESTIMATED QUANTITIES

	CAST-IN-PLACE CONCRETE C.Y.	PRESTRESSED CONCRETE BOX BEAMS (4'-0"x2'-9") EACH
ABUTMENT 1	63.2	———
ABUTMENT 2	63.2	———
SUPERSTRUCTURE	2.3	23
TOTAL	128.7	23

GENERAL NOTES

DESIGN SPECIFICATIONS: AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1977, INCLUDING INTERIM SPECIFICATIONS, 1978, 1979, 1980, 1981 & 1982.

LOADING: 737-200 AIRCRAFT LOADING. TOTAL GROSS LOAD 108,000 POUNDS - DUAL GEAR, 48,500 POUNDS EACH.

1 CONCRETE: CONCRETE IN PRESTRESSED MEMBERS - f'c = 5000 PSI. CONCRETE IN SUBSTRUCTURE - f'c = 4000 PSI.

REINFORCING STEEL: REINFORCING STEEL SHALL CONFORM TO ASTM A-615, GRADE 60. ALL REINFORCING BAR DIMENSIONS ON THE DETAILED DRAWINGS ARE TO CENTERS OF BARS EXCEPT WHERE OTHERWISE NOTED.

FOUNDATIONS: FOOTINGS FOR ABUTMENTS SHALL REST ON FIRM MATERIAL. BEARING CAPACITY OF FOUNDATION SHALL BE 3500 PSF.

CHAMFER: ALL EXPOSED CORNERS OF CONCRETE SHALL BE CHAMFERED WITH 3/4" x 3/4" MILLED CHAMFER STRIPS.

COMMONWEALTH OF VIRGINIA
LARRY L. ICHTER
LICENSE No. 11122
PROFESSIONAL ENGINEER
Larry L. Ichter 6/23/83

HDR
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DESIGNED BY: LLI	DRAWN BY: JCL	CHECKED BY: LLI	APPROVED BY: HFR
DATE: JULY 1983	SCALE: AS NOTED		CMP PROJ. NO. CMP 8305

NO.	DATE	REVISIONS	BY	APPR.
1	5/87	Rev. f'c	REM	KHG

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

TAXIWAY BRIDGE OVER BROAD RUN
PLAN & ELEVATION

PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
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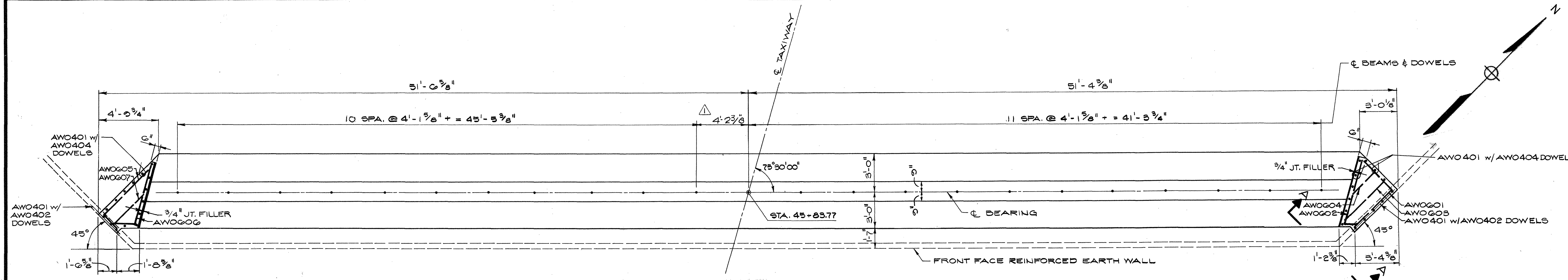
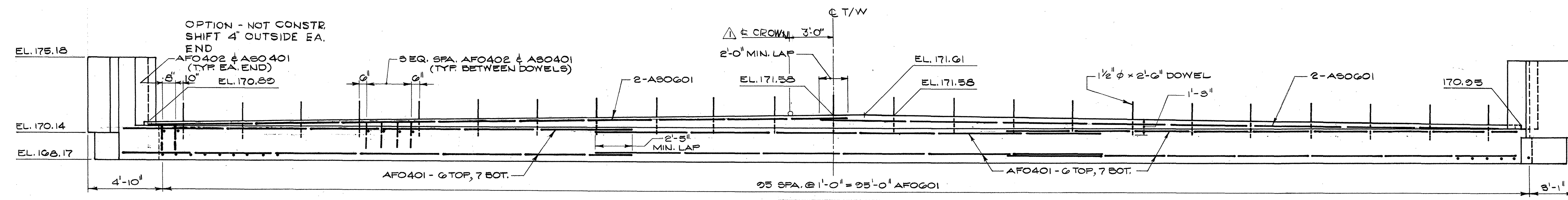


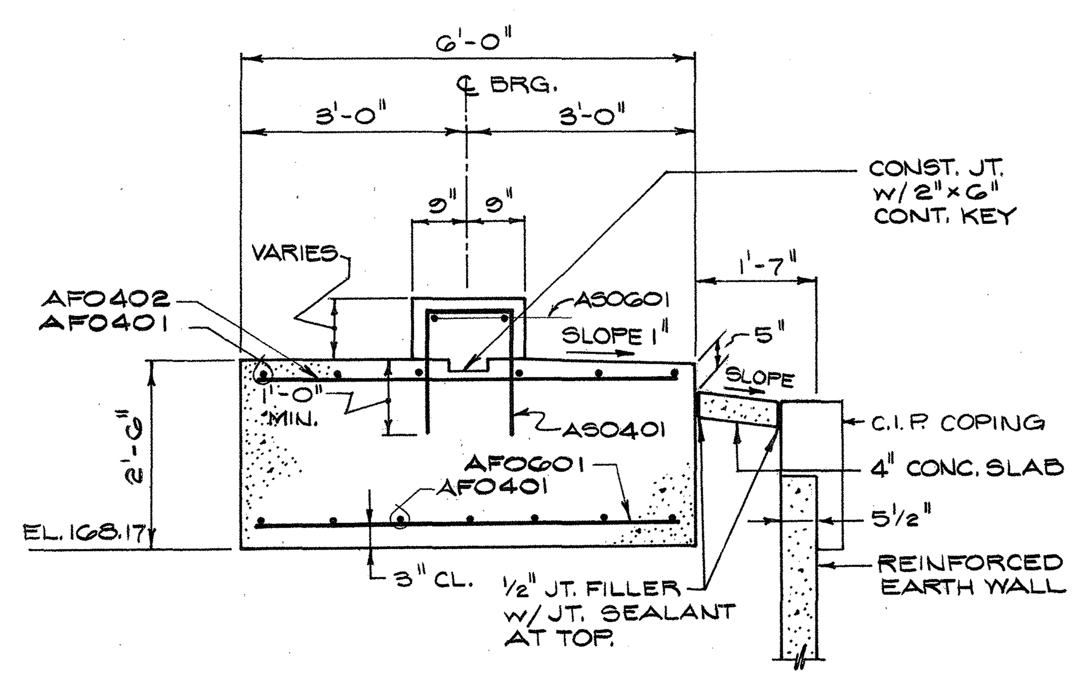
PLAN

SCALE: 1/4"=1'-0"



ELEVATION

SCALE: 1/4"=1'-0"



TYPICAL SECTION

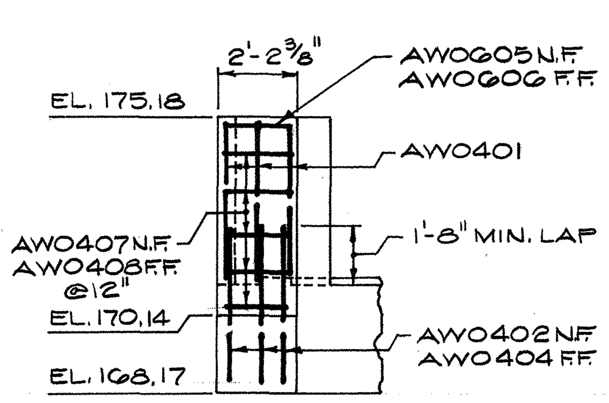
SCALE: 1/2"=1'-0"

NOTES:
1. MAINTAIN 2" CLR. UNLESS NOTED OTHERWISE.
2. PLACE 4" CONC. SLAB AS SOON AS POSSIBLE AFTER PLACING SPREAD FOOTING.
3. PLACE 3/4" JT. FILLER BETWEEN ALL INTERSECTING FACES OF CAST-IN-PLACE CONCRETE & REINFORCED EARTH WALL UNLESS NOTED OTHERWISE.
4. SEE NEXT SHEET FOR BEARING DETAILS.
5. PLACE SILICONE JOINT SEALANT IN ACCORDANCE WITH DETAIL THIS SHEET. ALL JOINTS SHALL BE SEALED WITH SILICONE SEAL.
6. ALL SILICONE SEAL SHALL COMPLY WITH ITEM P-605 TYPE B.

⚠ ABUTMENT CONSTRUCTION NOTES

1. CONSTRUCTION JOINTS SHALL BE SHALL SPACED AT INTERVALS NOT EXCEEDING 60 FEET. AT LEAST 48 HOURS SHALL ELAPSE BETWEEN PLACING ADJACENT SECTIONS.
2. THE 4" CONCRETE SLAB SHALL BE REINFORCED WITH WELDED WIRE FABRIC -WWF 6×6— W2.9 × W2.9. THE SLAB SHALL BE PLACED IN SECTIONS NOT EXCEEDING 50 FEET IN LENGTH. AT LEAST 48 HOURS SHALL ELAPSE BETWEEN PLACING ADJACENT SECTIONS. A 1/2" JOINT WITH JOINT SEALANT SHALL SEPARATE ADJACENT SECTIONS. REINFORCING SHALL NOT BE CONTINUOUS THROUGH THESE JOINTS.

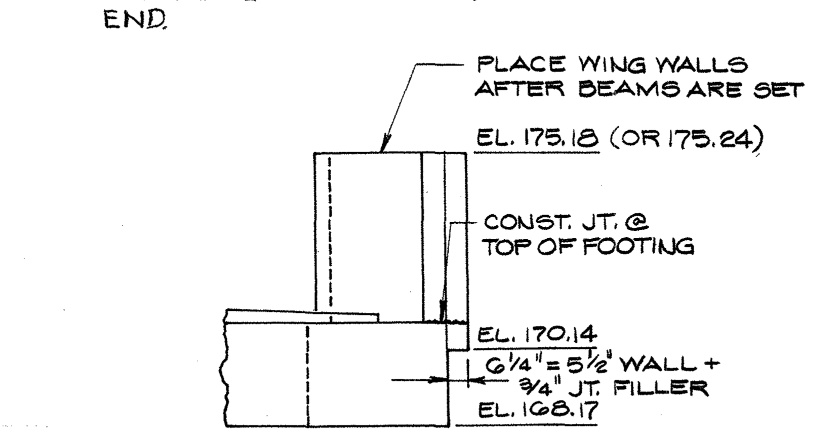
⚠ PER LTR. NO. 23
EXTEND BM. SEAT 4' EA. END.



ELEVATION

WEST WINGWALL - ABUT. 1
EAST WINGWALL - ABUT. 2

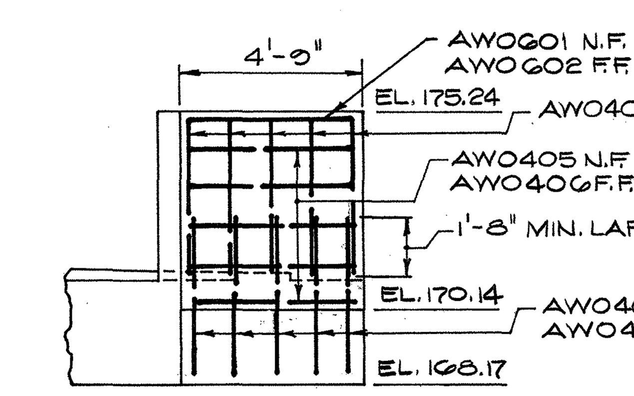
SCALE: 1/4"=1'-0"



SECTION A-A

WEST WINGWALL - ABUT. 1 SHOWN
OTHER WINGWALL SIMILAR

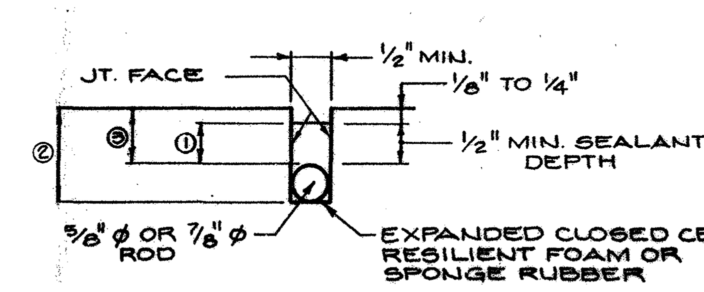
SCALE: 1/4"=1'-0"



ELEVATION

EAST WINGWALL - ABUT. 1
WEST WINGWALL - ABUT. 2

SCALE: 1/4"=1'-0"



JOINT WIDTH	SEALANT BEAD THICKNESS ①	EXPANDED CLOSED CELL ROD DIAMETER
1/2"	1/2"	5/8"
3/4"	1/2"	7/8"

JOINT WIDTH	MINIMUM JOINT DEPTH ②	EXPANDED CLOSED CELL ROD DEPTH ③
1/2"	1 3/8"	3/4"
3/4"	1 7/8"	3/4"

⚠ NOTE
POLYETHYLEME BOND BREAKING TAPE SUBSTITUTED BACKER ROD.

SILICONE JOINT SEALANT INSTALLATION DETAIL

N. T. S.

COMMONWEALTH OF VIRGINIA
LARRY L. ICHTER
LICENSE No. 11122
PROFESSIONAL ENGINEER
Larry L. Ichter 6/23/83

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Alexandria, Virginia

DESIGNED BY:	DRAWN BY:	CHECKED BY:	APPROVED BY:
LLI	JCL	LLI	HFR
DATE: JULY 1983	SCALE: AS NOTED		CMP PROJ. NO. CMP 8305

NO.	DATE	REVISIONS	BY	APPR.
⚠	5/87	Corr. Loc., Dim., Of Crown, Add. Notes	REM	KHG

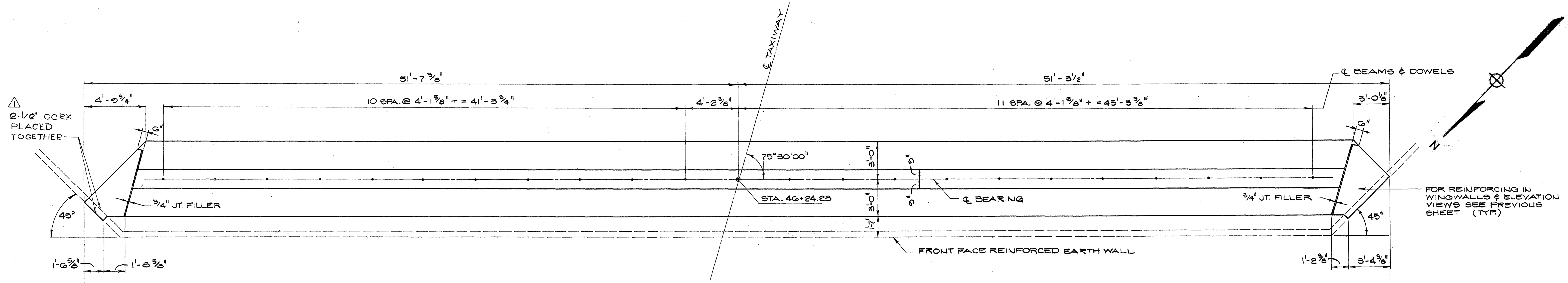
MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

TAXIWAY BRIDGE OVER BROAD RUN
ABUTMENT 1

PROJECT NO.
AIP 5-51-0030-01

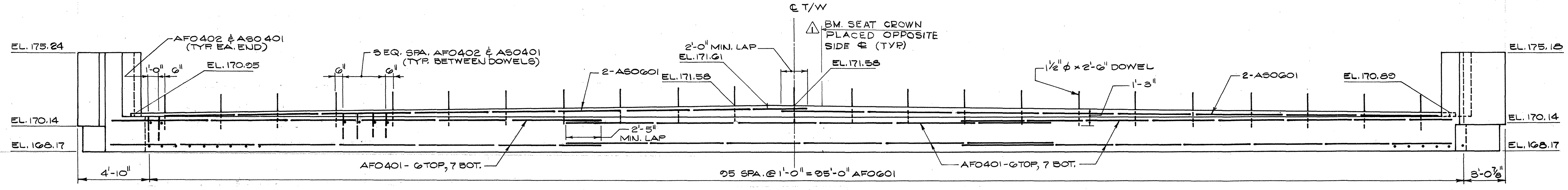
SHEET NO.
30 OF 41

K&E 19 1253 4-83 A1318*



PLAN

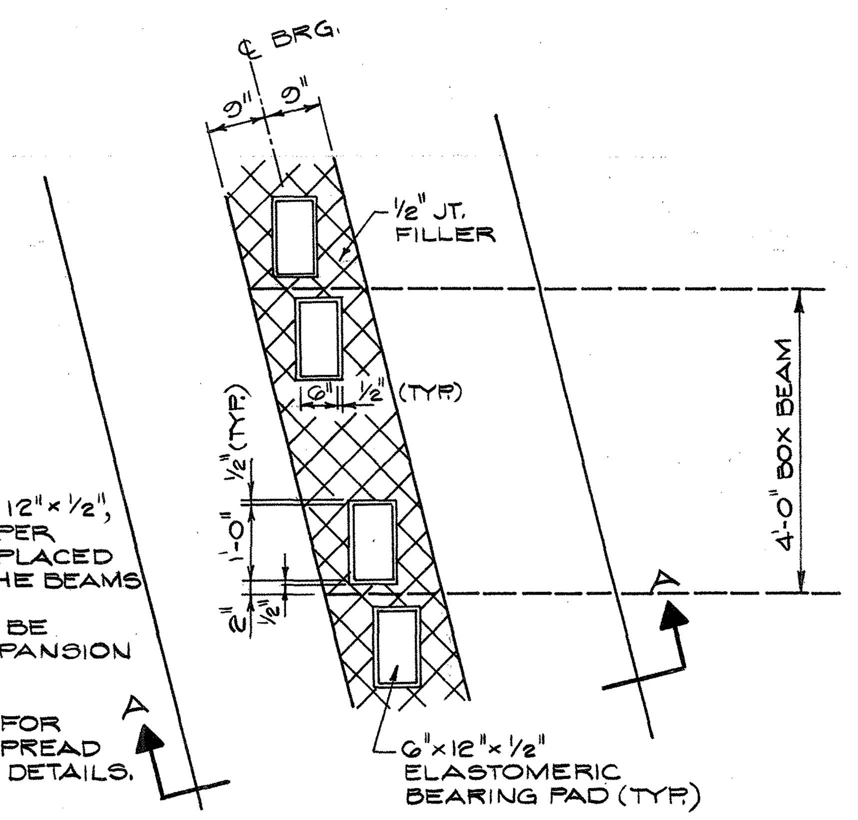
SCALE: 1/4"=1'-0"



ELEVATION

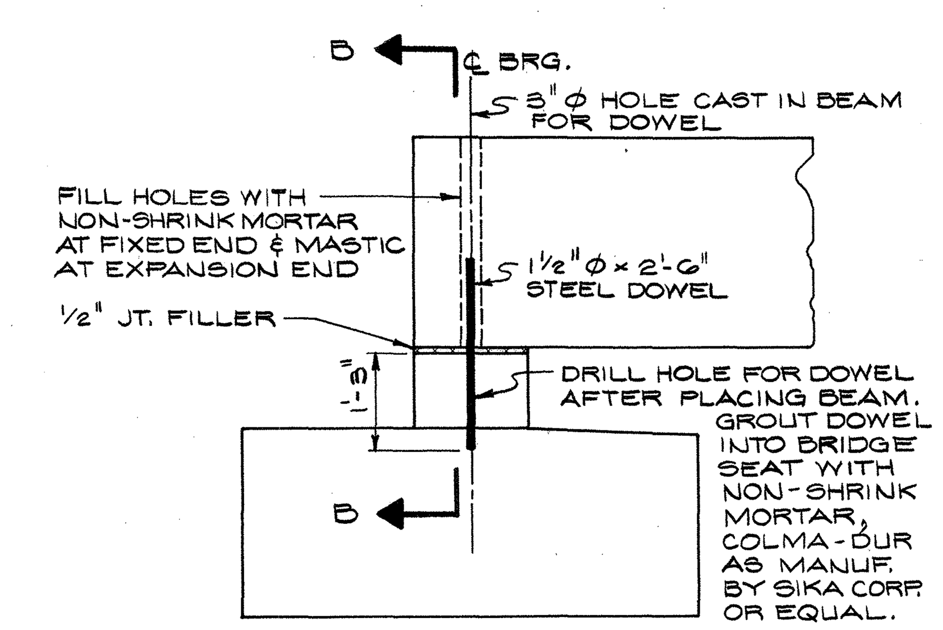
SCALE: 1/4"=1'-0"

NOTES:
1. FOUR BEARING PADS 6"×12"×1/2", 70 DUROMETER REQ'D. PER BEAM. BEARINGS ARE PLACED PERPENDICULAR TO THE BEAMS.
2. ALL JT. FILLER SHALL BE PREFORMED CORK EXPANSION JOINT MATERIAL.
3. SEE PREVIOUS SHEET FOR TYPICAL SECTION OF SPREAD FOOTING & WINGWALL DETAILS.



BEARING LAYOUT

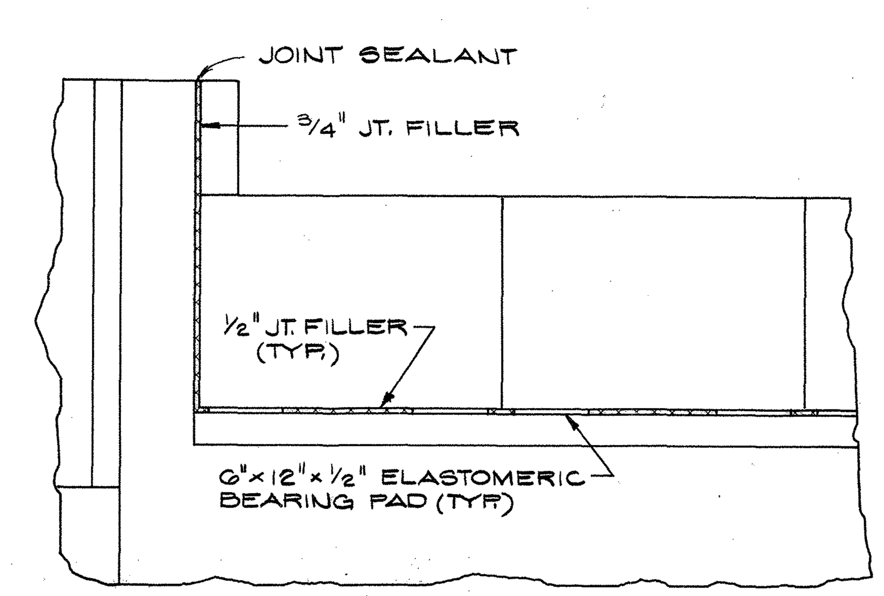
SCALE: 1/2"=1'-0"



SECTION A-A

SCALE: 1/2"=1'-0"

⚠ PER LTR. NO. 03
EXTEND BM. SEAT 4" EA. END.



SECTION B-B

SCALE: 1/2"=1'-0"

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LLI	JCL	LLI	HFR
DATE: JULY 1983	SCALE: AS NOTED		CMP PROJ. NO. CMP 8305

NO.	DATE	REVISIONS	BY	APPR
⚠	5/87	Corr. Loc., Dim. Of Crown, Add. Notes	REM	KHG

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

TAXIWAY BRIDGE OVER BROAD RUN
ABUTMENT 2

PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
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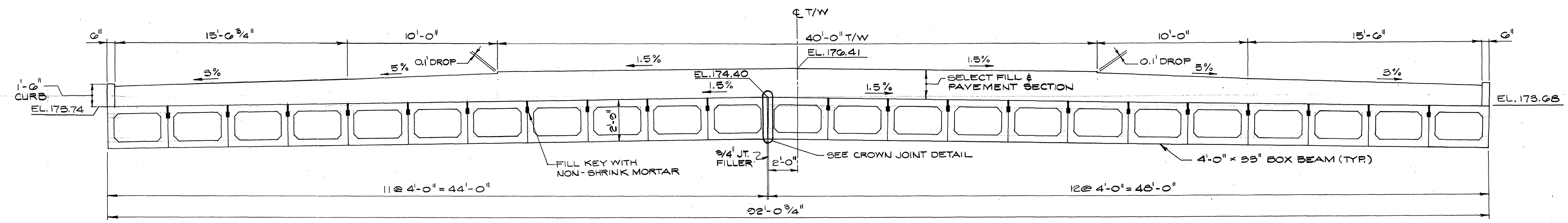
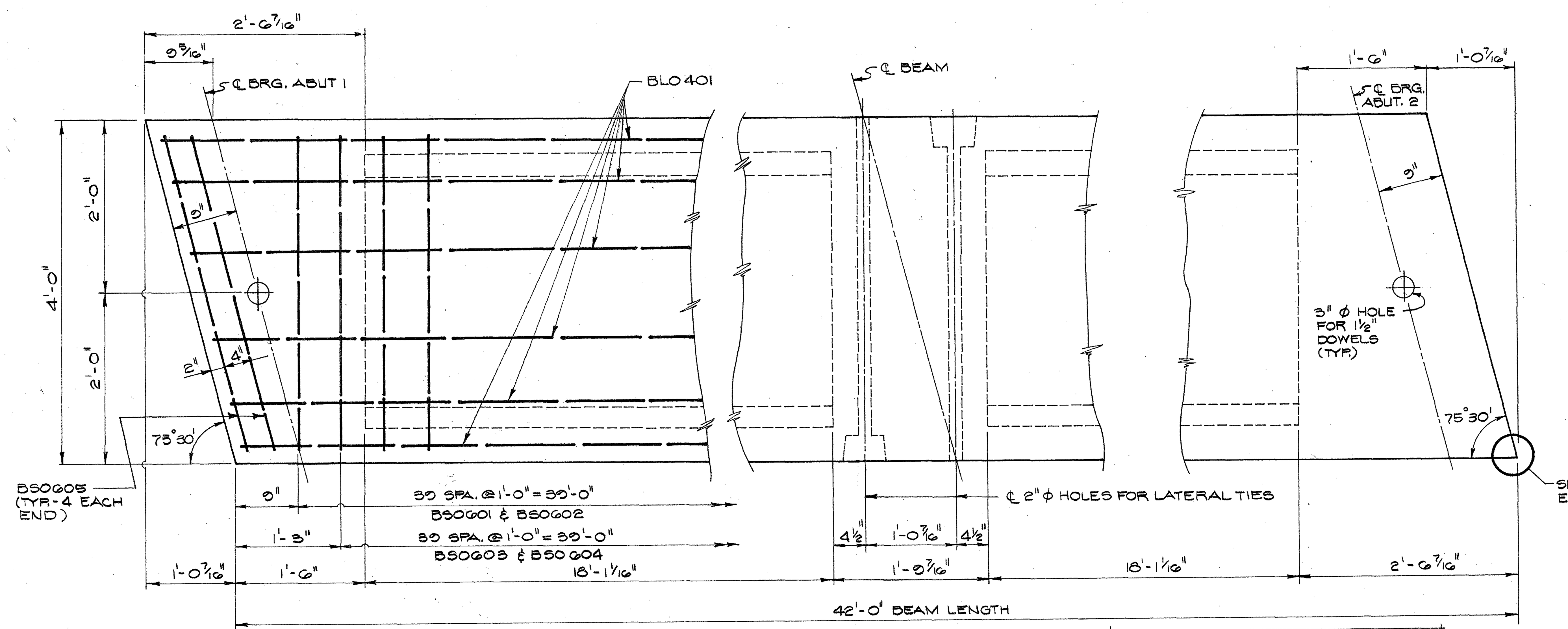
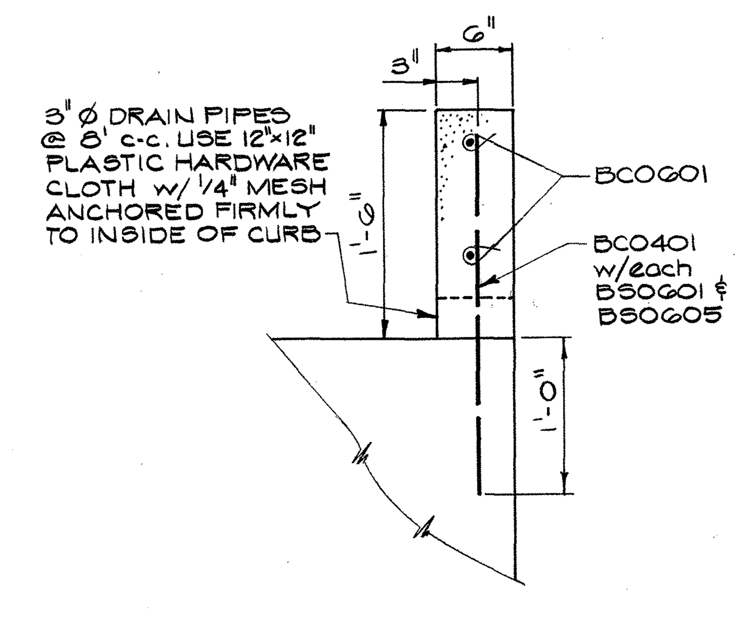


TYPICAL SECTION
SCALE: 1/4"=1'-0"

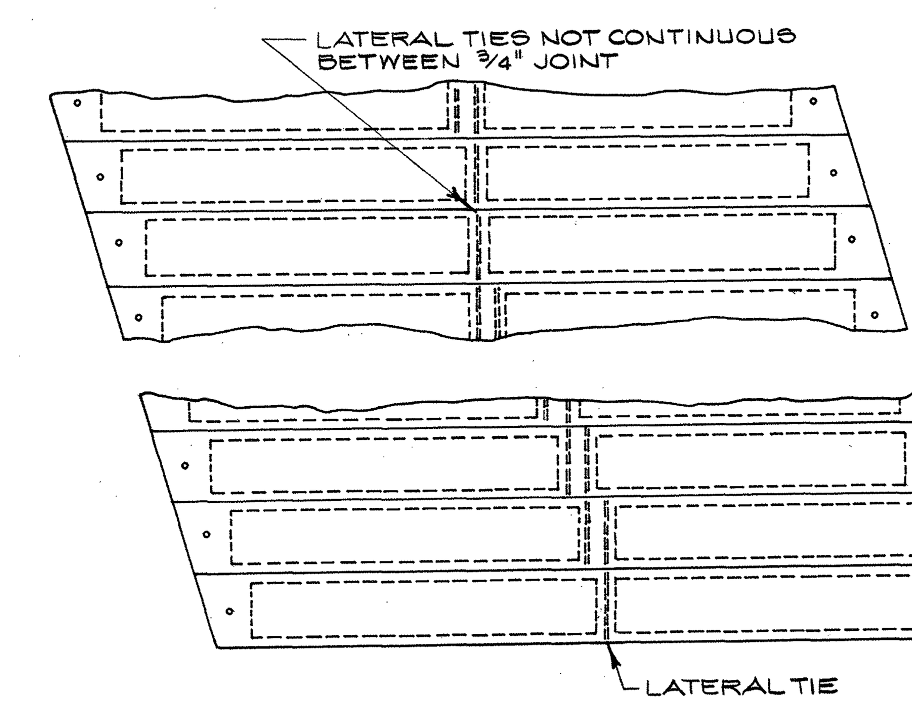


PLAN
SCALE: 1"=1'-0"

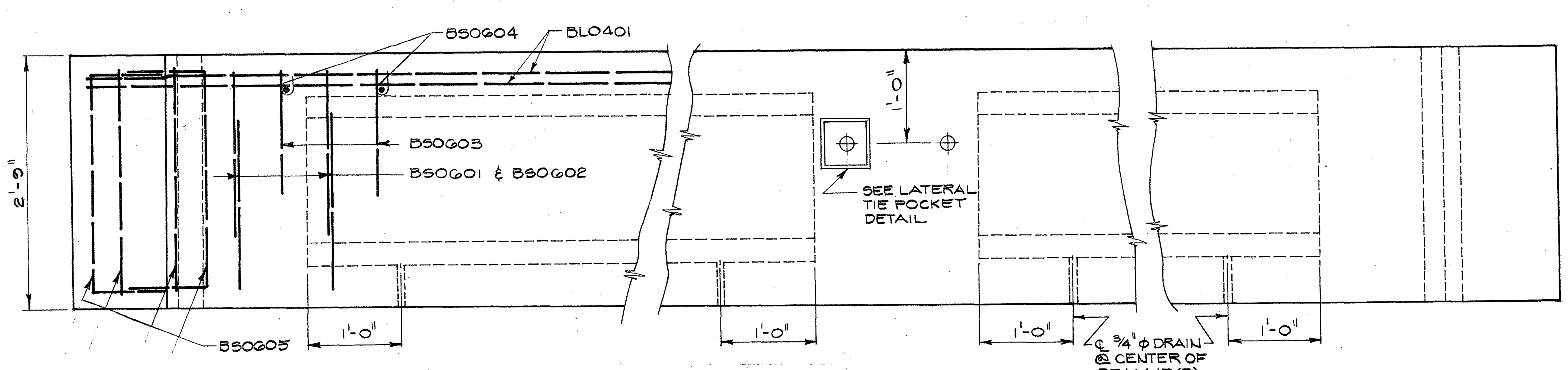
NOTE: KEYWAY & CHAMFERED EDGES NOT SHOWN FOR CLARITY.



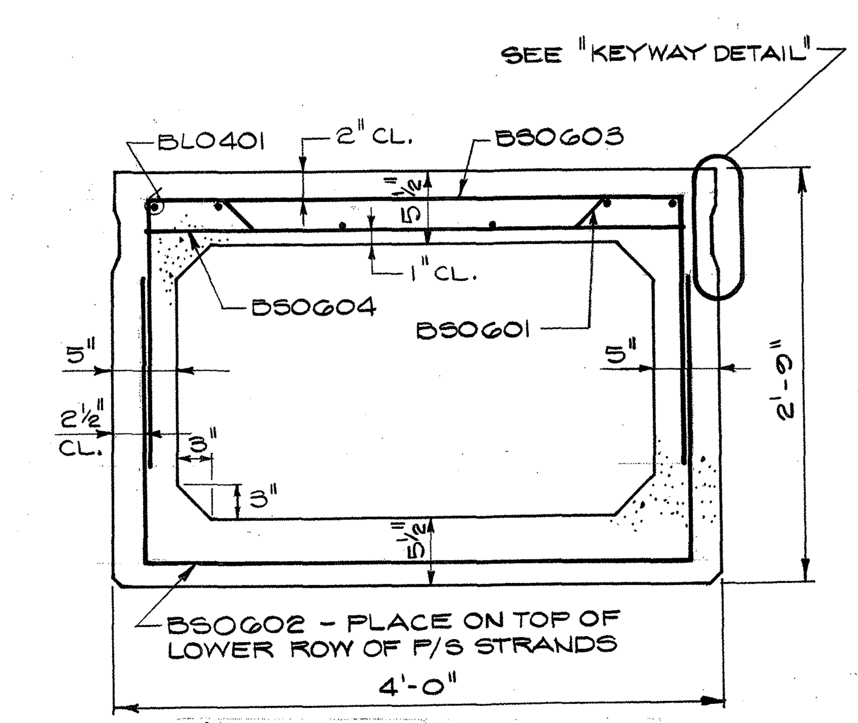
CURB DETAIL
(EXTERIOR BEAMS)
SCALE: 1"=1'-0"



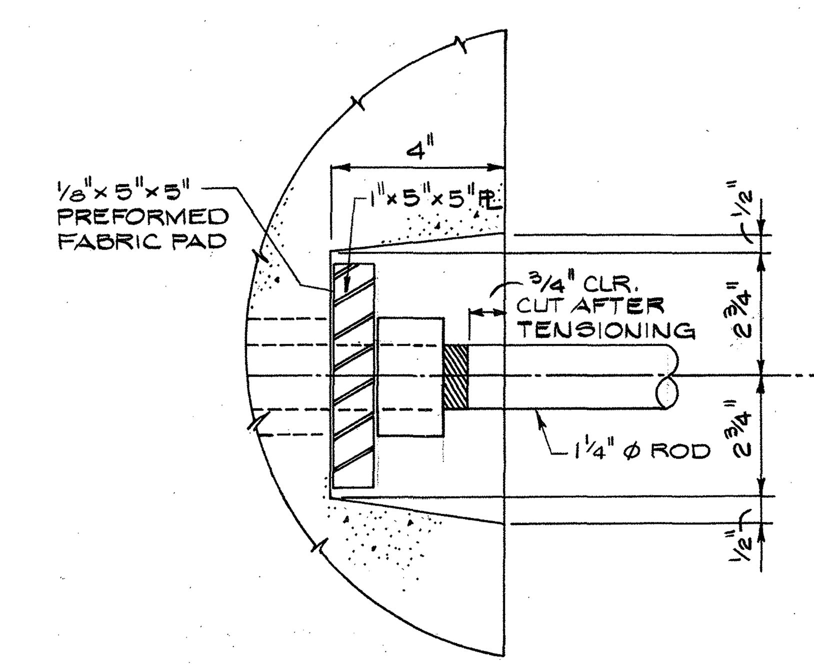
LATERAL TIE PLAN
N. T. S.



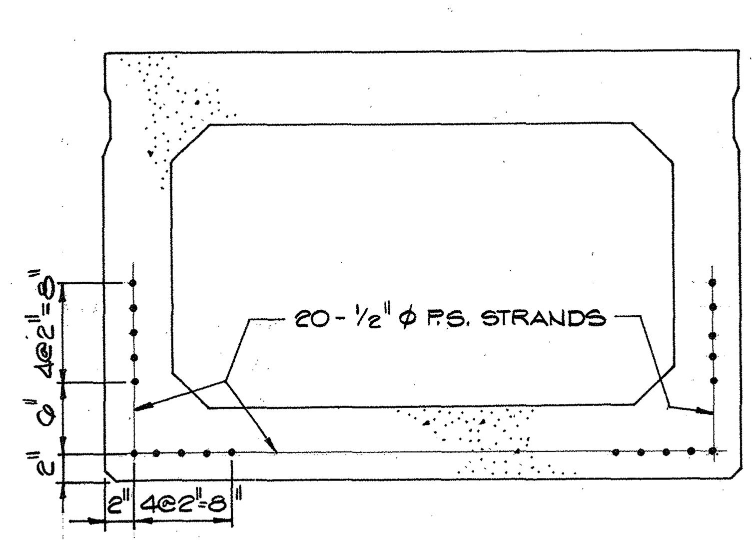
ELEVATION
SCALE: 1"=1'-0"



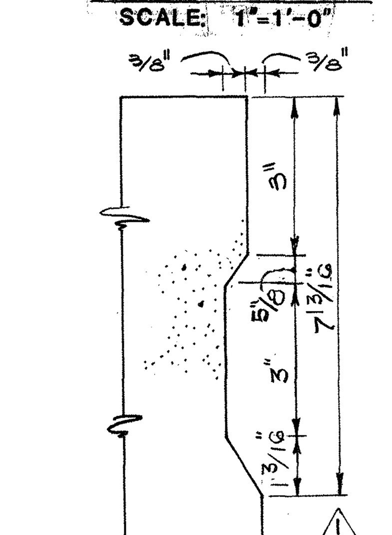
TYPICAL SECTION
SCALE: 1"=1'-0"
(INTERIOR BEAM SHOWN)



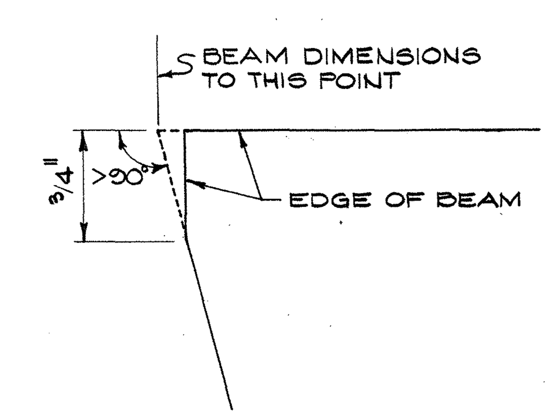
LATERAL TIE POCKET DETAIL
N. T. S.



STRAND PATTERN
SCALE: 1"=1'-0"



KEYWAY DETAIL
N. T. S.



BEAM END DETAIL
N. T. S.

NOTES:

1. PRESTRESSING STRANDS SHALL BE 1/2" Ø STRESS-RELIEVED STRANDS, GRADE 270. THE INITIAL PRESTRESSING FORCE SHALL BE 28,900 POUNDS PER STRAND.

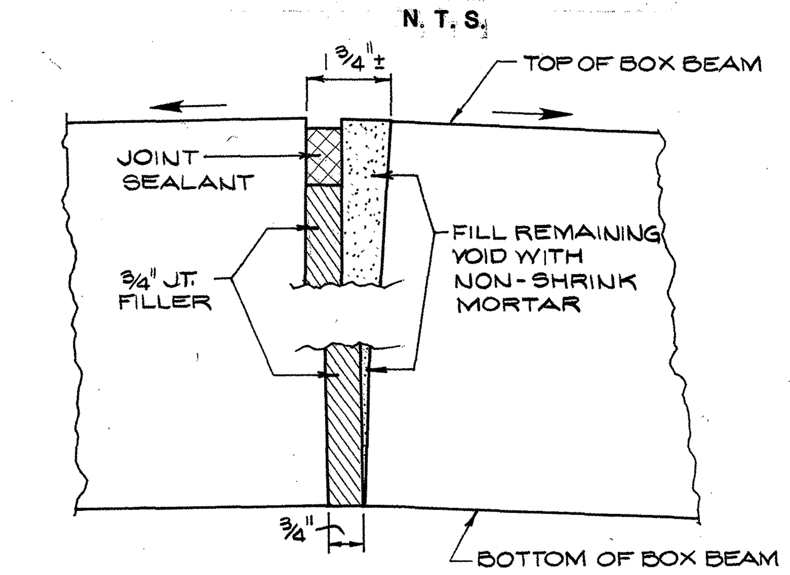
2. PRESTRESSING STRANDS SHALL NOT BE RELEASED UNTIL THE CONCRETE HAS REACHED A COMPRESSIVE STRENGTH OF 4000 PSI.

3. LATERAL TIES SHALL BE TENSIONED TO 30,000 POUNDS AND MAY BE EITHER 1-1/4" Ø PLAIN STRUCTURAL RODS OR 1/2" Ø PRESTRESSING STRANDS AS DESCRIBED IN NOTE 1. ALL LATERIAL TIE POCKET RECESSES SHALL BE FILLED WITH NON-SHRINK MORTAR AFTER LATERAL TIES ARE TENSIONED. LATERAL TIES SHALL BE TENSIONED PRIOR TO FILLING KEYS WITH NON-SHRINK MORTAR AND PLACING DOWELS.

4. CURBS SHALL BE CAST-IN-PLACE AFTER BEAMS ARE ERECTED.

5. THE VERTICAL 3/4" JOINT FILLER MATERIAL SHALL BE RIGIDLY ATTACHED TO THE SIDE OF ONE OF THE BOX BEAMS USING MECHANICAL FASTENERS OR NAILS.

6. SEE ABUTMENT SHEETS FOR SILICONE JOINT SEALANT DETAILS.



CROWN JOINT DETAIL
N. T. S.

AT THE CONTRACTOR'S OPTION, A BEVELED SECTION OF JOINT FILLER MAY BE USED IN PLACE OF NON-SHRINK MORTAR FOR THE AREA 6" BELOW THE TOP OF THE BEAM. NON-SHRINK MORTAR WILL BE REQUIRED FOR THE TOP 6".

COMMONWEALTH OF VIRGINIA
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DESIGNED BY: LLI	DRAWN BY: JCL	CHECKED BY: LLI	APPROVED BY: HFR
DATE: JULY 1983	SCALE: AS NOTED	CMP PROJ. NO. CMP 8305	

REVISIONS

NO.	DATE	REVISIONS	BY	APPR.
1	5/87	Corr. Keyway Dim.	REM	KHG

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

TAXIWAY BRIDGE OVER BROAD RUN
TYPICAL SECTION & DETAILS

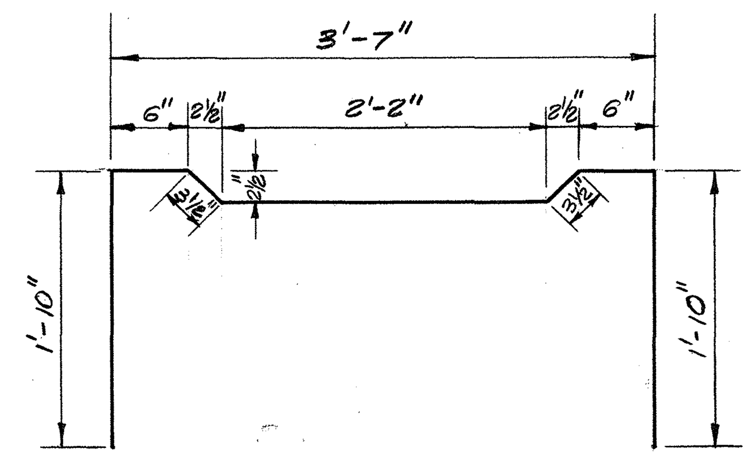
PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
32 OF 41

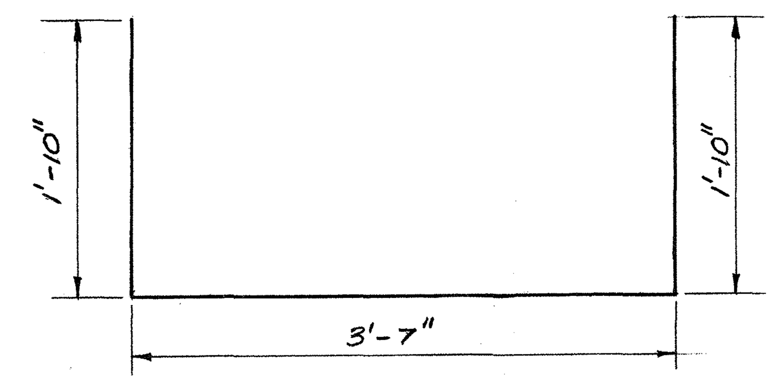
REINFORCING STEEL SCHEDULE

MARK	NO.	SIZE	LENGTH FT-IN	WEIGHT LBS.	TYPE STRAIGHT	TYPE BENT	PIN DIA. IN.	REMARKS
SUPERSTRUCTURE - TAXIWAY								
BC0401	88	4	2-4	137	X			CURB
BC0601	4	6	41-6	249	X			CURB
BL0401	138	4	41-8	3,841	X			
BS0601	920	6	7-0	9,673		X	3-3/4	
BS0602	920	6	6-11	9,558		X	4-1/2	
BS0603	920	6	5-11	8,176		X	4-1/2	
BS0604	920	6	3-7	4,952	X			
BS0605	184	6	8-6	2,349		X	4-1/2	END OF BEAMS
SUPERSTRUCTURE TOTAL				38,935				
SUBSTRUCTURE - ABUTMENTS 1 & 2 - TAXIWAY								
AF0401	78	4	35-6	1,850	X			
AF0402	184	4	5-6	676	X			
AF0601	192	6	5-6	1,586	X			
AS0401	184	4	4-8	574		X	2	BEARING SEAT
AS0601	8	6	48-5	582	X			BEARING SEAT
AW0401	58	4	4-3	165	X			WING
AW0402	16	4	4-11	53		X	3	WING
AW0404	42	4	4-7	129		X	3	WING
AW0405	10	4	7-11	53		X	3	WING
AW0406	10	4	6-5	43		X	3	WING
AW0407	10	4	7-1	47		X	3	WING
AW0408	10	4	6-7	44		X	3	WING
AW0601	2	6	7-10	24		X	4-1/2	WING
AW0602	2	6	6-4	19		X	4-1/2	WING
AW0603	2	6	4-6	14	X			WING
AW0604	2	6	2-6	8	X			WING
AW0605	2	6	7-0	21		X	4-1/2	WING
AS0606	2	6	6-7	20		X	4-1/2	WING
AW0607	2	6	3-6	11	X			WING
SUBSTRUCTURE TOTAL				5,919				

NOTES:
1. ALL BAR DIMENSIONS ARE OUT TO OUT.
2. LENGTH OF BARS INCLUDE REDUCTIONS FOR BENDS.



BSO601



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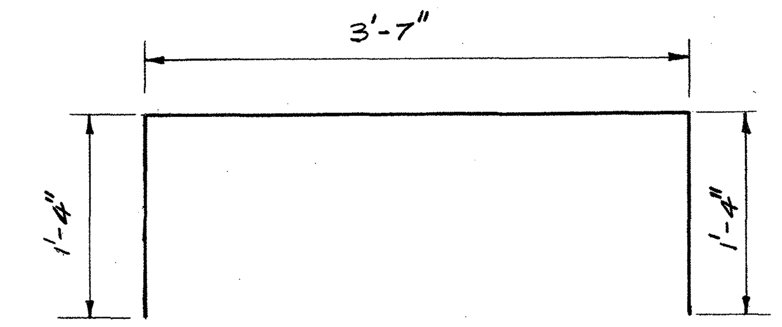

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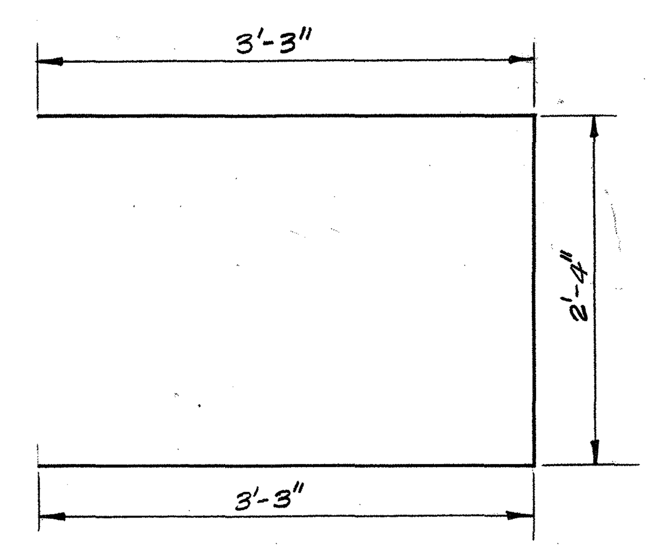

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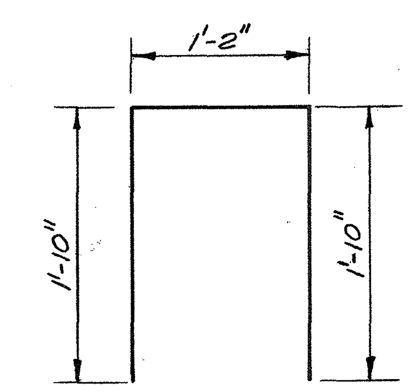
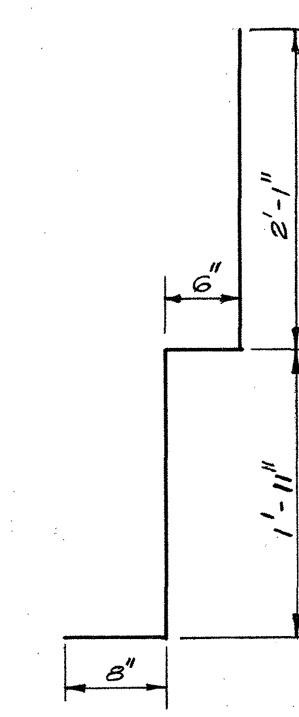
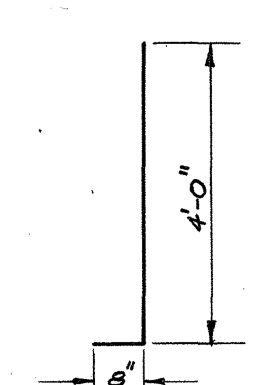


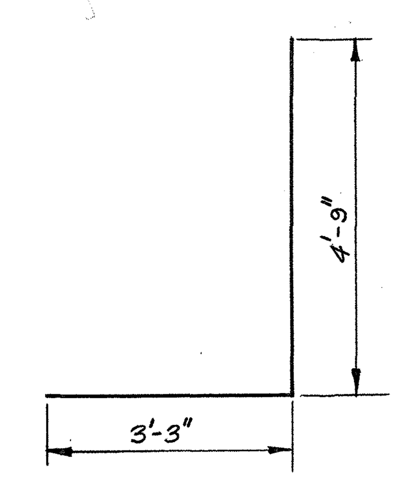
ASO401



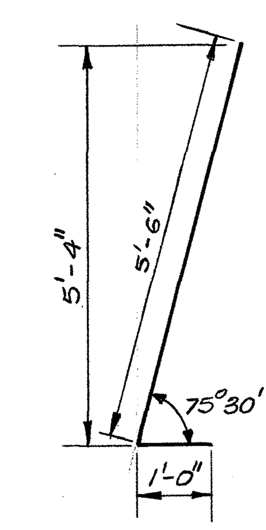
AWO402



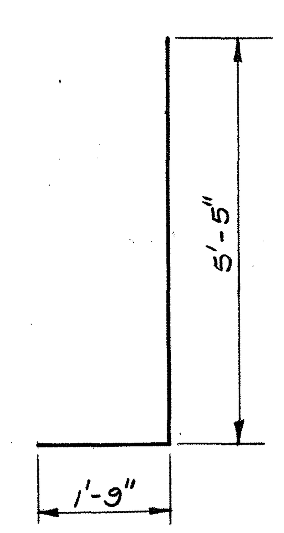
AWO404



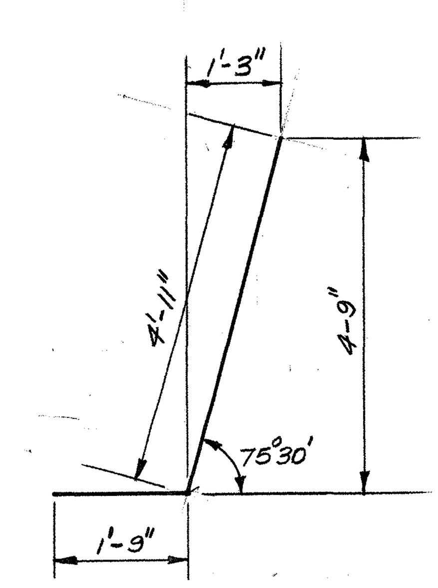
AWO405 & AWO601



AWO406 & AWO602



AWO407 & AWO605



AWO408 & AWO606

COMMONWEALTH OF VIRGINIA
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DESIGNED BY:	DRAWN BY:	CHECKED BY:	APPROVED BY:
LLI	UJS	LLI	HFR

DATE:	SCALE:	CMP PROJ. NO.
JULY 1983	NONE	CMP 8305

NO.	DATE	REVISIONS	BY	APPR.

MANASSAS MUNICIPAL AIRPORT
MANASSAS, VIRGINIA

TAXIWAY BRIDGE OVER BROAD RUN
REINFORCING SCHEDULE

PROJECT NO.
AIP 5-51-0030-01

SHEET NO.
33 OF 41

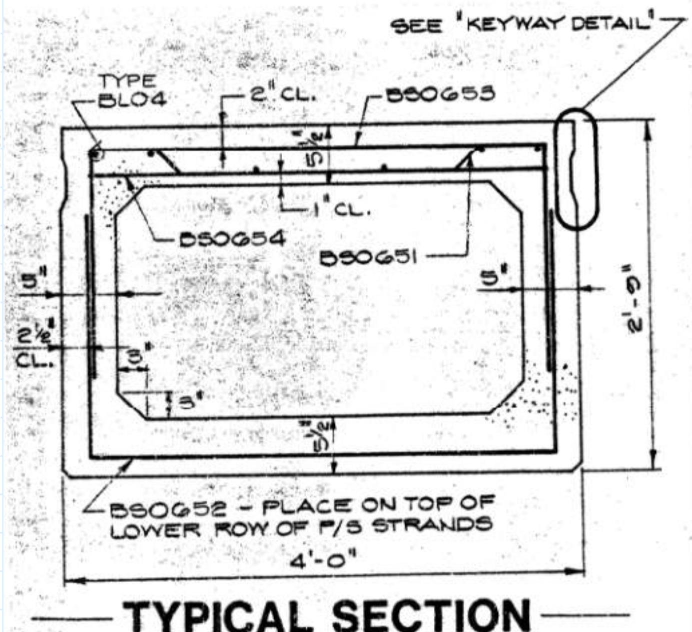
Abutment Check

This section will cover 3 di erent design checks:

Strip Footing Bearing Load Check
Dowel Connection Check
Bearing Pad Check

GENERAL NOTES

DESIGN SPECIFICATIONS:	AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1977, INCLUDING INTERIM SPECIFICATIONS, 1978, 1979, 1980, 1981 & 1982.
LOADING:	737-200 AIRCRAFT LOADING. TOTAL GROSS LOAD 108,000 POUNDS - DUAL GEAR, 48,500 POUNDS EACH.
CONCRETE:	CONCRETE IN PRESTRESSED MEMBERS - f'c = 5000 PSI. CONCRETE IN SUBSTRUCTURE - f'c = 3000 PSI.
REINFORCING STEEL:	REINFORCING STEEL SHALL CONFORM TO ASTM A-615, GRADE 60. ALL REINFORCING BAR DIMENSIONS ON THE DETAILED DRAWINGS ARE TO CENTERS OF BARS EXCEPT WHERE OTHERWISE NOTED.
FOUNDATIONS:	FOOTINGS FOR ABUTMENTS SHALL REST ON FIRM MATERIAL. BEARING CAPACITY OF FOUNDATION SHALL BE 3500 PSF.
CHAMFER:	ALL EXPOSED CORNERS OF CONCRETE SHALL BE CHAMFERED WITH 3/4" x 3/4" MILLED CHAMFER STRIPS.



TYPICAL SECTION
SCALE: 1"= 1'-0"
(INTERIOR BEAM SHOWN)

Site Geometry

Bridge Span (box beam total length) $L_{span} := 40\ ft + 8\ in$

Fill and Pavement Height $h_{pf} := 2.01\ ft$

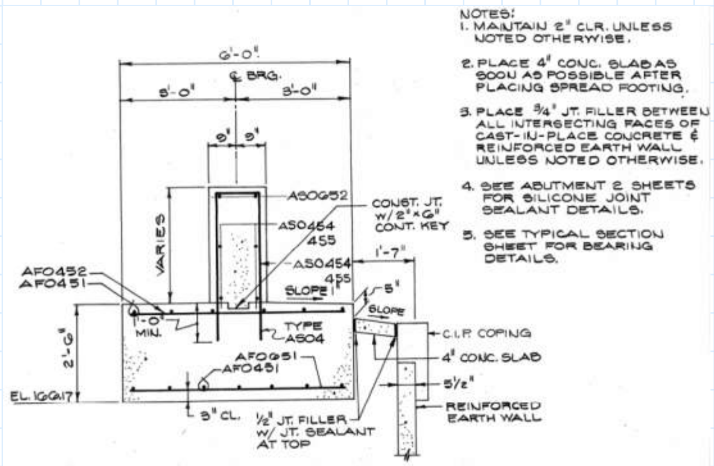
Box Beam Width $b_{box} := 4\ ft$

Box Beam Height $h_{box} := 2\ ft + 9\ in$

Box Beam Area (cross sectional area) $A_{box} := 766\ in^2$

Footing Thickness $h_{footing} := 2\ ft + 6\ in$

Footing Width $b_{footing} := 6\ ft$



TYPICAL SECTION
SCALE:1/2"=1'-0"

Max Stem Height $h_{stemmax} := 171.61\ ft - 168.17\ ft - h_{footing} = 11.28\ in$

Stem Height at Edge of Runway $h_{stemmin} := 170.95\ ft - 168.17\ ft - h_{footing} = 3.36\ in$

Average Stem Height $h_{stemavg} := \text{mean}\left(h_{stemmax}, h_{stemmin}\right) = 7.32\ in$

Stem Width $b_{stem} := 18\ in$

Clear Cover of Footing $cc_{bot} := 3\ in$ $d_{bot} := h_{footing} - \left(cc_{bot} + \frac{d_{b6}}{2}\right) = 26.625\ in$

$cc_{top} := 2\ in$ $d_{top} := h_{footing} - \left(cc_{top} + 1.5\ d_{b6}\right) = 26.875\ in$

Soil Height on Footing $h_{soil} := 176.41\ ft - 168.17\ ft - h_{footing} = 5.74\ ft$

Dead Load

Specific Weight of Concrete (and Fill) $\gamma_c := 150\ pcf$ Conservatively assumed heaviest material (i.e., concrete) for overburden dead weight

Live Load

Snippet below is from B737 aircraft specification sheets.

7.3 MAXIMUM PAVEMENT LOADS

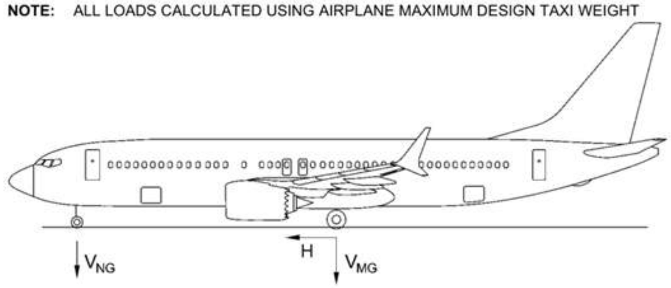
7.3.1 Maximum Pavement Loads: All Models

V_{NG} = MAXIMUM VERTICAL NOSE GEAR GROUND LOAD AT MOST FORWARD CENTER OF GRAVITY

V_{MG} = MAXIMUM VERTICAL MAIN GEAR GROUND LOAD AT MOST AFT CENTER OF GRAVITY

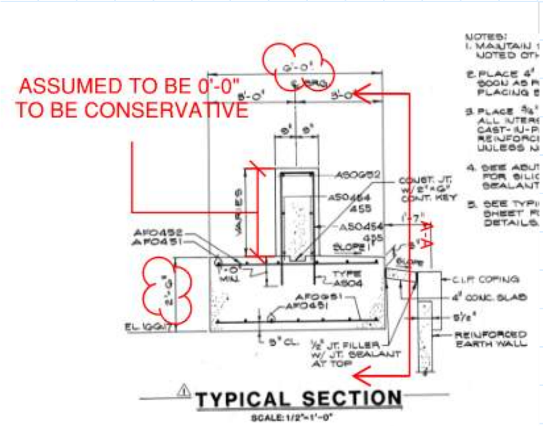
H = MAXIMUM HORIZONTAL GROUND LOAD FROM BRAKING

NOTE: ALL LOADS CALCULATED USING AIRPLANE MAXIMUM DESIGN TAXI WEIGHT



AIRPLANE MODEL	UNITS	MAX DESIGN TAXI WEIGHT	V_{NG} STATIC AT MOST FWD C.G.	V_{NG} STATIC + BRAKING 10 FT/SEC² DECEL	V_{MG} PER STRUT AT MAX LOAD AT STATIC AFT C.G.	H PER STRUT STEADY BRAKING 10 FT/SEC² DECEL	H PER STRUT AT INSTANTANEOUS BRAKING (μ = 0.8)
737-7	LB	177,500	18,918	30,637	82,866	27,566	66,293
	KG	80,512	8,581	13,897	37,587	12,504	30,070
737-8 / -8-200 / BBJ MAX 8	LB	182,700	15,894	26,282	85,258	28,373	68,206
	KG	82,871	7,209	11,921	38,672	12,870	30,938
737-9 / BBJ MAX 9	LB	195,200	15,514	25,639	91,868	30,315	73,494
	KG	88,541	7,037	11,630	41,671	13,751	33,336
737-10	LB	198,400	13,613	23,251	93,679	30,812	74,944
	KG	89,992	6,175	10,546	42,492	13,976	33,994

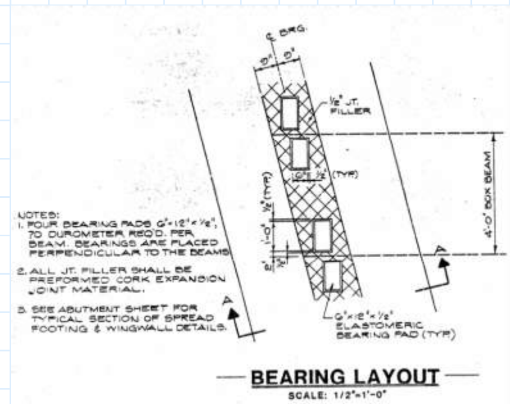
All the following calculations are relative to the sketch below for dimensions and distance



Plane Weight	$W_{max\_DF} := 564.29\ kip$
Distribution Factor	$DF := 0.35$
Gear Loading Per Strut	$P_{strut\_DF} := W_{max\_DF} \cdot 0.475 \cdot DF = 93.813\ kip$

Bearing Check

Bearing Capacity
Per General Notes $\quad q_{all} := 4000\ psf$



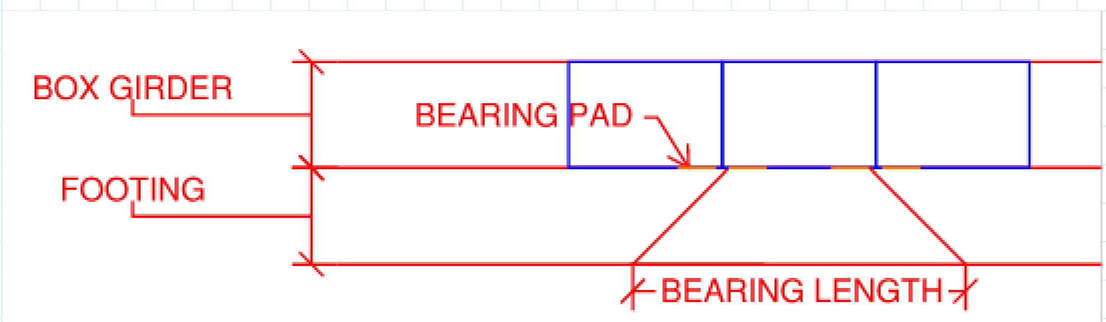
Bearing Pad Geometry

Bearing Pad Width $\quad b_{bp} := 6\ in$

Bearing Pad Length $\quad l_{bp} := 12\ in$

Bearing Area

Distribution Length - We've conservatively assumed 45 deg distribution within the concrete box beam down to footing elevation



$$l_{dw} := b_{box} - 2 \cdot 2\ in + 2\ \frac{h_{stemavg} + h_{footing}}{\tan(45\ °)} = 9.887\ ft$$

$$A_{bearing} := b_{footing} \cdot l_{dw} = 59.32\ ft^2$$

Length for Single Bearing Pad

$$l_{dwonepad} := 12\ in + 2\ \frac{h_{stemavg} + h_{footing}}{\tan(45\ °)} = 7.22\ ft$$

$$A_{bearingonepad} := b_{footing} \cdot l_{dwonepad} = 43.32\ ft^2$$

Loads below are divided by two to distribute total load between two abutments on either end

Weight of Pavement and Fill Above per foot $w_{pf} := \gamma_c \cdot h_{pf} \cdot \dfrac{L_{span}}{2} = 6.131 \ \boldsymbol{klf}$

Box Beam Weight per foot $w_{box} := \gamma_c \cdot A_{box} \cdot \dfrac{L_{span}}{2 \ b_{box}} = 4.056 \ \boldsymbol{klf}$

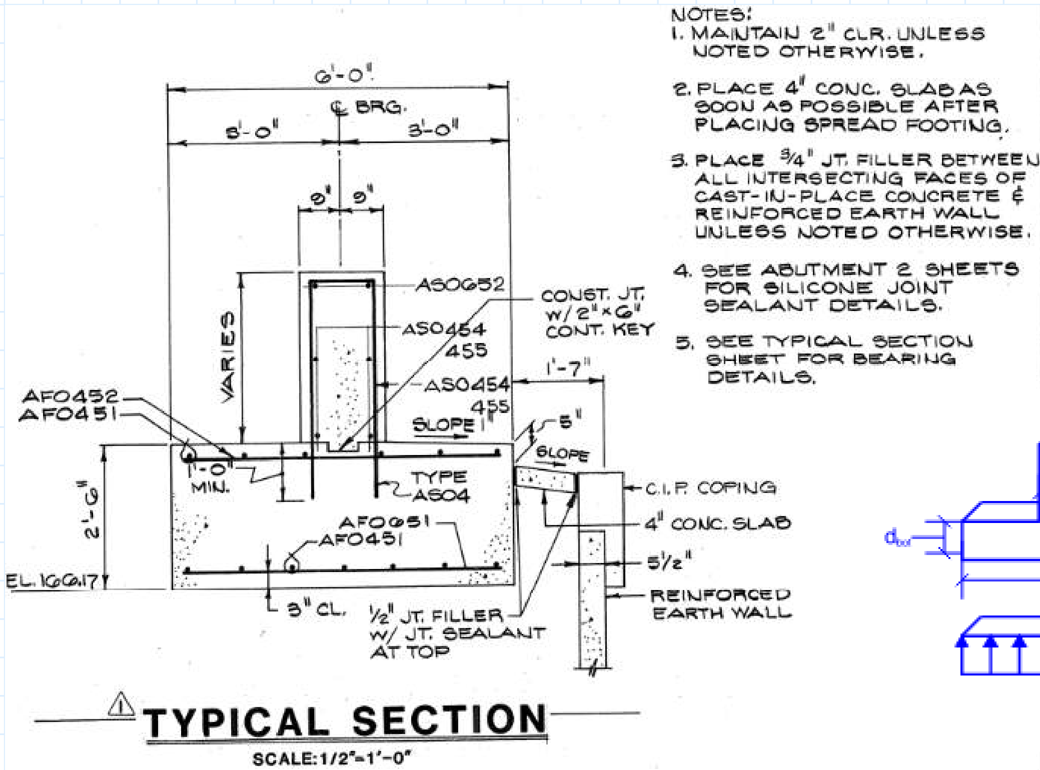
Footing Weight $w_{SW} := \gamma_c \cdot \left(b_{footing} \cdot h_{footing} + b_{stem} \cdot h_{stemavg}\right) = 2.387 \ \boldsymbol{klf}$

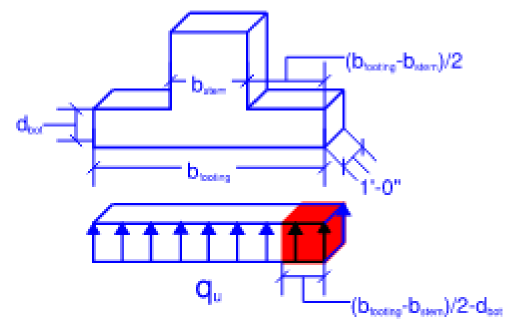
Soil on Footing Weight $w_{soil} := \gamma_c \cdot \dfrac{\left(b_{footing} - b_{stem}\right)}{2} \cdot h_{soil} = 1.937 \ \boldsymbol{klf}$

Bearing Pressure $q_{strut} := \dfrac{w_{pf} + w_{box} + w_{SW} + w_{soil}}{b_{footing}} + \dfrac{P_{strut\_DF}}{A_{bearing}} = 3999.99 \ \boldsymbol{psf}$

$\dfrac{q_{strut}}{q_{all}} = 1$ <1.0 **OK**

Strength Check — Strength of Foundation not Controlling





Distribution Factor

$DF = 0.35$

Max Strut Loading (based on moment and shear capacity)

$P_{strut\_DFM} := 5000\ kip \cdot 0.475 \cdot DF = 831.25\ kip$

$P_{strut\_DFV} := 5000\ kip \cdot 0.475 \cdot DF = 831.25\ kip$

Ultimate Soil Pressure Strength II

$$q_{uM} := 1.25\ \frac{w_{pf} + w_{box} + w_{SW} + w_{soil}}{b_{footing}} + 1.35\ \frac{P_{strut\_DFM}}{A_{bearing}} = 21.941\ ksf$$

$$q_{uV} := 1.25\ \frac{w_{pf} + w_{box} + w_{SW} + w_{soil}}{b_{footing}} + 1.35\ \frac{P_{strut\_DFV}}{A_{bearing}} = 21.941\ ksf$$

Moment Demand for 1 ft section

$$M_u := q_{uM} \cdot 1\ ft \cdot \frac{b_{footing} - 18\ in}{2} \cdot \frac{b_{footing} - 18\ in}{4} = 55.537\ kip \cdot ft$$

Shear Demand for 1 ft section

$$V_u := q_{uV} \cdot 1\ ft \cdot \left(\frac{b_{footing} - 18\ in}{2} - d_{bot}\right) = 0.686\ kip$$

$$M_{uV} := q_{uV} \cdot 1\ ft \cdot \frac{b_{footing} - 18\ in}{2} \cdot \frac{b_{footing} - 18\ in}{4} = 55.537\ kip \cdot ft$$

Concrete Properties

Initial Concrete Strength $f'_{ci} := 4000\ psi$

28-Day Concrete Strength $f'_c := 5000\ psi$

Concrete Specific Weight (Table 3.5.1-1) $w_c := 145\ pcf$

$w_{cDL} := 150\ pcf$

Correction Factor $K_1 := 1$

Modulus of Elasticity (Eq. 5.4.2.4-1)

$$E_{ci} := 120000\ K_1 \cdot \left(\frac{w_c}{1000\ pcf}\right)^2 \left(\frac{f'_{ci}}{ksi}\right)^{0.33} ksi = 3986.548\ ksi$$

$$E_c := 120000\ K_1 \cdot \left(\frac{w_c}{1000\ pcf}\right)^2 \left(\frac{f'_c}{ksi}\right)^{0.33} ksi = 4291.186\ ksi$$

Stress Block Factors (5.6.2.2)

$$\alpha_1(x) := \begin{array}{l} \text{if } x \le 10\ ksi \\ \quad 0.85 \\ \text{else} \\ \quad \max\left(0.85 - \frac{0.02\ (x - 10\ ksi)}{1\ ksi}, 0.75\right) \end{array}$$

$$\alpha_1 := \alpha_1(f'_c) = 0.85$$

$$\beta_1(x) := \begin{array}{l} \text{if } x \le 4\ ksi \\ \quad 0.85 \\ \text{else} \\ \quad \max\left(0.85 - \frac{0.05\ (x - 4\ ksi)}{1\ ksi}, 0.65\right) \end{array}$$

$$\beta_1 := \beta_1(f'_c) = 0.8$$

Ultimate Compressive Strain $\varepsilon_{cu} := 0.003$

Steel Reinforcement, Grade 60

Modulus of Elasticity $f_y := 60\ ksi$

Yield Strength $E_y := 29000\ ksi$

Yield Strain $\varepsilon_y := min\left(\frac{f_y}{E_y}, 0.002\right) = 0.002$

Reinforcement Detailing Check

AASHTO 12.14 is related to Precast Reinforced Concrete Three-Sided Structures. This section may not be the most applicable to the bridge we are analyzing, but let's use it as a baseline,

Justifications:
- Bridge is 3-sided
- precast box girders

Per 12.14.5.8:

Minimum Reinforcement Short Direction, per ft $A_{sminshort} := 0.002 \cdot 1\ \mathbf{ft} \cdot h_{footing} = 0.72\ \mathbf{in}^2$

Provided in Drawings $A_{sprovshort} := A_{b6} + A_{b4} = 0.64\ \mathbf{in}^2$ $A_{sminshort} \le A_{sprovshort} = 0$

Minimum Reinforcement Long Direction, per ft $A_{sminlong} := 0.002 \cdot b_{footing} \cdot h_{footing} = 4.32\ \mathbf{in}^2$

Provided in Drawings $A_{sprovlong} := 12\ A_{b4} = 2.4\ \mathbf{in}^2$ $A_{sminlong} \le A_{sprovlong} = 0$

Provided Reinforcement not meeting the minimum.

Moment Capacity

Reinforcement Geometry, Assuming spacing for top and bottom reinforcement is @1'-0" C.C.

Top Reinforcement $A_{top} := A_{b4}$

Bottom Reinforcement $A_{bot} := A_{b6}$

Assuming:

Bottom Reinforcement Yielding
Top Reinforcement Yielding, on Tension Side
No PT

$A_s \cdot f_s - A'_s \cdot f'_s = \alpha_1 \cdot f'_c \cdot \beta_1 \cdot b \cdot c$ Eqn. 5.6.3.1.1-4

$A_s := A_{bot}$ $A'_s := A_{top}$ $b := 1\ ft$

$f_s := f_y$ $f'_s := f_y$

$c := \dfrac{A_s \cdot f_s - A'_s \cdot f'_s}{\alpha_1 \cdot f'_c \cdot \beta_1 \cdot b} = 0.353\ in$ $\dfrac{c}{d_{bot}} \le \dfrac{0.003}{0.003 + \varepsilon_{cu}} = 1$ (5.6.2.1-1)

Checks Reinforcement Assumption $c > d_{bot} = 0$
- Correct Assumptions

$c > h_{footing} - d_{top} = 0$

Check steel yielding assumptions $\varepsilon_{sbot} := \dfrac{d_{bot} - c}{c} \cdot \varepsilon_{cu} = 0.223$ $\varepsilon'_{stop} := \dfrac{(h_{footing} - d_{top}) - c}{c} \cdot \varepsilon_{cu} = 0.024$
- Correct Assumptions

$\varepsilon_{sbot} > \varepsilon_y = 1$ $\varepsilon'_{stop} > \varepsilon_y = 1$

$$M_n := A_s \cdot f_s \cdot \left(d_{bot} - \frac{\beta_1 \cdot c}{2}\right) + A'_s \cdot f'_s \cdot \left((h_{footing} - d_{top}) - \frac{\beta_1 \cdot c}{2}\right) = 61.248\ kip \cdot ft$$

$$\phi(\varepsilon) := \begin{cases} \text{if } \varepsilon \le \varepsilon_y & 0.75 \\ \text{else if } \varepsilon < 0.005 & 0.75 + 0.25 \dfrac{(\varepsilon - \varepsilon_y)}{0.005 - \varepsilon_y} \\ \text{else} & 1 \end{cases}$$

$\phi := \phi(\varepsilon_{sbot}) = 1$

$\phi M_n := \phi \cdot M_n = 61.248\ kip \cdot ft$ $\dfrac{M_u}{\phi M_n} = 0.907$

Shear Capacity

$$d_v := \max\left(0.9 \cdot d_{bot}, 0.72\ h_{box}\right) = 23.963\ \boldsymbol{in}$$

$$b_v := 1\ \boldsymbol{ft}$$

Area of Concrete
on Flexural Tension Side

$$A_{ct} := \frac{A_{box}}{2}$$

Figure B5.2-1 shows half of cross-section to be considered as concrete on the flexural tension side

No Transverse Reinforcement,
none required per 5.7.2.3

$$\phi_v := 0.9$$

$$A_s := A_{sprovshort}$$

β Determination

$$M_u := \max\left(M_{uV}, V_u \cdot d_v\right) = 55.537\ \boldsymbol{kip \cdot ft}$$

$$V_u = 0.686\ \boldsymbol{kip}$$

$$v_u := \frac{V_u}{\phi_v \cdot b_v \cdot d_v} = 0.003\ \boldsymbol{ksi}$$

$$\frac{v_u}{f'_c} = 0.001$$

Iteration 1

$$\varepsilon_s := \frac{\left(\frac{M_u}{d_v} + 0.5\ V_u\right)}{2\ E_y \cdot A_s} = 0.001$$

$$\theta := \left(29 + 3500\ \varepsilon_s\right)\ ° = 31.655\ °$$

Eq. 5.7.3.4.2-3

$$\theta_{before} := \theta$$

Iteration 2

$$\varepsilon_s := \frac{\left(\frac{M_u}{d_v} + 0.5\ V_u \cdot \cot(\theta)\right)}{2\ E_y \cdot A_s} = 0.001$$

$$\theta := \left(29 + 3500\ \varepsilon_s\right)\ ° = 31.675\ °$$

$$\frac{\theta - \theta_{before}}{\theta} = 0.001$$

$$\theta_{before} := \theta$$

Iteration 3

$$\varepsilon_s := \frac{\left(\frac{M_u}{d_v} + 0.5\ V_u \cdot \cot(\theta)\right)}{2\ E_y \cdot A_s} = 0.001$$

$$\theta := (29 + 3500\ \varepsilon_s)\ ° = 31.675\ °$$

$$\frac{\theta - \theta_{before}}{\theta} = 0$$

End of Iterations

$$\beta := \frac{4.8}{1 + 750\ \varepsilon_s} = 3.051$$

Eq. 5.7.3.4.2-1,
for meeting minimum shear reinforcement

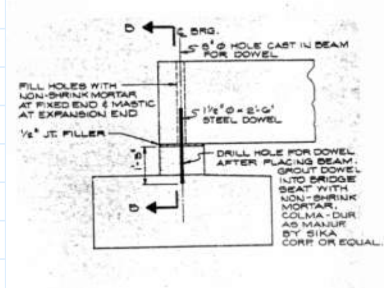
Concrete Shear Capacity $V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{f'_c}{ksi}}\ ksi \cdot b_v \cdot d_v = 61.994\ kip$

Total Capacity $\phi V_{nend} := \phi_v \cdot V_c = 55.795\ kip$

$$\frac{V_u}{\phi V_{nend}} = 0.012$$

Dowel Check

At the one end the box beam is free but on the other end it is fixed to the abutment with a 1-1/2" steel down shown in the sketch here. We need to check the capacity of this dowel to withstand braking forces from the aircraft



SECTION A-A
SCALE: 1/2"=1'-0"

Dowel Properties

Dowel Diameter	$d_{dowel} := 1.5\ in$
Yield Stress (Assumed)	$f_y := 60\ ksi$
Plastic Section	$Z := \dfrac{d_{dowel}^{\ 3}}{6} = 0.563\ in^3$
Elastic Section	$S := \dfrac{\pi \cdot d_{dowel}^{\ 3}}{32} = 0.331\ in^3$
	$\phi_v := 1 \qquad \phi_m := 1$
Shear Capacity per AISC	$\phi V_n := \phi_v \cdot 0.6\ f_y\ \dfrac{\pi}{4} \cdot d_{dowel}^{\ 2} = 63.617\ kip$
Moment Capacity per AISC	$\phi M_n := \phi_m \cdot min\left(f_y \cdot Z, 1.6\ f_y \cdot S\right) = 2.651\ kip \cdot ft$

Passive Pressure

Consideration of the passive pressure from soil will allow for increase in dowel capacity.

Passive Pressure From Geotechnical Report	$q_{pass} := 300\ \dfrac{psf}{ft}$
Passive Force	$V_{passive} := 0.5\ q_{pass} \cdot \left(h_{pf} + h_{box}\right)^2 \cdot b_{box} = 13.595\ kip$
Factored Passive Force	$'0.9V_{passive} := V_{passive} \cdot 0.9 = 12.235\ kip$

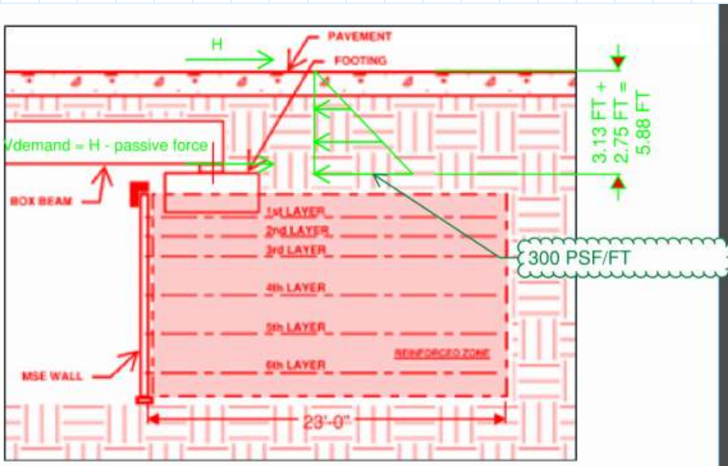


Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
○ Thermoplastic Culverts		1.30	0.90
○ All others		1.95	0.90
ES: Earth Surcharge		1.50	0.75

AASHTO LOAD FACTOR TABLE

Determining the shear demand acting on the existing dowels.

Coe icient of friction $\mu := 0.3$

Conservatively, using 16 inches as the pavement thickness $TW\&RW.H_{pavement} := 16\ in$

Pavement selftweight $PavementWeight := TW\&RW.H_{pavement} \cdot \gamma_c = 200\ psf$

Friction alculations between pavement weight and soil $'\mu := F \cdot N$ ——> $N := '\mu \cdot F$

Horizontal resisting force acting at bottom of pavement $N1 := \mu \cdot PavementWeight = 60\ psf$

Shear demand $V_{demand} := 60\ psf \cdot b_{box} \cdot L_{span} = 9.76\ kip$

Final Shear demand $V_U := \left(V_{demand} \cdot 1.75\right) - \left('0.9V_{passive}\right) = 4.845\ kip$

Based on Concrete Breakout Capacity

Breakout capacity was determined using Hilti Profis. See attached Hilti Profis calculation package.

Breakout Capacity $\phi V_{nbreakout} := 16839\ lbf$

$V_U = 4.845\ kip$ $<$ $\phi V_{nbreakout} = 16.839\ kip$

Bearing Pad Check

Demands

Values below divided by two because there are two bearing pads per box beam at each end

Airplane Load on Bearing Pad — To Be Determined Based on Compressive Service Stress Capacity

Dead Load on Bearing Pad $P_{DLbp} := \frac{(w_{pf} + w_{box}) \cdot b_{box}}{2} = 20.373\ kip$

Geometry

Width (perpendicular to tra ic direction) $pad_{width} := l_{bp} = 12\ in$

Length (parallel to tra ic direction) $pad_{length} := b_{bp} = 6\ in$

Number of Steel Layers $n_{steel\_layers} := 0$

Thickness of Exterior Elastomeric Layers $h_{elast\_ext} := 0.5 \cdot in$

Thickness of Interior Elastomeric Layers $h_{elast\_int} := 0.5 \cdot in$

Thickness of Steel Plates $h_s := 0 \cdot in$

Minimum Shear Modulus of Elastomerer
AASHTO Table 14.7.6.2.1 $G_{min} := 0.200 \cdot ksi$

Maximum Shear Modulus of Elastomerer
AASHTO Table 14.7.6.2.1 $G_{max} := 0.300 \cdot ksi$

Total Elastomeric Material Thickness $h_{rt} := 2 \cdot h_{elast\_ext} + (n_{steel\_layers} - 1) \cdot h_{elast\_int} = 0.5\ in$

Total Pad Thickness $h_{pad} := h_{rt} + n_{steel\_layers} \cdot h_s = 0.5\ in$

Bearing Area of Pad $A_{pad} := pad_{width} \cdot pad_{length} = 72\ in^2$

Compressive Stress Check

Shape factor of the ith elastomeric layer $S_{i\_ext} := \dfrac{pad_{width} \cdot pad_{length}}{2 \cdot h_{elast\_ext} \cdot \left(pad_{width} + pad_{length}\right)} = 4$

Shape factor of the ith elastomeric layer $S_{i\_int} := \dfrac{pad_{width} \cdot pad_{length}}{2 \cdot h_{elast\_int} \cdot \left(pad_{width} + pad_{length}\right)} = 4$

Max Compressive Stress Limit for PEP (Plain Elastomeric Pad) AASHTO 14.7.6.3.2 $\sigma_s := min\left(1.0\ G_{min} \cdot S_{i\_ext}, 800\ psi\right) = 800\ psi$

ACI 343R-95 10.5 Impact-Live Load $i_{highspeed} := 0.40$

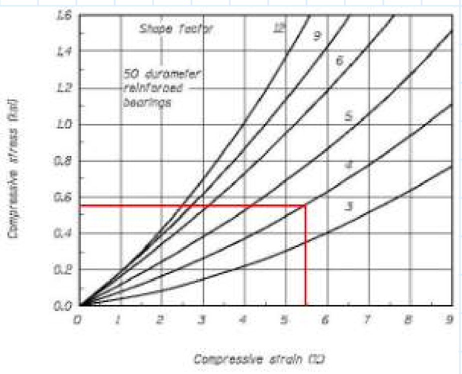
$i_{touchdown} := 1.00$ $i_{use} := i_{touchdown}$

Maximum Airplane Load on Bearing Pad Based on Max Compressive Stress Limit for PEP ACI 343R-95 10.5.2 $P_{planebp} := \dfrac{\sigma_s \cdot A_{pad} - P_{DLbp}}{\left(1 + i_{use} \cdot \left(1 - \dfrac{h_{pf}}{10\ ft}\right)\right) \cdot 0.475 \cdot 0.25} = 174.257\ kip$

Airplane Braking Force $V_{uInstantmax} := P_{planebp} \cdot 0.375$

Compressive Deflection Check

No graph for 70 Durometer, Use 60 Durometer graph to be conservative.



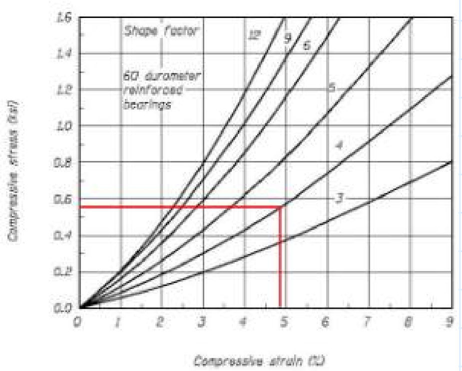


Figure C14.7.6.3.3-1—Stress-strain Curves

$S_{i\_ext} = 4$ $S_{i\_int} = 4$

$\sigma_L := \sigma_s = 0.8\ ksi$

At 60 Durometer, Approximately: $\varepsilon_{cPEP} := 7.1\%$

AASHTO 8th Edition 14.7.6.3.3 $\varepsilon_{cPEPLimit} := 0.09 = 0.09$

$\dfrac{\varepsilon_{cPEP}}{\varepsilon_{cPEPLimit}} = 0.79$ <1, OK

Estimate Bearing Pad Horizontal Deformations

Estimate the axial shortening of girders to check the bearing pads. Exclude elastic shortening from calculations because this occurred in the precast yard.

$$\Delta_s = \Delta_{CR} + \Delta_{SH} + \Delta_{TU} + \Delta_{BR}$$

$$\Delta_s = \left(\frac{P \cdot L}{A_g \cdot E_c} \cdot \psi_{cr} + \varepsilon_{sh} \cdot L_{span} + \varepsilon_{temp} \cdot L_{span} + \frac{F_{BR} \cdot h_{rt}}{G_{min} \cdot A_{pad}} \right)$$

Loads:
PS: Post tensioning
CR: Creep
SH: Shrinkage
TU: Uniform Temperature
BR: Braking forces

Uniform Temperature Deformation

Uniform Temperature Change
Table 3.12.3-1, Let's have T1-T3 — $\Delta_{temp} := (41 - 0)\ \Delta°F$

Temperature Strain — $\varepsilon_{temp} := \frac{6.0 \cdot 10^{-6}}{\Delta°F} \cdot \Delta_{temp} = 0$

Deformation — $\Delta_{TU} := \varepsilon_{temp} \cdot L_{span} = 0.12\ in$

Braking Forces Deformation

Longitudinal Braking Force — $F_{BR} := \frac{V_{uInstantmax}}{4} = 16.337\ kip$

Breaking Force Deformation — $\Delta_{BR} := \frac{F_{BR} \cdot h_{rt}}{G_{min} \cdot A_{pad}} = 0.567\ in$

Creep Deformation (AASHTO 8th Edition 5.4.2.3.2)

Initial Strength — $f'_{ci} := 4.0 \cdot ksi$

28-Day Strength — $f'_c := 5.0 \cdot ksi$

Correction Factor — $K_1 := 1$

Initial Modulus of Elasticity (AASHTO Eq. 5.4.2.4-1) $E_{ciAASHTO} := 120000\ K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^2 \left(\frac{f'_{ci}}{ksi}\right)^{0.33} ksi = 4266.223\ ksi$

Initial Modulus of Elasticity (ACI 318) $E_{ciACI} := 33000 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi = 3834.3\ ksi$

$E_{ciuse} := min\left(E_{ciAASHTO}, E_{ciACI}\right) = 3834.254\ ksi$

28-Day Modulus of Elasticity (AASHTO Eq. 5.4.2.4-1) $E_{cAASHTO} := 120000\ K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^2 \left(\frac{f'_c}{ksi}\right)^{0.33} ksi = 4592.232\ ksi$

28-Day Modulus of Elasticity (ACI 318) $E_{cACI} := 33000 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi = 4286.8\ ksi$

$E_{cuse} := min\left(E_{cAASHTO}, E_{cACI}\right) = 4286.826\ ksi$

Girder Area $A_{girder} := 766\ in^2$

Girder Width $b_{width} := 48\ in$

Volume to Surface Ratio $V\_S := \frac{A_{girder}}{b_{width}} = 15.958\ in$

Assume for Release Age $t_i := 1$ days Critical to Assume Earlier Age

Assume for Service $t_s := 41 \cdot 365$ days Current Age of Bridge ~ 41 years

$k_s := \max\left(1, 1.45 - 0.13 \cdot \left(\frac{V\_S}{in}\right)\right) = 1$

Relative humidity for Manassas, Figure 5.4.2.3.3-1 $H := 75\%$

$k_{hc} := 1.56 - 0.008 \cdot \dfrac{H}{1\%} = 0.96$

$k_f := \dfrac{5}{1 + \dfrac{f'_{ci}}{ksi}} = 1$

$k_{td}(t) := \dfrac{t}{61 - 4 \cdot \dfrac{f'_{ci}}{ksi} + t}$

$\psi(t, t_i) := 1.9 \cdot k_s \cdot k_{hc} \cdot k_f \cdot k_{td}(t) \cdot t_i^{-0.118}$

Creep Coe icient at Service $\psi_{cr} := \psi(t_s, t_i) = 1.819$

$A_{0.5pt} := 0.153 \cdot in^2$

$f_{pu} := 270\ ksi$

E ective Forces
Assume 25 ksi Loss per
PCI Handbook 7th edition 5.7.2 $F_e := 30 \cdot A_{0.5pt} \cdot (0.75 \cdot f_{pu} - 25 \cdot ksi) = 814.7\ kip$

Creep Deformation $\Delta_{CR} := \dfrac{F_e \cdot L_{span}}{A_{girder} \cdot E_{cuse}} \cdot \psi_{cr} = 0.22\ in$

Shrinkage Deformation (AASHTO 8th Edition 5.4.2.3.3)

$k_{hs} := 2 - 0.014 \cdot \dfrac{H}{1\%} = 0.95$ (Eq. 5.4.2.3.3-1)

Shrinkage Strain at Service $\varepsilon_{sh} := k_s \cdot k_{hs} \cdot k_f \cdot k_{td}(t_s) \cdot 0.48 \cdot 10^{-3} = 0$

Shrinkage deformation $\Delta_{SH} := \varepsilon_{sh} \cdot L_{span} = 0.222\ in$

Total Horizontal Deformation (per pad)

$\Delta_s := 1.0 \cdot \Delta_{CR} + 1.0 \cdot \Delta_{SH} + 1.0 \cdot \Delta_{TU} + 1.0 \cdot \Delta_{BR} = 1.129\ in$

Shear Deformation Check

For PEP (14.7.6.3.4-1)

$$\frac{\Delta_s}{0.5 \cdot h_{rt}} = 4.517$$

>1, Not OK

Without Braking Force (no plane load), the deformation check fails.

$$\frac{1.0 \cdot \Delta_{CR} + 1.0 \cdot \Delta_{SH} + 1.0 \cdot \Delta_{TU}}{0.5 \cdot h_{rt}} = 2.248$$

Rotation Check (14.7.6.3.5)

No Specific Limit for PEP

Stability Check (14.7.6.3.6)

$$\frac{h_{rt}}{\frac{1}{3} \, min\left(pad_{width}, pad_{length}\right)} = 0.25$$

<1, OK

Reinforcement Check (14.7.6.3.7)

Not Applicable to PEP

Bearing pad analysis summary: Max service load for compression determined. Per calculations, bearing pads fail in shear deformation. On site inspection, the bearing pads are ok. We will need to closely monitor performance of the bearing pads.

Spec Check Report

For Span 1 Girder A

May 22, 2025 11:30:37 am

PGSuper™ (x64)

Version 8.0.6 - Built on Jul 9 2024

Project Properties

Bridge Name	Manassas Airport Bridge
Bridge ID	
Company	WPM
Engineer	Jeffrey Liu
Job Number	D012400400
Comments	
File	C:\Users\JeffreyL\Walter P. Moore and Associates\D01-24004-00 HEF Manassas Bridge Assessments - Documents\General\Diagnostics\2-Design\Analysis\Superstructure\JL Working\DF = 0.35\PGSuper1 - Manassas Airport - Taxiway Bridge.pgs

Configuration

Configuration Server: Default libraries installed with PGSuper
Configuration Name: PGSuper.com Community
Configuration Source: C:\PROGRAM FILES\WSDOT\BRIDGELINK\Configurations\WSDOT.lbr
Configuration Date Stamp: March 19, 2024 3:42:28 pm

Library	Entry	Source
Girders	Manassas	Project Library
Traffic Barriers	none	Project Library
Project Criteria	TxDOT 2020 based on AASHTO LRFD Bridge Design Specifications, 9th Edition 2020	Project Library
Vehicular Live Load	Max Load - Design	Project Library
Load Rating Criteria	MBE 2020 based on The Manual for Bridge Evaluation, Third Edition 2018, with 2020 interim provisions	Project Library
Haul Trucks	Old Haul Truck -0	Project Library

Trans. Reinforcement data for Girder A does not match Girder Library entry Manassas
Long. Reinforcement data for Girder A does not match Girder Library entry Manassas

Analysis Controls

Structural Analysis Method: Simple Span
Section Properties: Gross
Losses: Refined estimate per TxDOT Research Report 0-6374-2

Notes

Symbol	Definition
L_r	Span Length of Girder at Release
L_l	Span Length of Girder during Lifting
L_{st}	Span Length of Girder during Storage
L_h	Span Length of Girder during Hauling
L_e	Span Length of Girder after Erection
L_s	Length of Span
Debond	Point where bond begins for a debonded strand
PSXFR	Point of prestress transfer
FoS	Face of Support in final bridge configuration
ST	Section Transitions
STLF	Section Transitions, Left Face
STRF	Section Transitions, Right Face
SDCR	Start of Deck Casting Region
EDCR	End of Deck Casting Region
Diaphragm	Location of a precast or cast in place diaphragm
Bar Cutoff	End of a reinforcing bar in the girder
Deck Bar Cutoff	End of a reinforcing bar in the deck
CS	Critical Section for Shear
SZB	Stirrup Zone Boundary
H	H from end of girder or face of support
1.5H	1.5H from end of girder or face of support
HP	Harp Point
Pick Point	Support point where girder is lifted from form
Bunk Point	Point where girder is supported during transportation

Status Items

Level	Description
Information	Live Load Distribution Factors were User-Input.

Level	Description
Warning	Span 1, Girder A: Either the Jacking stress is not equal to 0.75Fpu, or Debonded strands are present, or Temporary strands are present, or the girder is Not Prismatic. Therefore, for the calculation of elastic shortening; an iterative solution was used to find Fcgp after release rather than assuming 0.7*Fpu per the TxDOT design manual.

Specification Checks

Specification: TxDOT 2020

Stress Limitations on Prestressing Tendons [5.9.2.2]

Strand Stresses

Loss Stage	Stress Limit (KSI)	Straight		Adjustable Straight	
		Strand Stress (KSI)	Status (C/D)	Strand Stress (KSI)	Status (C/D)
At Jacking (f_{pj})	202.500	202.500	Pass (1.00)	0.000	Pass (∞)
After All Losses and Elastic Gains including Live Load (f_{pe})	194.400	173.230	Pass (1.12)	0.000	Pass (∞)

Required Minimum Concrete Strengths

Required f'_{ci} = 2.364 KSI ⇒ 2.400 KSI
Provided f'_{ci} = 4.000 KSI

Required f'_c = 2.988 KSI ⇒ 3.000 KSI
Provided f'_c = 5.000 KSI

Interval 2: Prestress Release : Service I Compression

For Temporary Stresses before Losses (LRFD 5.9.2.3.1)
Compression Stresses (LRFD 5.9.2.3.1a)

f'_{ci} = 4.000 KSI
Compression stress limit = $-0.65f'_{ci}$ = -2.600 KSI
Concrete strength required to satisfy this requirement = 2.364 KSI

Location from Left Support (ft)	Location from End of Girder (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
		f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(STRF) -0.750	(STRF, 0.0L_r) 0.000	0.000	0.000	0.000	0.000	0.000	0.000	No	Yes	Pass (∞)
(STLF) 0.750	(STLF) 1.500	0.166	-0.623	-0.034	0.034	0.133	-0.590	No	Yes	Pass (4.41)
(STRF) 0.750	(STRF) 1.500	0.034	-0.963	-0.043	0.043	-0.009	-0.920	No	Yes	Pass (2.83)
(PSXFR) 1.750	(PSXFR) 2.500	0.057	-1.606	-0.069	0.069	-0.012	-1.536	No	Yes	Pass (1.69)
3.317	(0.1L_r) 4.067	0.057	-1.607	-0.107	0.107	-0.050	-1.500	No	Yes	Pass (1.73)
7.383	(0.2L_r) 8.133	0.057	-1.609	-0.189	0.189	-0.131	-1.421	No	Yes	Pass (1.83)
11.450	(0.3L_r) 12.200	0.057	-1.611	-0.247	0.247	-0.190	-1.364	No	Yes	Pass (1.91)
15.517	(0.4L_r) 16.267	0.057	-1.613	-0.282	0.282	-0.225	-1.330	No	Yes	Pass (1.95)
(0.5L_s) 19.583	(0.5L_r) 20.333	0.057	-1.613	-0.294	0.294	-0.236	-1.319	No	Yes	Pass (1.97)
23.650	(0.6L_r) 24.400	0.057	-1.613	-0.282	0.282	-0.225	-1.330	No	Yes	Pass (1.95)
27.717	(0.7L_r) 28.467	0.057	-1.611	-0.247	0.247	-0.190	-1.364	No	Yes	Pass (1.91)
31.783	(0.8L_r) 32.533	0.057	-1.609	-0.189	0.189	-0.131	-1.421	No	Yes	Pass (1.83)
35.850	(0.9L_r) 36.600	0.057	-1.607	-0.107	0.107	-0.050	-1.500	No	Yes	Pass (1.73)
(PSXFR) 37.417	(PSXFR) 38.167	0.057	-1.606	-0.069	0.069	-0.012	-1.536	No	Yes	Pass (1.69)

Location from Left Support (ft)	Location from End of Girder (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
		f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(STLF) 38.417	(STLF) 39.167	0.034	-0.963	-0.043	0.043	-0.009	-0.920	No	Yes	Pass (2.83)
(STRF) 38.417	(STRF) 39.167	0.166	-0.623	-0.034	0.034	0.133	-0.590	No	Yes	Pass (4.41)
(STLF) 39.917	(STLF, 1.0L_r) 40.667	0.000	0.000	0.000	0.000	0.000	0.000	No	Yes	Pass (∞)

Interval 2: Prestress Release : Service I Tension

For Temporary Stresses before Losses (LRFD 5.9.2.3.1)
Tension Stresses (LRFD 5.9.2.3.1b)

f'_{ci} = 4.000 KSI
Tension stress limit in precompressed tensile zone without bounded reinforcement = N/A
Tension stress limit in areas other than the precompressed tensile zone and without bonded reinforcement = $0.2400\lambda\sqrt{f'_{ci}}$ = 0.480 KSI
Tension stress limit in areas with sufficient bonded reinforcement = $0.2400\lambda\sqrt{f'_{ci}}$ = 0.480 KSI
Concrete strength required to satisfy this requirement = 0.305 KSI

Location from Left Support (ft)	Location from End of Girder (ft)	Pre-tension		Service I		Demand		Tension Limit		Precompressed Tensile Zone		Status (C/D)
		f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top (KSI)	Bottom (KSI)	Top	Bottom	
(STRF) -0.750	(STRF, 0.0L_r) 0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.480	-	No	Yes	Pass (∞)
(STLF) 0.750	(STLF) 1.500	0.166	-0.623	-0.034	0.034	0.133	-0.590	0.480	-	No	Yes	Pass (3.62)
(STRF) 0.750	(STRF) 1.500	0.034	-0.963	-0.043	0.043	-0.009	-0.920	0.480	-	No	Yes	Pass (-)
(PSXFR) 1.750	(PSXFR) 2.500	0.057	-1.606	-0.069	0.069	-0.012	-1.536	0.480	-	No	Yes	Pass (-)
3.317	(0.1L_r) 4.067	0.057	-1.607	-0.107	0.107	-0.050	-1.500	0.480	-	No	Yes	Pass (-)
7.383	(0.2L_r) 8.133	0.057	-1.609	-0.189	0.189	-0.131	-1.421	0.480	-	No	Yes	Pass (-)
11.450	(0.3L_r) 12.200	0.057	-1.611	-0.247	0.247	-0.190	-1.364	0.480	-	No	Yes	Pass (-)
15.517	(0.4L_r) 16.267	0.057	-1.613	-0.282	0.282	-0.225	-1.330	0.480	-	No	Yes	Pass (-)
(0.5L_s) 19.583	(0.5L_r) 20.333	0.057	-1.613	-0.294	0.294	-0.236	-1.319	0.480	-	No	Yes	Pass (-)
23.650	(0.6L_r) 24.400	0.057	-1.613	-0.282	0.282	-0.225	-1.330	0.480	-	No	Yes	Pass (-)
27.717	(0.7L_r) 28.467	0.057	-1.611	-0.247	0.247	-0.190	-1.364	0.480	-	No	Yes	Pass (-)
31.783	(0.8L_r) 32.533	0.057	-1.609	-0.189	0.189	-0.131	-1.421	0.480	-	No	Yes	Pass (-)
35.850	(0.9L_r) 36.600	0.057	-1.607	-0.107	0.107	-0.050	-1.500	0.480	-	No	Yes	Pass (-)
(PSXFR) 37.417	(PSXFR) 38.167	0.057	-1.606	-0.069	0.069	-0.012	-1.536	0.480	-	No	Yes	Pass (-)
(STLF) 38.417	(STLF) 39.167	0.034	-0.963	-0.043	0.043	-0.009	-0.920	0.480	-	No	Yes	Pass (-)
(STRF) 38.417	(STRF) 39.167	0.166	-0.623	-0.034	0.034	0.133	-0.590	0.480	-	No	Yes	Pass (3.62)
(STLF) 39.917	(STLF, 1.0L_r) 40.667	0.000	0.000	0.000	0.000	0.000	0.000	0.480	-	No	Yes	Pass (∞)

Interval 10: Install composite overlay, Apply User Defined Loads : Service I Compression

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = $-0.6f'_c$ = -3.000 KSI

Concrete strength required to satisfy this requirement = 2.126 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (10+)
(STLF) 0.750	0.150	-0.561	-0.045	0.045	0.104	-0.516	No	Yes	Pass (5.82)
(STRF) 0.750	0.030	-0.842	-0.058	0.058	-0.028	-0.784	No	Yes	Pass (3.83)
(PSXFR) 1.750	0.050	-1.407	-0.131	0.131	-0.081	-1.276	No	Yes	Pass (2.35)
($0.1L_s$) 3.917	0.050	-1.413	-0.276	0.276	-0.226	-1.137	No	Yes	Pass (2.64)
($0.2L_s$) 7.833	0.051	-1.423	-0.490	0.490	-0.440	-0.933	No	Yes	Pass (3.22)
($0.3L_s$) 11.750	0.051	-1.430	-0.643	0.643	-0.592	-0.786	No	Yes	Pass (3.82)
($0.4L_s$) 15.667	0.051	-1.434	-0.735	0.735	-0.684	-0.699	No	Yes	Pass (4.29)
($0.5L_s$) 19.583	0.051	-1.435	-0.766	0.766	-0.715	-0.669	No	Yes	Pass (4.20)
($0.6L_s$) 23.500	0.051	-1.434	-0.735	0.735	-0.684	-0.699	No	Yes	Pass (4.29)
($0.7L_s$) 27.417	0.051	-1.430	-0.643	0.643	-0.592	-0.786	No	Yes	Pass (3.82)
($0.8L_s$) 31.333	0.051	-1.423	-0.490	0.490	-0.440	-0.933	No	Yes	Pass (3.22)
($0.9L_s$) 35.250	0.050	-1.413	-0.276	0.276	-0.226	-1.137	No	Yes	Pass (2.64)
(PSXFR) 37.417	0.050	-1.407	-0.131	0.131	-0.081	-1.276	No	Yes	Pass (2.35)
(STLF) 38.417	0.030	-0.842	-0.058	0.058	-0.028	-0.784	No	Yes	Pass (3.83)
(STRF) 38.417	0.150	-0.561	-0.045	0.045	0.104	-0.516	No	Yes	Pass (5.82)
($1.0L_s$) 39.167	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (10+)

Interval 10: Install composite overlay, Apply User Defined Loads : Service I Tension

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Tension Stresses (LRFD 5.9.2.3.2b)

f'_c = 5.000 KSI

Tension stress limit for components with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions = $0.2400\lambda\sqrt{f'_c}$ = 0.537 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Tension Limit		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top (KSI)	Bottom (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.075	-0.280	0.000	0.000	0.075	-0.280	-	0.537	No	Yes	Pass (-)

Location from Left Support (ft)	Pre-tension		Service I		Demand		Tension Limit		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top (KSI)	Bottom (KSI)	Top	Bottom	
(STLF) 0.750	0.150	-0.561	-0.045	0.045	0.104	-0.516	-	0.537	No	Yes	Pass (-)
(STRF) 0.750	0.030	-0.842	-0.058	0.058	-0.028	-0.784	-	0.537	No	Yes	Pass (-)
(PSXFR) 1.750	0.050	-1.407	-0.131	0.131	-0.081	-1.276	-	0.537	No	Yes	Pass (-)
($0.1L_s$) 3.917	0.050	-1.413	-0.276	0.276	-0.226	-1.137	-	0.537	No	Yes	Pass (-)
($0.2L_s$) 7.833	0.051	-1.423	-0.490	0.490	-0.440	-0.933	-	0.537	No	Yes	Pass (-)
($0.3L_s$) 11.750	0.051	-1.430	-0.643	0.643	-0.592	-0.786	-	0.537	No	Yes	Pass (-)
($0.4L_s$) 15.667	0.051	-1.434	-0.735	0.735	-0.684	-0.699	-	0.537	No	Yes	Pass (-)
($0.5L_s$) 19.583	0.051	-1.435	-0.766	0.766	-0.715	-0.669	-	0.537	No	Yes	Pass (-)
($0.6L_s$) 23.500	0.051	-1.434	-0.735	0.735	-0.684	-0.699	-	0.537	No	Yes	Pass (-)
($0.7L_s$) 27.417	0.051	-1.430	-0.643	0.643	-0.592	-0.786	-	0.537	No	Yes	Pass (-)
($0.8L_s$) 31.333	0.051	-1.423	-0.490	0.490	-0.440	-0.933	-	0.537	No	Yes	Pass (-)
($0.9L_s$) 35.250	0.050	-1.413	-0.276	0.276	-0.226	-1.137	-	0.537	No	Yes	Pass (-)
(PSXFR) 37.417	0.050	-1.407	-0.131	0.131	-0.081	-1.276	-	0.537	No	Yes	Pass (-)
(STLF) 38.417	0.030	-0.842	-0.058	0.058	-0.028	-0.784	-	0.537	No	Yes	Pass (-)
(STRF) 38.417	0.150	-0.561	-0.045	0.045	0.104	-0.516	-	0.537	No	Yes	Pass (-)
($1.0L_s$) 39.167	0.075	-0.280	0.000	0.000	0.075	-0.280	-	0.537	No	Yes	Pass (-)

Interval 15: Open to Traffic, Roadway Geometry Control : Service I Compression without live load

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = $-0.45f'_c$ = -2.250 KSI

Concrete strength required to satisfy this requirement = 2.835 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (8.04)
(STLF) 0.750	0.150	-0.561	-0.045	0.045	0.104	-0.516	No	Yes	Pass (4.36)
(STRF) 0.750	0.030	-0.842	-0.058	0.058	-0.028	-0.784	No	Yes	Pass (2.87)
(PSXFR) 1.750	0.050	-1.407	-0.131	0.131	-0.081	-1.276	No	Yes	Pass (1.76)
($0.1L_s$) 3.917	0.050	-1.413	-0.276	0.276	-0.226	-1.137	No	Yes	Pass (1.98)
($0.2L_s$) 7.833	0.051	-1.423	-0.490	0.490	-0.440	-0.933	No	Yes	Pass

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
									(2.41)
($0.3L_s$) 11.750	0.051	-1.430	-0.643	0.643	-0.592	-0.786	No	Yes	Pass (2.86)
($0.4L_s$) 15.667	0.051	-1.434	-0.735	0.735	-0.684	-0.699	No	Yes	Pass (3.22)
($0.5L_s$) 19.583	0.051	-1.435	-0.766	0.766	-0.715	-0.669	No	Yes	Pass (3.15)
($0.6L_s$) 23.500	0.051	-1.434	-0.735	0.735	-0.684	-0.699	No	Yes	Pass (3.22)
($0.7L_s$) 27.417	0.051	-1.430	-0.643	0.643	-0.592	-0.786	No	Yes	Pass (2.86)
($0.8L_s$) 31.333	0.051	-1.423	-0.490	0.490	-0.440	-0.933	No	Yes	Pass (2.41)
($0.9L_s$) 35.250	0.050	-1.413	-0.276	0.276	-0.226	-1.137	No	Yes	Pass (1.98)
(PSXFR) 37.417	0.050	-1.407	-0.131	0.131	-0.081	-1.276	No	Yes	Pass (1.76)
(STLF) 38.417	0.030	-0.842	-0.058	0.058	-0.028	-0.784	No	Yes	Pass (2.87)
(STRF) 38.417	0.150	-0.561	-0.045	0.045	0.104	-0.516	No	Yes	Pass (4.36)
($1.0L_s$) 39.167	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (8.04)

Interval 15: Open to Traffic, Roadway Geometry Control : Service I Compression

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = $-0.6f'_c$ = -3.000 KSI

Concrete strength required to satisfy this requirement = 2.988 KSI

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (10+)
(STLF) 0.750	0.150	-0.561	-0.108	0.045	0.041	-0.516	No	Yes	Pass (5.82)
(STRF) 0.750	0.030	-0.842	-0.139	0.058	-0.109	-0.784	No	Yes	Pass (3.83)
(PSXFR) 1.750	0.050	-1.407	-0.315	0.131	-0.265	-1.276	No	Yes	Pass (2.35)
($0.1L_s$) 3.917	0.050	-1.413	-0.664	0.276	-0.614	-1.137	No	Yes	Pass (2.64)
($0.2L_s$) 7.833	0.051	-1.423	-1.180	0.490	-1.130	-0.933	No	Yes	Pass (2.66)
($0.3L_s$) 11.750	0.051	-1.430	-1.549	0.643	-1.498	-0.786	No	Yes	Pass (2.00)
($0.4L_s$) 15.667	0.051	-1.434	-1.770	0.735	-1.719	-0.699	No	Yes	Pass (1.75)
($0.5L_s$) 19.583	0.051	-1.435	-1.844	0.766	-1.793	-0.669	No	Yes	Pass (1.67)
($0.6L_s$) 23.500	0.051	-1.434	-1.770	0.735	-1.719	-0.699	No	Yes	Pass (1.75)

Location from Left Support (ft)	Pre-tension		Service I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(0.7L_s) 27.417	0.051	-1.430	-1.549	0.643	-1.498	-0.786	No	Yes	Pass (2.00)
(0.8L_s) 31.333	0.051	-1.423	-1.180	0.490	-1.130	-0.933	No	Yes	Pass (2.66)
(0.9L_s) 35.250	0.050	-1.413	-0.664	0.276	-0.614	-1.137	No	Yes	Pass (2.64)
(PSXFR) 37.417	0.050	-1.407	-0.315	0.131	-0.265	-1.276	No	Yes	Pass (2.35)
(STLF) 38.417	0.030	-0.842	-0.139	0.058	-0.109	-0.784	No	Yes	Pass (3.83)
(STRF) 38.417	0.150	-0.561	-0.108	0.045	0.041	-0.516	No	Yes	Pass (5.82)
(1.0L_s) 39.167	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (10+)

Interval 15: Open to Traffic, Roadway Geometry Control : Service III Tension

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)

Tension Stresses (LRFD 5.9.2.3.2b)

f'_c = 5.000 KSI

Tension stress limit for components with bonded prestressing tendons that are subjected to not worse than moderate corrosion conditions = $0.1900\lambda\sqrt{f'_c} \leq 0.600$ KSI = 0.425 KSI

Concrete strength required to satisfy this requirement = 1.173 KSI

Location from Left Support (ft)	Pre-tension		Service III		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(0.0L_s) 0.000	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (-)
(STLF) 0.750	0.150	-0.561	-0.045	0.096	0.104	-0.465	No	Yes	Pass (-)
(STRF) 0.750	0.030	-0.842	-0.058	0.124	-0.028	-0.718	No	Yes	Pass (-)
(PSXFR) 1.750	0.050	-1.407	-0.131	0.280	-0.081	-1.126	No	Yes	Pass (-)
(0.1L_s) 3.917	0.050	-1.413	-0.276	0.591	-0.226	-0.822	No	Yes	Pass (-)
(0.2L_s) 7.833	0.051	-1.423	-0.490	1.050	-0.440	-0.372	No	Yes	Pass (-)
(0.3L_s) 11.750	0.051	-1.430	-0.643	1.378	-0.592	-0.051	No	Yes	Pass (-)
(0.4L_s) 15.667	0.051	-1.434	-0.735	1.575	-0.684	0.142	No	Yes	Pass (3.00)
(0.5L_s) 19.583	0.051	-1.435	-0.766	1.641	-0.715	0.206	No	Yes	Pass (2.06)
(0.6L_s) 23.500	0.051	-1.434	-0.735	1.575	-0.684	0.142	No	Yes	Pass (3.00)
(0.7L_s) 27.417	0.051	-1.430	-0.643	1.378	-0.592	-0.051	No	Yes	Pass (-)
(0.8L_s) 31.333	0.051	-1.423	-0.490	1.050	-0.440	-0.372	No	Yes	Pass (-)
(0.9L_s) 35.250	0.050	-1.413	-0.276	0.591	-0.226	-0.822	No	Yes	Pass (-)
(PSXFR) 37.417	0.050	-1.407	-0.131	0.280	-0.081	-1.126	No	Yes	Pass (-)

Location from Left Support (ft)	Pre-tension		Service III		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
(STLF) 38.417	0.030	-0.842	-0.058	0.124	-0.028	-0.718	No	Yes	Pass (-)
(STRF) 38.417	0.150	-0.561	-0.045	0.096	0.104	-0.465	No	Yes	Pass (-)
($1.0L_s$) 39.167	0.075	-0.280	0.000	0.000	0.075	-0.280	No	Yes	Pass (-)

Interval 15: Open to Traffic, Roadway Geometry Control : Fatigue I Compression

Stresses at Service Limit State after Losses (LRFD 5.9.2.3.2)
Compression Stresses (LRFD 5.9.2.3.2a)

f'_c = 5.000 KSI

Compression stress limit = $-0.4f'_c$ = -2.000 KSI

Concrete strength required to satisfy this requirement = 1.595 KSI

Location from Left Support (ft)	Pre-tension		Fatigue I		Demand		Precompressed Tensile Zone		Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	Top	Bottom	
($0.0L_s$) 0.000	0.075	-0.280	0.000	0.000	0.037	-0.140	No	Yes	Pass (10+)
(STLF) 0.750	0.150	-0.561	-0.023	0.023	0.052	-0.258	No	Yes	Pass (7.76)
(STRF) 0.750	0.030	-0.842	-0.029	0.029	-0.014	-0.392	No	Yes	Pass (5.10)
(PSXFR) 1.750	0.050	-1.407	-0.066	0.066	-0.041	-0.638	No	Yes	Pass (3.14)
($0.1L_s$) 3.917	0.050	-1.413	-0.138	0.138	-0.113	-0.569	No	Yes	Pass (3.52)
($0.2L_s$) 7.833	0.051	-1.423	-0.245	0.245	-0.220	-0.466	No	Yes	Pass (4.29)
($0.3L_s$) 11.750	0.051	-1.430	-0.322	0.322	-0.296	-0.393	No	Yes	Pass (5.09)
($0.4L_s$) 15.667	0.051	-1.434	-0.367	0.367	-0.342	-0.349	No	Yes	Pass (5.73)
($0.5L_s$) 19.583	0.051	-1.435	-0.383	0.383	-0.357	-0.335	No	Yes	Pass (5.60)
($0.6L_s$) 23.500	0.051	-1.434	-0.367	0.367	-0.342	-0.349	No	Yes	Pass (5.73)
($0.7L_s$) 27.417	0.051	-1.430	-0.322	0.322	-0.296	-0.393	No	Yes	Pass (5.09)
($0.8L_s$) 31.333	0.051	-1.423	-0.245	0.245	-0.220	-0.466	No	Yes	Pass (4.29)
($0.9L_s$) 35.250	0.050	-1.413	-0.138	0.138	-0.113	-0.569	No	Yes	Pass (3.52)
(PSXFR) 37.417	0.050	-1.407	-0.066	0.066	-0.041	-0.638	No	Yes	Pass (3.14)
(STLF) 38.417	0.030	-0.842	-0.029	0.029	-0.014	-0.392	No	Yes	Pass (5.10)
(STRF) 38.417	0.150	-0.561	-0.023	0.023	0.052	-0.258	No	Yes	Pass (7.76)
($1.0L_s$) 39.167	0.075	-0.280	0.000	0.000	0.037	-0.140	No	Yes	Pass (10+)

Ultimate Moment Capacity

Positive Moment Capacity for Strength I Limit State [5.6]

Location from Left Support (ft)	M_u (kip-ft)	ϕM_n (kip-ft)	ϕM_n Min (kip-ft)	Status	
				ϕM_n Min ≤ ϕM_n ($\phi M_n/\phi M_n$ Min)	$M_u \le \phi M_n$ ($\phi M_n/M_u$)
($0.0L_s$) 0.000	0.00	348.77	0.00	Pass (∞)	Pass (∞)
($0.1L_s$) 3.917	586.51	1273.91	780.06	Pass (1.63)	Pass (2.17)
($0.2L_s$) 7.833	1042.46	1543.72	1375.83	Pass (1.12)	Pass (1.48)
($0.3L_s$) 11.750	1368.13	1628.68	1379.54	Pass (1.18)	Pass (1.19)
($0.4L_s$) 15.667	1563.53	1628.76	1381.77	Pass (1.18)	Pass (1.04)
($0.5L_s$) 19.583	1628.67	1628.79	1382.51	Pass (1.18)	Pass (1.00)
($0.6L_s$) 23.500	1563.53	1628.76	1381.77	Pass (1.18)	Pass (1.04)
($0.7L_s$) 27.417	1368.13	1628.68	1379.54	Pass (1.18)	Pass (1.19)
($0.8L_s$) 31.333	1042.46	1543.72	1375.83	Pass (1.12)	Pass (1.48)
($0.9L_s$) 35.250	586.51	1273.91	780.06	Pass (1.63)	Pass (2.17)
($1.0L_s$) 39.167	0.00	348.77	0.00	Pass (∞)	Pass (∞)

Ultimate Shear Capacity

Ultimate Shears for Strength I Limit State [5.8]

Location from Left Support (ft)	Stirrups Required	Stirrups Provided	$\|V_u\|$ (kip)	ϕV_n (kip)	Status ($\phi V_n/V_u$)
(CS) 2.318	Yes	Yes	153.19	264.25	Pass (1.72)
(H) 3.000	Yes	Yes	149.33	260.93	Pass (1.75)
($0.1L_s$) 3.917	Yes	Yes	144.15	257.03	Pass (1.78)
(1.5H) 4.375	Yes	Yes	141.56	255.24	Pass (1.80)
($0.2L_s$) 7.833	Yes	Yes	122.00	202.30	Pass (1.66)
($0.3L_s$) 11.750	Yes	Yes	99.85	154.77	Pass (1.55)
($0.4L_s$) 15.667	Yes	Yes	77.70	136.57	Pass (1.76)
($0.5L_s$) 19.583	Yes	Yes	55.56	134.38	Pass (2.42)
($0.6L_s$) 23.500	Yes	Yes	77.70	136.57	Pass (1.76)
($0.7L_s$) 27.417	Yes	Yes	99.85	154.77	Pass (1.55)
($0.8L_s$) 31.333	Yes	Yes	122.00	202.30	Pass (1.66)
(1.5H) 34.792	Yes	Yes	141.56	255.24	Pass (1.80)
($0.9L_s$) 35.250	Yes	Yes	144.15	257.03	Pass (1.78)

Location from Left Support (ft)	Stirrups Required	Stirrups Provided	$\lvert V_u \rvert$ (kip)	ϕV_n (kip)	Status ($\phi V_n/V_u$)
(H) 36.167	Yes	Yes	149.33	260.93	Pass (1.75)
(CS) 36.849	Yes	Yes	153.19	264.25	Pass (1.72)

[LRFD 5.7.3.2] The reaction introduces compression into the end of the girder. Load between the CSS and the support is transferred directly to the support by compressive arching action without causing additional stresses in the stirrups. Hence, A_V/S in this region must be equal or greater than A_V/S at the critical section.

Longitudinal Reinforcement for Shear Check - Strength I [5.7.3.5]

$$A_s f_y + A_{ps} f_{ps} \geq \left[\frac{M_u}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_a} + \left(\left| \frac{V_u}{\varphi_v} - V_p \right| - 0.5V_s \right) \cot\theta \right] \quad 5.7.3.5\text{-}1$$

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\varphi_v} - V_p - 0.5V_s \right) \cot\theta \quad 5.7.3.5\text{-}2$$

$$A_{ps} f_{ps} > A_s f_y$$

Location from Left Support (ft)	Capacity (kip)	Demand (kip)	Equation	Status (C/D)	$A_{ps}f_{ps}$ (kip)	$A_s f_y$ (kip)	Status
(FoS) 0.250	217.24	156.37	5.7.3.5-2	Pass (1.39)	217.24	0.00	Pass
(Bar Develop.) 1.933	546.94	156.37	5.7.3.5-2	Pass (3.50)	546.94	0.00	Pass
(CS) 2.318	561.20	331.89	5.7.3.5-1	Pass (1.69)	561.20	0.00	Pass
(H) 3.000	586.31	375.37	5.7.3.5-1	Pass (1.56)	586.31	0.00	Pass
($0.1L_s$) 3.917	620.10	431.37	5.7.3.5-1	Pass (1.44)	620.10	0.00	Pass
(1.5H) 4.375	637.00	458.29	5.7.3.5-1	Pass (1.39)	637.00	0.00	Pass
($0.2L_s$) 7.833	761.08	623.41	5.7.3.5-1	Pass (1.22)	761.08	0.00	Pass
($0.3L_s$) 11.750	806.32	751.00	5.7.3.5-1	Pass (1.07)	806.32	0.00	Pass
($0.4L_s$) 15.667	806.37	825.57	5.7.3.5-1	Pass* (0.98)	806.37	0.00	Pass
($0.5L_s$) 19.583	806.39	842.62	5.7.3.5-1	Pass* (0.96)	806.39	0.00	Pass
($0.6L_s$) 23.500	806.37	825.57	5.7.3.5-1	Pass* (0.98)	806.37	0.00	Pass
($0.7L_s$) 27.417	806.32	751.00	5.7.3.5-1	Pass (1.07)	806.32	0.00	Pass
($0.8L_s$) 31.333	761.08	623.41	5.7.3.5-1	Pass (1.22)	761.08	0.00	Pass
(1.5H) 34.792	637.00	458.29	5.7.3.5-1	Pass (1.39)	637.00	0.00	Pass
($0.9L_s$) 35.250	620.10	431.37	5.7.3.5-1	Pass (1.44)	620.10	0.00	Pass
(H) 36.167	586.31	375.37	5.7.3.5-1	Pass (1.56)	586.31	0.00	Pass
(CS) 36.849	561.20	331.89	5.7.3.5-1	Pass (1.69)	561.20	0.00	Pass
(Bar Develop.) 37.233	546.94	156.37	5.7.3.5-2	Pass (3.50)	546.94	0.00	Pass

Location from Left Support (ft)	Capacity (kip)	Demand (kip)	Equation	Status (C/D)	$A_{ps}f_{ps}$ (kip)	A_sf_y (kip)	Status
(FoS) 38.917	217.24	156.37	5.7.3.5-2	Pass (1.39)	217.24	0.00	Pass

* The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone

Horizontal Interface Shear for Strength I Limit State

AASHTO LRFD BDS 5.7.4

Location from Left Support (ft)	5.7.4.5			5.7.4.2			5.7.4.1		
	s (in)	s_{max} (in)	Status	a_{vf} (in^2/ft)	$a_{vf\ min}$ (in^2/ft)	Status	$\|v_{ui}\|$ (kip/ft)	ϕv_{ni} (kip/ft)	Status ($\phi v_{ni}/\|v_{ui}\|$)
($0.0L_s$) 0.000	12.000	33.100	Pass	0.800	0.000	Pass	1.167	64.801	Pass (10+)
(FoS) 0.250	12.000	33.100	Pass	0.800	0.000	Pass	1.167	64.801	Pass (10+)
(Bar Develop.) 1.933	12.000	33.100	Pass	0.800	0.000	Pass	1.490	64.801	Pass (10+)
(CS) 2.318	12.000	33.100	Pass	0.800	0.000	Pass	1.490	64.801	Pass (10+)
(H) 3.000	12.000	33.100	Pass	0.800	0.000	Pass	1.453	64.801	Pass (10+)
($0.1L_s$) 3.917	12.000	33.100	Pass	0.800	0.000	Pass	1.402	64.801	Pass (10+)
(1.5H) 4.375	12.000	33.100	Pass	0.800	0.000	Pass	1.377	64.801	Pass (10+)
($0.2L_s$) 7.833	12.000	33.100	Pass	0.800	0.000	Pass	1.187	64.801	Pass (10+)
($0.3L_s$) 11.750	12.000	33.100	Pass	0.800	0.000	Pass	0.971	64.801	Pass (10+)
($0.4L_s$) 15.667	12.000	33.100	Pass	0.800	0.000	Pass	0.756	64.801	Pass (10+)
($0.5L_s$) 19.583	12.000	33.100	Pass	0.800	0.000	Pass	0.540	64.801	Pass (10+)
($0.6L_s$) 23.500	12.000	33.100	Pass	0.800	0.000	Pass	0.756	64.801	Pass (10+)
($0.7L_s$) 27.417	12.000	33.100	Pass	0.800	0.000	Pass	0.971	64.801	Pass (10+)
($0.8L_s$) 31.333	12.000	33.100	Pass	0.800	0.000	Pass	1.187	64.801	Pass (10+)
(1.5H) 34.792	12.000	33.100	Pass	0.800	0.000	Pass	1.377	64.801	Pass (10+)
($0.9L_s$) 35.250	12.000	33.100	Pass	0.800	0.000	Pass	1.402	64.801	Pass (10+)
(H) 36.167	12.000	33.100	Pass	0.800	0.000	Pass	1.453	64.801	Pass (10+)
(CS) 36.849	12.000	33.100	Pass	0.800	0.000	Pass	1.490	64.801	Pass (10+)
(Bar Develop.) 37.233	12.000	33.100	Pass	0.800	0.000	Pass	1.490	64.801	Pass (10+)
(FoS) 38.917	12.000	33.100	Pass	0.800	0.000	Pass	1.167	64.801	Pass (10+)
($1.0L_s$) 39.167	12.000	33.100	Pass	0.800	0.000	Pass	1.167	64.801	Pass (10+)

Principal Tensile Stresses in Webs

Principal Tensile Stresses in Webs limitations are not applicable.
Concrete strength does not exceed the 10.000 KSI threshold

Live Load Deflection Check [2.5.2.6.2]

Allowable deflection span ratio = L/800

Allowable maximum deflection = ± 0.587 in
Minimum live load deflection along girder = -0.476 in
Maximum live load deflection along girder = 0.000 in
Status = Pass

Check for Lifting in Casting Yard

Lifting analysis disabled in Project Criteria. No analysis performed.
Per LRFD 5.5.4.3, "Buckling and stability of precast members during handling, transportation, and erection shall be investigated." Also see C5.5.4.3 and C5.12.3.2.1.

Check for Hauling to Bridge Site

Hauling analysis disabled in Project Criteria. No analysis performed.
Per LRFD 5.5.4.3, "Buckling and stability of precast members during handling, transportation, and erection shall be investigated." Also see C5.5.4.3 and C5.12.3.2.1.

Constructability Checks

Girder Dimensions Detailing Check [5.12.3.2.2]

Dimension	Minimum (in)	Actual (in)	Status
Top Flange Thickness	2.000	5.500	Pass
Web Thickness	5.000	5.000	Pass
Bottom Flange Thickness	5.000	5.500	Pass

Stirrup Detailing Check: Strength I [5.7.2.5, 5.7.2.6, 5.10.3.1.2]

Location from Left Support (ft)	Bar Size	S (in)	S_{max} (in)	S_{min} (in)	A_v/S (in^2/ft)	A_v/S_{min} (in^2/ft)	Status
($0.0L_s$) 0.000	#6	12.000	10.275	2.745	0.880	0.141	Fail
(FoS) 0.250	#6	12.000	10.225	2.745	0.880	0.141	Fail
(Bar Develop.) 1.933	#6	12.000	9.942	2.745	0.880	0.141	Fail
(CS) 2.318	#6	12.000	9.927	2.745	0.880	0.141	Fail
(H) 3.000	#6	12.000	9.899	2.745	0.880	0.141	Fail
($0.1L_s$) 3.917	#6	12.000	9.861	2.745	0.880	0.141	Fail
(1.5H) 4.375	#6	12.000	9.842	2.745	0.880	0.141	Fail
($0.2L_s$) 7.833	#6	12.000	19.472	2.745	0.880	0.141	Pass
($0.3L_s$) 11.750	#6	12.000	19.391	2.745	0.880	0.141	Pass
($0.4L_s$) 15.667	#6	12.000	19.391	2.745	0.880	0.141	Pass
($0.5L_s$) 19.583	#6	12.000	19.391	2.745	0.880	0.141	Pass
($0.6L_s$) 23.500	#6	12.000	19.391	2.745	0.880	0.141	Pass
($0.7L_s$) 27.417	#6	12.000	19.391	2.745	0.880	0.141	Pass
($0.8L_s$) 31.333	#6	12.000	19.472	2.745	0.880	0.141	Pass
(1.5H) 34.792	#6	12.000	9.842	2.745	0.880	0.141	Fail
($0.9L_s$) 35.250	#6	12.000	9.861	2.745	0.880	0.141	Fail
(H) 36.167	#6	12.000	9.899	2.745	0.880	0.141	Fail
(CS) 36.849	#6	12.000	9.927	2.745	0.880	0.141	Fail
(Bar Develop.) 37.233	#6	12.000	9.942	2.745	0.880	0.141	Fail
(FoS) 38.917	#6	12.000	10.225	2.745	0.880	0.141	Fail
($1.0L_s$) 39.167	#6	12.000	10.275	2.745	0.880	0.141	Fail

Haunch Geometry Checks

Slab Offset ("A" Dimension)

This table compares the input slab offset to the rounded computed slab offset required to have the least haunch depth be equal to the Fillet dimension. A failed status indicates that the top of the girder will encroach into the deck slab and the Slab Offset dimension should be refined.

Span	Girder	Provided (in)	Required (in)	Status	Notes
1	A	5.000	0.500	Excessive	The difference between the minimum and maximum CL haunch depths along the girder is 0.299 in. This exceeds one half of the slab depth. Check stirrup lengths to ensure they engage the deck in all locations. Refer to the Haunch Details chapter in the Details report for more information. Provided Slab Offset exceeded Required by allowable tolerance of 0.500 in

Excess Camber Check

Haunch dead load is affected by variable haunch depth along the girder. Haunch depth along a girder is defined by the roadway geometry, slab offset ("A"), and the parabolic girder camber defined by the user input Assumed Excess Camber at mid-span. The table below compares the Assumed Excess Camber with the Computed Excess Camber. A failed status indicates the assumed value is not within tolerance of the computed value - meaning that results dependent on haunch dead load may be inaccurate. See the Haunch Details and Loading Details chapters in Details Report for more information.

Span	Girder	Computed Excess Camber (in)	Assumed Excess Camber (in)	Difference (in)	Allowable Difference (in)	Status	Notes
1	A	0.299	0.000	0.299	± 0.500	Pass	Assumed Excess Camber is within tolerance

Camber

	Camber (in)
Screed Camber, C at mid-span	0.107
Camber at 21 days, D_{21}	0.406

Appendix D – Bridge Substructure Structural Analysis Calculations by Walter P Moore

01. DESCRIPTION

1.1 Objective.

Conduct an analysis of the existing MSE retaining walls at the HEF Manassas Regional Airport, located at the taxiway and runway bridges, to evaluate their structural utilization under anticipaed increased live loads from aircraft. The assessment will consider the projected performance and capacity of the MSE walls for both 50-year and 100-year design lifespans. The bridge structures are approximately 43 years old, and the design lifespans of 50 years and 100 years are measured from the original time of construction.

1.2 Code References

AASHTO LRFD Bridge Design Specifications 9th Edition 2020
AASHTO The Manual for Bridge Evaluation 3rd Edition 2018
ACI 343R-95 Analysis and Design of Reinforced Concrete Bridge Structures Section 5.10 - Airport runway bridge loads
AISC 360 Steel Construction Manual 15th Edition
ACI 318-14 Building Code Requirements for Structural Concrete

1.3 Additional References

Manassas Municipal Airport original drawings by Campbell, McQueen, & Paris, Engineers dated July 1983.
Manassas Regional Airport Evaluation Report by Athavale, Lystad & Associates dated November 2001.
Manassas Regional Airport - Runway 16L34R & Taxiway"B" Rehabilitation and Miscellaneous Improvements Drawings (partial drawing set) by Campbell & Paris Engineers dated 07/09/2002.
Manassas Regional Airport - Runway 34R Extension and Related work Package 2 Bridge Structure Extensions (partial drawing set) by RS & H dated 03/18/2013.
A321 Aircraft Characteristics - Airport and Maintenance Planning dated 06/01/2024.
WPM Draft HEF Manassas Airport Bridge Assessments Preliminary Structural Analysis dated 01/24/2025.
Original reinforcing strip report of chemical and physical properties by Reservco, Inc. dated 10/25/1983.
Record of Soil Exploration by Entech Engineers dated 03/26/2025.

1.4 Assumptions

Non aggresive soils for the reduction of reinforcing strips assumed per AASHTO section 11.10.6.4.2a)
Assuming 50 years of bridge lifespan
Assuming 100 years of bridge lifespan
AASHTO - Inextensible broken slip surface
Coefficient of friction = 0.3 (between pavement and soil)

1.5 Software Utilized

MSE Wall Geo5 Software - This specialized module from the GEO5 suite enables detailed analysis and verification of mechanically stabilized earth (MSE) and segmental retaining walls reinforced with geogrids. With a robust set of features—including built-in material and geogrid libraries, support for multiple design codes (such as EN 1997-1, LRFD, AASHTO, FHWA), sophisticated stability checks (slip, overturning, global, internal), bearing-capacity assessments, construction stage modeling, and seismic loading analysis—the software delivers a comprehensive and efficient design workflow

1.6 Summary Results

MSE Wall Analysis – Runway & Taxiway (165-kip Aircraft Load)

Runway – 50-Year Lifespan

All verification checks satisfactory.
Controlling factor: reinforcing strip-to-panel connection (≈99.6% utilization, governed by concrete breakout).
Overall capacity acceptable for 50-year design life.

Runway – 100-Year Lifespan

Reinforcement strip strength exceeds capacity (≈110%) → NOT OK.
100-year design life not feasible without strengthening/retrofit.

Taxiway – 50-Year Lifespan

All verification checks satisfactory.
Controlling factor: reinforcing strip-to-panel connection (≈87.4% utilization, governed by concrete breakout).
Acceptable for 50-year design life.

Taxiway – 100-Year Lifespan

All verification checks satisfactory.
Reinforcement strength increases to ≈95.5% utilization, still within capacity.
100-year design life is feasible.

Executive Summary:
The MSE wall evaluation under a 165-kip aircraft load indicates that both the Runway and Taxiway walls are capable of meeting a 50-year design life, with performance governed by the reinforcing strip-to-panel connection at the facing panels. At a 100-year design life, the Taxiway walls remain adequate with reinforcement strength utilization approaching but not exceeding capacity. However, the Runway walls experience reinforcement overstress at this horizon, making the 100-year lifespan not feasible without strengthening or retrofit measures.

2.0 MATERIALS, GEOMETRY, AND LOADS

2.1 Material Densities

Per existing drawings "Manassas Municipal Airport original drawings by Campbell, McQueen, &paris, Engineers dated July 1983.

$Fy_{Reinf.Strip} := 36000\ psi$ $Fu_{Reinf.Strip} := 58000\ psi$ $Fy_{Rebar} := 60000\ psi$ $\rho_{conc\&fill} := 150\ pcf = 150.000\ pcf$

$Existing.f'c := 4000\ psi$ MSE Wall panel per original drawings

2.2 Geometry

Per existing drawings "Manassas Municipal Airport original drawings by Campbell, McQueen, & Paris, Engineers dated July 1983.

RunWay Geometry: see below and previous calculation package

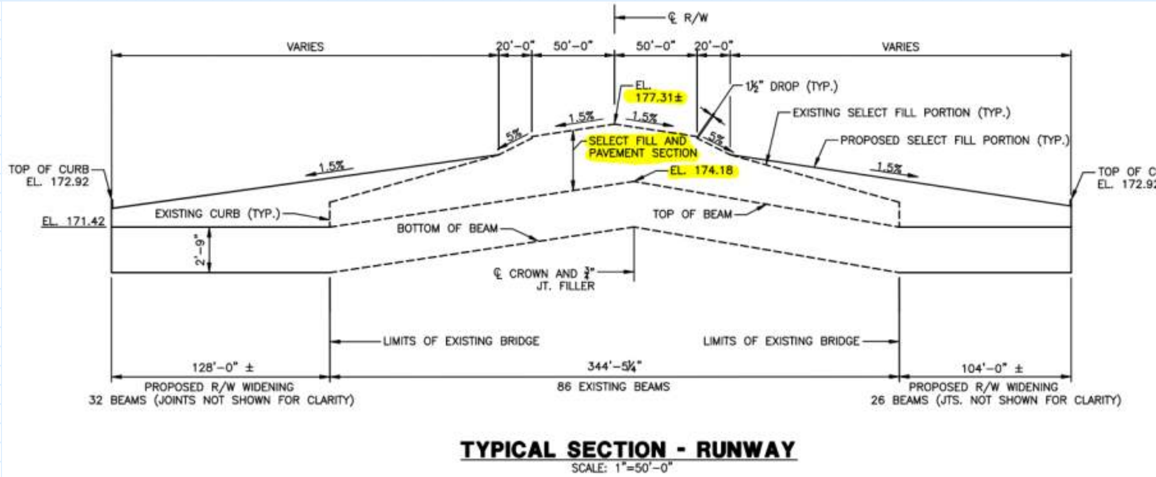
$Wall.Height_{RunWay} := 168.08\ ft - 155\ ft = 13.080\ ft$ See below the highlighted elevations

$Fil\&Pavement_{RunWay} := 177.31\ ft - 174.18\ ft = 3.130\ ft$ See below the highlighted elevations

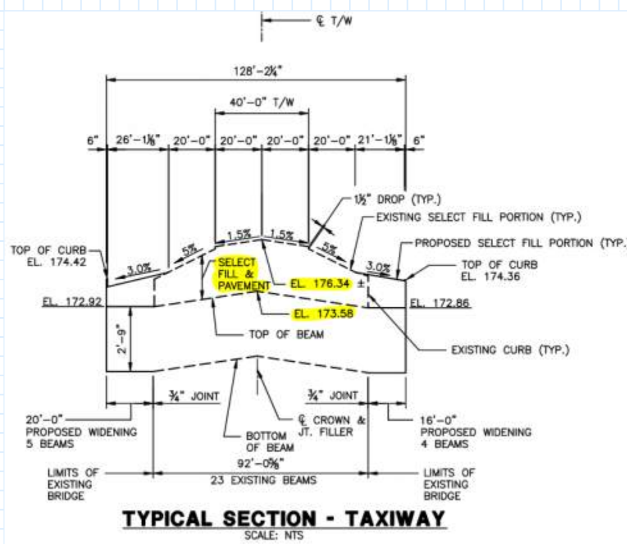
TaxiWay Geometry: similar to RunWay with less overburden height therefore we are using RunWay overburden height

$Wall.Height_{TaxiWay} := 170.08\ ft - 155\ ft = 15.080\ ft$ See below the highlighted elevations

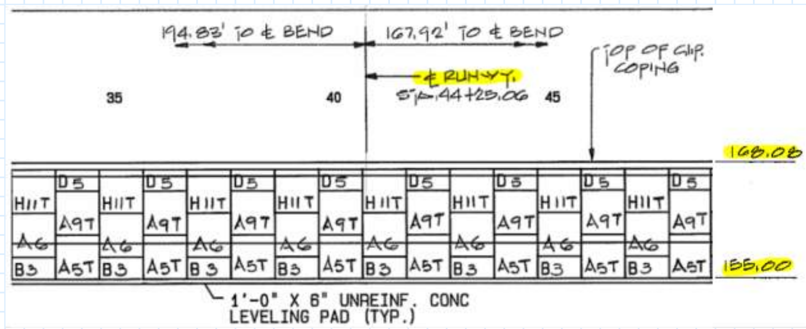
$Fil\&Pavement_{TaxiWay} := 176.34\ ft - 173.58\ ft = 2.760\ ft$ See below the highlighted elevations



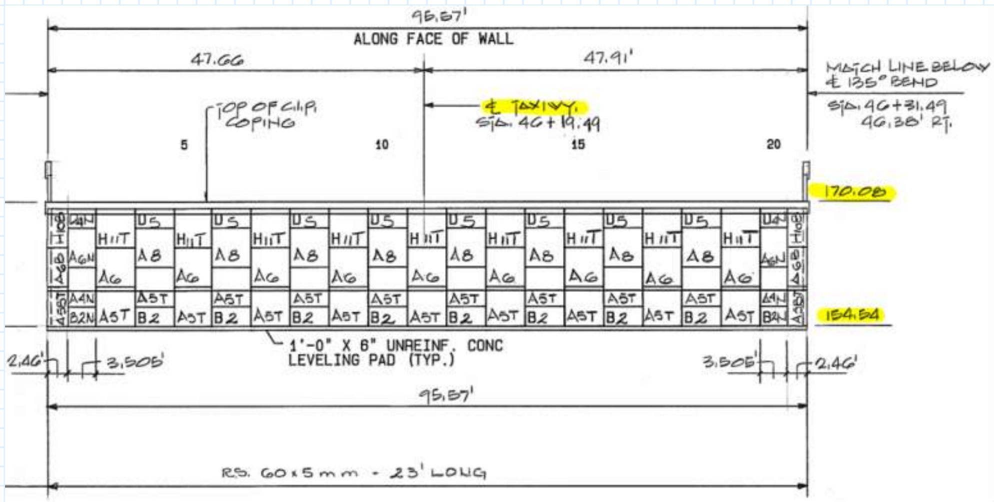
Runway 34R Extension and Related Work Package 2
Bridge Structure Extensions dwgs - Sheet S112



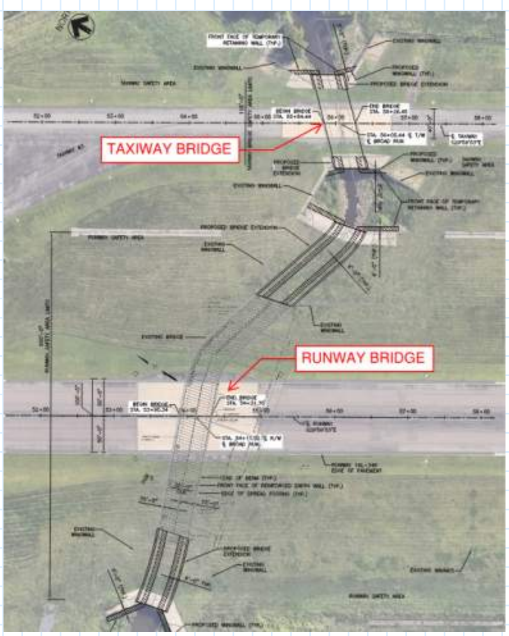
Runway 34R Extension and Related Work Package 2
Bridge Structure Extensions dwgs - Sheet S112



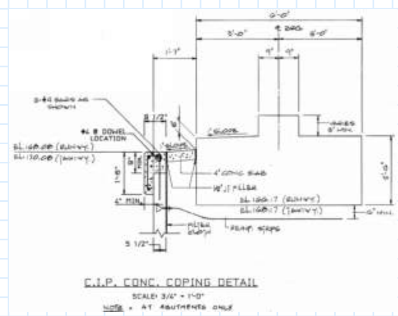
RUNWAY WALL ELEVATION
(per original dwgs - sheet 36)



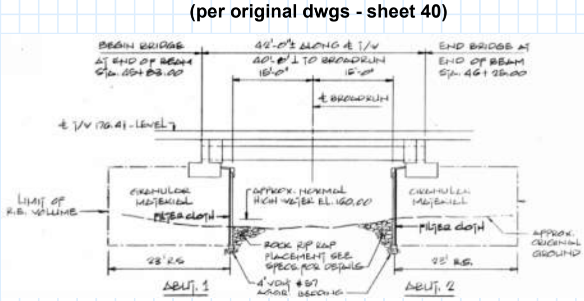
TAXIWAY WALL ELEVATION
(per original dwgs - sheet 39)



OVERALL TAXIWAY & RUNWAY BRIDGE PLAN
(satellite & Runway 34R Extension dwgs S128 overlay)



TOP OF MSE WALL - TYPICAL DETAIL
(per original dwgs - sheet 40)



TYPICAL RUNWAY & TAXIWAY BRIDGE SECTION
(per original dwgs - sheet 39)

2.3 Loads

For the dead load calculation below, we used the concrete density to account for the overburden soil - this is a conservative calculation to determine the total DL above the bridge γ_c = 150 pcf concrete density

2.3.1 RunWay Gravity Loads

Dead Load Determination from Previous Calculations

$L_{spanRW} := 40\ ft + 8\ in$	Bridge Span (box beam total length)	$b_{footing} := 6\ ft$	Footing Width
$RW.h_{pf} := 3.13\ ft$	Fill and Pavement Height	$b_{stem} := 18\ in$	Stem Width
$b_{box} := 4\ ft$	Box Beam Width	$h_{footing} := 2\ ft + 6\ in$	Footing Thickness
$h_{box} := 2.75\ ft$	Box Beam height		

$A_{box} := 766\ in^2$ Box Beam Area (cross sectional area)

$\gamma_c := 150\ pcf$ Specific Weight of Concrete (and Fill)
Conservatively assumed heaviest material (i.e., concrete) for overburden dead weight

$h_{stemmax} := 172\ ft + 3\ in - (166\ ft + 2\ in) - h_{footing} = 43.000\ in$ Max Stem Height

$h_{stemmin} := h_{stemmax} - 0.015 \cdot 50\ ft = 34.000\ in$ Stem Height at Edge of Runway

$h_{stemavg} := \text{mean}\left(h_{stemmax}, h_{stemmin}\right) = 38.500\ in$ Average Stem Height

$h_{soil} := 178\ ft + 1.5\ in - (166\ ft + 2\ in) - h_{footing} = 9.458\ ft$ Soil Height on Footing

$w_{pf} := \gamma_c \cdot RW.h_{pf} \cdot \frac{L_{spanRW}}{2} = 9.547\ klf$ Weight of Pavement and Fill Above per foot

$w_{box} := \gamma_c \cdot A_{box} \cdot \frac{L_{spanRW}}{2\ b_{box}} = 4.056\ klf$ Box Beam Weight per foot

$w_{SW} := \gamma_c \cdot \left(b_{footing} \cdot h_{footing} + b_{stem} \cdot h_{stemavg}\right) = 2.972\ klf$ Footing & Stem Weight

$w_{soil} := \gamma_c \cdot \frac{\left(b_{footing} - b_{stem}\right)}{2} \cdot h_{soil} = 3.192\ klf$ Soil on Footing Weight

$Total.DL := \frac{\left(w_{pf} + w_{box} + w_{SW} + w_{soil}\right)}{b_{footing}} = 3294.440\ psf$ Total dead load along bottom of footing from previous calculations

Refining Overburden Dead Loads
(by using soil density for fill material)

$H_{pavement} := 17\ in$ $Fill1_{RW} := 6\ in$ $Fill2_{RW} := 15\ in$

$\rho_{soil} := 120\ pcf$ Based on Emtech Engineers soil boring exploration dated 03-26-2025

$L_{Bridge} := 40.67\ ft$ Length btw stem walls

Max Stem Height $h_{stemmax} := 172\ ft + 3\ in - (166\ ft + 2\ in) - h_{footing} = 43\ in$

Loads below are divided by two to distribute total load between two abutments on either end

~~Weight of Pavement and Fill Above per foot~~ $\cancel{w_{pf} := \gamma_c \cdot h_{pf} \cdot \frac{L_{span}}{2} = 9.547\ klf}$

Box Beam Weight per foot $w_{box} := \gamma_c \cdot A_{box} \cdot \frac{L_{span}}{2\ b_{box}} = 4.056\ klf$

Footing Weight and Stem Weight $w_{SW} := \gamma_c \cdot \left(b_{footing} \cdot h_{footing} + b_{stem} \cdot h_{stemavg}\right) = 2.972\ klf$

~~Soil on Footing Weight~~ $\cancel{w_{soil} := \gamma_c \cdot \frac{\left(b_{footing} - b_{stem}\right)}{2} \cdot h_{soil} = 3.192\ klf}$

REFINING LOADS FROM PREVIOUS CALCULATIONS

$DL_{RW.Bridge.Overburden} := \left(H_{pavement} \cdot \gamma_c\right) + \left(Fill1_{RW} \cdot \rho_{soil}\right) + \left(Fill2_{RW} \cdot \rho_{soil}\right) = 422.500\ psf$

$DL_{RW.Soil.on.Ftg} := \left(H_{pavement} \cdot \gamma_c\right) + \left(\left(Fill1_{RW} + Fill2_{RW}\right) \cdot \rho_{soil}\right) + \left(h_{stemmax} \cdot \rho_{soil}\right) = 852.500\ psf$

$DL_{RW} := \left(DL_{RW.Bridge.Overburden} \cdot \frac{\left(L_{Bridge}\right)}{2}\right) + \left(DL_{RW.Soil.on.Ftg} \cdot \frac{\left(b_{footing} - b_{stem}\right)}{2}\right) = 10.510\ klf$

$TotalDL_{RW} := \frac{\left(DL_{RW} + w_{box} + w_{SW}\right)}{b_{footing}} = 2922.936\ psf$

Total updated dead load along bottom of footing

2.3.2 TaxiWay Gravity Loads

Refining Overburden Dead Loads
(by using soil density for fill material)

$H_{pavement} := 17\ in$ $\quad Fill_{TW} := Fil\&Pavement_{TaxiWay} - H_{pavement} = 1.343\ ft$

$\rho_{soil} := 120\ pcf$ $\quad$ Based on Emtech Engineers soil boring exploration dated 03-26-2025

$L_{Bridge} := 40.67\ ft$ $\quad$ Length btw stem walls

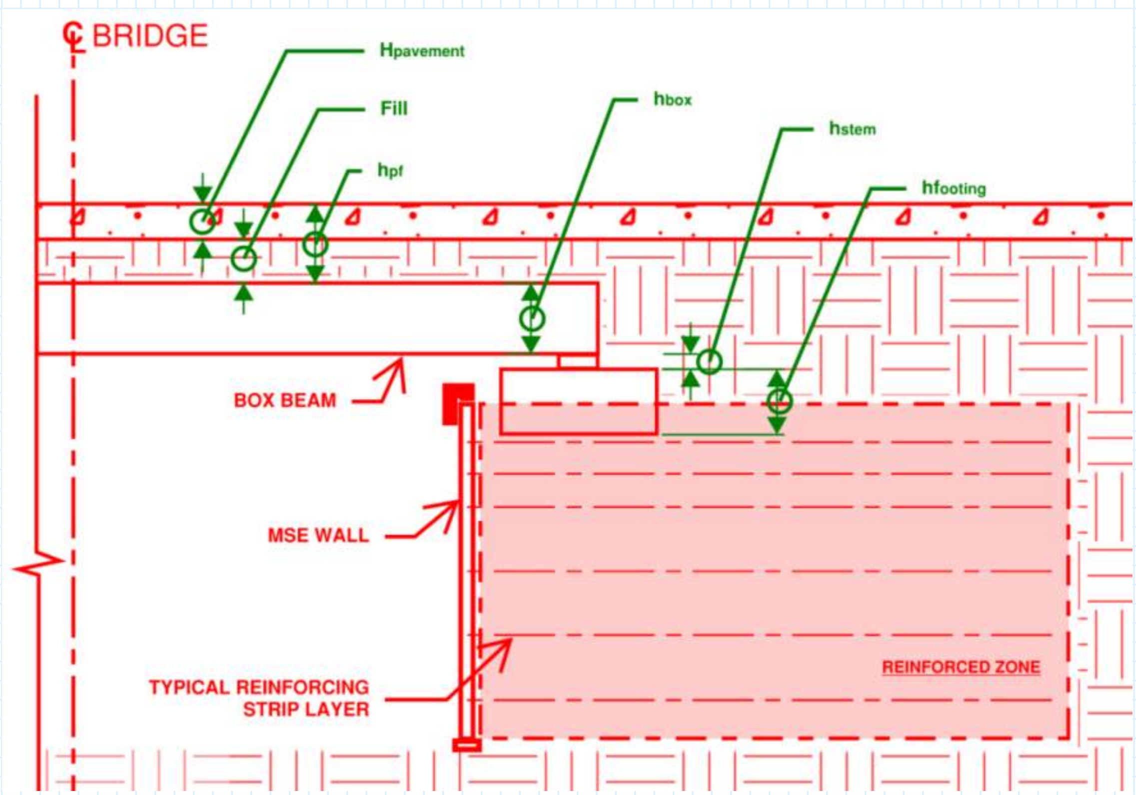
$$DL_{TW.Bridge.Overburden} := (H_{pavement}\ \gamma_c) + (Fill_{TW}\ \rho_{soil}) = 373.700\ psf$$

$$DL_{TW.Soil.on.Ftg} := (H_{pavement}\ \gamma_c) + (Fill_{TW}\ \rho_{soil}) + (h_{stemmax}\ \rho_{soil}) = 803.700\ psf$$

$$DL_{TW} := \left(DL_{TW.Bridge.Overburden} \cdot \frac{(L_{Bridge})}{2}\right) + \left(DL_{TW.Soil.on.Ftg} \cdot \frac{(b_{footing} - b_{stem})}{2}\right) = 9.408\ klf$$

$$TotalDL_{TW} := \frac{(DL_{TW} + w_{box} + w_{SW})}{b_{footing}} = 2739.244\ psf$$

Total updated dead load along bottom of footing



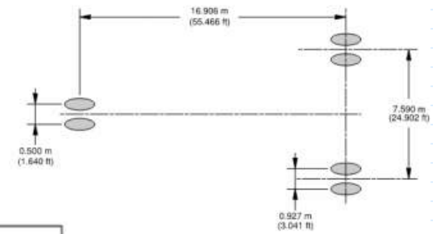
TYPICAL BRIDGE ANNOTATIONS

2.3.3 RunWay & TaxiWay Gravity Live Loads

See below airplane loads per A321 Aircraft Characteristics - Airport and Maintenance Planning

WEIGHT VARIANT	MAXIMUM RAMP WEIGHT	PERCENTAGE OF WEIGHT ON MAIN GEAR GROUP	NOSE GEAR TIRE SIZE	NOSE GEAR TIRE PRESSURE	WING GEAR TIRE SIZE	WING GEAR TIRE PRESSURE
A321NEO WV080 (CG 36.53%)	95 400 kg (210 325 lb)	95.2%	30x8.8R15	11.6 bar (168 psi)	1 270x455R22	15.7 bar (228 psi)
A321NEO WV080 (CG 36.46%)	95 400 kg (210 325 lb)	95.1%	30x8.8R15	11.6 bar (168 psi)	1 270x455R22	15.7 bar (228 psi)

ON A/C A321neo-XLR

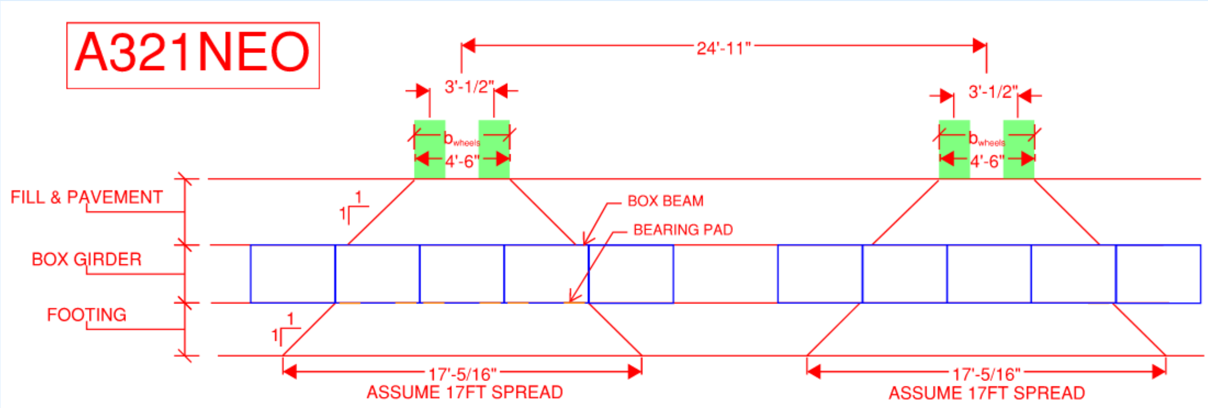


**ON A/C A321neo

3. The following table gives characteristics of A321NEO models, these data are specific to each weight variant:

Aircraft Characteristics			
	WV067	WV070	WV080
Maximum Take-Off Weight (MTOW)	90 000 kg (198 416 lb)	80 000 kg (176 370 lb)	95 000 kg (209 439 lb)

AIRPLANE A321NEO



Load Distribuition from Rear Plane Wheels to Pavement

$Total.LL_{PointLoad} := 165\ \mathbf{kip} = 165.000\ \mathbf{kip}$

Plane load not to exceed 165 kips load

$RW\&TW.MaxLL := \left(\frac{770\ \mathbf{psf} \cdot b_{footing} \cdot 17\ \mathbf{ft}}{0.952}\right) \cdot 2 = 165.000\ \mathbf{kip}$

95.2% or 0.952 = percentage of weight on main gear group (rear wheels) as shown above in the A321 Aircraft Characteristics - Airport and Maintenance Planning

Using 770 psf load acting on base of footing will equal 165kip plane load

2.3.4 RunWay & TaxiWay Horizontal Live Loads

Total Live Load Braking Force:

1	2	3		4	5		6	
		$V_{(NG)}$			$V_{(MG)}$ (PER STRUT)		H (PER STRUT)	
WEIGHT VARIANT	MAXIMUM RAMP WEIGHT	STATIC LOAD AT FWD CG		STATIC BRAKING AT 10 ft/s² DECELERATION	STATIC LOAD AT AFT CG		STEADY BRAKING AT 10 ft/s² DECELERATION	AT INSTANTANEOUS BRAKING COEFFICIENT = 0.8
A321NEO WV080 (CG 36.46%)	95 400 kg (210 325 lb)	8 640 kg (19 050 lb)	17.5% MAC (b)	14 100 kg (31 100 lb)	45 380 kg (100 050 lb)	36.46% MAC (a)	14 830 kg (32 675 lb) (c)	36 300 kg (80 025 lb) (c)

$LL_{Steady.BrakeForce} := 32675\ \textbf{lbf} = 32.675\ \textbf{kip}$ Steady braking force as shown on A321NEO Aircraft Characteristics above

$LL_{Instantaneous.BrakeForce} := 80025\ \textbf{lbf} = 80.025\ \textbf{kip}$ Instantaneous braking force as shown on A321NEO Aircraft Characteristics above

$Max.Ramp.WT := 210325\ \textbf{lbf}$ Max Ramp weight as shown on A321NEO Aircraft Characteristics above

Determining % of Steady & Instantaneous Braking Force for 165 kips Plane Load:

$$Percentage_{Steady.Brake} := \frac{LL_{Steady.BrakeForce}}{Max.Ramp.WT} = 0.155$$

Using this percentage, we can calculate the amount of horizontal force (braking force) using 165 kip plane load

$$Percentage_{Insta.Brake} := \frac{LL_{Instantaneous.BrakeForce}}{Max.Ramp.WT} = 0.380$$

Horiztonal Load Using 165 kips Plane:

$'165kip.Plane := 165\ \textbf{kip}$

$\mu := 0.3$ Coefficient of friction

$TW\&RW.H_{pavement} := 16\ \textbf{in}$ Conservatively, using 16 inches as the pavement thickness

$PavementWeight := TW\&RW.H_{pavement} \cdot \gamma_c = 200.000\ \textbf{psf}$ Pavement selftweight

$LL_{SBF165kip} := ('165kip.Plane \cdot Percentage_{Steady.Brake}) \cdot 2 = 51.267\ \textbf{kip}$ Steady braking force for 2 set of struts

$LL_{IBF165kip} := ('165kip.Plane \cdot Percentage_{Insta.Brake}) \cdot 2 = 125.559\ \textbf{kip}$ Instantaneous braking force for 2 set of struts

Partially Counteracting Horizontal Load with Pavement Selftweight Friction

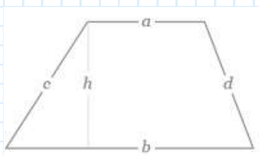
$'\mu := F \cdot N \quad \xrightarrow{} \quad N := {'\mu} \cdot F$

$N1 := \mu \cdot PavementWeight = 60.000\ \textbf{psf}$ Horizontal resisting force acting at bottom of pavement

$$Area_{SBF} := \frac{LL_{SBF165kip}}{N1} = 854.451\ \textbf{ft}^2$$ Area needed to counteract plane horizontal force for Steady Braking

$$Area_{IBF} := \frac{LL_{IBF165kip}}{N1} = 2092.654\ \textbf{ft}^2$$ Area needed to counteract plane horizontal force for Instantaneous Braking

Calculating Minimum Trapezoidal Pavement Area to Resist Braking Force



$$'A_{SBF} := \frac{(a+b)}{2} \cdot h$$

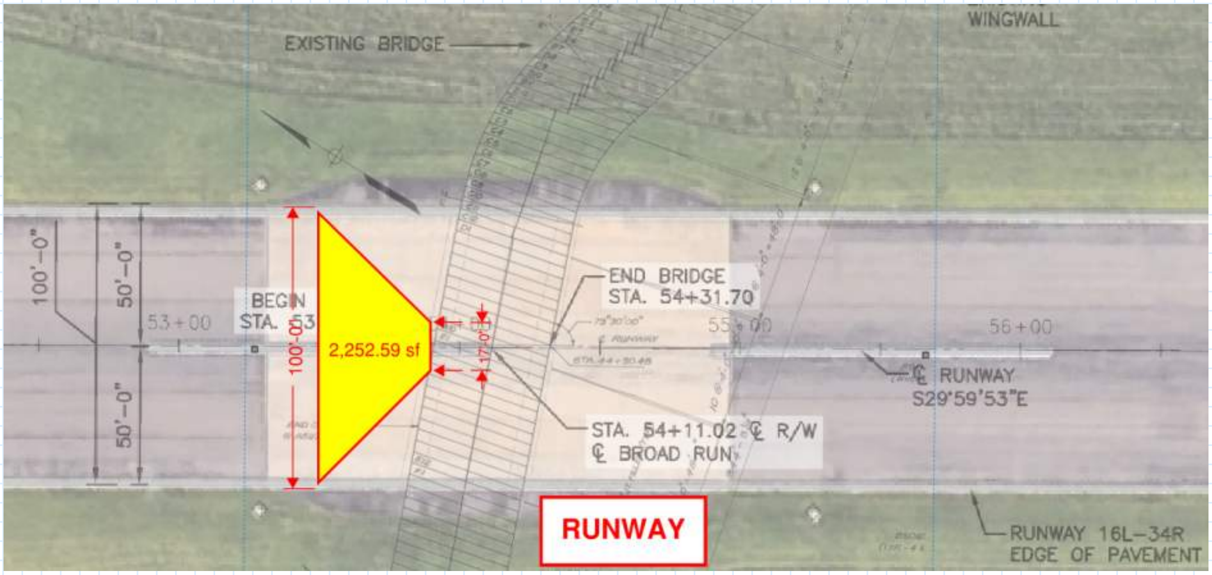
$a := 17\ \textbf{ft}$

$h_{SBF} := 26\ \textbf{ft}$ $b_{SBF} := 2 \cdot h_{SBF} = 52.000\ \textbf{ft}$ Height for Steady braking force

$h_{IBF} := 38\ \textbf{ft}$ $b_{IBF} := 100\ \textbf{ft} = 100.000\ \textbf{ft}$ Height for Instantaneous braking force

$$'A_{SBF} := \frac{(a+b_{SBF})}{2} \cdot h_{SBF} = 897.000\ \textbf{ft}^2 \quad > \quad Area_{SBF} = 854.451\ \textbf{ft}^2$$

$$'A_{IBF} := \frac{(a+b_{IBF})}{2} \cdot h_{IBF} = 2223.000\ \textbf{ft}^2 \quad > \quad Area_{IBF} := 2092.654\ \textbf{ft}^2$$



VISUALLY SHOWING APPROXIMATE PAVEMENT AREA ENGAGED TO RESIST INSTANTANEOUS BRAKING FORCE

$N1 = 60.000\ psf$ Horizontal resisting force acting at bottom of pavement. This horizontal resisting force will be added in the MSE wall software as lateral loads

3.0 MSE Wall Analysis

3.1 RunWay & TaxiWay - Reinforcing Strip Capacity

Per existing drawings "Manassas Municipal Airport original drawings by Campbell, McQueen, &paris, Engineers dated July 1983.

$Width_{Reinf.Strip} := 60\ mm = 2.362\ in$

$Thickness_{Reinf.Strip} := 5\ mm = 0.197\ in$

Per AASHTO "Es" Sacrificial Thickness of metal for 50 years life duration

Per AASHTO section 11.10.6.4.2a, we are determining the structural capacity of the reinforcing strips for a life duration of 50 years located at the runway MSE wall

$Galv.Thickness := 3.5$ Per reinforcing strip report (galvanization applied to metal is in mills)

$ThicknessLoss_{First.2years} := 0.58 \cdot 2 = 1.160$ 3.5 mills - 1.16 mills = 2.34 mills

$ThicknessLoss_{Remaining.Galv.Loss} := \frac{2.34}{0.16} = 14.625$ 3.5 mills of galvanization = 14 years

$ThicknessLoss_{CarbonSection50} := (50 - 16) \cdot 0.47 = 15.980$ 16 mills loss in the steel section per face = 0.016 inch

$Es_{50} := (0.016\ in) \cdot 2 = 0.032\ in$ section loss per strip

$Ec_{50} := Thickness_{Reinf.Strip} - Es_{50} = 0.165\ in$ new thickness of reinforcing strip for 50 years

Per AASHTO "Es" Sacrificial Thickness of metal for 100 years life duration

$ThicknessLoss_{CarbonSection100} := (100 - 16) \cdot 0.47 = 39.480$ 40 mills loss in the steel section per face = 0.040 inch

$Es_{100} := (0.040\ in) \cdot 2 = 0.080\ in$ section loss per strip

$Ec_{100} := Thickness_{Reinf.Strip} - Es_{100} = 0.117\ in$ new thickness of reinforcing strip for 100 years

Yield & Ultimate Strength of Reinforcing Strips:

$Fy_{Reinf.Strip.36ksi} := 36000\ psi$ Based on specifications

$Fu_{Reinf.Strip.58ksi} := 58000\ psi$ Based on specifications

$T_{al.50Yrs.ReinfStrip36ksi} := (Width_{Reinf.Strip} \cdot Ec_{50}) \cdot Fy_{Reinf.Strip.36ksi} = 14.019\ kip$

$T_{ult.50Yrs.ReinfStrip58ksi} := (Width_{Reinf.Strip} \cdot Ec_{50}) \cdot Fu_{Reinf.Strip.58ksi} = 22.586\ kip$

$T_{al.100Yrs.ReinfStrip36ksi} := (Width_{Reinf.Strip} \cdot Ec_{100}) \cdot Fy_{Reinf.Strip.36ksi} = 9.937\ kip$

$T_{ult.100Yrs.ReinfStrip58ksi} := (Width_{Reinf.Strip} \cdot Ec_{100}) \cdot Fu_{Reinf.Strip.58ksi} = 16.009\ kip$

11.10.6.4.2a—Steel Reinforcements

Steel soil reinforcements shall comply with the provisions of AASHTO *LRFD Bridge Construction Specifications*, Article 7.6.4.2, "Steel Reinforcements."

The structural design of steel soil reinforcements and connections shall be made on the basis of a thickness, E_c, as follows:

$$E_c = E_n - E_s \quad (11.10.6.4.2a\text{-}1)$$

where:

E_c = thickness of metal reinforcement at end of service life as shown in Figure 11.10.6.4.1-1 (mil.)

E_n = nominal thickness of steel reinforcement at construction (mil.)

E_s = sacrificial thickness of metal expected to be lost by uniform corrosion during service life of structure (mil.)

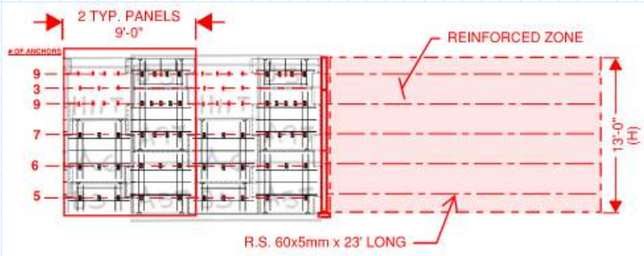
For structural design, sacrificial thicknesses shall be computed for each exposed surface as follows, assuming that the soil backfill used is nonaggressive:

- Loss of galvanizing = 0.58 mil./yr. for first 2 years
  = 0.16 mil./yr. for subsequent years
- Loss of carbon steel = 0.47 mil./yr. after zinc depletion

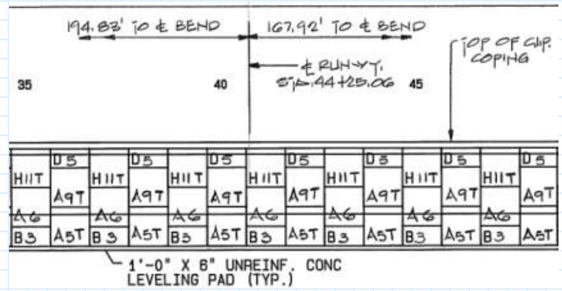
AASHTO Section 11.10.6.4.2a

RUNWAY MSE WALL REINFORCING STRIP LAYERS

$TribWidth_{Reinf.Strip} := 9\ ft$ MSE Panels are repetitive at the taxiway & runway - we are using 9 ft as the average length to calculate the strength of reinforcing strip per foot basis (see figure below)



TYPICAL RUNWAY PANEL LAYOUT



Laying out MSE Panels next to MSE Wall to determine the typical # of reinforcing strips per foot

TYPICAL RUNWAY MSE WALL PANEL LAYOUT

$First.Reinf.Layer_{\#ofStrips} := 9$ # of anchors found on typical MSE wall for 2 panels - 1st layer from the top of wall

$Second.Reinf.Layer_{\#ofStrips} := 3$ # of anchors found on typical MSE wall for 2 panels - 2nd layer from the top of wall

$Third.Reinf.Layer_{\#ofStrips} := 9$ # of anchors found on typical MSE wall for 2 panels - 3rd layer from the top of wall

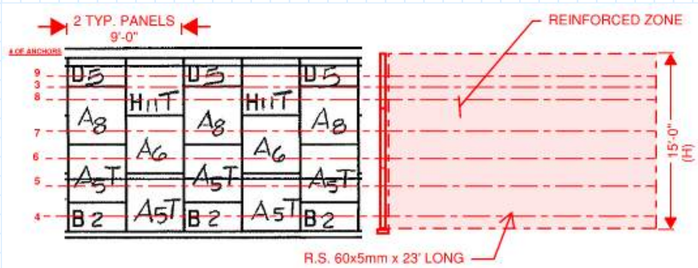
$Fourth.Reinf.Layer_{\#ofStrips} := 7$ # of anchors found on typical MSE wall for 2 panels - 4th layer from the top of wall

$Fifth.Reinf.Layer_{\#ofStrips} := 6$ # of anchors found on typical MSE wall for 2 panels - 5th layer from the top of wall

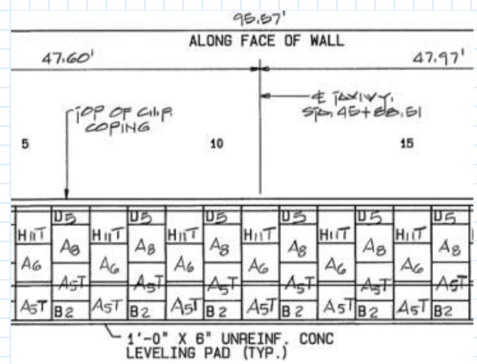
$Sixth.Reinf.Layer_{\#ofStrips} := 5$ # of anchors found on typical MSE wall for 2 panels - 6th layer from the top of wall

TAXIWAY MSW WALL REINFORCING STRIP LAYERS

$TribWidth_{Reinf.Strip} := 9\ ft$ MSE Panels are repetitive at the taxiway & runway - we are using 9 ft as the average length to calculate the strength of reinforcing strip per foot basis (see figure below)



TYPICAL TAXIWAY PANEL LAYOUT



Laying out MSE Panels next to MSE Wall to determine the typical # of reinforcing strips per foot

TYPICAL TAXIWAY MSE WALL PANEL LAYOUT

$TW.First.Reinf.Layer_{\#ofStrips} := 9$ # of anchors found on typical MSE wall for 2 panels - 1st layer from the top of wall

$TW.Second.Reinf.Layer_{\#ofStrips} := 3$ # of anchors found on typical MSE wall for 2 panels - 2nd layer from the top of wall

$TW.Third.Reinf.Layer_{\#ofStrips} := 8$ # of anchors found on typical MSE wall for 2 panels - 3rd layer from the top of wall

$TW.Fourth.Reinf.Layer_{\#ofStrips} := 7$ # of anchors found on typical MSE wall for 2 panels - 4th layer from the top of wall

$TW.Fifth.Reinf.Layer_{\#ofStrips} := 6$ # of anchors found on typical MSE wall for 2 panels - 5th layer from the top of wall

$TW.Sixth.Reinf.Layer_{\#ofStrips} := 5$ # of anchors found on typical MSE wall for 2 panels - 6th layer from the top of wall

$TW.Seventh.Reinf.Layer_{\#ofStrips} := 4$ # of anchors found on typical MSE wall for 2 panels - 7th layer from the top of wall

50 YRS RUNWAY REINFORCING STRIP CAPACITY USING 45 KSI YIELD STRENGTH AND 62 KSI ULTIMATE STRENGTH:

Ultimate tensile strength of reinforcing strips per each layer:

$'50Yrs.T_{ult9} := \frac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \; First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 22585.802 \; \frac{lbf}{ft}$ At the 1st reinforcement layer from top of wall, we have 9 anchors

$'50Yrs.T_{ult3} := \frac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \; Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7528.601 \; \frac{lbf}{ft}$ At the 2nd reinforcement layer from top of wall, we have 3 anchors

$'50Yrs.T_{ult9} = 22585.802 \; \frac{lbf}{ft}$ At the 3rd reinforcement layer from top of wall, we have 9 anchors

$'50Yrs.T_{ult7} := \frac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \; Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 17566.735 \; \frac{lbf}{ft}$ At the 4th reinforcement layer from top of wall, we have 7 anchors

$'50Yrs.T_{ult6} := \frac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \; Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 15057.201 \; \frac{lbf}{ft}$ At the 5th reinforcement layer from top of wall, we have 6 anchors

$'50Yrs.T_{ult5} := \frac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \; Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 12547.668 \; \frac{lbf}{ft}$ At the 6th reinforcement layer from top of wall, we have 5 anchors

Tensile design strength of reinforcing strips per each layer:

$'50Yrs.R_{t9} := \frac{\left(T_{al.50Yrs.ReinfStrip36ksi} \; First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 14018.774 \; \frac{lbf}{ft}$ At the 1st reinforcement layer from top of wall, we have 9 anchors

$'50Yrs.R_{t3} := \frac{\left(T_{al.50Yrs.ReinfStrip36ksi} \; Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 4672.925 \; \frac{lbf}{ft}$ At the 2nd reinforcement layer from top of wall, we have 3 anchors

$'50Yrs.R_{t9} = 14018.774 \; \frac{lbf}{ft}$ At the 3rd reinforcement layer from top of wall, we have 9 anchors

$'50Yrs.R_{t7} := \frac{\left(T_{al.50Yrs.ReinfStrip36ksi} \; Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 10903.491 \; \frac{lbf}{ft}$ At the 4th reinforcement layer from top of wall, we have 7 anchors

$'50Yrs.R_{t6} := \frac{\left(T_{al.50Yrs.ReinfStrip36ksi} \; Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 9345.849 \; \frac{lbf}{ft}$ At the 5th reinforcement layer from top of wall, we have 6 anchors

$'50Yrs.R_{t5} := \frac{\left(T_{al.50Yrs.ReinfStrip36ksi} \; Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7788.208 \; \frac{lbf}{ft}$ At the 6th reinforcement layer from top of wall, we have 5 anchors

100 YRS RUNWAY REINFORCING STRIP CAPACITY USING 36 KSI YIELD STRENGTH AND 58 KSI ULTIMATE STRENGTH:

Ultimate tensile strength of reinforcing strips per each layer:

$'100Yrs.T_{ult9} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 16009.424 \frac{lbf}{ft}$ At the 1st reinforcement layer from top of wall, we have 9 anchors

$'100Yrs.T_{ult3} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 5336.475 \frac{lbf}{ft}$ At the 2nd reinforcement layer from top of wall, we have 3 anchors

$'100Yrs.T_{ult9} = 16009.424 \frac{lbf}{ft}$ At the 3rd reinforcement layer from top of wall, we have 9 anchors

$'100Yrs.T_{ult7} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 12451.774 \frac{lbf}{ft}$ At the 4th reinforcement layer from top of wall, we have 7 anchors

$'100Yrs.T_{ult6} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 10672.949 \frac{lbf}{ft}$ At the 5th reinforcement layer from top of wall, we have 6 anchors

$'100Yrs.T_{ult5} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 8894.124 \frac{lbf}{ft}$ At the 6th reinforcement layer from top of wall, we have 5 anchors

Tensile design strength of reinforcing strips per each layer:

$'100Yrs.R_{t9} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 9936.884 \frac{lbf}{ft}$ At the 1st reinforcement layer from top of wall, we have 9 anchors

$'100Yrs.R_{t3} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 3312.295 \frac{lbf}{ft}$ At the 2nd reinforcement layer from top of wall, we have 3 anchors

$'100Yrs.R_{t9} = 9936.884 \frac{lbf}{ft}$ At the 3rd reinforcement layer from top of wall, we have 9 anchors

$'100Yrs.R_{t7} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7728.687 \frac{lbf}{ft}$ At the 4th reinforcement layer from top of wall, we have 7 anchors

$'100Yrs.R_{t6} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 6624.589 \frac{lbf}{ft}$ At the 5th reinforcement layer from top of wall, we have 6 anchors

$'100Yrs.R_{t5} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 5520.491 \frac{lbf}{ft}$ At the 6th reinforcement layer from top of wall, we have 5 anchors

50 YRS TAXIWAY REINFORCING STRIP CAPACITY USING 45 KSI YIELD STRENGTH AND 62 KSI ULTIMATE STRENGTH:

Ultimate tensile strength of reinforcing strips per each layer:

$TW.50YrsT_{ult9} := \dfrac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \cdot TW.First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 22585.802 \; \dfrac{lbf}{ft}$

At the 1st reinforcement layer from top of wall, we have 9 anchors

$TW.50YrsT_{ult3} := \dfrac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \cdot TW.Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7528.601 \; \dfrac{lbf}{ft}$

At the 2nd reinforcement layer from top of wall, we have 3 anchors

$TW.50YrsT_{ult8} := \dfrac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \cdot TW.Third.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 20076.268 \; \dfrac{lbf}{ft}$

At the 3th reinforcement layer from top of wall, we have 8 anchors

$TW.50YrsT_{ult7} := \dfrac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \cdot TW.Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 17566.735 \; \dfrac{lbf}{ft}$

At the 4th reinforcement layer from top of wall, we have 7 anchors

$TW.50YrsT_{ult6} := \dfrac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \cdot TW.Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 15057.201 \; \dfrac{lbf}{ft}$

At the 5th reinforcement layer from top of wall, we have 6 anchors

$TW.50YrsT_{ult5} := \dfrac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \cdot TW.Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 12547.668 \; \dfrac{lbf}{ft}$

At the 6th reinforcement layer from top of wall, we have 5 anchors

$TW.50Yrs.T_{ult4} := \dfrac{\left(T_{ult.50Yrs.ReinfStrip58ksi} \cdot TW.Seventh.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 10038.134 \; \dfrac{lbf}{ft}$

At the 7th reinforcement layer from top of wall, we have 4 anchors

Tensile design strength of reinforcing strips per each layer:

$TW.50Yrs.R_{t9} := \dfrac{\left(T_{al.50Yrs.ReinfStrip36ksi} \cdot TW.First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 14018.774 \; \dfrac{lbf}{ft}$

At the 1st reinforcement layer from top of wall, we have 9 anchors

$TW.50Yrs.R_{t3} := \dfrac{\left(T_{al.50Yrs.ReinfStrip36ksi} \cdot TW.Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 4672.925 \; \dfrac{lbf}{ft}$

At the 2nd reinforcement layer from top of wall, we have 3 anchors

$TW.50Yrs.R_{t8} := \dfrac{\left(T_{al.50Yrs.ReinfStrip36ksi} \cdot TW.Third.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 12461.132 \; \dfrac{lbf}{ft}$

At the 3th reinforcement layer from top of wall, we have 8 anchors

$TW.50Yrs.R_{t7} := \dfrac{\left(T_{al.50Yrs.ReinfStrip36ksi} \cdot TW.Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 10903.491 \; \dfrac{lbf}{ft}$

At the 4th reinforcement layer from top of wall, we have 7 anchors

$TW.50Yrs.R_{t6} := \dfrac{\left(T_{al.50Yrs.ReinfStrip36ksi} \cdot TW.Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 9345.849 \; \dfrac{lbf}{ft}$

At the 5th reinforcement layer from top of wall, we have 6 anchors

$TW.50Yrs.R_{t5} := \dfrac{\left(T_{al.50Yrs.ReinfStrip36ksi} \cdot TW.Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7788.208 \; \dfrac{lbf}{ft}$

At the 6th reinforcement layer from top of wall, we have 5 anchors

$TW.50Yrs.R_{t4} := \dfrac{\left(T_{al.50Yrs.ReinfStrip36ksi} \cdot TW.Seventh.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 6230.566 \; \dfrac{lbf}{ft}$

At the 7th reinforcement layer from top of wall, we have 4 anchors

100 YRS TAXIWAY REINFORCING STRIP CAPACITY USING 45 KSI YIELD STRENGTH AND 62 KSI ULTIMATE STRENGTH:

Ultimate tensile strength of reinforcing strips per each layer:

$$TW.100YrsT_{ult9} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot TW.First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 16009.424 \ \frac{lbf}{ft}$$

At the 1st reinforcement layer from top of wall, we have 9 anchors

$$TW.100YrsT_{ult3} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot TW.Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 5336.475 \ \frac{lbf}{ft}$$

At the 2nd reinforcement layer from top of wall, we have 3 anchors

$$TW.100YrsT_{ult8} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot TW.Third.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 14230.599 \ \frac{lbf}{ft}$$

At the 3th reinforcement layer from top of wall, we have 8 anchors

$$TW.100YrsT_{ult7} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot TW.Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 12451.774 \ \frac{lbf}{ft}$$

At the 4th reinforcement layer from top of wall, we have 7 anchors

$$TW.100YrsT_{ult6} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot TW.Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 10672.949 \ \frac{lbf}{ft}$$

At the 5th reinforcement layer from top of wall, we have 6 anchors

$$TW.100YrsT_{ult5} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot TW.Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 8894.124 \ \frac{lbf}{ft}$$

At the 6th reinforcement layer from top of wall, we have 5 anchors

$$TW.100Yrs.T_{ult4} := \frac{\left(T_{ult.100Yrs.ReinfStrip58ksi} \cdot TW.Seventh.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7115.300 \ \frac{lbf}{ft}$$

At the 7th reinforcement layer from top of wall, we have 4 anchors

Tensile design strength of reinforcing strips per each layer:

$$TW.100Yrs.R_{t9} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot TW.First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 9936.884 \ \frac{lbf}{ft}$$

At the 1st reinforcement layer from top of wall, we have 9 anchors

$$TW.100Yrs.R_{t3} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot TW.Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 3312.295 \ \frac{lbf}{ft}$$

At the 2nd reinforcement layer from top of wall, we have 3 anchors

$$TW.100Yrs.R_{t8} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot TW.Third.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 8832.786 \ \frac{lbf}{ft}$$

At the 3th reinforcement layer from top of wall, we have 8 anchors

$$TW.100Yrs.R_{t7} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot TW.Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7728.687 \ \frac{lbf}{ft}$$

At the 4th reinforcement layer from top of wall, we have 7 anchors

$$TW.100Yrs.R_{t6} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot TW.Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 6624.589 \ \frac{lbf}{ft}$$

At the 5th reinforcement layer from top of wall, we have 6 anchors

$$TW.100Yrs.R_{t5} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot TW.Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 5520.491 \ \frac{lbf}{ft}$$

At the 6th reinforcement layer from top of wall, we have 5 anchors

$$TW.100Yrs.R_{t4} := \frac{\left(T_{al.100Yrs.ReinfStrip36ksi} \cdot TW.Seventh.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 4416.393 \ \frac{lbf}{ft}$$

At the 7th reinforcement layer from top of wall, we have 4 anchors

3.4 MSE Wall Panel Connection Checks

Per existing drawings "Manassas Municipal Airport original drawings by Campbell, McQueen, &paris, Engineers dated July 1983.

$Fy_{Reinf.Strip} := 36000\ psi$ $Fu_{Reinf.Strip} := 58000\ psi$

$Es_{50.Bolt} := 0.016\ in$ section loss per strip for 50 year corrosion reduction

$Es_{100.Bolt} := 0.04\ in$ section loss per strip for 100 year corrosion reduction

$TieStrip.Width := 2.375\ in$

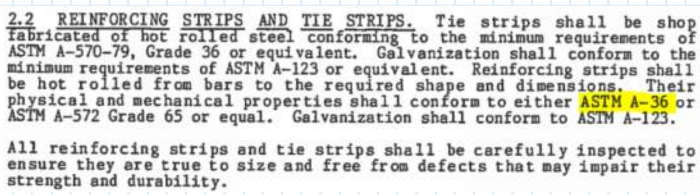
$TieStrip.Thickness := 0.1345\ in$ Tie strip thickness #10GA with no reduction

$TieStrip.Thickness_{50yrs} := (2 \cdot TieStrip.Thickness) - (2 \cdot Es_{50}) = 0.205\ in$ Reducing the thickness per AASHTO 50 year corrosion reduction for 2 tie strips

$TieStrip.Thickness_{100yrs} := (2 \cdot TieStrip.Thickness) - (2 \cdot Es_{100}) = 0.109\ in$ Reducing the thickness per AASHTO 50 year corrosion reduction for 2 tie strips

$TieStrip.Area_{50yrs} := TieStrip.Thickness_{50yrs} \cdot TieStrip.Width = 0.487\ in^2$

$TieStrip.Area_{100yrs} := TieStrip.Thickness_{100yrs} \cdot TieStrip.Width = 0.259\ in^2$



2.2 REINFORCING STRIPS AND TIE STRIPS. Tie strips shall be shop fabricated of hot rolled steel conforming to the minimum requirements of ASTM A-570-79, Grade 36 or equivalent. Galvanization shall conform to the minimum requirements of ASTM A-123 or equivalent. Reinforcing strips shall be hot rolled from bars to the required shape and dimensions. Their physical and mechanical properties shall conform to either ASTM A-36 or ASTM A-572 Grade 65 or equal. Galvanization shall conform to ASTM A-123.

All reinforcing strips and tie strips shall be carefully inspected to ensure they are true to size and free from defects that may impair their strength and durability.

MSE WALL SPECIFICATIONS

Tie Strip Connection Capacity

$'50YRS.T2_{ult.ReinfStrip} := (TieStrip.Area_{50yrs}) \cdot Fu_{Reinf.Strip} = 28.239\ kip$ $>$ $T_{ult.50Yrs.ReinfStrip58ksi} = 22.586\ kip$

$'50YRS.T_{al2} := TieStrip.Area_{50yrs} \cdot Fy_{Reinf.Strip} = 17.528\ kip$ $>$ $T_{al.50Yrs.ReinfStrip36ksi} = 14.019\ kip$

$'100YRS.T2_{ult.ReinfStrip} := (TieStrip.Area_{100yrs}) \cdot Fu_{Reinf.Strip} = 15.015\ kip$ $<$ $T_{ult.100Yrs.ReinfStrip58ksi} = 16.009\ kip$

$'100YRS.T_{al2} := TieStrip.Area_{100yrs} \cdot Fy_{Reinf.Strip} = 9.320\ kip$ $>$ $T_{al.100Yrs.ReinfStrip36ksi} = 9.937\ kip$

$Ec_{50tiestrip} := TieStrip.Thickness_{50yrs} = 0.205\ in$

$Ec_{100tiestrip} := TieStrip.Thickness_{100yrs} = 0.109\ in$

$Fu_{Plate} := Fu_{Reinf.Strip} = 58.000\ ksi$ (per specifications)

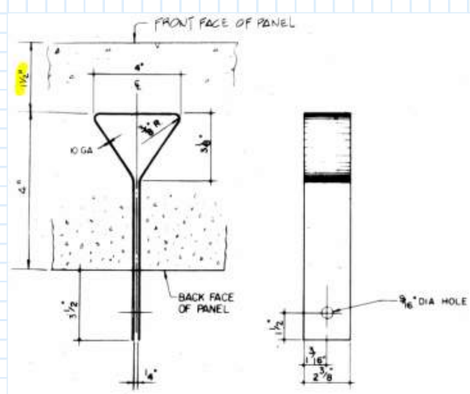
$Fu_{Bolt} := 120\ ksi$ (per specifications)

$lc := 1.5\ in$ (edge distance see tie strip detail)

$\phi := 0.75$

$'50d_{bolt} := (0.5\ in - Es_{50.Bolt}) = 0.484\ in$ $'100d_{bolt} := (0.5\ in - Es_{100.Bolt}) = 0.460\ in$

$'50A_{bolt} := \dfrac{\pi \cdot ('50d_{bolt}^{\ 2})}{4} = 0.184\ in^2$ $'100A_{bolt} := \dfrac{\pi \cdot ('100d_{bolt}^{\ 2})}{4} = 0.166\ in^2$



TIE STRIP DETAIL
(per original dwgs - sheet 41)

<u>50Yr Bolt Shear Strength Check:</u>

Bearing Check = $Rn_{Bearing50} := 3.0 \ \ '50d_{bolt} \ \ Ec_{50tiestrip} \ \ Fu_{Plate} = 17.264 \ \textbf{kip}$ Per AISC J.3.11a

Tearout Check = $Rn_{Tearout50} := 1.5 \ \ lc \ \ Ec_{50tiestrip} \ \ Fu_{Plate} = 26.753 \ \textbf{kip}$ Per AISC J.3.11a

<u>50Yr Reinforcing Strip and Tie Strip Tear out Check:</u>

Bearing Check = $Rn_{Bolt50} := 0.75 \ \ '50A_{bolt} \ \ Fu_{Bolt} = 16.559 \ \textbf{kip}$ Per AISC J.3.8

We will be using Bearing Check capacity for our connection strength $Rn_{Bolt50} = 16.559 \ \textbf{kip}$

$$'50YrsTIESTRIP.R_{con9} := \frac{\left(Rn_{Bolt50} \ \ First.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 16558.581 \ \frac{\textbf{lbf}}{\textbf{ft}}$$ Layer with 9 anchors

$$'50YrsTIESTRIP.R_{con8} := \frac{\left(Rn_{Bearing50} \ \ TW.Third.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 15346.027 \ \frac{\textbf{lbf}}{\textbf{ft}}$$ Layer with 8 anchors

$$'50YrsTIESTRIP.R_{con3} := \frac{\left(Rn_{Bolt50} \ \ Second.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 5519.527 \ \frac{\textbf{lbf}}{\textbf{ft}}$$ Layer with 3 anchors

$$'50YrsTIESTRIP.R_{con7} := \frac{\left(Rn_{Bolt50} \ \ Fourth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 12878.896 \ \frac{\textbf{lbf}}{\textbf{ft}}$$ Layer with 7 anchors

$$'50YrsTIESTRIP.R_{con6} := \frac{\left(Rn_{Bolt50} \ \ Fifth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 11039.054 \ \frac{\textbf{lbf}}{\textbf{ft}}$$ Layer with 6 anchors

$$'50YrsTIESTRIP.R_{con5} := \frac{\left(Rn_{Bolt50} \ \ Sixth.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 9199.212 \ \frac{\textbf{lbf}}{\textbf{ft}}$$ Layer with 5 anchors

$$'50YrsTIESTRIP.R_{con4} := \frac{\left(Rn_{Bearing50} \ \ TW.Seventh.Reinf.Layer_{\#ofStrips}\right)}{TribWidth_{Reinf.Strip}} = 7673.013 \ \frac{\textbf{lbf}}{\textbf{ft}}$$ Layer with 4 anchors

COMPARE THESE VALUES TO THE CONCRETE CAPACITY CONNECTION BREAK OUT VALUES

100Yr Bolt Shear Strength Check:

Bearing Check = $Rn_{Bearing100} := 3.0 \cdot \phi100d_{bolt} \cdot Ec_{50tiestrip} \cdot Fu_{Plate} = 16.408\ kip$ Per AISC J.3.11a

Tearout Check = $Rn_{Tearout100} := 1.5 \cdot lc \cdot Ec_{100tiestrip} \cdot Fu_{Plate} = 14.225\ kip$ Per AISC J.3.11a

100Yr Reinforcing Strip and Tie Strip Tear out Check:

Bearing Check = $Rn_{Bolt100} := 0.75 \cdot \phi100A_{bolt} \cdot Fu_{Bolt} = 14.957\ kip$ Per AISC J.3.8

We will be using Bearing Check capacity for our connection strength $Rn_{Tearout100} = 14.225\ kip$

$\phi100YrsTIESTRIP.R_{con9} := \frac{(Rn_{Tearout100} \cdot First.Reinf.Layer_{\#ofStrips})}{TribWidth_{Reinf.Strip}} = 14224.500\ \frac{lbf}{ft}$ Layer with 9 anchors

$\phi100YrsTIESTRIP.R_{con8} := \frac{(Rn_{Tearout100} \cdot TW.Third.Reinf.Layer_{\#ofStrips})}{TribWidth_{Reinf.Strip}} = 12644.000\ \frac{lbf}{ft}$ Layer with 8 anchors

$\phi100YrsTIESTRIP.R_{con3} := \frac{(Rn_{Tearout100} \cdot Second.Reinf.Layer_{\#ofStrips})}{TribWidth_{Reinf.Strip}} = 4741.500\ \frac{lbf}{ft}$ Layer with 3 anchors

$\phi100YrsTIESTRIP.R_{con7} := \frac{(Rn_{Tearout100} \cdot Fourth.Reinf.Layer_{\#ofStrips})}{TribWidth_{Reinf.Strip}} = 11063.500\ \frac{lbf}{ft}$ Layer with 7 anchors

$\phi100YrsTIESTRIP.R_{con6} := \frac{(Rn_{Tearout100} \cdot Fifth.Reinf.Layer_{\#ofStrips})}{TribWidth_{Reinf.Strip}} = 9483.000\ \frac{lbf}{ft}$ Layer with 6 anchors

$\phi100YrsTIESTRIP.R_{con5} := \frac{(Rn_{Tearout100} \cdot Sixth.Reinf.Layer_{\#ofStrips})}{TribWidth_{Reinf.Strip}} = 7902.500\ \frac{lbf}{ft}$ Layer with 5 anchors

$\phi100YrsTIESTRIP.R_{con4} := \frac{(Rn_{Tearout100} \cdot TW.Seventh.Reinf.Layer_{\#ofStrips})}{TribWidth_{Reinf.Strip}} = 6322.000\ \frac{lbf}{ft}$ Layer with 4 anchors

COMPARE THESE VALUES TO THE CONCRETE CAPACITY CONNECTION BREAK OUT VALUES

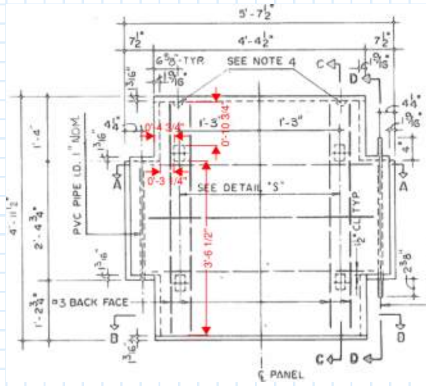
Concrete Break out Capacity:

Assuming tie strips to be a headed anchor to determine its capacity per ACI 318-14 chapter 17

$A_{brg} := (TieStrip.Thickness \cdot 2)\ \ TieStrip.Width = 0.639\ in^2$ (tie strip area)

$Existing.f'c = 4.000\ ksi$ $\qquad \phi_{anchor} := 0.7$ per ACI 318-14 section 17.3.3.3

$N_P := 8 \cdot A_{brg}\ \ Existing.f'c = 20.444\ kip$ Pull out strength of a single anchor per ACI 318-14 section 17.4.3.4



TYPICAL MSE WALL PANEL DETAIL
(per original dwgs - sheet 41)

Concrete Break Strength of Anchors in Tension:

$K_C := 24$ Nominal concrete breakout strength per ACI 17.4.2.6

$\lambda_a := 1.0$ Nominal factor for lightweight concrete

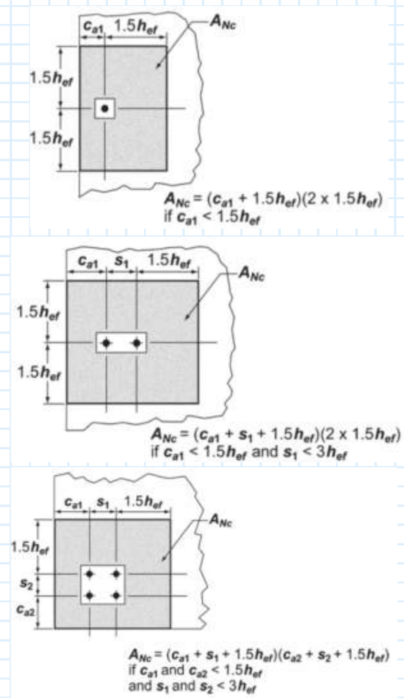
$H_{ef} := 4\ in$ Effective embedment depth of anchor

$e_N := 0.0\ in$ Eccentricity of tensile load

$\psi_{c.N} := 1.0$ Modificattion factor for cracked concrete

$\psi_{cp.N} := 1.0$ Modificattion factor for post installed anchor

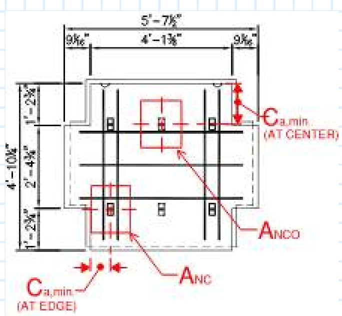
$\psi_{ec.N} := \dfrac{1}{\left(1 + \dfrac{(2 \cdot e_N)}{(3\ \ H_{ef})}\right)} = 1.000$ Nominal breakout strenght when tensile force is eccentric



ACI FIG. R17.4.2.1

$C_{a.min1} := (6.625\ in - 1.5625\ in) = 5.063\ in$ (edge anchor)

$C_{a.min2} := 14.375\ in - \dfrac{(2.375\ in)}{2} = 13.188\ in$ (center anchor)



$'4Anchor_{spacing} := 10\ in$

$'5Anchor_{spacing} := 7.5\ in$

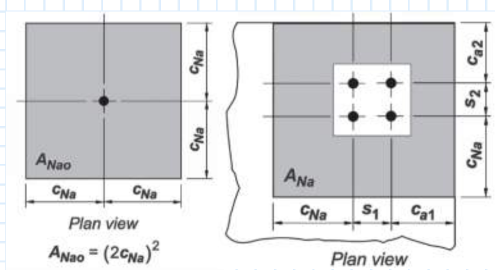
if $C_{a.min1} = 5.063\ in < 1.5 \cdot H_{ef} = 6.000\ in$

$\psi_{ed.N.edge} := 0.7 + 0.3\ \dfrac{(C_{a.min1})}{(1.5 \cdot H_{ef})} = 0.953$

if $C_{a.min2} = 13.188\ in > 1.5 \cdot H_{ef} = 6.000\ in$ then $\psi_{ed.N.center} := 1.0$

$N_b := K_C\ \ \lambda_a\ \sqrt[2]{4000}\ \ (4)^{1.5}\ (lbf) = 12143.146\ lbf$ Basic conc break out strenght for a single anchor per ACI 17.4.2.6

$A_{NCO.1} := 9\ \ H_{ef}^{\ 2} = 144.000\ in^2$ Max. area of concrete cone break out around a single anchor under tension laod per ACI 17.4.2.1c



ACI FIG. R17.4.5.1

Determining Concrete Breakout Capacity using 4 ksi Concrete Strength

Concrete Break Strength of 1 Anchor located at the center:

$A_{NC.1} := ((2 \cdot 1.5 \cdot H_{ef})) \cdot (2 \cdot 1.5 \cdot H_{ef}) = 144.000\ in^2$ Actual breakout area per ACI 17.4.2.1

$N_{cb1.4ksi} := \frac{A_{NC.1}}{A_{NCO.1}} \cdot \psi_{ed.N.center} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 12.143\ kip$ Calculated tensile strength to resist cone break out per ACI 17.4.2.1

Concrete Break Strength for 2 Anchor located at edges of panel:

$A_{NC.edge} := ((C_{a.min1}) + (1.5 \cdot H_{ef})) \cdot (2 \cdot 1.5 \cdot H_{ef}) = 132.750\ in^2$ Actual breakout area per ACI 17.4.2.1

$N_{cbg2.4ksi} := \left(\frac{A_{NC.edge}}{A_{NCO.1}} \cdot \psi_{ed.N.edge} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b\right) \cdot 2 = 21.339\ kip$ Calculated tensile strength to resist cone break out per ACI 17.4.2.1

Concrete Break Strength for 4 Anchor in a row:

$A_{NC.4} := (((2 \cdot C_{a.min1}) + (3 \cdot 4Anchor_{spacing}))) \cdot ((2 \cdot 1.5 \cdot H_{ef}) + 4\ in) = 642.000\ in^2$ Actual breakout area per ACI 17.4.2.1

$N_{cbg4.4ksi} := \frac{A_{NC.4}}{A_{NCO.1}} \cdot \psi_{ed.N.edge} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 51.600\ kip$ Calculated tensile strength to resist cone break out per ACI 17.4.2.1

Concrete Break Strength for 5 Anchor in a row:

$A_{NC.5} := (((2 \cdot C_{a.min1}) + (3 \cdot 5Anchor_{spacing}))) \cdot ((2 \cdot 1.5 \cdot H_{ef}) + 4\ in) = 522.000\ in^2$ Actual breakout area

$N_{cbg5.4ksi} := \frac{A_{NC.5}}{A_{NCO.1}} \cdot \psi_{ed.N.edge} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 41.956\ kip$ Calculated tensile strength to resist cone break out per ACI 17.4.2.1

Concrete Break out Capacity of Tie Strips in MSE Conc Panel:

$4ksi.R_{con9} := \frac{(N_{cbg4.4ksi} + N_{cbg5.4ksi})}{TribWidth_{Reinf.Strip}} = 10395.109\ \frac{lbf}{ft}$ Layer with 9 anchors

$4ksi.R_{con8} := \frac{(N_{cbg4.4ksi} + N_{cbg4.4ksi})}{TribWidth_{Reinf.Strip}} = 11466.770\ \frac{lbf}{ft}$ Layer with 8 anchors

$4ksi.R_{con3} := \frac{(N_{cbg2.4ksi} + N_{cb1.4ksi})}{TribWidth_{Reinf.Strip}} = 3720.288\ \frac{lbf}{ft}$ Layer with 3 anchors

$4ksi.R_{con7} := \frac{(N_{cbg4.4ksi} + N_{cbg2.4ksi} + N_{cb1.4ksi})}{TribWidth_{Reinf.Strip}} = 9453.673\ \frac{lbf}{ft}$ Layer with 7 anchors

$4ksi.R_{con6} := \frac{(N_{cbg2.4ksi} + N_{cb1.4ksi} + N_{cbg2.4ksi} + N_{cb1.4ksi})}{TribWidth_{Reinf.Strip}} = 7440.576\ \frac{lbf}{ft}$ Layer with 6 anchors

$4ksi.R_{con5} := \frac{(N_{cbg2.4ksi} + N_{cb1.4ksi} + N_{cbg2.4ksi})}{TribWidth_{Reinf.Strip}} = 6091.337\ \frac{lbf}{ft}$ Layer with 5 anchors

$4ksi.R_{con4} := \frac{(N_{cbg2.4ksi} + N_{cbg2.4ksi})}{TribWidth_{Reinf.Strip}} = 4742.099\ \frac{lbf}{ft}$ Layer with 4 anchors

USE CONCRETE BREAK OUT CAPACITY VALUES SINCE THESE VALUES ARE LOWER THAN THE TIE STRIP CONNECTION VALUES

3.2 Internal Stability - RUNWAY 100 & 50 Year Results

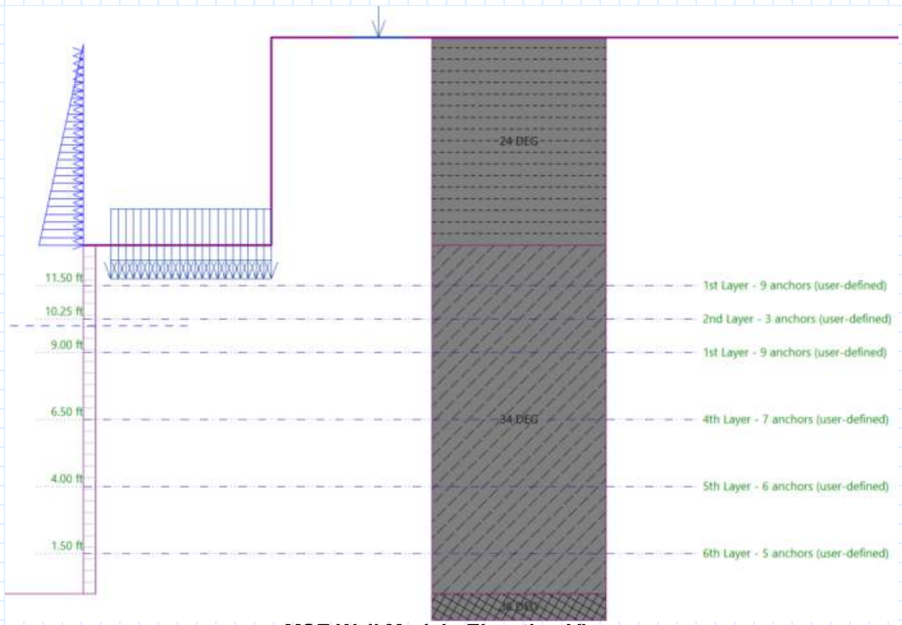
Soil parameters per Engineering & Materials Technologies, inc.

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient		R_{con}
				T_{ult} [lbf/ft]	R_t [lbf/ft]	C_{ds} [–]	C_i [–]	[lbf/ft]
1	1st Layer - 9 anchors	user-defined		16009.42	9936.88	0.60	0.70	10395.11
2	2nd Layer - 3 anchors	user-defined		5336.48	3312.30	0.60	0.70	3720.29
3	4th Layer - 7 anchors	user-defined		12451.77	7728.69	0.60	0.70	9453.67
4	5th Layer - 6 anchors	user-defined		10672.95	6624.59	0.60	0.70	7440.58
5	6th Layer - 5 anchors	user-defined		8894.12	5520.49	0.60	0.70	6091.34

Reinforcing Strip & Connection Input Values for 100 years

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient		R_{con}
				T_{ult} [lbf/ft]	R_t [lbf/ft]	C_{ds} [–]	C_i [–]	[lbf/ft]
1	1st Layer - 9 anchors	user-defined		22585.80	14018.77	0.60	0.70	10395.11
2	2nd Layer - 3 anchors	user-defined		7528.60	4672.92	0.60	0.70	3720.29
3	4th Layer - 7 anchors	user-defined		17566.74	10903.49	0.60	0.70	9453.67
4	5th Layer - 6 anchors	user-defined		15057.20	9345.85	0.60	0.70	7440.58
5	6th Layer - 5 anchors	user-defined		12547.67	7788.21	0.60	0.70	6091.34

Reinforcing Strip & Connection Input Values for 50 years



MSE Wall Model - Elevation View

Verification

OVERTURNING : SATISFACTORY (12.4%)
SLIP : SATISFACTORY (11.6%)

Verification

ECCENTRICITY: SATISFACTORY (6.8%)
FOUNDATION SOIL: SATISFACTORY (42.8%)

Reinforcement capacity

STRENGTH NOT OK. (109.9%)
PULLOUT SATISFACTORY (21.0%)
CONNECTION SATISFACTORY (99.6%)

Dimensioning

SLIP SATISFACTORY (13.9%)

100 Year Summary Resuls

Verification

OVERTURNING : SATISFACTORY (12.4%)
SLIP : SATISFACTORY (11.6%)

Verification

ECCENTRICITY: SATISFACTORY (6.8%)
FOUNDATION SOIL: SATISFACTORY (42.8%)

Reinforcement capacity

STRENGTH SATISFACTORY (77.9%)
PULLOUT SATISFACTORY (21.0%)
CONNECTION SATISFACTORY (99.6%)

Dimensioning

SLIP SATISFACTORY (13.9%)

50 Year Summary Resuls

3.4 Internal Stability - TAXIWAY 50 Year Results

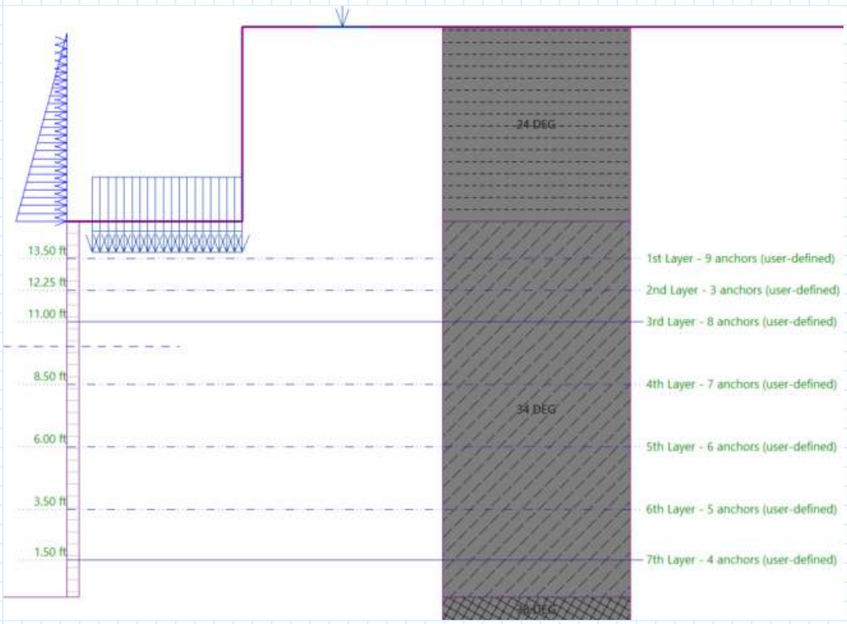
Soil parameters per Engineering & Materials Technologies, inc.

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient		R_{con}
				T_{ult} [lbf/ft]	R_t [lbf/ft]	C_{ds} [–]	C_i [–]	[lbf/ft]
1	1st Layer - 9 anchors	user-defined	– ·· – ·· – ··	16009.42	9936.88	0.60	0.70	10395.11
2	2nd Layer - 3 anchors	user-defined	– ·· – ·· – ··	5336.48	3312.30	0.60	0.70	3720.29
3	4th Layer - 7 anchors	user-defined	– ·· – ·· – ··	12451.77	7728.69	0.60	0.70	9453.67
4	5th Layer - 6 anchors	user-defined	– ·· – ·· – ··	10672.95	6624.59	0.60	0.70	7440.58
5	6th Layer - 5 anchors	user-defined	– ·· – ·· – ··	8894.12	5520.49	0.60	0.70	6091.34

Reinforcing Strip & Connection Input Values for 100 years

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient		R_{con}
				T_{ult} [lbf/ft]	R_t [lbf/ft]	C_{ds} [–]	C_i [–]	[lbf/ft]
1	1st Layer - 9 anchors	user-defined	– ·· – ·· – ··	22585.80	14018.77	0.60	0.70	10395.11
2	2nd Layer - 3 anchors	user-defined	– ·· – ·· – ··	7528.60	4672.92	0.60	0.70	3720.29
3	4th Layer - 7 anchors	user-defined	– ·· – ·· – ··	17566.74	10903.49	0.60	0.70	9453.67
4	5th Layer - 6 anchors	user-defined	– ·· – ·· – ··	15057.20	9345.85	0.60	0.70	7440.58
5	6th Layer - 5 anchors	user-defined	– ·· – ·· – ··	12547.67	7788.21	0.60	0.70	6091.34

Reinforcing Strip & Connection Input Values for 50 years



MSE Wall Model - Elevation View

1

Verification

OVERTURNING :	SATISFACTORY	(11.3%)
SLIP :	SATISFACTORY	(10.9%)

Verification

ECCENTRICITY:	SATISFACTORY	(0.5%)
FOUNDATION SOIL:	SATISFACTORY	(44.1%)

Reinforcement capacity

STRENGTH	SATISFACTORY	(95.5%)
PULLOUT	SATISFACTORY	(20.4%)
CONNECTION	SATISFACTORY	(85.1%)

Dimensioning

SLIP	SATISFACTORY	(13.5%)

100 Year Summary Resuls

Verification

OVERTURNING :	SATISFACTORY	(11.3%)
SLIP :	SATISFACTORY	(10.9%)

Verification

ECCENTRICITY:	SATISFACTORY	(0.5%)
FOUNDATION SOIL:	SATISFACTORY	(44.1%)

Reinforcement capacity

STRENGTH	SATISFACTORY	(68.4%)
PULLOUT	SATISFACTORY	(20.8%)
CONNECTION	SATISFACTORY	(87.4%)

Dimensioning

SLIP	SATISFACTORY	(13.5%)

50 Year Summary Resuls

Analysis of reinforced slopes

Input data

Date : 3/3/2025

Settings

(input for current task)

Materials and standards

Concrete structures : ACI 318-19

Wall analysis

Verification methodology : according to LRFD
Active earth pressure calculation : Coulomb
Passive earth pressure calculation : Mazindrani (Rankine)
Earthquake analysis : Mononobe-Okabe
Shape of earth wedge : Consider always vertical
Allowable eccentricity : 0.333
Internal stability : AASHTO - Inextensible (broken slip surface)

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Dead load of structural components :	DC =	0.90	[–]	1.25	[–]
Dead load of wearing surfaces :	DW =	0.65	[–]	1.50	[–]
Earth pressure - active :	EH_A =	0.90	[–]	1.50	[–]
Earth pressure - at rest :	EH_R =	0.90	[–]	1.35	[–]
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Vertical pressure of earth fill :	EV =	1.00	[–]	1.35	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]
Water load :	WA =	1.00	[–]	1.00	[–]

Resistance factors			
Design situation - Strength I			
Resistance factor on overturning :	ϕ_o =	0.90	[–]
Resistance factor on sliding :	ϕ_t =	1.00	[–]
Resistance factor on bearing capacity :	ϕ_b =	0.65	[–]
Resistance factor on passive pressure :	ϕ_{VE} =	0.75	[–]

Stability analysis

Verification methodology : according to LRFD

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]

Resistance factors		
Design situation - Strength I		
Resistance factor on stability :	ϕ_{SS} =	0.65 [–]

Geometry of structure

Embankment height h_n = 13.00 ft
Embankment length l_n = 0.00 ft
Cover thickness t_c = 0.46 ft

Material

Cover material
Unit weight γ = 150.00 pcf
Shear resistance R_s = 0.0 psf

Types of reinforcements

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient	
				T_{ult}[lbf/ft]	R_t[lbf/ft]	C_{ds}[–]	C_i[–]
1	1st Layer - 9 anchors (user-defined)	user-defined	–··–··–··	22585.80	14018.77	0.60	0.70
2	2nd Layer - 3 anchors (user-defined)	user-defined	–··–··–··	7528.60	4672.92	0.60	0.70
3	4th Layer - 7 anchors (user-defined)	user-defined	–··–··–··	17566.74	10903.49	0.60	0.70
4	5th Layer - 6 anchors (user-defined)	user-defined	–··–··–··	15057.20	9345.85	0.60	0.70
5	6th Layer - 5 anchors (user-defined)	user-defined	–··–··–··	12547.67	7788.21	0.60	0.70

Reinforcement details

1. 1st Layer - 9 anchors (user-defined)
Short-term char. strength T_{ult} = 22585.80 lbf/ft
Long-term design strength R_t = 14018.77 lbf/ft

Design connection strength R_{con} = 10395.11 lbf/ft
2. 2nd Layer - 3 anchors (user-defined)
Short-term char. strength T_{ult} = 7528.60 lbf/ft
Long-term design strength R_t = 4672.92 lbf/ft

Design connection strength R_{con} = 3720.29 lbf/ft
3. 4th Layer - 7 anchors (user-defined)
Short-term char. strength T_{ult} = 17566.74 lbf/ft
Long-term design strength R_t = 10903.49 lbf/ft

Design connection strength R_{con} = 9453.67 lbf/ft
4. 5th Layer - 6 anchors (user-defined)
Short-term char. strength T_{ult} = 15057.20 lbf/ft
Long-term design strength R_t = 9345.85 lbf/ft

Design connection strength R_{con} = 7440.58 lbf/ft
5. 6th Layer - 5 anchors (user-defined)
Short-term char. strength T_{ult} = 12547.67 lbf/ft
Long-term design strength R_t = 7788.21 lbf/ft

Design connection strength R_{con} = 6091.34 lbf/ft

Reinforcement

No.	Number of reinforcements	Type of reinforcement	Spacing of reinforcements h_r[ft]	Height of first reinforcement y[ft]	Reinforcements geometry
1	1	6th Layer - 5 anchors (user-defined)	1.00	1.50	identical length of reinforcements
2	1	5th Layer - 6 anchors (user-defined)	1.00	4.00	identical length of reinforcements
3	1	4th Layer - 7 anchors (user-defined)	1.00	6.50	identical length of reinforcements
4	1	1st Layer - 9 anchors (user-defined)	1.00	9.00	identical length of reinforcements
5	1	1st Layer - 9 anchors (user-defined)	1.00	11.50	identical length of reinforcements
6	1	2nd Layer - 3 anchors (user-defined)	1.00	10.25	identical length of reinforcements

Reinforcement details

Reinforcement No. 1

Reinforcement type : 6th Layer - 5 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	1.50	23.00

Reinforcement No. 2

Reinforcement type : 5th Layer - 6 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	4.00	23.00

Reinforcement No. 3

Reinforcement type : 4th Layer - 7 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	6.50	23.00

Reinforcement No. 4

Reinforcement type : 1st Layer - 9 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	9.00	23.00

Reinforcement No. 5

Reinforcement type : 1st Layer - 9 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

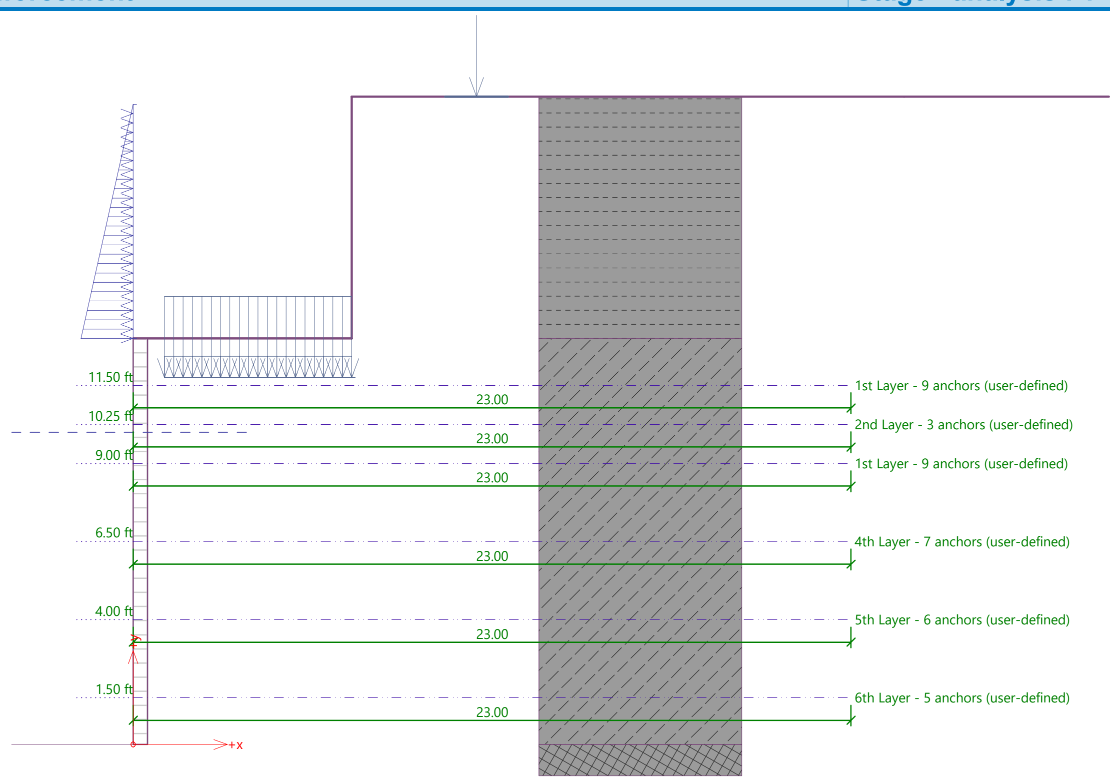
No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	11.50	23.00

Reinforcement No. 6

Reinforcement type : 2nd Layer - 3 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	10.25	23.00

Name : Reinforcement | **Stage - analysis : 1 - 0**



Soil parameters

34 DEG

Basic data

Unit weight : γ = 125.0 [pcf]

Internal friction angle : φ_{ef} = 34.00 [°]

Cohesion : c_{ef} = 0.0 [psf]

Friction angle structure-soil : δ = 23.00 [°]

Uplift pressure

Uplift calculation : standard

Unit weight of saturated soil : γ_{sat} = 130.0 [pcf]

View

Soil pattern :

38 DEG

Basic data

Unit weight : γ = 140.0 [pcf]

Internal friction angle : φ_{ef} = 38.00 [°]

Cohesion : c_{ef} = 0.0 [psf]

Friction angle structure-soil : δ = 20.00 [°]

Uplift pressure

Uplift calculation : standard

Unit weight of saturated soil : γ_{sat} = 140.0 [pcf]

View

Soil pattern :

24 DEG

Basic data

Unit weight : γ = 120.0 [pcf]

Internal friction angle : φ_{ef} = 24.00 [°]

Cohesion : c_{ef} = 0.0 [psf]

Friction angle structure-soil : δ = 24.00 [°]

Uplift pressure

Uplift calculation : standard

Unit weight of saturated soil : γ_{sat} = 130.0 [pcf]

View

Soil pattern :

Geological profile and assigned soils

Position information

Terrain elevation = -9.75 ft

Geological profile and assigned soils

No.	Thickness of layer t [ft]	Depth z [ft]	Elevation [ft]	Assigned soil	Pattern
1	0.01	0.00 .. 0.01	-9.75 .. -9.76	24 DEG	
2	12.99	0.01 .. 13.00	-9.76 .. -22.75	34 DEG	

No.	Thickness of layer t [ft]	Depth z [ft]	Elevation [ft]	Assigned soil	Pattern
3	7.00	13.00 .. 20.00	-22.75 .. -29.75	38 DEG	
4	-	20.00 .. ∞	-29.75 .. -	38 DEG	

Terrain profile

No.	Coordinates x [ft]	Depth z [ft]
1	0.00	0.00
2	7.00	0.00
3	7.00	-7.75
4	8.00	-7.75

Origin [0,0] is located in upper right edge of construction.
Positive coordinate +z has downward direction.

Water influence

Ground water table at height 10.00 ft from structure heel.

Input surface surcharges

No.	Surcharge new	Surcharge change	Action	Mag.1 [lbf/ft^2]	Mag.2 [lbf/ft^2]	Ord.x x [ft]	Length l [ft]	Depth z [ft]
1	Yes		permanent	2922.90		1.00	6.00	1.25
2	Yes		variable	770.00		1.00	6.00	1.25

No.	Name
1	DL
2	LL

Input concentrated surcharges

No.	Surcharge new	Surcharge change	Action	Magnitude [lbf]	Ord.x x [ft]	Length l [ft]	Width b[ft]	Depth z [ft]
3	Yes		variable	0.10	10.00	2.00	19.00	on terrain

No.	Name
3	LL

Resistance on front face of the structure

Resistance on front face of the structure: at rest
Soil on front face of the structure - 34 DEG
Soil thickness in front of structure h = 1.00 ft

Terrain in front of structure is flat.

Applied forces acting on the structure

No.	Force new	Force edit	Name	Action	Type	l [ft]	q_{x1} [lbf/ft^2]	q_{x2} [lbf/ft^2]
1	Yes		Earth Passive Pressure	permanent	trapezoid	7.50	0.0	1875.0

Settings of the stage of construction

Reduction of soil/soil friction angle : do not reduce
Design situation : Strength I

Verification No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-10.96	38684.5	12.85	1.000	1.000	1.350
Active pressure	5451.6	-7.98	3398.5	23.00	1.500	1.500	1.500
Water pressure	0.0	-20.75	0.0	46.00	1.000	1.000	1.000
DL	0.0	-11.75	17537.4	4.00	0.750	0.750	1.500
LL	0.0	-11.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-20.75	0.0	11.00	0.000	0.000	1.750
Earth Passive Pressure	-7031.2	-15.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability

Resisting moment M_{res} = 663888.9 lbfft/ft
Overturning moment M_{ovr} = 65248.5 lbfft/ft

Capacity demand ratio CDR = 10.17
Wall for overturning is SATISFACTORY

Check for slip

Resisting horizontal force H_{res} = 44482.75 lbf/ft
Active horizontal force H_{act} = 3607.03 lbf/ft

Capacity demand ratio CDR = 12.33
Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Verification No. 2

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-10.96	38684.5	12.85	1.000	1.000	1.350
Active pressure	5451.6	-7.98	3398.5	23.00	1.500	1.500	1.500
Water pressure	0.0	-20.75	0.0	46.00	1.000	1.000	1.000
DL	0.0	-11.75	17537.4	4.00	0.750	0.750	1.500
LL	0.0	-11.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-20.75	0.0	11.00	0.000	0.000	1.750
Earth Passive Pressure	-7031.2	-15.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability

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Active horizontal force H_{act} = 3607.03 lbf/ft

Capacity demand ratio CDR = 12.33

Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Bearing capacity of foundation soil

Design load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]	Eccentricity [–]	Stress [psf]
1	30770.3	91712.97	-2369.54	0.015	4107.4
2	-17649.6	56935.33	3607.03	0.000	2475.4
3	30770.3	91712.97	-2369.54	0.015	4107.4
4	-17649.6	56935.33	3607.03	0.000	2475.4

Service load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]
1	9530.3	64240.43	-1579.69
2	-25119.7	59620.43	-1579.69
3	9530.3	64240.43	-1579.69
4	-25119.7	59620.43	-1579.69

Verification of foundation soil

Stress in the footing bottom : rectangle

Eccentricity verification

Max. eccentricity of normal force e = 0.015

Maximum allowable eccentricity e_{alw} = 0.333

Eccentricity of the normal force is SATISFACTORY

Verification of bearing capacity

Ultimate bearing capacity of found. soil R = 15000.0 psf

Partial factor on bearing capacity γ_{Rv} = 0.65

Max. stress at footing bottom σ = 4107.4 psf

Allowable bearing capacity of foundation soil R_d = 9750.0 psf

Capacity demand ratio CDR = 2.4

Bearing capacity of foundation soil is SATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Verification of slip on georeinforcement No. 1

Forces acting on construction (verification of most utilized reinforcement)

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Design coefficient
Active pressure	4159.1	-7.09	3011.1	23.00	1.500
Earth Passive Pressure	-7031.2	-14.00	0.0	0.00	0.650
Weight - reinforced soil	0.0	-10.11	36698.1	12.92	1.000
DL	0.0	-10.25	17537.4	4.00	0.650
LL	0.0	-10.25	4620.0	4.00	0.000

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Design coefficient
LL	0.0	-19.25	0.0	11.00	0.000

Check for slip along geo-reinforcement with the maximal utilization (Reinforc. No.: 1)

Inclination of slip surface	=	90.00	°
Overall normal force acting on reinforcement	=	52614.05	lbf/ft
Coefficient of reduction of slip along geo-textile	=	0.60	
Resistance along geo-reinforcement	=	21293.18	lbf/ft
Wall resistance	=	0.00	lbf/ft
Overall bearing capacity of reinforcements	=	0.00	lbf/ft

Check for slip:

Resisting horizontal force H_{res} = 21293.18 lbf/ft

Active horiz. force H_{act} = 1668.38 lbf/ft

Capacity demand ratio CDR = 12.76

Slip along geotextile is SATISFACTORY

Calculation of internal stability No. 1

Calculated forces and strength of reinforcements

No.	Name	F_x [lbf/ft]	Depth z[ft]	R_t [lbf/ft]	Utiliz. [%]	T_p [lbf/ft]	Utiliz. [%]	R_{con} [lbf/ft]	Utiliz. [%]
1	6th Layer - 5 anchors (user-defined)	-4352.09	11.51	5841.16	74.51	37084.89	11.74	4568.50	95.26
2	5th Layer - 6 anchors (user-defined)	-4575.01	9.01	7009.39	65.27	30007.91	15.25	5580.44	81.98
3	4th Layer - 7 anchors (user-defined)	-4738.33	6.51	8177.62	57.94	23594.91	20.08	7090.25	66.83
4	1st Layer - 9 anchors (user-defined)	-3678.05	4.01	10514.08	34.98	19360.23	19.00	7796.33	47.18
5	2nd Layer - 3 anchors (user-defined)	-2483.33	2.76	3504.69	70.86	17246.59	14.40	2790.22	89.00
6	1st Layer - 9 anchors (user-defined)	-536.70	1.51	10514.08	5.10	15132.96	3.55	7796.33	6.88

Check for tensile strength (reinforcement No.1)

Tension strength R_t = 5841.16 lbf/ft

Force in reinforcement F_x = 4352.09 lbf/ft

Capacity demand ratio CDR = 1.34

Reinforcement for tensile strength is SATISFACTORY

Check for pull out resistance (reinforcement No.3)

Pull out resistance T_p = 23594.91 lbf/ft

Force in reinforcement F_x = 4738.33 lbf/ft

Capacity demand ratio CDR = 4.98

Reinforcement for pull out resistance is SATISFACTORY

Verification of connection strength (reinforcement No.1)

Connection strength R_{con} = 4568.50 lbf/ft

Force in reinforcement F_x = 4352.09 lbf/ft

Capacity demand ratio CDR = 1.05

Connection strength is SATISFACTORY

Overall verification - reinforcement is SATISFACTORY

Global stability analysis No. 1

Parameters of input slip surface

Center	S	=	(-5.87;-8.98)	ft
Radius	r	=	22.77	ft
Angle	α_1	=	0.00	°
	α_2	=	0.00	°

Analysis has not been performed.

Analysis of reinforced slopes

Input data

Date : 3/3/2025

Settings

(input for current task)

Materials and standards

Concrete structures : ACI 318-19

Wall analysis

Verification methodology : according to LRFD
Active earth pressure calculation : Coulomb
Passive earth pressure calculation : Mazindrani (Rankine)
Earthquake analysis : Mononobe-Okabe
Shape of earth wedge : Consider always vertical
Allowable eccentricity : 0.333
Internal stability : AASHTO - Inextensible (broken slip surface)

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Dead load of structural components :	DC =	0.90	[–]	1.25	[–]
Dead load of wearing surfaces :	DW =	0.65	[–]	1.50	[–]
Earth pressure - active :	EH_A =	0.90	[–]	1.50	[–]
Earth pressure - at rest :	EH_R =	0.90	[–]	1.35	[–]
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Vertical pressure of earth fill :	EV =	1.00	[–]	1.35	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]
Water load :	WA =	1.00	[–]	1.00	[–]

Resistance factors			
Design situation - Strength I			
Resistance factor on overturning :	ϕ_o =	0.90	[–]
Resistance factor on sliding :	ϕ_t =	1.00	[–]
Resistance factor on bearing capacity :	ϕ_b =	0.65	[–]
Resistance factor on passive pressure :	ϕ_{VE} =	0.75	[–]

Stability analysis

Verification methodology : according to LRFD

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]

Resistance factors			
Design situation - Strength I			
Resistance factor on stability :	ϕ_{SS} =	0.65	[–]

Geometry of structure

Embankment height h_n = 13.00 ft
Embankment length l_n = 0.00 ft
Cover thickness t_c = 0.46 ft

Material

Cover material
Unit weight γ = 150.00 pcf
Shear resistance R_s = 0.0 psf

Types of reinforcements

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient	
				T_{ult}[lbf/ft]	R_t[lbf/ft]	C_{ds}[–]	C_i[–]
1	1st Layer - 9 anchors (user-defined)	user-defined	–··–··–··	16009.42	9936.88	0.60	0.70
2	2nd Layer - 3 anchors (user-defined)	user-defined	–··–··–··	5336.48	3312.30	0.60	0.70
3	4th Layer - 7 anchors (user-defined)	user-defined	–··–··–··	12451.77	7728.69	0.60	0.70
4	5th Layer - 6 anchors (user-defined)	user-defined	–··–··–··	10672.95	6624.59	0.60	0.70
5	6th Layer - 5 anchors (user-defined)	user-defined	–··–··–··	8894.12	5520.49	0.60	0.70

Reinforcement details

1. 1st Layer - 9 anchors (user-defined)
Short-term char. strength T_{ult} = 16009.42 lbf/ft
Long-term design strength R_t = 9936.88 lbf/ft

Design connection strength R_{con} = 10395.11 lbf/ft
2. 2nd Layer - 3 anchors (user-defined)
Short-term char. strength T_{ult} = 5336.48 lbf/ft
Long-term design strength R_t = 3312.30 lbf/ft

Design connection strength R_{con} = 3720.29 lbf/ft
3. 4th Layer - 7 anchors (user-defined)
Short-term char. strength T_{ult} = 12451.77 lbf/ft
Long-term design strength R_t = 7728.69 lbf/ft

Design connection strength R_{con} = 9453.67 lbf/ft
4. 5th Layer - 6 anchors (user-defined)
Short-term char. strength T_{ult} = 10672.95 lbf/ft
Long-term design strength R_t = 6624.59 lbf/ft

Design connection strength R_{con} = 7440.58 lbf/ft
5. 6th Layer - 5 anchors (user-defined)
Short-term char. strength T_{ult} = 8894.12 lbf/ft
Long-term design strength R_t = 5520.49 lbf/ft

Design connection strength R_{con} = 6091.34 lbf/ft

Reinforcement

No.	Number of reinforcements	Type of reinforcement	Spacing of reinforcements h_r[ft]	Height of first reinforcement y[ft]	Reinforcements geometry
1	1	6th Layer - 5 anchors (user-defined)	1.00	1.50	identical length of reinforcements
2	1	5th Layer - 6 anchors (user-defined)	1.00	4.00	identical length of reinforcements
3	1	4th Layer - 7 anchors (user-defined)	1.00	6.50	identical length of reinforcements
4	1	1st Layer - 9 anchors (user-defined)	1.00	9.00	identical length of reinforcements
5	1	1st Layer - 9 anchors (user-defined)	1.00	11.50	identical length of reinforcements
6	1	2nd Layer - 3 anchors (user-defined)	1.00	10.25	identical length of reinforcements

Reinforcement details

Reinforcement No. 1
Reinforcement type : 6th Layer - 5 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	1.50	23.00

Reinforcement No. 2
Reinforcement type : 5th Layer - 6 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	4.00	23.00

Reinforcement No. 3
Reinforcement type : 4th Layer - 7 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	6.50	23.00

Reinforcement No. 4
Reinforcement type : 1st Layer - 9 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	9.00	23.00

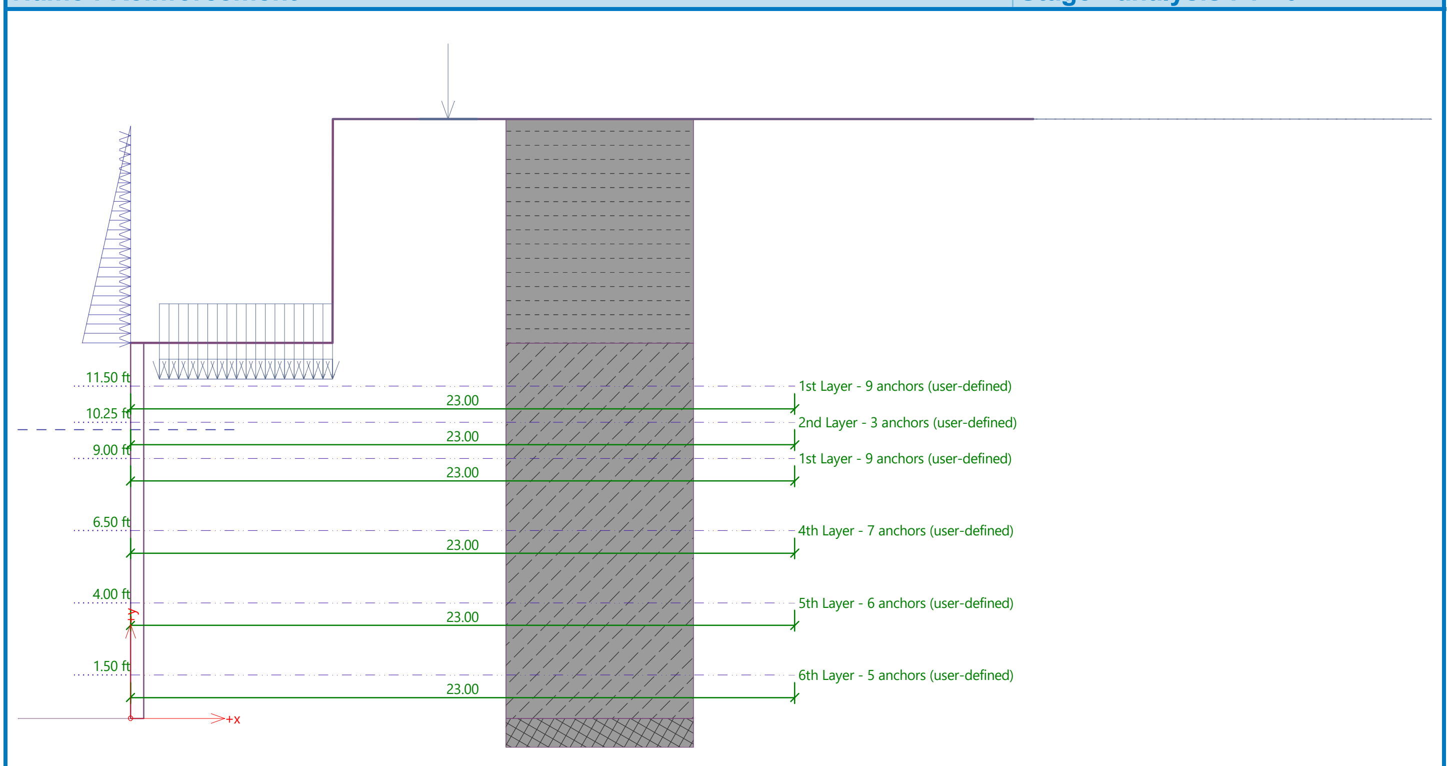
Reinforcement No. 5
Reinforcement type : 1st Layer - 9 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	11.50	23.00

Reinforcement No. 6
Reinforcement type : 2nd Layer - 3 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	10.25	23.00

Name : Reinforcement **Stage - analysis : 1 - 0**



Soil parameters

34 DEG

Basic data

Unit weight : γ = 125.0 [pcf]
Internal friction angle : φ_{ef} = 34.00 [°]
Cohesion : c_{ef} = 0.0 [psf]
Friction angle structure-soil : δ = 23.00 [°]

Uplift pressure

Uplift calculation : standard
Unit weight of saturated soil : γ_{sat} = 130.0 [pcf]

View

Soil pattern :

38 DEG

Basic data

Unit weight : γ = 140.0 [pcf]
Internal friction angle : φ_{ef} = 38.00 [°]
Cohesion : c_{ef} = 0.0 [psf]
Friction angle structure-soil : δ = 20.00 [°]

Uplift pressure

Uplift calculation : standard
Unit weight of saturated soil : γ_{sat} = 140.0 [pcf]

View

Soil pattern :

24 DEG

Basic data

Unit weight : γ = 120.0 [pcf]
Internal friction angle : φ_{ef} = 24.00 [°]
Cohesion : c_{ef} = 0.0 [psf]
Friction angle structure-soil : δ = 24.00 [°]

Uplift pressure

Uplift calculation : standard
Unit weight of saturated soil : γ_{sat} = 130.0 [pcf]

View

Soil pattern :

Geological profile and assigned soils

Position information

Terrain elevation = -9.75 ft

Geological profile and assigned soils

No.	Thickness of layer t [ft]	Depth z [ft]	Elevation [ft]	Assigned soil	Pattern
1	0.01	0.00 .. 0.01	-9.75 .. -9.76	24 DEG	
2	12.99	0.01 .. 13.00	-9.76 .. -22.75	34 DEG	

No.	Thickness of layer t [ft]	Depth z [ft]	Elevation [ft]	Assigned soil	Pattern
3	7.00	13.00 .. 20.00	-22.75 .. -29.75	38 DEG	
4	-	20.00 .. ∞	-29.75 .. -	38 DEG	

Terrain profile

No.	Coordinates x [ft]	Depth z [ft]
1	0.00	0.00
2	7.00	0.00
3	7.00	-7.75
4	8.00	-7.75

Origin [0,0] is located in upper right edge of construction.
Positive coordinate +z has downward direction.

Water influence

Ground water table at height 10.00 ft from structure heel.

Input surface surcharges

No.	Surcharge new	Surcharge change	Action	Mag.1 [lbf/ft2]	Mag.2 [lbf/ft2]	Ord.x x [ft]	Length l [ft]	Depth z [ft]
1	Yes		permanent	2922.90		1.00	6.00	1.25
2	Yes		variable	770.00		1.00	6.00	1.25

No.	Name
1	DL
2	LL

Input horizontal surcharges

No.	Surcharge new	Surcharge change	Action	Mag.1 [lbf/ft2]	Ord.x x [ft]	Length l [ft]	Depth z [ft]
4	Yes		variable	60.00	7.00	38.00	on terrain

No.	Name
4	Braking Force

Input concentrated surcharges

No.	Surcharge new	Surcharge change	Action	Magnitude [lbf]	Ord.x x [ft]	Length l [ft]	Width b[ft]	Depth z [ft]
3	Yes		variable	0.10	10.00	2.00	19.00	on terrain

No.	Name
3	LL

Resistance on front face of the structure

Resistance on front face of the structure: at rest
Soil on front face of the structure - 34 DEG
Soil thickness in front of structure h = 1.00 ft

Terrain in front of structure is flat.

Applied forces acting on the structure

No.	Force new	Force edit	Name	Action	Type	l [ft]	q_{x1} [lbf/ft²]	q_{x2} [lbf/ft²]
1	Yes		Earth Passive Pressure	permanent	trapezoid	7.50	0.0	1875.0

Settings of the stage of construction

Reduction of soil/soil friction angle : do not reduce
Design situation : Strength I

Verification No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-10.96	38684.5	12.85	1.000	1.000	1.350
Active pressure	5451.6	-7.98	3398.5	23.00	1.500	1.500	1.500
Water pressure	0.0	-20.75	0.0	46.00	1.000	1.000	1.000
Braking Force	890.4	-10.93	0.0	23.00	1.750	1.750	1.750
DL	0.0	-11.75	17537.4	4.00	0.750	0.750	1.500
LL	0.0	-11.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-20.75	0.0	11.00	0.000	0.000	1.750
Earth Passive Pressure	-7031.2	-15.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 663888.9 lbfft/ft
Overturning moment M_{ovr} = 82278.0 lbfft/ft

Capacity demand ratio CDR = 8.07
Wall for overturning is SATISFACTORY

Check for slip
Resisting horizontal force H_{res} = 44482.75 lbf/ft
Active horizontal force H_{act} = 5165.20 lbf/ft

Capacity demand ratio CDR = 8.61
Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Verification No. 2

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-10.96	38684.5	12.85	1.000	1.000	1.350
Active pressure	5451.6	-7.98	3398.5	23.00	1.500	1.500	1.500
Water pressure	0.0	-20.75	0.0	46.00	1.000	1.000	1.000
Braking Force	890.4	-10.93	0.0	23.00	1.750	1.750	1.750
DL	0.0	-11.75	17537.4	4.00	0.750	0.750	1.500
LL	0.0	-11.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-20.75	0.0	11.00	0.000	0.000	1.750

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Earth Passive Pressure	-7031.2	-15.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability

Resisting moment M_{res} = 663888.9 lbfft/ft
Overturning moment M_{ovr} = 82278.0 lbfft/ft

Capacity demand ratio CDR = 8.07
Wall for overturning is SATISFACTORY

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Resisting horizontal force H_{res} = 44482.75 lbf/ft
Active horizontal force H_{act} = 5165.20 lbf/ft

Capacity demand ratio CDR = 8.61
Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Bearing capacity of foundation soil

Design load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]	Eccentricity [–]	Stress [psf]
1	47799.9	91712.97	-811.37	0.023	4176.8
2	-620.1	56935.33	5165.20	0.000	2475.4
3	47799.9	91712.97	-811.37	0.023	4176.8
4	-620.1	56935.33	5165.20	0.000	2475.4

Service load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]
1	19261.5	64240.43	-689.31
2	-15388.5	59620.43	-689.31
3	19261.5	64240.43	-689.31
4	-15388.5	59620.43	-689.31

Verification of foundation soil

Stress in the footing bottom : rectangle

Eccentricity verification

Max. eccentricity of normal force e = 0.023
Maximum allowable eccentricity e_{alw} = 0.333

Eccentricity of the normal force is SATISFACTORY

Verification of bearing capacity

Ultimate bearing capacity of found. soil R = 15000.0 psf
Partial factor on bearing capacity γ_{Rv} = 0.65
Max. stress at footing bottom σ = 4176.8 psf
Allowable bearing capacity of foundation soil R_d = 9750.0 psf
Capacity demand ratio CDR = 2.3

Bearing capacity of foundation soil is SATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Verification of slip on georeinforcement No. 1

Forces acting on construction (verification of most utilized reinforcement)

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Design coefficient
Active pressure	4159.1	-7.09	3011.1	23.00	1.500
Braking Force	732.6	-9.81	0.0	23.00	1.750
Earth Passive Pressure	-7031.2	-14.00	0.0	0.00	0.650
Weight - reinforced soil	0.0	-10.11	36698.1	12.92	1.000
DL	0.0	-10.25	17537.4	4.00	0.650
LL	0.0	-10.25	4620.0	4.00	0.000
LL	0.0	-19.25	0.0	11.00	0.000

Check for slip along geo-reinforcement with the maximal utilization (Reinforc. No.: 1)

Inclination of slip surface = 90.00 °
Overall normal force acting on reinforcement = 52614.05 lbf/ft
Coefficient of reduction of slip along geo-textile = 0.60
Resistance along geo-reinforcement = 21293.18 lbf/ft
Wall resistance = 0.00 lbf/ft
Overall bearing capacity of reinforcements = 0.00 lbf/ft

Check for slip:
Resisting horizontal force H_{res} = 21293.18 lbf/ft
Active horiz. force H_{act} = 2950.48 lbf/ft

Capacity demand ratio CDR = 7.22
Slip along geotextile is SATISFACTORY

Calculation of internal stability No. 1

Calculated forces and strength of reinforcements

No.	Name	F_x [lbf/ft]	Depth z[ft]	R_t [lbf/ft]	Utiliz. [%]	T_p [lbf/ft]	Utiliz. [%]	R_{con} [lbf/ft]	Utiliz. [%]
1	6th Layer - 5 anchors (user-defined)	-4552.03	11.51	4140.37	109.94	37084.89	12.27	4568.50	99.64
2	5th Layer - 6 anchors (user-defined)	-4770.00	9.01	4968.44	96.01	30007.91	15.90	5580.44	85.48
3	4th Layer - 7 anchors (user-defined)	-4946.01	6.51	5796.52	85.33	23594.91	20.96	7090.25	69.76
4	1st Layer - 9 anchors (user-defined)	-3725.83	4.01	7452.66	49.99	19360.23	19.24	7796.33	47.79
5	2nd Layer - 3 anchors (user-defined)	-2483.33	2.76	2484.23	99.96	17246.59	14.40	2790.22	89.00
6	1st Layer - 9 anchors (user-defined)	-536.70	1.51	7452.66	7.20	15132.96	3.55	7796.33	6.88

Check for tensile strength (reinforcement No.1)
Tension strength R_t = 4140.37 lbf/ft
Force in reinforcement F_x = 4552.03 lbf/ft

Capacity demand ratio CDR = 0.91
Reinforcement for tensile strength is NOT SATISFACTORY

Check for pull out resistance (reinforcement No.3)
Pull out resistance T_p = 23594.91 lbf/ft
Force in reinforcement F_x = 4946.01 lbf/ft

Capacity demand ratio CDR = 4.77

Reinforcement for pull out resistance is SATISFACTORY

Verification of connection strength (reinforcement No.1)

Connection strength	R_{con}	=	4568.50	lbf/ft
Force in reinforcement	F_x	=	4552.03	lbf/ft

Capacity demand ratio CDR = 1.00

Connection strength is SATISFACTORY

Overall verification - reinforcement is NOT SATISFACTORY

Global stability analysis No. 1

Parameters of input slip surface

Center	S	=	(-5.87;-8.98)	ft
Radius	r	=	22.77	ft
Angle	α_1	=	0.00	°
	α_2	=	0.00	°

Analysis has not been performed.

Analysis of reinforced slopes

Input data

Date : 3/3/2025

Settings

(input for current task)

Materials and standards

Concrete structures : ACI 318-19

Wall analysis

Verification methodology :	according to LRFD
Active earth pressure calculation :	Coulomb
Passive earth pressure calculation :	Mazindrani (Rankine)
Earthquake analysis :	Mononobe-Okabe
Shape of earth wedge :	Consider always vertical
Allowable eccentricity :	0.333
Internal stability :	AASHTO - Inextensible (broken slip surface)

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Dead load of structural components :	DC =	0.90	[–]	1.25	[–]
Dead load of wearing surfaces :	DW =	0.65	[–]	1.50	[–]
Earth pressure - active :	EH_A =	0.90	[–]	1.50	[–]
Earth pressure - at rest :	EH_R =	0.90	[–]	1.35	[–]
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Vertical pressure of earth fill :	EV =	1.00	[–]	1.35	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]
Water load :	WA =	1.00	[–]	1.00	[–]

Resistance factors			
Design situation - Strength I			
Resistance factor on overturning :	ϕ_o =	0.90	[–]
Resistance factor on sliding :	ϕ_t =	1.00	[–]
Resistance factor on bearing capacity :	ϕ_b =	0.65	[–]
Resistance factor on passive pressure :	ϕ_{VE} =	0.75	[–]

Stability analysis

Verification methodology : according to LRFD

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]

Resistance factors			
Design situation - Strength I			
Resistance factor on stability :	ϕ_{SS} =	0.65	[–]

Geometry of structure

Embankment height h_n = 15.00 ft
Embankment length l_n = 0.00 ft
Cover thickness t_c = 0.46 ft

Material

Cover material
Unit weight γ = 150.00 pcf
Shear resistance R_s = 0.0 psf

Types of reinforcements

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient	
				T_{ult}[lbf/ft]	R_t[lbf/ft]	C_{ds}[–]	C_i[–]
1	1st Layer - 9 anchors (user-defined)	user-defined		22585.80	14018.77	0.60	0.70
2	2nd Layer - 3 anchors (user-defined)	user-defined		7528.60	4672.92	0.60	0.70
3	4th Layer - 7 anchors (user-defined)	user-defined		17566.74	10903.49	0.60	0.70
4	5th Layer - 6 anchors (user-defined)	user-defined		15057.20	9345.85	0.60	0.70
5	6th Layer - 5 anchors (user-defined)	user-defined		12547.67	7788.21	0.60	0.70
6	3rd Layer - 8 anchors (user-defined)	user-defined		20076.27	12461.13	0.60	0.70
7	7th Layer - 4 anchors (user-defined)	user-defined		10038.13	6230.57	0.60	0.70

Reinforcement details

1. 1st Layer - 9 anchors (user-defined)
Short-term char. strength T_{ult} = 22585.80 lbf/ft
Long-term design strength R_t = 14018.77 lbf/ft

Design connection strength R_{con} = 10395.11 lbf/ft
2. 2nd Layer - 3 anchors (user-defined)
Short-term char. strength T_{ult} = 7528.60 lbf/ft
Long-term design strength R_t = 4672.92 lbf/ft

Design connection strength R_{con} = 3720.29 lbf/ft
3. 4th Layer - 7 anchors (user-defined)
Short-term char. strength T_{ult} = 17566.74 lbf/ft
Long-term design strength R_t = 10903.49 lbf/ft

Design connection strength R_{con} = 9453.67 lbf/ft
4. 5th Layer - 6 anchors (user-defined)
Short-term char. strength T_{ult} = 15057.20 lbf/ft
Long-term design strength R_t = 9345.85 lbf/ft

Design connection strength R_{con} = 7440.58 lbf/ft

5. 6th Layer - 5 anchors (user-defined)

Short-term char. strength T_{ult} = 12547.67 lbf/ft
Long-term design strength R_t = 7788.21 lbf/ft

Design connection strength R_{con} = 6091.34 lbf/ft

6. 3rd Layer - 8 anchors (user-defined)

Short-term char. strength T_{ult} = 20076.27 lbf/ft
Long-term design strength R_t = 12461.13 lbf/ft

Design connection strength R_{con} = 11466.77 lbf/ft

7. 7th Layer - 4 anchors (user-defined)

Short-term char. strength T_{ult} = 10038.13 lbf/ft
Long-term design strength R_t = 6230.57 lbf/ft

Design connection strength R_{con} = 4742.10 lbf/ft

Reinforcement

No.	Number of reinforcements	Type of reinforcement	Spacing of reinforcements h_r[ft]	Height of first reinforcement y[ft]	Reinforcements geometry
1	1	6th Layer - 5 anchors (user-defined)	1.00	3.50	identical length of reinforcements
2	1	5th Layer - 6 anchors (user-defined)	1.00	6.00	identical length of reinforcements
3	1	4th Layer - 7 anchors (user-defined)	1.00	8.50	identical length of reinforcements
4	1	3rd Layer - 8 anchors (user-defined)	1.00	11.00	identical length of reinforcements
5	1	1st Layer - 9 anchors (user-defined)	1.00	13.50	identical length of reinforcements
6	1	2nd Layer - 3 anchors (user-defined)	1.00	12.25	identical length of reinforcements
7	1	7th Layer - 4 anchors (user-defined)	1.00	1.50	identical length of reinforcements

Reinforcement details

Reinforcement No. 1

Reinforcement type : 6th Layer - 5 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	3.50	23.00

Reinforcement No. 2

Reinforcement type : 5th Layer - 6 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	6.00	23.00

Reinforcement No. 3
Reinforcement type : 4th Layer - 7 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	8.50	23.00

Reinforcement No. 4
Reinforcement type : 3rd Layer - 8 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	11.00	23.00

Reinforcement No. 5
Reinforcement type : 1st Layer - 9 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	13.50	23.00

Reinforcement No. 6
Reinforcement type : 2nd Layer - 3 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

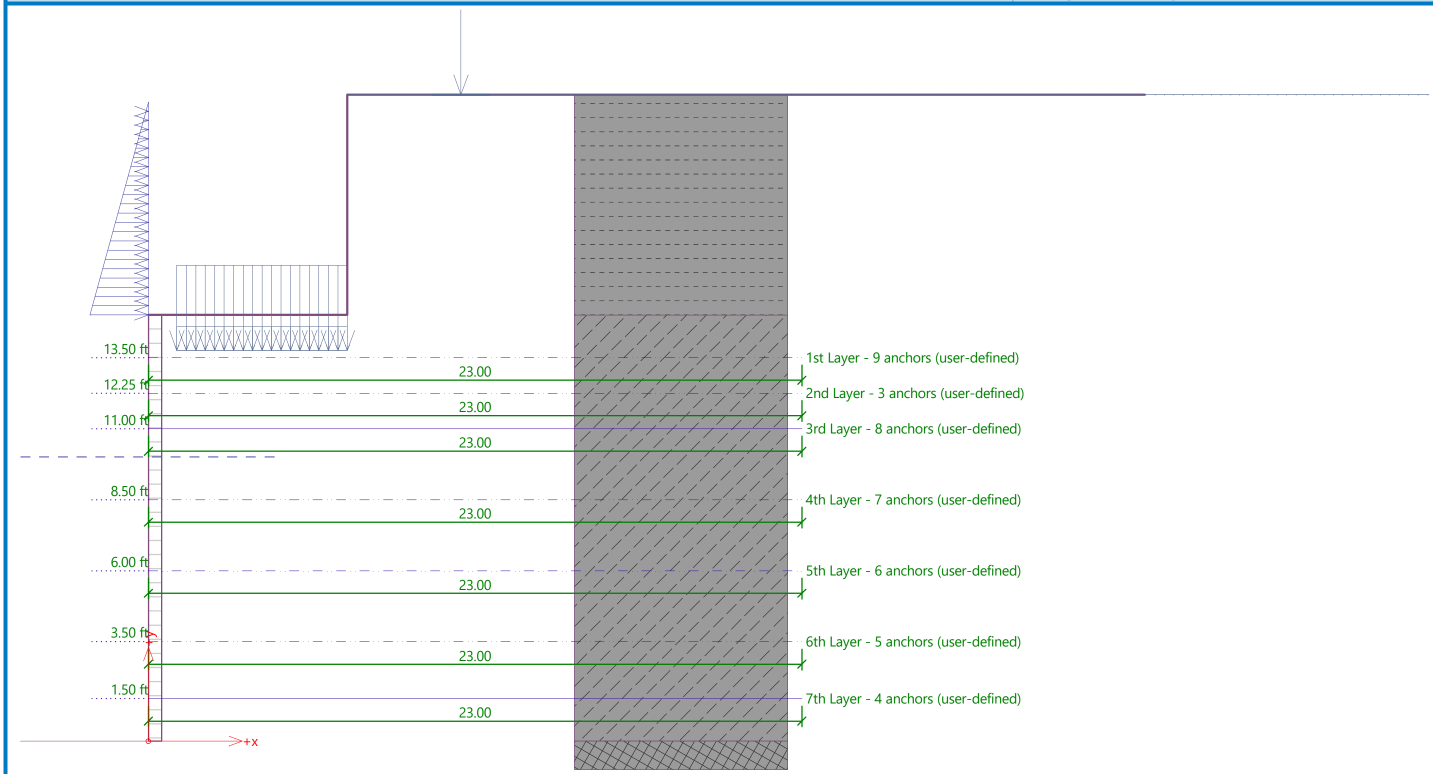
No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	12.25	23.00

Reinforcement No. 7
Reinforcement type : 7th Layer - 4 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	1.50	23.00

Name : Reinforcement

Stage - analysis : 1 - 0



Soil parameters

34 DEG

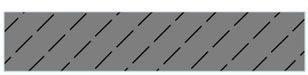
Basic data

Unit weight :	γ = 125.0 [pcf]
Internal friction angle :	φ_{ef} = 34.00 [°]
Cohesion :	c_{ef} = 0.0 [psf]
Friction angle structure-soil :	δ = 23.00 [°]

Uplift pressure

Uplift calculation :	standard
Unit weight of saturated soil :	γ_{sat} = 130.0 [pcf]

View

Soil pattern : 

38 DEG

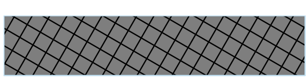
Basic data

Unit weight :	γ = 140.0 [pcf]
Internal friction angle :	φ_{ef} = 38.00 [°]
Cohesion :	c_{ef} = 0.0 [psf]
Friction angle structure-soil :	δ = 20.00 [°]

Uplift pressure

Uplift calculation :	standard
Unit weight of saturated soil :	γ_{sat} = 140.0 [pcf]

View

Soil pattern : 

24 DEG

Basic data

Unit weight : γ = 120.0 [pcf]

Internal friction angle : φ_{ef} = 24.00 [°]

Cohesion : c_{ef} = 0.0 [psf]

Friction angle structure-soil : δ = 24.00 [°]

Uplift pressure

Uplift calculation : standard

Unit weight of saturated soil : γ_{sat} = 130.0 [pcf]

View

Soil pattern :

Geological profile and assigned soils

Position information

Terrain elevation = -9.75 ft

Geological profile and assigned soils

No.	Thickness of layer t [ft]	Depth z [ft]	Elevation [ft]	Assigned soil	Pattern
1	0.01	0.00 .. 0.01	-9.75 .. -9.76	24 DEG	
2	14.99	0.01 .. 15.00	-9.76 .. -24.75	34 DEG	
3	5.00	15.00 .. 20.00	-24.75 .. -29.75	38 DEG	
4	-	20.00 .. ∞	-29.75 .. -	38 DEG	

Terrain profile

No.	Coordinates x [ft]	Depth z [ft]
1	0.00	0.00
2	7.00	0.00
3	7.00	-7.75
4	8.00	-7.75

Origin [0,0] is located in upper right edge of construction.
Positive coordinate +z has downward direction.

Water influence

Ground water table at height 10.00 ft from structure heel.

Input surface surcharges

No.	Surcharge		Action	Mag.1	Mag.2	Ord.x	Length	Depth
	new	change		[lbf/ft2]	[lbf/ft2]	x [ft]	l [ft]	z [ft]
1	Yes		permanent	2739.20		1.00	6.00	1.25
2	Yes		variable	770.00		1.00	6.00	1.25

No.	Name
1	DL
2	LL

Input horizontal surcharges

No.	Surcharge new	Surcharge change	Action	Mag.1 [lbf/ft²]	Ord.x x [ft]	Length l [ft]	Depth z [ft]
4	Yes		variable	60.00	7.00	38.00	on terrain

No.	Name
4	Braking Force

Input concentrated surcharges

No.	Surcharge new	Surcharge change	Action	Magnitude [lbf]	Ord.x x [ft]	Length l [ft]	Width b[ft]	Depth z [ft]
3	Yes		variable	0.10	10.00	2.00	19.00	on terrain

No.	Name
3	LL

Resistance on front face of the structure

Resistance on front face of the structure: at rest
Soil on front face of the structure - 34 DEG
Soil thickness in front of structure h = 1.00 ft

Terrain in front of structure is flat.

Applied forces acting on the structure

No.	Force new	Force edit	Name	Action	Type	l [ft]	q_{x1} [lbf/ft²]	q_{x2} [lbf/ft²]
1	Yes		Earth Passive Pressure	permanent	trapezoid	7.50	0.0	1875.0

Settings of the stage of construction

Reduction of soil/soil friction angle : do not reduce
Design situation : Strength I

Verification No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-12.01	44204.5	12.68	1.000	1.000	1.350
Active pressure	6600.2	-8.60	4173.3	23.00	1.500	1.500	1.500
Water pressure	0.0	-22.75	0.0	46.00	1.000	1.000	1.000
Braking Force	968.8	-11.96	0.0	23.00	1.750	1.750	1.750
DL	0.0	-13.75	16435.2	4.00	0.750	0.750	1.500
LL	0.0	-13.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-22.75	0.0	11.00	0.000	0.000	1.750
Earth Passive Pressure	-7031.2	-17.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 750328.6 lbfft/ft
Overturning moment M_{ovr} = 105430.8 lbfft/ft

Capacity demand ratio CDR = 7.12
Wall for overturning is SATISFACTORY

Check for slip

Resisting horizontal force H_{res} = 49057.59 lbf/ft
Active horizontal force H_{act} = 7025.35 lbf/ft

Capacity demand ratio CDR = 6.98
Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Verification No. 2

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-12.01	44204.5	12.68	1.000	1.000	1.350
Active pressure	6600.2	-8.60	4173.3	23.00	1.500	1.500	1.500
Water pressure	0.0	-22.75	0.0	46.00	1.000	1.000	1.000
Braking Force	968.8	-11.96	0.0	23.00	1.750	1.750	1.750
DL	0.0	-13.75	16435.2	4.00	0.750	0.750	1.500
LL	0.0	-13.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-22.75	0.0	11.00	0.000	0.000	1.750
Earth Passive Pressure	-7031.2	-17.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability

Resisting moment M_{res} = 750328.6 lbfft/ft
Overturning moment M_{ovr} = 105430.8 lbfft/ft

Capacity demand ratio CDR = 7.12
Wall for overturning is SATISFACTORY

Check for slip

Resisting horizontal force H_{res} = 49057.59 lbf/ft
Active horizontal force H_{act} = 7025.35 lbf/ft

Capacity demand ratio CDR = 6.98
Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Bearing capacity of foundation soil

Design load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]	Eccentricity [–]	Stress [psf]
1	24094.1	98673.85	1048.79	0.011	4383.2
2	-6172.8	62790.85	7025.35	0.000	2730.0
3	24094.1	98673.85	1048.79	0.011	4383.2
4	-6172.8	62790.85	7025.35	0.000	2730.0

Service load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]
1	3147.8	69433.02	537.73

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]
2	-31502.2	64813.01	537.73
3	3147.8	69433.02	537.73
4	-31502.2	64813.01	537.73

Verification of foundation soil

Stress in the footing bottom : rectangle

Eccentricity verification
Max. eccentricity of normal force e = 0.011
Maximum allowable eccentricity e_{alw} = 0.333

Eccentricity of the normal force is SATISFACTORY

Verification of bearing capacity
Ultimate bearing capacity of found. soil R = 15000.0 psf
Partial factor on bearing capacity γ_{Rv} = 0.65
Max. stress at footing bottom σ = 4383.2 psf
Allowable bearing capacity of foundation soil R_d = 9750.0 psf
Capacity demand ratio CDR = 2.2

Bearing capacity of foundation soil is SATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Verification of slip on georeinforcement No. 1

Forces acting on construction (verification of most utilized reinforcement)

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Design coefficient
Active pressure	5186.4	-7.77	3733.6	23.00	1.500
Braking Force	811.1	-10.82	0.0	23.00	1.750
Earth Passive Pressure	-7031.2	-16.00	0.0	0.00	0.650
Weight - reinforced soil	0.0	-11.14	42448.1	12.73	1.000
DL	0.0	-12.25	16435.2	4.00	0.650
LL	0.0	-12.25	4620.0	4.00	0.000
LL	0.0	-21.25	0.0	11.00	0.000

Check for slip along geo-reinforcement with the maximal utilization (Reinforc. No.: 1)

Inclination of slip surface = 90.00 °
Overall normal force acting on reinforcement = 58731.37 lbf/ft
Coefficient of reduction of slip along geo-textile = 0.60
Resistance along geo-reinforcement = 23768.88 lbf/ft
Wall resistance = 0.00 lbf/ft
Overall bearing capacity of reinforcements = 0.00 lbf/ft

Check for slip:
Resisting horizontal force H_{res} = 23768.88 lbf/ft
Active horiz. force H_{act} = 4628.61 lbf/ft

Capacity demand ratio CDR = 5.14
Slip along geotextile is SATISFACTORY

Calculation of internal stability No. 1

Calculated forces and strength of reinforcements

No.	Name	F_x [lbf/ft]	Depth z[ft]	R_t [lbf/ft]	Utiliz. [%]	T_p [lbf/ft]	Utiliz. [%]	R_{con} [lbf/ft]	Utiliz. [%]
1	7th Layer - 4 anchors (user-defined)	-1842.26	13.51	4672.93	39.42	40998.94	4.49	3556.58	51.80
2	6th Layer - 5 anchors (user-defined)	-3993.02	11.51	5841.16	68.36	35071.77	11.39	4568.50	87.40
3	5th Layer - 6 anchors (user-defined)	-4605.81	9.01	7009.39	65.71	28260.39	16.30	5580.44	82.54
4	4th Layer - 7 anchors (user-defined)	-4760.71	6.51	8177.62	58.22	22846.53	20.84	7090.25	67.14
5	3rd Layer - 8 anchors (user-defined)	-3571.27	4.01	9345.85	38.21	18752.05	19.04	8600.08	41.53
6	2nd Layer - 3 anchors (user-defined)	-2373.66	2.76	3504.69	67.73	16704.81	14.21	2790.22	85.07
7	1st Layer - 9 anchors (user-defined)	-517.46	1.51	10514.08	4.92	14657.57	3.53	7796.33	6.64

Check for tensile strength (reinforcement No.2)

Tension strength R_t = 5841.16 lbf/ft
Force in reinforcement F_x = 3993.02 lbf/ft

Capacity demand ratio CDR = 1.46
Reinforcement for tensile strength is SATISFACTORY

Check for pull out resistance (reinforcement No.4)

Pull out resistance T_p = 22846.53 lbf/ft
Force in reinforcement F_x = 4760.71 lbf/ft

Capacity demand ratio CDR = 4.80
Reinforcement for pull out resistance is SATISFACTORY

Verification of connection strength (reinforcement No.2)

Connection strength R_{con} = 4568.50 lbf/ft
Force in reinforcement F_x = 3993.02 lbf/ft

Capacity demand ratio CDR = 1.14
Connection strength is SATISFACTORY

Overall verification - reinforcement is SATISFACTORY

Global stability analysis No. 1

Parameters of input slip surface

Center S = (-6.16;-8.02) ft
Radius r = 23.85 ft
Angle α_1 = 0.00 °
α_2 = 0.00 °

Analysis has not been performed.

Analysis of reinforced slopes

Input data

Date : 3/3/2025

Settings

(input for current task)

Materials and standards

Concrete structures : ACI 318-19

Wall analysis

Verification methodology : according to LRFD
Active earth pressure calculation : Coulomb
Passive earth pressure calculation : Mazindrani (Rankine)
Earthquake analysis : Mononobe-Okabe
Shape of earth wedge : Consider always vertical
Allowable eccentricity : 0.333
Internal stability : AASHTO - Inextensible (broken slip surface)

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Dead load of structural components :	DC =	0.90	[–]	1.25	[–]
Dead load of wearing surfaces :	DW =	0.65	[–]	1.50	[–]
Earth pressure - active :	EH_A =	0.90	[–]	1.50	[–]
Earth pressure - at rest :	EH_R =	0.90	[–]	1.35	[–]
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Vertical pressure of earth fill :	EV =	1.00	[–]	1.35	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]
Water load :	WA =	1.00	[–]	1.00	[–]

Resistance factors			
Design situation - Strength I			
Resistance factor on overturning :	ϕ_o =	0.90	[–]
Resistance factor on sliding :	ϕ_t =	1.00	[–]
Resistance factor on bearing capacity :	ϕ_b =	0.65	[–]
Resistance factor on passive pressure :	ϕ_{VE} =	0.75	[–]

Stability analysis

Verification methodology : according to LRFD

Load factors					
Design situation - Strength I					
		Minimum		Maximum	
Earth surcharge load (permanent) :	ES =	0.75	[–]	1.50	[–]
Live load surcharge :	LL =	0.00	[–]	1.75	[–]

Resistance factors		
Design situation - Strength I		
Resistance factor on stability :	ϕ_{SS} =	0.65 [–]

Geometry of structure

Embankment height h_n = 15.00 ft
Embankment length l_n = 0.00 ft
Cover thickness t_c = 0.46 ft

Material

Cover material
Unit weight γ = 150.00 pcf
Shear resistance R_s = 0.0 psf

Types of reinforcements

No.	Name	Type of reinforcement	Line type	Reinforcement strength		Coefficient	
				T_{ult}[lbf/ft]	R_t[lbf/ft]	C_{ds}[–]	C_i[–]
1	1st Layer - 9 anchors (user-defined)	user-defined	–··–··–··–	16009.42	9936.88	0.60	0.70
2	2nd Layer - 3 anchors (user-defined)	user-defined	–··–··–··–	5336.48	3312.30	0.60	0.70
3	4th Layer - 7 anchors (user-defined)	user-defined	–··–··–··–	12451.77	7728.69	0.60	0.70
4	5th Layer - 6 anchors (user-defined)	user-defined	–··–··–··–	10672.95	6624.59	0.60	0.70
5	6th Layer - 5 anchors (user-defined)	user-defined	–··–··–··–	8894.12	5520.49	0.60	0.70
6	3rd Layer - 8 anchors (user-defined)	user-defined	———	14230.60	8832.79	0.60	0.70
7	7th Layer - 4 anchors (user-defined)	user-defined	———	7115.30	4416.39	0.60	0.70

Reinforcement details

1. 1st Layer - 9 anchors (user-defined)
Short-term char. strength T_{ult} = 16009.42 lbf/ft
Long-term design strength R_t = 9936.88 lbf/ft

Design connection strength R_{con} = 10395.11 lbf/ft

2. 2nd Layer - 3 anchors (user-defined)
Short-term char. strength T_{ult} = 5336.48 lbf/ft
Long-term design strength R_t = 3312.30 lbf/ft

Design connection strength R_{con} = 3720.29 lbf/ft

3. 4th Layer - 7 anchors (user-defined)
Short-term char. strength T_{ult} = 12451.77 lbf/ft
Long-term design strength R_t = 7728.69 lbf/ft

Design connection strength R_{con} = 9453.67 lbf/ft

4. 5th Layer - 6 anchors (user-defined)
Short-term char. strength T_{ult} = 10672.95 lbf/ft
Long-term design strength R_t = 6624.59 lbf/ft

Design connection strength R_{con} = 7440.58 lbf/ft

5. 6th Layer - 5 anchors (user-defined)

Short-term char. strength T_{ult} = 8894.12 lbf/ft
Long-term design strength R_t = 5520.49 lbf/ft

Design connection strength R_{con} = 6091.34 lbf/ft

6. 3rd Layer - 8 anchors (user-defined)

Short-term char. strength T_{ult} = 14230.60 lbf/ft
Long-term design strength R_t = 8832.79 lbf/ft

Design connection strength R_{con} = 11466.77 lbf/ft

7. 7th Layer - 4 anchors (user-defined)

Short-term char. strength T_{ult} = 7115.30 lbf/ft
Long-term design strength R_t = 4416.39 lbf/ft

Design connection strength R_{con} = 4742.10 lbf/ft

Reinforcement

No.	Number of reinforcements	Type of reinforcement	Spacing of reinforcements h_r[ft]	Height of first reinforcement y[ft]	Reinforcements geometry
1	1	6th Layer - 5 anchors (user-defined)	1.00	3.50	identical length of reinforcements
2	1	5th Layer - 6 anchors (user-defined)	1.00	6.00	identical length of reinforcements
3	1	4th Layer - 7 anchors (user-defined)	1.00	8.50	identical length of reinforcements
4	1	3rd Layer - 8 anchors (user-defined)	1.00	11.00	identical length of reinforcements
5	1	1st Layer - 9 anchors (user-defined)	1.00	13.50	identical length of reinforcements
6	1	2nd Layer - 3 anchors (user-defined)	1.00	12.25	identical length of reinforcements
7	1	7th Layer - 4 anchors (user-defined)	1.00	1.50	identical length of reinforcements

Reinforcement details

Reinforcement No. 1

Reinforcement type : 6th Layer - 5 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	3.50	23.00

Reinforcement No. 2

Reinforcement type : 5th Layer - 6 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	6.00	23.00

Reinforcement No. 3
Reinforcement type : 4th Layer - 7 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	8.50	23.00

Reinforcement No. 4
Reinforcement type : 3rd Layer - 8 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	11.00	23.00

Reinforcement No. 5
Reinforcement type : 1st Layer - 9 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	13.50	23.00

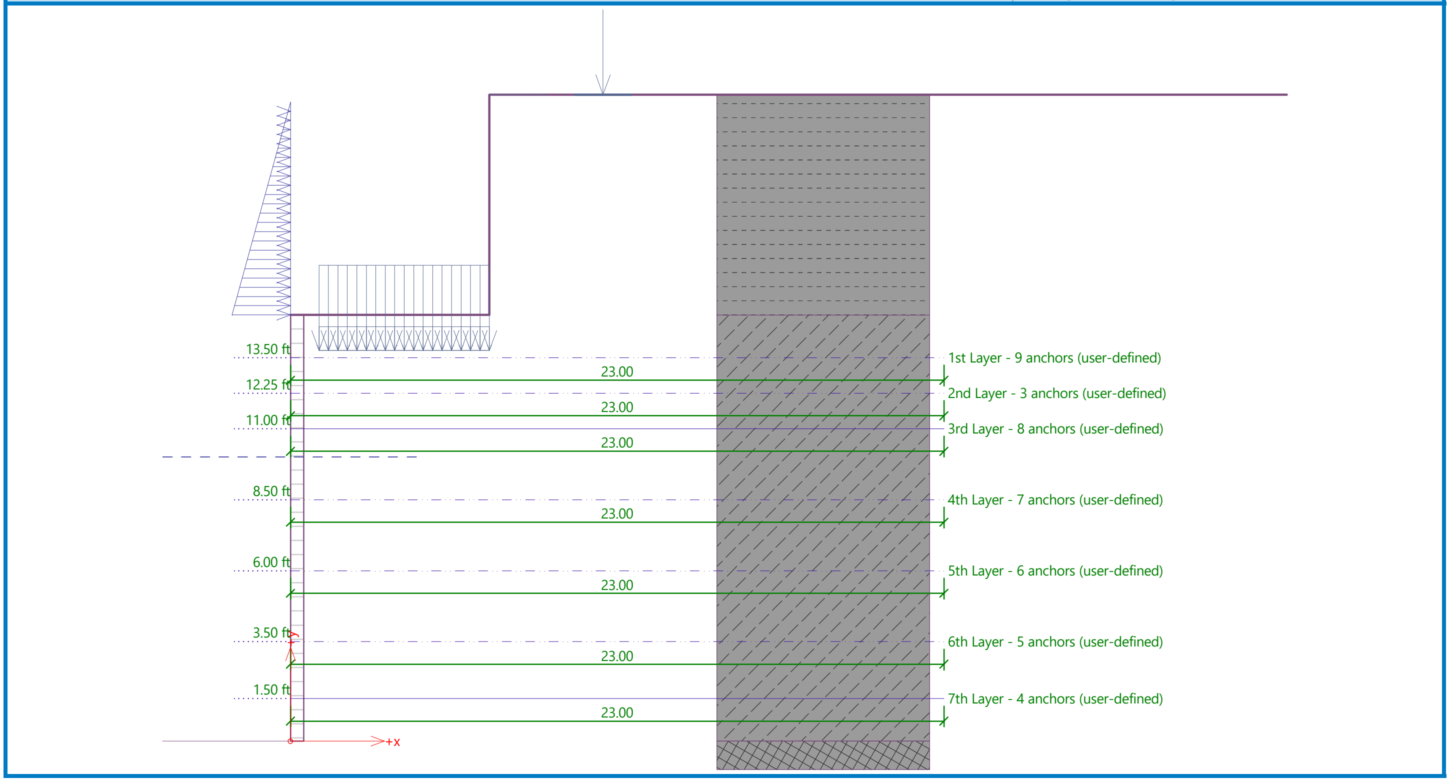
Reinforcement No. 6
Reinforcement type : 2nd Layer - 3 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	12.25	23.00

Reinforcement No. 7
Reinforcement type : 7th Layer - 4 anchors (user-defined)
Number of reinforcements 1
Reinforcement geometry : identical length of reinforcements
Reinforcement length : 23.00 ft

No.	Origin l_1[ft]	End l_2[ft]	Height from bottom y[ft]	Length l[ft]
1	0.00	23.00	1.50	23.00

Name : Reinforcement | **Stage - analysis : 1 - 0**



Soil parameters

34 DEG

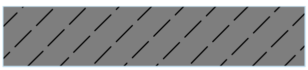
<u>Basic data</u>

Unit weight : γ = 125.0 [pcf]
Internal friction angle : φ_{ef} = 34.00 [°]
Cohesion : c_{ef} = 0.0 [psf]
Friction angle structure-soil : δ = 23.00 [°]

<u>Uplift pressure</u>

Uplift calculation : standard
Unit weight of saturated soil : γ_{sat} = 130.0 [pcf]

<u>View</u>

Soil pattern : 

38 DEG

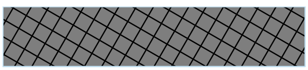
<u>Basic data</u>

Unit weight : γ = 140.0 [pcf]
Internal friction angle : φ_{ef} = 38.00 [°]
Cohesion : c_{ef} = 0.0 [psf]
Friction angle structure-soil : δ = 20.00 [°]

<u>Uplift pressure</u>

Uplift calculation : standard
Unit weight of saturated soil : γ_{sat} = 140.0 [pcf]

<u>View</u>

Soil pattern : 

24 DEG

Basic data

Unit weight : γ = 120.0 [pcf]
Internal friction angle : φ_{ef} = 24.00 [°]
Cohesion : c_{ef} = 0.0 [psf]
Friction angle structure-soil : δ = 24.00 [°]

Uplift pressure

Uplift calculation : standard
Unit weight of saturated soil : γ_{sat} = 130.0 [pcf]

View

Soil pattern :

Geological profile and assigned soils

Position information

Terrain elevation = -9.75 ft

Geological profile and assigned soils

No.	Thickness of layer t [ft]	Depth z [ft]	Elevation [ft]	Assigned soil	Pattern
1	0.01	0.00 .. 0.01	-9.75 .. -9.76	24 DEG	
2	14.99	0.01 .. 15.00	-9.76 .. -24.75	34 DEG	
3	5.00	15.00 .. 20.00	-24.75 .. -29.75	38 DEG	
4	-	20.00 .. ∞	-29.75 .. -	38 DEG	

Terrain profile

No.	Coordinates x [ft]	Depth z [ft]
1	0.00	0.00
2	7.00	0.00
3	7.00	-7.75
4	8.00	-7.75

Origin [0,0] is located in upper right edge of construction.
Positive coordinate +z has downward direction.

Water influence

Ground water table at height 10.00 ft from structure heel.

Input surface surcharges

No.	Surcharge new	Surcharge change	Action	Mag.1 [lbf/ft^2]	Mag.2 [lbf/ft^2]	Ord.x x [ft]	Length l [ft]	Depth z [ft]
1	Yes		permanent	2739.20		1.00	6.00	1.25
2	Yes		variable	770.00		1.00	6.00	1.25

No.	Name
1	DL
2	LL

Input horizontal surcharges

No.	Surcharge new	Surcharge change	Action	Mag.1 [lbf/ft^2]	Ord.x x [ft]	Length l [ft]	Depth z [ft]
4	Yes		variable	60.00	7.00	2.00	on terrain

No.	Name
4	Braking Force

Input concentrated surcharges

No.	Surcharge new	Surcharge change	Action	Magnitude [lbf]	Ord.x x [ft]	Length l [ft]	Width b[ft]	Depth z [ft]
3	Yes		variable	0.10	10.00	2.00	19.00	on terrain

No.	Name
3	LL

Resistance on front face of the structure

Resistance on front face of the structure: at rest
Soil on front face of the structure - 34 DEG
Soil thickness in front of structure h = 1.00 ft

Terrain in front of structure is flat.

Applied forces acting on the structure

No.	Force new	Force edit	Name	Action	Type	l [ft]	q_{x1} [lbf/ft^2]	q_{x2} [lbf/ft^2]
1	Yes		Earth Passive Pressure	permanent	trapezoid	7.50	0.0	1875.0

Settings of the stage of construction

Reduction of soil/soil friction angle : do not reduce
Design situation : Strength I

Verification No. 1

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-12.01	44204.5	12.68	1.000	1.000	1.350
Active pressure	6600.2	-8.60	4173.3	23.00	1.500	1.500	1.500
Water pressure	0.0	-22.75	0.0	46.00	1.000	1.000	1.000
DL	0.0	-13.75	16435.2	4.00	0.750	0.750	1.500
LL	0.0	-13.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-22.75	0.0	11.00	0.000	0.000	1.750
Earth Passive Pressure	-7031.2	-17.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 750328.6 lbfft/ft
Overturning moment M_{ovr} = 85147.8 lbfft/ft

Capacity demand ratio CDR = 8.81
Wall for overturning is SATISFACTORY

Check for slip

Resisting horizontal force H_{res} = 49057.59 lbf/ft
Active horizontal force H_{act} = 5330.02 lbf/ft

Capacity demand ratio CDR = 9.20
Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Verification No. 2

Forces acting on construction

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Coeff. overtur.	Coeff. sliding	Coeff. stress
Weight - reinforced soil	0.0	-12.01	44204.5	12.68	1.000	1.000	1.350
Active pressure	6600.2	-8.60	4173.3	23.00	1.500	1.500	1.500
Water pressure	0.0	-22.75	0.0	46.00	1.000	1.000	1.000
DL	0.0	-13.75	16435.2	4.00	0.750	0.750	1.500
LL	0.0	-13.75	4620.0	4.00	0.000	0.000	1.750
LL	0.0	-22.75	0.0	11.00	0.000	0.000	1.750
Earth Passive Pressure	-7031.2	-17.50	0.0	0.00	0.650	0.650	1.500

Verification of complete wall

Check for overturning stability
Resisting moment M_{res} = 750328.6 lbfft/ft
Overturning moment M_{ovr} = 85147.8 lbfft/ft

Capacity demand ratio CDR = 8.81
Wall for overturning is SATISFACTORY

Check for slip
Resisting horizontal force H_{res} = 49057.59 lbf/ft
Active horizontal force H_{act} = 5330.02 lbf/ft

Capacity demand ratio CDR = 9.20
Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

Bearing capacity of foundation soil

Design load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]	Eccentricity [–]	Stress [psf]
1	3811.1	98673.85	-646.54	0.002	4304.6
2	-26455.8	62790.85	5330.02	0.000	2730.0
3	3811.1	98673.85	-646.54	0.002	4304.6
4	-26455.8	62790.85	5330.02	0.000	2730.0

Service load acting at the center of footing bottom

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]
1	-8442.5	69433.02	-431.03
2	-43092.5	64813.01	-431.03
3	-8442.5	69433.02	-431.03

No.	Moment [lbfft/ft]	Norm. force [lbf/ft]	Shear Force [lbf/ft]
4	-43092.5	64813.01	-431.03

Verification of foundation soil

Stress in the footing bottom : rectangle

Eccentricity verification
Max. eccentricity of normal force e = 0.002
Maximum allowable eccentricity e_{alw} = 0.333

Eccentricity of the normal force is SATISFACTORY

Verification of bearing capacity
Ultimate bearing capacity of found. soil R = 15000.0 psf
Partial factor on bearing capacity γ_{Rv} = 0.65
Max. stress at footing bottom σ = 4304.6 psf
Allowable bearing capacity of foundation soil R_d = 9750.0 psf
Capacity demand ratio CDR = 2.3

Bearing capacity of foundation soil is SATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Verification of slip on georeinforcement No. 1

Forces acting on construction (verification of most utilized reinforcement)

Name	F_{hor} [lbf/ft]	App.Pt. z [ft]	F_{vert} [lbf/ft]	App.Pt. x [ft]	Design coefficient
Active pressure	5186.4	-7.77	3733.6	23.00	1.500
Earth Passive Pressure	-7031.2	-16.00	0.0	0.00	0.650
Weight - reinforced soil	0.0	-11.14	42448.1	12.73	1.000
DL	0.0	-12.25	16435.2	4.00	0.650
LL	0.0	-12.25	4620.0	4.00	0.000
LL	0.0	-21.25	0.0	11.00	0.000

Check for slip along geo-reinforcement with the maximal utilization (Reinforc. No.: 1)

Inclination of slip surface = 90.00 °
Overall normal force acting on reinforcement = 58731.37 lbf/ft
Coefficient of reduction of slip along geo-textile = 0.60
Resistance along geo-reinforcement = 23768.88 lbf/ft
Wall resistance = 0.00 lbf/ft
Overall bearing capacity of reinforcements = 0.00 lbf/ft

Check for slip:
Resisting horizontal force H_{res} = 23768.88 lbf/ft
Active horiz. force H_{act} = 3209.27 lbf/ft

Capacity demand ratio CDR = 7.41
Slip along geotextile is SATISFACTORY

Calculation of internal stability No. 1

Calculated forces and strength of reinforcements

No.	Name	F_x [lbf/ft]	Depth z[ft]	R_t [lbf/ft]	Utiliz. [%]	T_p [lbf/ft]	Utiliz. [%]	R_{con} [lbf/ft]	Utiliz. [%]
1	7th Layer - 4 anchors (user-defined)	-1688.32	13.51	3312.29	50.97	40998.94	4.12	3556.58	47.47
2	6th Layer - 5 anchors (user-defined)	-3867.23	11.51	4140.37	93.40	35071.77	11.03	4568.50	84.65
3	5th Layer - 6 anchors (user-defined)	-4479.56	9.01	4968.44	90.16	28260.39	15.85	5580.44	80.27
4	4th Layer - 7 anchors (user-defined)	-4651.50	6.51	5796.52	80.25	22846.53	20.36	7090.25	65.60
5	3rd Layer - 8 anchors (user-defined)	-3549.63	4.01	6624.59	53.58	18752.05	18.93	8600.08	41.27
6	2nd Layer - 3 anchors (user-defined)	-2373.66	2.76	2484.23	95.55	16704.81	14.21	2790.22	85.07
7	1st Layer - 9 anchors (user-defined)	-517.46	1.51	7452.66	6.94	14657.57	3.53	7796.33	6.64

Check for tensile strength (reinforcement No.6)
Tension strength R_t = 2484.23 lbf/ft
Force in reinforcement F_x = 2373.66 lbf/ft

Capacity demand ratio CDR = 1.05
Reinforcement for tensile strength is SATISFACTORY

Check for pull out resistance (reinforcement No.4)
Pull out resistance T_p = 22846.53 lbf/ft
Force in reinforcement F_x = 4651.50 lbf/ft

Capacity demand ratio CDR = 4.91
Reinforcement for pull out resistance is SATISFACTORY

Verification of connection strength (reinforcement No.6)
Connection strength R_{con} = 2790.22 lbf/ft
Force in reinforcement F_x = 2373.66 lbf/ft

Capacity demand ratio CDR = 1.18
Connection strength is SATISFACTORY

Overall verification - reinforcement is SATISFACTORY

Global stability analysis No. 1

Parameters of input slip surface
Center S = (-6.16;-8.02) ft
Radius r = 23.85 ft
Angle α_1 = 0.00 °
α_2 = 0.00 °

Analysis has not been performed.

Appendix E – Slide Deck on Conceptual Strengthening, Opinion of Probable Costs, and Construction Schedule by Walter P Moore

Updated 8/22/25

HEF Manassas Bridge Assessments

Conceptual Strengthening, Opinion of Probable Costs, and Construction Schedule

Steven Treser, PE, AIA

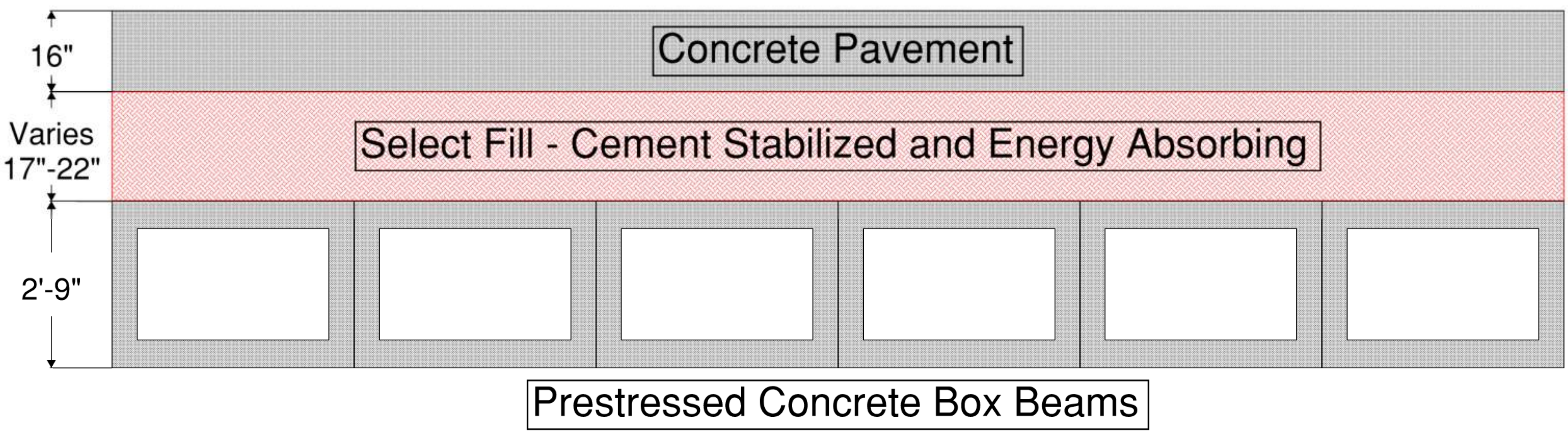
Managing Director – Walter P Moore

Amir Manafpour, PE, SE

Senior Project Manager – Walter P Moore

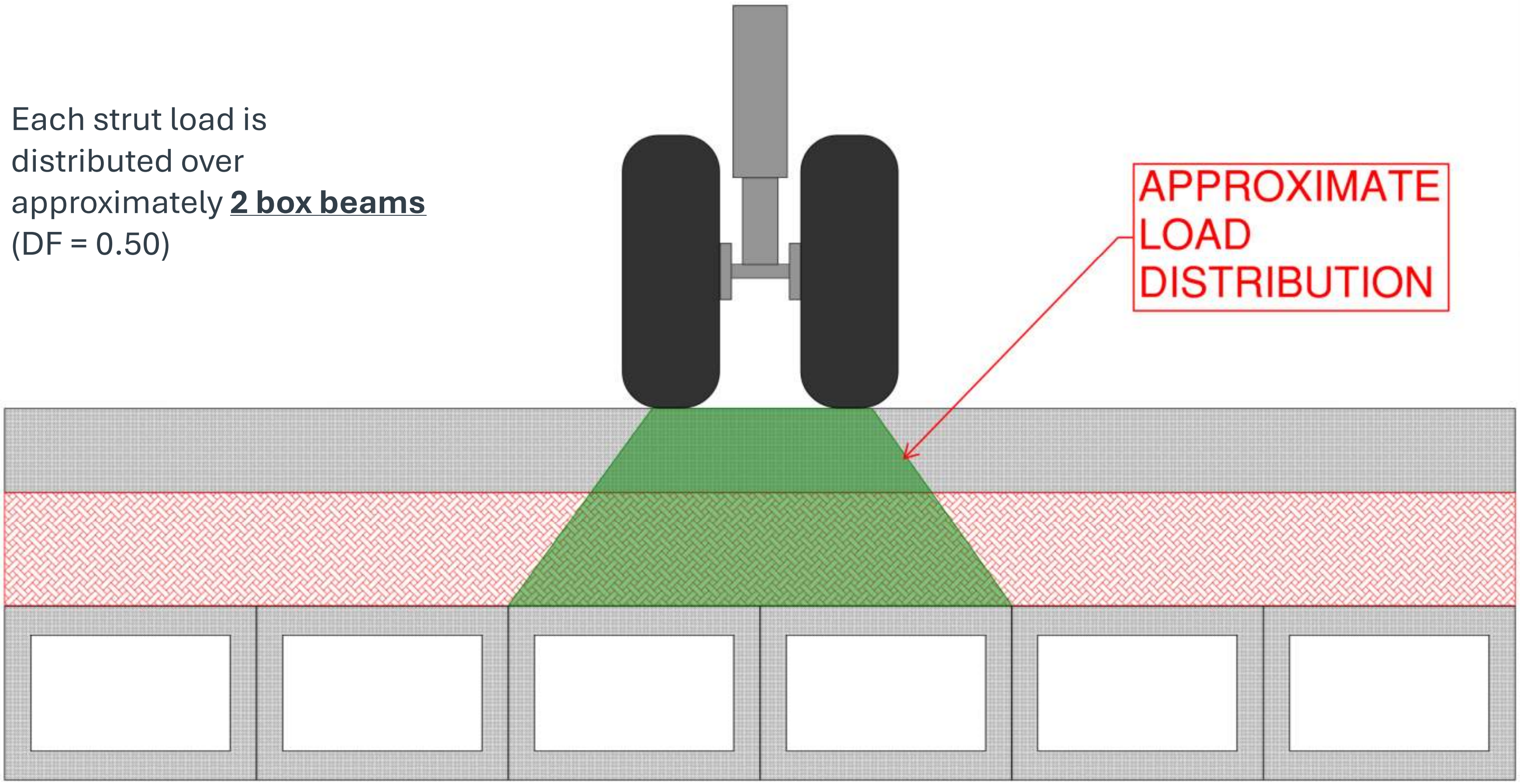
Walter P Moore Project No. D01-24004-00

Existing Taxiway and Runway Bridge Cross Section



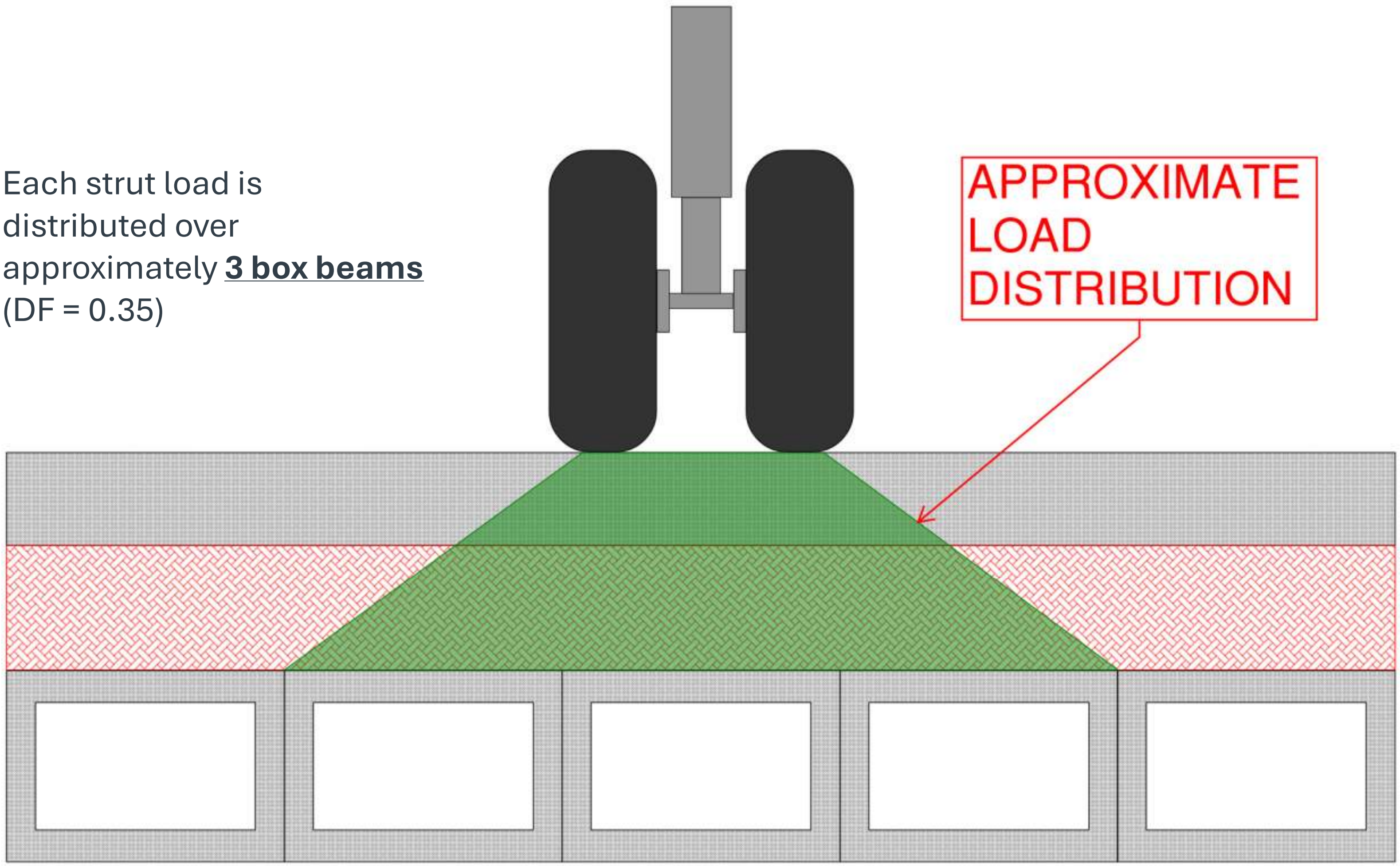
Original Load Distribution – AASHTO Method

Each strut load is distributed over approximately **2 box beams** (DF = 0.50)

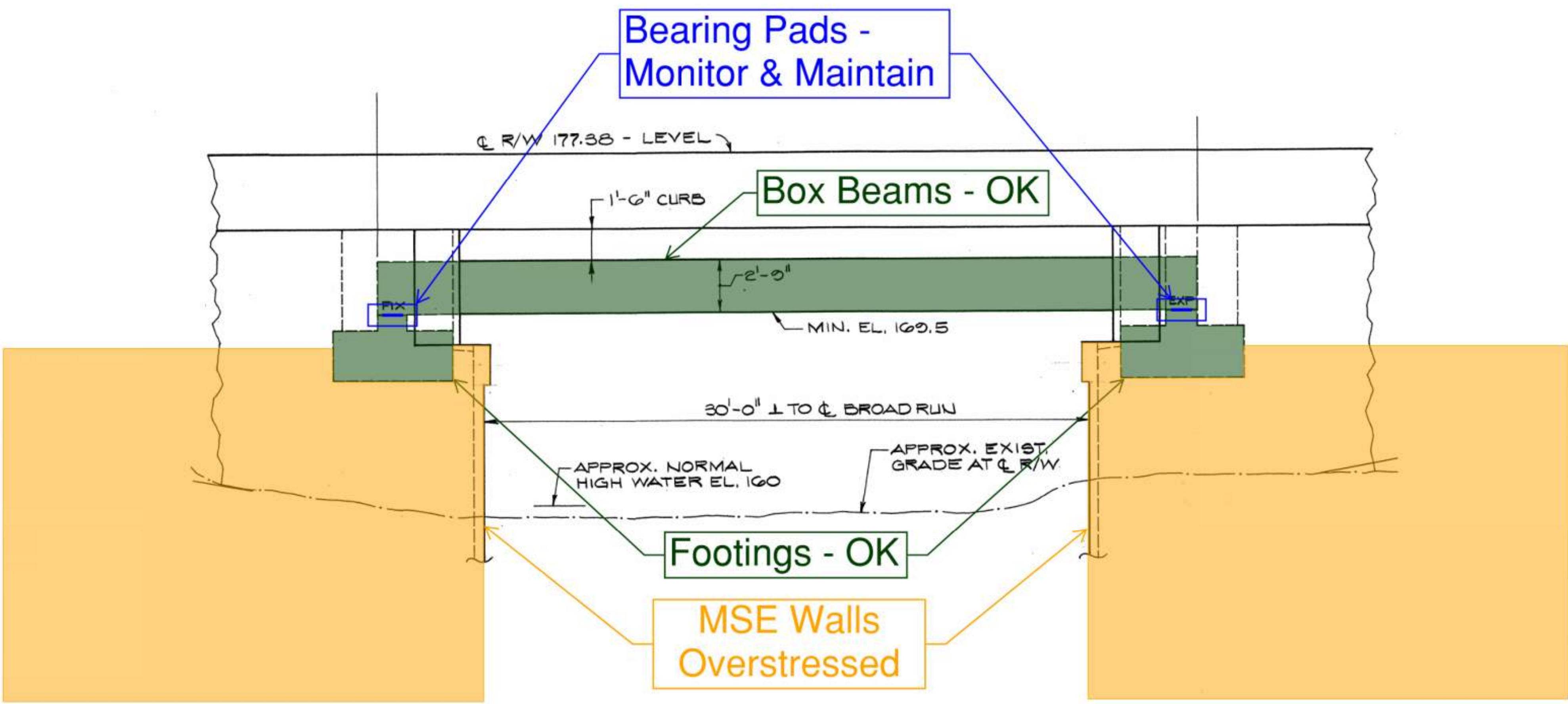


New Load Distribution – Based on Load Tests Performed by WPM

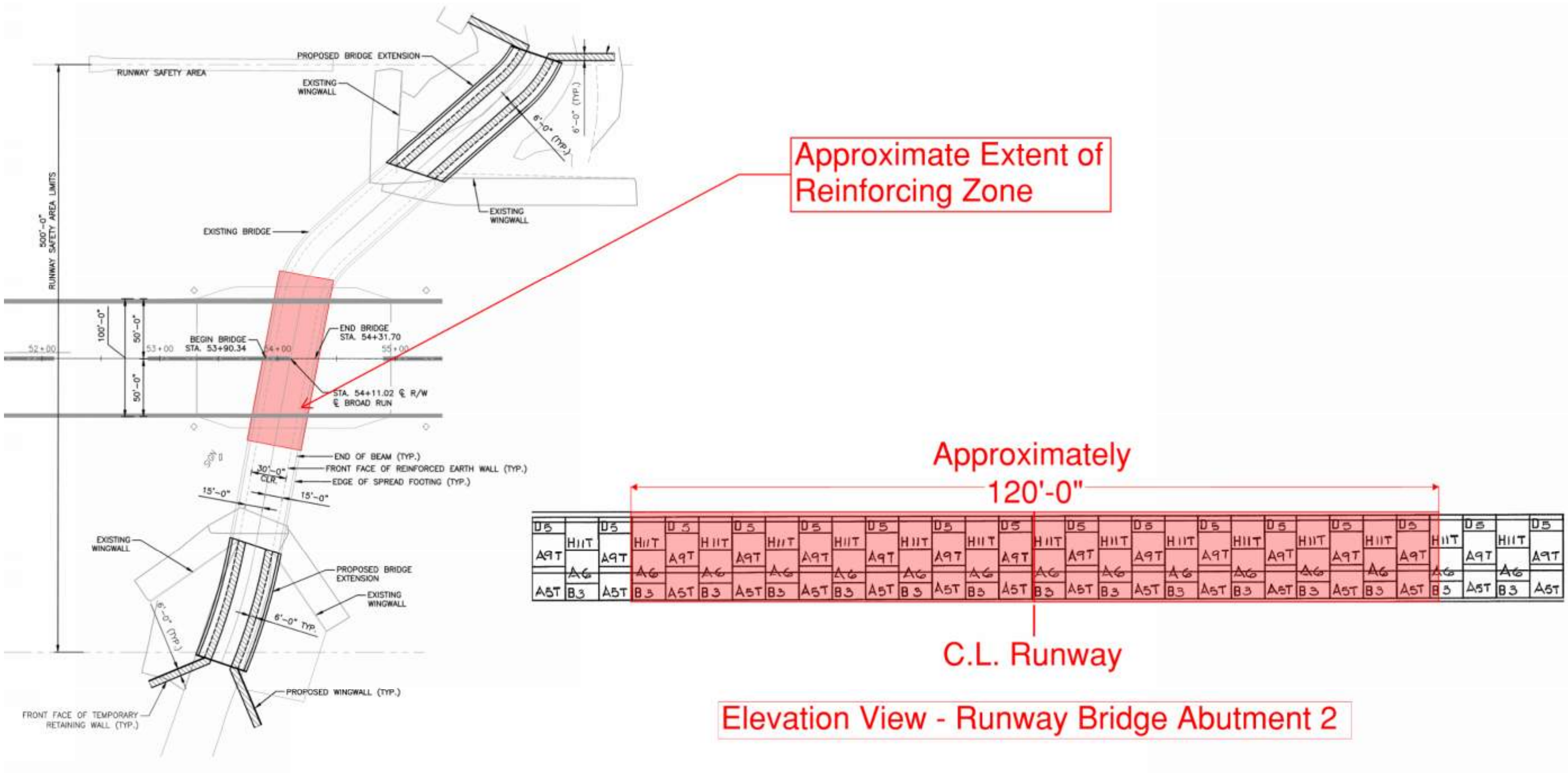
Each strut load is distributed over approximately **3 box beams** (DF = 0.35)



Analysis and Investigation Summary



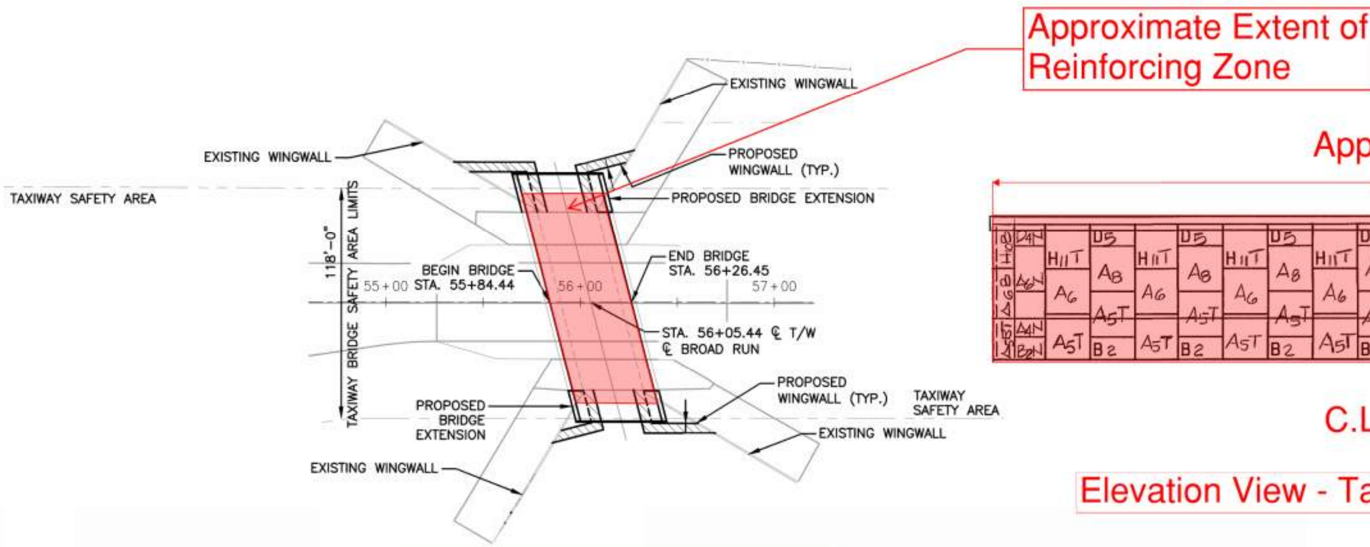
MSE Wall – Conceptual Strengthening Extents (Runway)



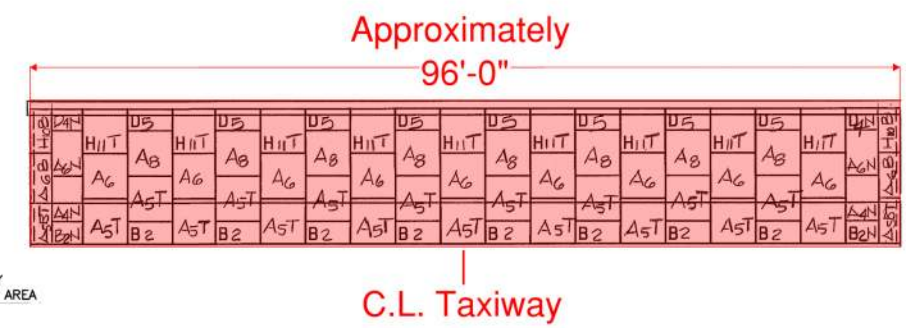
Plan View - Runway Bridge

Elevation View - Runway Bridge Abutment 2

MSE Wall – Conceptual Strengthening Extents (Taxiway)

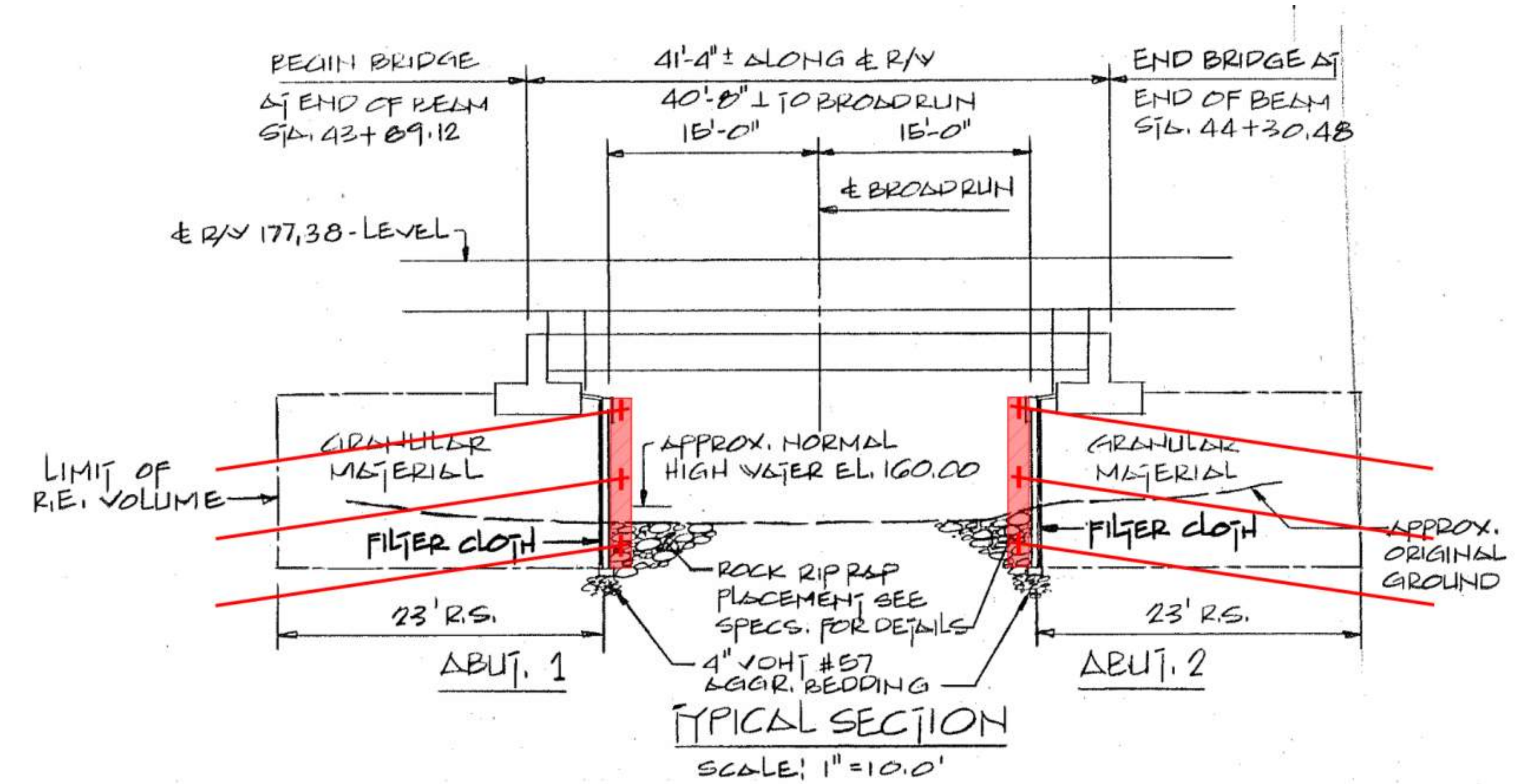


Plan View - Taxiway Bridge



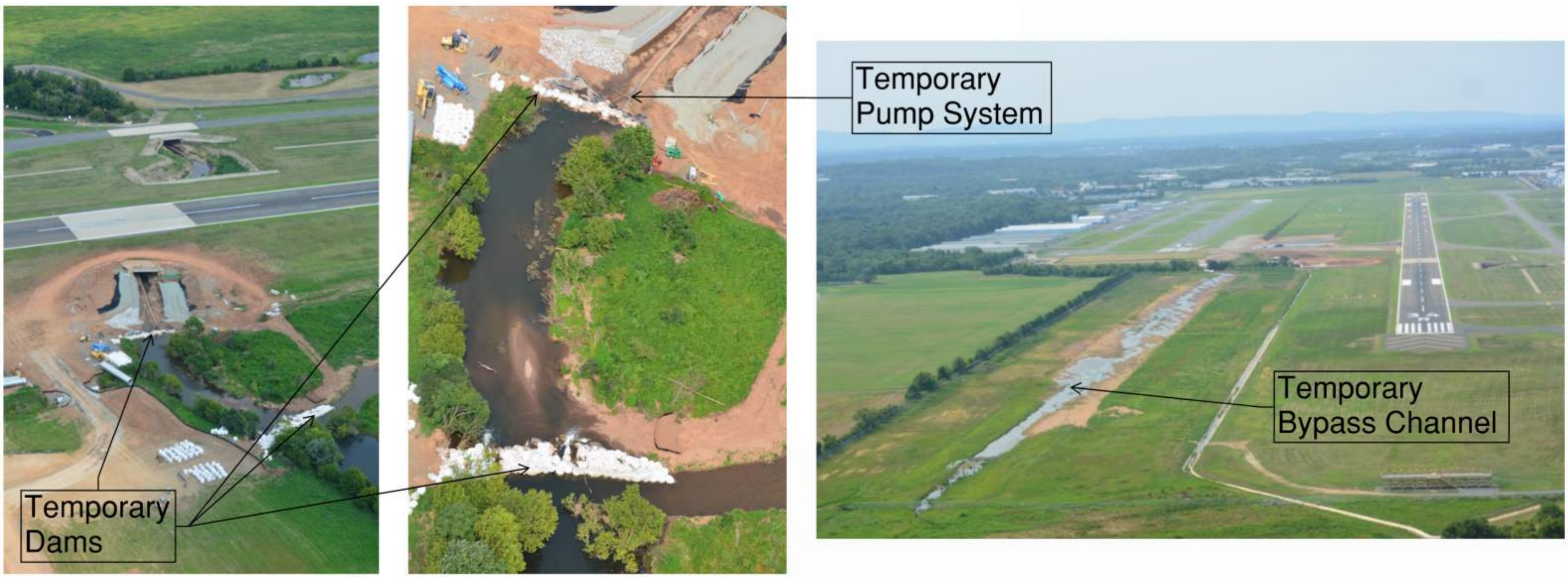
Elevation View - Taxiway Bridge Abutment 1

MSE Wall – Conceptual Strengthening Soil Nail Wall



MSE Wall – Conceptual Rerouting of Broad Run

MSE Wall – Rerouting Broad Run from circa 2013



PRELIMINARY Opinion of Probable Costs – Summary (Updated 8/22/25)

Task Group	Description	Low Cost	High Cost	Duration
Engineering	Engineering Design and Construction Administration	$450k (~10%)	$900k (~20%)	2-3 months
Site Work	Re-routing Broad Run, Remove Rip-Rap, Scaffolding			1-2 months
Soil Nailing and Wall Shotcrete	Installing, Tensioning Soil Nails, Reinforced Shotcrete Wall, Drainage Mat, Weep Holes	$1.45MM	$2.2MM	2-3 months
Finish Site Work	Earthwork, Replace Rip-Rap	$150K	$225k	1-2 months
	Subtotal =	$3.1MM	$4.9MM	
Night Contingency (30%)	Nighttime Work Instead of Daytime	$900k	$1.5MM	1-2 months
	TOTAL =	**$3.75MM**	**$6.025MM**	**8-14 months**

Thank You

Attachment #2 – Preliminary Concept for Maintaining Broad Run Minimum Water Flow During Construction at HEF Manassas Airport

202.481.7685 main

December 12, 2025

Abel Garcia
Principal
V-1 Consulting, LLC
Denver, Colorado

Re: Memorandum – Preliminary Concept for Maintaining Broad Run Minimum Water Flow During Construction at HEF Manassas Airport
Walter P Moore Project Number: D01.24004.00

Dear Abel:

Walter P Moore is pleased to submit this Memorandum to describe Preliminary Concepts for maintaining Broad Run minimum water flow during bridge strengthening construction at the HEF Manassas Airport.

Walter P Moore Engineers have conducted assessments, testing and observations of the runway and taxiway bridges over Broad Run. As a result of our investigation, we have recommended strengthening the retaining walls supporting the above-mentioned bridges, as an alternative to full replacement of each bridge.

While strengthening each bridge's foundation provides a significant economic benefit, doing so without adversely affecting both the quantity and quality of base flow in Broad Run presents a challenge. Protecting Broad Run both within the work zone and immediately downstream will be a primary focus throughout design and construction.

Recent base flowrate measurements estimate Broad Run's base flow to be in a range of 7 to 8 cubic feet per second (cfs). To help minimize the environmental impact on the stream ecosystem, including the mussels recently identified, we have discussed the following considerations during construction:

1. Design stream modifications to temporarily maintain minimum Broad Run base flow through the work zone area and divert remaining stream flow around the work zone, reconnecting to Broad Run further downstream.
2. Construct a temporary working surface within the work zone utilizing precast concrete Double-Tees (see Image #1).
3. In the event Broad Run base flow can not be maintained through the work zone due to flow rate or water quality limitations, an alternative will be developed to pipe Broad Run's base flow instead using either gravity flow through a minimum of two pipes or mechanical pumping with back up.

The above considerations will at a minimum ensure Broad Run's base flow will be protected and maintained immediately downstream of the work zone. Walter P Moore will strive to devise methods to also maintain Broad Run's flow through the work zone. However, these Preliminary considerations are provided prior to the following steps:

1. Conducting additional investigations regarding the elevation of the retaining wall foundation relative to the stream height during low flow

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2. Providing preliminary design details

3. Consultation with a qualified Contractor for Means and Methods during construction

Additional concepts will be explored once the above steps are completed and throughout the design process.

Image #1 – Preliminary Concept – Double-Tees as working surface to maintain stream flow underneath.

We very much appreciate the opportunity to provide this Memorandum and look forward to working with you on this Project.

Sincerely,

WALTER P. MOORE AND ASSOCIATES, INC.

Steven Treser

Steven Treser, PE, AIA
Principal

cc: Managing Director
Principal in Charge

Attachment #3 – VADWR Concurrence

Peter Byrne

From: Ryan Schwegman <Ryan@BioSurveyGroup.com>
Sent: Thursday, December 18, 2025 4:00 PM
To: Peter Byrne
Cc: Jamie Morgan (); Carol Weed
Subject: FW: ESSLog# 46422_Manassas Regional Airport Terminal Redevelopment Project_DWR_AEM20251218

See below.

From: Martin, Amy (DWR) <Amy.Martin@dwr.virginia.gov>
Sent: Thursday, December 18, 2025 2:59 PM
To: Ryan Schwegman <Ryan@BioSurveyGroup.com>
Cc: Schul, Hannah (DWR) <Hannah.Schul@dwr.virginia.gov>; Watson, Brian (DWR) <Brian.Watson@dwr.virginia.gov>; Strawderman, Nicole (DWR) <Nicole.Strawderman@dwr.virginia.gov>; AKing@avports.com
Subject: ESSLog# 46422_Manassas Regional Airport Terminal Redevelopment Project_DWR_AEM20251218

Mr. Schwegman,
We have received and reviewed the *Manassas Regional Airport – Broad Run Mussel Survey Report* prepared by Davey Resource Group along with the *Preliminary Concept for Maintaining Broad Run Minimum Water Flow During Construction at the HEF Manassas Airport* for the subject project and offer the following comments and recommendations regarding the protection of mussels known from Broad Run downstream of the proposed work area.

We concur with the findings of the mussel survey, believe it was performed appropriately, and that it was done in coordination with our State Malacologist, Brian Watson. We also are agreeable to the maintenance of a minimum flows within Broad Run of at least 7.48 cfs and the preliminary plan to achieve that goal. We would like to see a final plan to maintain minimum flows in Broad Run that includes a plan for monitoring flows (monitoring locations and frequency of monitoring) and a plan to address any interruptions to flow that includes contacting DWR's Environmental Program Manager, Hannah Schul, at Hannah.Schul@dwr.virginia.gov or 804-968-8546.

Assuming the following, we have determined that the project has minimized impacts upon freshwater mussels to the greatest extent practicable and therefore has satisfied our concerns for their protection:

1. That DWR has the opportunity to review and weigh in on the final plan to maintain minimum flows;
2. That the final plan includes the information we requested above; and
3. That minimum flows of 7.48 cfs are maintained in Broad Run within and downstream of the work site.

We appreciate your willingness to work cooperatively to minimize impacts upon wildlife associated with the project. Please do not hesitate to reach out if you need anything additional.

Thank you, Amy

VIRGINIA DWR

Amy Martin
(she/her/hers)
Manager, Nongame and Endangered Species Program
Cell: 804-481-5296
Virginia Department of Wildlife Resources
www.dwr.virginia.gov