FORENSIC GEOTECHNICAL EVALUATION
US 36 RETAINING WALL B1-10R
WESTMINSTER, COLORADO

Prepared for:

OFFICE OF THE ATTORNEY GENERAL
COLORADO DEPARTMENT OF LAW
Ralph L. Carr Judicial Building
1300 Broadway, 10th Floor
Denver, Colorado 80203

Attention: Andrew Gomez

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SCOPE

This report presents the results of our Forensic Geotechnical Evaluation of Retaining Wall B1-10R along US 36 in Westminster, Colorado (Fig. 1) which failed in July 2019. We were requested to evaluate conditions associated with the wall to form opinions regarding the cause(s) of damage. Our scope included review of data supplied by the Colorado Department of Transportation (CDOT) regarding the wall failure, observations during demolition and exploratory drilling, laboratory testing of soils obtained during demolition and drilling, review of a portion of the available documents regarding the wall design and construction, review of FHWA design methodology, and analysis. This report includes a summary of our understanding of the failure progression, observations, the field and laboratory data, analysis, and opinions.
This report was prepared based on limited data. There may be other documents regarding the wall design and construction which were not provided to us, and there is certainly data from forensic investigations conducted by others. Additional data and analyses by others, including competing opinions, could result in changes to our understanding of conditions, analysis and opinions. We have considered comments provided by some of the project design team during confidential settlement negotiations.

SUMMARY OF PROJECT HISTORY

Wall B1-10R is located immediately west of a bridge which spans the Burlington Northern Santa Fe (BNSF) railroad alignment, between the Church Ranch Boulevard and Wadsworth Boulevard interchanges along US 36 (Fig. 1). US 36 is known as the Boulder Turnpike because a toll road was constructed along the alignment in the early 1950’s. Photo 1 shows the site prior to construction of the toll road. It appears a ditch was present at the northeast corner of Lower Church Lake, at the location of the BNSF bridge and Wall B1-10R. Photo 2 shows site conditions in 1954, after the Turnpike construction.

Photo 1: 1937 Aerial Photo – Source: Colorado Aerial Photo
Wall B1-10R was designed and constructed by Ames/Granite Joint Venture (Ames/Granite) during Phase I of the US 36 improvements. Consulting engineering companies designed the highway, provided geotechnical engineering recommendations, designed the retaining wall, and provided construction observation and testing services. Reference to Ames/Granite in this report is intended to include the entire design/build team.

(The next paragraph is based on an information provided by Mr. Anthony Meneghetti, P.E. with High Performance Transportation Enterprise (HPTE))

Ames/Granite had a Quality Assurance/Quality Control process in place that was approved by CDOT. CDOT completed audits to ensure the design was completed by Ames/Granite in accordance with the Design-Build contract requirements and followed Ames/Granite’s QA/QC (Quality Assurance/Quality Control) process and procedures. CDOT completed audits on certain aspects of construction in the field and process audits to ensure Ames/Granite was following the QA/QC processes.

A submittal dated April 4, 2013 (Exhibit A) indicates the wall height ranged up to about 35 feet. The wall extended from Project Station 2167 to 2174+60. Our observations indicate the portion which failed extended from approximate Project Station 2171
to the BNSF bridge (approx. Station 2173+90). The wall consisted of a mechanically re-
reinforced earth system with pre-cast concrete face panels and GS11 grid strips embed-
ded in CDOT Class I structural fill. The bottom elevation of the wall panels in the failed 
portion ranged from elevation (el.) 5354.07 to 5351.57, with the drop occurring at ap-
proximate Station 2173+60. The construction included a cut into the existing Turnpike fill 
embankment. The retained soil between the cut slope and reinforced zone was also 
Class I material (Ames/Granite As-Built Sheet WD-201, May 16, 2016, Exhibit E).

The site is between a detention pond to the north, and Lower Church Lake to the 
south. A storm sewer discharges water from the detention pond to the Lake; this pipe is 
located at Station 2170+76. Prior to the failure, plans indicate the ground surface be-
tween the wall and Lake sloped gradually for about 10 feet south of the wall face, and 
then steeper toward the south.

The Geotechnical Roadway Design Report was dated October 30, 2012. The ap-
proximate locations of the geotechnical borings and logs are shown in Exhibit B. These 
plans show a bike path was planned south of the Wall, separate from the Wall. The bike 
path was subsequently moved to the top of the wall. The logs indicate soil conditions in 
borings drilled at the top of the existing embankment (BNSF-B7-KA and BNSF-B8-KA) 
included existing pavement underlain by lean clay fill which extended to a depth of 29 
feet or el. 5354-5. The fill was underlain by lean clay with sand to el. 5345 in BNSF-B7-
KA and then by claystone bedrock. The fill was underlain by fat clay to el. 5334 in 
BNSF-B8-KA and then by claystone. Penetration resistance test blow counts ranged 
from 7 to 22 blows to advance samplers 12 inches in the fill, with most tests ranging 
from 7 to 12 blows. Groundwater was reported at el. 5358 in BNSF-B7-KA at the time of 
drilling. No groundwater was reported in BNSF-B8-KA and no water measurements 
were made after drilling. Water was reported at el. 5337 in boring DP-KA-10 in the de-
tention pond area to the north.

Borings RE-KA-23, RE-KA-24 and RE-KA-25 were drilled south of the planned 
retaining wall. Six to 9 feet of lean clay and clayey sand were encountered in RE-KA-23
and RE-KA-24 from the ground surface (el. 5345 and 5341, respectively) over weathered and relatively unweathered claystone. Sandstone was found below el. 5309 in RE-KA-25 and groundwater was reported in RE-KA-25 at el. 5308.

Global slope stability of Wall B1-10R was evaluated during design using a cross-section at Station 2168+50 from plans provided in June 2012. This location is about 300 feet west of the actual failure area. The analysis output shows the retaining wall height was about 25 feet. Factors of safety of 1.6 and 1.7 were reported for the drained condition. Soil strength was modeled using an angle of internal friction (\(\phi\), \(\Phi\)) and cohesion (c). For long term analysis, soils are assumed “drained” and corresponding values are used (\(\Phi'\) and c'). A sample of clay from BNSF-B7-KA at 39 feet had an unconfined compressive strength of 677 pounds per square foot (psf). A direct shear test performed on a sample of fill from BNSF-B8-KA at 14 feet indicated peak strength cohesion of approximately 525 psf and angle of internal friction of 35 degrees; the test was run at natural moisture. There were no tests to evaluate the clay fraction of the embankment fill or native clay. On page 19 of the 10/30/12 Geotechnical report, a drained internal angle of friction of 30 degrees, cohesion of 500 psf and unit weight of 122 pcf were shown for analysis of bearing capacity for the foundation soils at Wall B1-10R, and \(\Phi'=38\) degrees for the reinforced fill and retained soil. In the global stability analysis, the values used were \(\Phi'=38\) degrees for the retained soil, \(\Phi'=50\) degrees and c'=10,000 psf for the reinforced soil, \(\Phi'=26\) degrees and 400 pcf cohesion for the embankment fill, \(\Phi'=26\) degrees and 200 psf cohesion for the native clay, and unit weights of 140, 140, 122 and 126 pounds per cubic foot (pcf), respectively. We do not know the basis for selection of the assumed strength parameters.

The geotechnical design consultant analyzed bearing capacity for the walls on the project and provided graphs which indicate the relationship of factored bearing resistance with varying reinforcing strap lengths. The graph for Wall B1-10R from the 2012 report is included in Exhibit B and indicates factored bearing resistance of about 7500 to 7900 psf for strap lengths of 8 to 20 feet.
The April 4, 2013 submittal (Exhibit A) indicates the wall design was revised; this significantly affected the design. A subsequent field design change (FDC) form dated August 2, 2013 (FDC #177, Exhibit C) states the bike path was moved to cross the BNSF bridge, “hence the wall is significantly taller than previously expected.” An FDC form dated March 8, 2013 (Exhibit D) indicates review of shop drawings for Wall B2-10R revealed the bearing pressures were higher than the range provided by the geotechnical design consultant in 2012. The form heading indicates this actually applies to Wall B1-10R, as does an enclosed email. The geotechnical consultant recommended over-excavation and placement of 2 feet of Class I fill below the wall system, underlain by 2 feet of impermeable material to allow use of a higher bearing pressure and the accompanying graph shows factored bearing resistance of about 10,500 to 9,800 psf for strap lengths of 8 to 24 feet. The April 4, 2013 submittal indicates factored bearing resistance of 8390 and 8740 psf and strap lengths of 24 and 25 feet; respectively, were used in the design within the failed portion of the wall. The strap length increase occurred at approximate Station 2173+60 where the base of the wall panels stepped down. We were not provided information which indicates global slope stability was re-evaluated for the increased wall height documented in FDC #177.

The wall was designed with an internal drain system which consisted of 1-foot wide geo-composite strip drains on 10 feet maximum horizontal spacing which extended upward from a 4-inch perforated pipe bedded in Class B filter material and wrapped in fabric at the base of the cut slope (the “toe drain”). Two options are detailed on As-Built Sheet WD-201 (Exhibit E), one with the strip drains on the cut slope and one with the drains extending vertically above the toe drain. We found details within the CDOT website which are similar to the drain detail showing the strip drains on the cut slope. We did not find a detail on the website which showed vertical strip drains. The two drain options share most requirements. The details indicate the strip drains penetrate an impermeable geomembrane below the pavement; this membrane is sloped to direct water to the strip drains. Four-inch, non-perforated pipes are shown extending from the toe drain to discharge on the slope below the wall at 100-feet centers (the “lateral drains”). No bedding is indicated for the lateral drains. No specifications are shown for the drain
pipe. We believe the primary purpose of either drain option is to collect water which infiltrates the pavement after construction. The strip drains would also transmit surface water downward during construction, prior to paving. The perforated pipe in the toe drain could also collect water. Construction photos provided by CDOT and observation during demolition confirm the vertical option was used (Photo 3). The solid lateral drains would not control water which seeped into the Class I fill south of the toe drain (toward the wall).

When the wall design was changed to include over-excavation and placement of 2 feet of Class I material underlain by 2 feet of comparatively impermeable soil, it increased the thickness of comparatively permeable Class I fill below the drain system by 2 feet. This increased the risk that water which penetrated the wall system would be trapped below the drain system. We have not found any indication the drain system was modified to control this water.

The wall was constructed in 2013. Mr. Meneghetti provided a Construction Timeline (Exhibit F) which shows construction started in March 2013. The leveling pad below the wall panels was poured in June, and the subsurface drains were placed in June.
The impermeable geomembrane was installed in December. Roadway paving occurred in April 2014.

Extremely heavy rainfall occurred in September 2013. Records from nearby National Oceanic & Atmospheric Administration (NOAA) stations show 4.4 to 6.9 inches of precipitation occurred between September 10 & 16 (Exhibit G). This resulted in severe flooding in northeastern Colorado. Construction photos indicate the wall was substantially complete and pavement was not present at this time. The roadway area flooded (Photo 4), and erosion of fill occurred adjacent to the wall, near the BNSF bridge abutment (Photo 5), and at the west end of the wall. Photo 5 indicates erosion also caused downward deflection and/or complete exposure of the upper straps adjacent to the wall. An Ames/Granite As-Built plan (Sheet WD-706, Exhibit H) reportedly shows the pavement within 400 feet west of the BNSF bridge settled. The plan does not indicate the date when this was observed. We believe settlement was probably caused by wetting which occurred during the September 2013 storms and other precipitation events before the pavement was fully in place. The approximate limits of the mapped settlement area are shown on Fig. 15; the limits correspond strongly with the failed portion of the wall.

On September 10, 2014 (one year after the 2013 floods), RFI No. 316 (Exhibit I) was issued, requesting approval to place flow fill in the voids caused by erosion adjacent to the retaining wall. The RFI document includes three un-dated, non-conformance notifications related to a hazard created by the erosion, including the inability of the temporary drainage design “to accommodate the ability to move water from the newly constructed bridge to a (sic “an”) inlet before causing damage to existing work” and failure to “maintain the previously constructed work during all construction phases.” We understand the voids created by the erosion were subsequently filled with flowable fill. We do not know if the affected straps were replaced.
A subsequent Request for Information dated 7/10/2015 (RFI 432A, Exhibit I), almost 2 years after the September 2013 floods, states “Ames/Granite has recently discovered that MSE Wall B1-10R experienced erosion which is believed to be from the heavy rains in 2013 than (sic “that”) was unknown and not referenced in RFI 316. The attached plan sheet shows the approximate limits of the visible erosion. It is difficult to see the extents of the erosion as there is only an approximate 1.5’ gap between the sidewalk and the wall.” An attached email states “This appears to be the same issue as before as (sic “as far as”) the wall is concerned.” A subsequent email states “I am concerned that this continues to be an issue during heavy rainfall events. Steps need to be taken to limit the potential for exposure of backfill do (sic “to”)
concentrated runoff.” “The 1’ gap gets 4” flatwork between the coping and the bikepath to protect the backfill and the eliminate this problem. I believe this had not been done previously as they were waiting for the fence to be installed prior to putting (sic “placing”) the flatwork.”

Very wet weather also occurred during late spring and early summer of 2019. NOAA records indicate 1 to 2-inch (+/-) accumulations around May 20-22, June 16-23 and July 6, with frequent lower rainfall events (Exhibit G). This wet weather could have contributed to wetting of soils which support the wall and the failure in July 2019.

WALL FAILURE

We were first contacted by a representative of HPTE on July 15, 2019 regarding Wall B1-10R and spoke with a representative of the Attorney General’s office on July 16. Our Mr. McOmber visited the site that afternoon (July 16) and met briefly with Mr. Meneghetti. We discussed the events which had occurred during the prior 5 days as the wall failure evolved and asked Mr. Meneghetti to prepare a written summary which is provided in Exhibit J.

- July 11 - A 2-inch deep (sic “vertical offset” based on verbal confirmation with Mr. Meneghetti) crack was discovered in the eastbound pavement, just west of the BNSF bridge. A 2 to 4-inch separation was observed between the bike path and adjacent barrier wall. No cracking or buckling of the ground in front of the wall was noticed.

- July 12 - The pavement crack had spread and displaced vertically. The gap between the bike path and barrier wall widened. Ground penetrating radar was used to evaluate if there was a void below the pavement. The results indicated a void south of the crack (Exhibit K). Some pavement panels were removed.

- July 13 – The roadway continued to move and cracks appeared in the exposed soil where pavement had been removed. Buckling and cracks in soil in front of the wall were seen. A wet area developed below the wall. Movement accelerated through the day. Bowing of the wall was evident, and panels at the BNSF bridge abutment were beginning to “fall out.” CDOT placed fill on the ground surface below the wall in an attempt to prevent additional movement.
July 15 – CDOT completed cutting steel connections between the approach slab and abutment and, shortly thereafter, MSE wall panels fell adjacent to the bridge.

CDOT performed surveys during the wall failure which included Light Detection and Ranging (LIDAR) measurements between July 13 and 27. CDOT surveyors provided data developed from the surveys on July 13, 16, 22 and 27 which allowed us to develop estimates of topography in the area south of the wall which are shown in Appendix A (Figures A-1 through A-4). We used the topography and CDOT data to develop cross-sections which show the progression of wall and ground movement between July 13 and 27 at Stations 2172+50 and 2173 which are shown on Figures A-5 and A-6. Figure 1 shows a comparison of the ground surface topography on July 13 and July 22. As indicated above, fill was placed on the site on July 13 (after the LIDAR survey) which influenced the changes in the ground surface.

CDOT also performed ground-based surveying on July 13 which documented the area where wet soils developed south of the wall. This area occurred between Stations 2172+15 and 2172+95, and elevations 5345 and 5347, and is identified on Fig. A-1, Fig. A-4, and Fig. 1. CDOT also surveyed the locations and elevations of exposed lateral drain outlets (Fig. A-1).

OBSERVATIONS

Our Mr. McOmber visited the site multiple times between July 20 and August 14, 2019 to observe conditions exposed during demolition. The demolition was performed using large excavators to remove the wall panels, straps, and reinforced and retained Class I soils. The northern edge of the excavation was cut in a series of benches which generally exposed the pre-2013 embankment fill, although some Class I materials remained on a few benches. We focused on evaluation of whether it appeared the wall was constructed in substantial conformance with the plans and understanding the potential cause(s) and modes of failure. We also sampled exposed soils and drain materials and measured some straps. We requested CDOT surveyors mark US 36 project
stationing on the center median, which was completed by July 30 and used to document findings. The following sections summarize our observations and sampling.

**Straps**

An intact strap was found during sampling on July 30 at Station 2171+50. We exposed and partially straightened the strap to allow measurement (23 feet, Photos 6 & 7).

On August 1, the re-construction contractor (Kraemer) provided a skid loader and laborers to expose straps near the west end of the wall, west of the failed portion. These straps were attached to existing panels. The number of straps per panel and length of straps were evaluated at Stations 2167+90, 2168+35 and 2168+50. Where the panels were higher than 6 feet above the existing grade in front of the wall and a fall hazard existed, strap length measurements were made by throwing a weighted tape to the back of the wall. Exemplary Photo 8 shows the straps exposed at Station 2167+90; additional photos are in our files.
On August 4, an excavator was used to expose the buried end of a strap at Station 2172+50. The end of the strap was approximately 25 feet from the back of the wall panel.

On August 10, excavation to remove reinforced soil below the bridge had occurred (Photo 8). We pulled an intact strap down from the excavation face; it measured 19 feet (Photos 9 through 11).
On August 8, a test pit was excavated to observe a lateral drain at the face of the wall. The excavation exposed numerous straps (Photo 12).

![Photo 12: Straps exposed in test pit near base of Wall at Station 2172+65 +/-](image)

Based on observations during demolition and our measurements, we believe the straps were installed in substantial conformance with the plans.

**Drains**

We observed three lateral drains along the base of the wall panels when the panels were removed. On August 7, water was encountered at approximate Sta. 2172+65 while Mr. McOmber was not on-site. When he arrived, the area had been backfilled. On August 8, a test pit was excavated to expose the area where water had been encountered. The excavation revealed the end of a lateral drain (Photo 13). The pipe was partially crushed. We estimated the initial flow was about 2 to 5 gallons per minute. No bedding was apparent around the flattened pipe other than Class I material. We obtained a sample of excavated pipe.

![Photo 13: Lateral drain exposed in test pit at base of Wall at Station 2172+65 +/-](image)
Video 1\(^1\) was taken about 15 minutes after the pipe was exposed.

Video 2 was taken about 40 minutes after the pipe was exposed.

A second lateral drain was exposed at approximate station 2171+65 during the afternoon of August 8. We returned to the site and recorded Video 3; very little water was flowing from this drain.

On August 9, a third lateral drain was exposed at approximate Station 2169+60. A trickle of water was observed (Photo 14). We obtained a sample of the pipe.

A second test pit was excavated at the location of the lateral drain at 2172+65 (+/-) on August 13 to attempt to expose the connection of this lateral drain to the toe drain. Considerable water again flowed from the lateral drain as shown in Videos 4 and 5 taken shortly after exposing the pipe and about 1 hour later. The excavation was terminated before exposing the toe drain because of Kraemer’s concern it would jeopardize the stability of the cut slope to the north.

The toe drain was exposed on August 13 at approximate Station 2171+20 (Photo 15) and on August 14 at Station 2169. In both instances, the drain pipe was surrounded by bedding wrapped with geotextile. The pipe was not crushed. We obtained a sample of the pipe, bedding and geotextile at 2171+20.

\(^1\) Videos are provided in a thumb drive provided separately.
Vertical drain strips were exposed during several of our site visits, at spacing of about 10 feet (Photos 16 & 17).

Our observations confirm the drains were generally installed in conformance with the plans. However, the drain design and installation were not sufficient to control water which seeped into the Class I fill during and following construction; in particular the Class I fill added with over-excavation below the wall. We have noted the locations and elevations of the lateral drain pipes surveyed by CDOT on July 13 on Figure A-1, as well as the design elevations of the bottom of the wall and the bottom of Class I fill about 2 feet below the base of the wall which resulted from the design change to increase bearing resistance. The easternmost lateral drain was found at about Station 2173+40 at elevation 5354.1 during the CDOT survey on July 13. The design bottom of the wall stepped down from 5354.1 to 5351.6 at about Station 2173+60, just east of this pipe and the bottom of Class I material stepped down to about 5349.6 or about 4.5 feet below the easternmost lateral drain. If there was no additional drain east of 2173+60, then water which
seeped into the Class I fill near the bridge abutment would collect at the bottom of the Class I fill placed after over-excavation. In addition, the survey data indicate the lateral drains and the toe drain west of Station 2173+40 were installed near the bottom of the wall and at least 2 feet above the bottom of Class I structural fill.

The conditions observed when the lateral drain was exposed at 2172+65 (+/-) during demolition confirmed there was considerable water stored in the Class I material and drain system below the eastern portion of the wall. This is consistent with development of the wet area south of the wall on July 13, 2019 during the failure.

A Field Report from September 2012 (Exhibit L) indicates a Heavy Duty pipe manufactured by Advanced Drainage Systems (ADS) was approved by Ames/Granite for the toe and lateral drains. The drain pipes found during demolition had a green stripe which is indicative of products manufactured by ADS.

Photo 18 shows the two pipe types. The toe drain pipe is labeled as Heavy Duty and ASTM F405. The lateral pipe sample has no labeling. The toe drain pipe has 19 ribs per foot and the lateral pipe has 17. We contacted ADS and requested they identify the pipe. They responded (Exhibit L) and stated they could not identify the lateral pipe because the sample does not have labeling. The toe drain is Heavy Duty pipe, and we suspect the lateral drain may be ADS Highway pipe. ADS literature (Exhibit M) indicates if either of these pipes are surrounded by Class I bedding material as defined by ADS, the maximum burial depth is 41 feet; this applies to the toe drain which was bedded in CDOT Class B bedding. The lateral drains were “bedded” in CDOT Class I structural fill, which corresponds to an ADS Class 3 bedding and ADS shows a maximum recommended burial depth of 19 feet for this bedding condition. The actual burial depths were about 30 to 35 feet in the area of the Wall failure. It is possible the lateral drain pipes crushed prior to wall failure.
Flowable Fill

On August 1, we obtained a sample of the flowable fill which had fallen from the excavation near the bridge abutment. Photo 19 shows the material at the back of a wall panel. Our observations confirm flowable fill was used to attempt to fill voids created by erosion.
Soils

We sampled soils exposed during demolition on several occasions, including Class 1 structure fill and the pre-2013 embankment materials. In most instances field moisture/density tests were performed during sampling. Results of the field and laboratory tests are summarized in Appendix C. Our observations confirmed Class I materials were used for the reinforced zone and the retained soils between the reinforced zone and the cut slope into the pre-2013 embankment, as indicated on plans. The cut slope was not benched into the pre-2013 embankment. The Class I materials appeared wetter at the base and near the bridge abutment. For example, Photo 20 shows the benched cut near the west portion of the wall, at the interface between the Class I material (left portion of photo) and pre-2013 embankment soils (right portion of photo). The lower edge of the Class I is visibly darker (wetter) than the overlying Class I materials. The Class I material near the abutment is shown on Photo 22.

![Photo 20: Cut slope at interface between Class I and pre-2013 embankment (Station 2169 +/-)](image)

During the morning of August 9, 2019 failure of the excavation slope occurred on the transition between the 7th and 8th excavation benches. We were provided Video 6 which shows a portion of the failure at approximate Station 2172+95. As the video shows, a smooth surface results after the ground movement, which is referred to as a slickenside. We understand the slickensided failure surface extended for over 200 feet.

We visited the site later on August 9 and photographed a small portion of the failure area (Photo 21). The failure occurred in the pre-2013 embankment material just below the contact with Class I fill, which can be seen in Photo 21. Photo 22 shows the exposed contact in the excavation face just west of the BNSF bridge abutment. The failure angle and the angle of the contact were both measured at about 36 to 38 degrees. We believe this failure surface was formed when the retaining wall failed, prior to demolition.
During site visits on August 13 and 14, the contractor was excavating to remove straps along the alignment of the drilled piers planned for the wall replacement. We observed the elevation of the lowest level of straps in the excavation rose rapidly at about Station 2171+20 up to the approximate pre-failure bottom elevation of the reinforced zone. We believe this corresponds with the western limit of deep-seated wall/slope failure which is consistent with the topography of the failed pavement on Fig. 1.

**Storm Sewer and Waterline**

Ames/Granite installed a 24-inch storm sewer and 16-inch raw water line below Wall B1-10R. Video examinations of the storm sewer conducted on July 31, 2019 revealed a fracture in the sewer pipe about 93 feet north of the outlet. The videos indicate the pipe south of the break had displaced downward. Water was present in the pipe from about 55 to 93 feet north of the outlet. The location of the sewer and break are shown on Figures 1 and A-1 through A-4. The break location corresponds well with the failed pavement area. We believe that as the wall east of the pipe settled and slid, it pulled the wall system west of the failed area,
pressure on the pipe increased and the pipe broke. We do not believe the broken storm sewer pipe was a source of water which caused failure of the wall.

The water line was pressure tested on August 8 and 9 (email from Kurt Kionka, CDOT, Exhibit M). The test did not indicate a leak.

FIELD AND LABORATORY EVALUATION

A representative of CTL | Thompson was present while eight exploratory borings were drilled by CDOT between August 6 and 9, 2019. Samples obtained during this drilling were split with another consultant acting on behalf of CDOT, and our representatives. We also drilled one additional boring on September 11. Drilling locations are shown on Figure 1 and logs of the borings are shown in Appendix B. Figures 2 through 4 show cross-sectional views. Borings B-1 through B-4 and B-8 were drilled on the excavation surface during demolition. B-5 through B-7 were drilled on the pavement near the center median barrier. Drill rig access was not permitted east of B-5 due to safety concerns. B-10 was drilled on the construction pad near the elevation of the bottom of the replacement wall.

Samples of soil and bedrock were obtained during drilling at intervals of 2 to 5 feet by driving modified California sampling barrels (2.5-inch O.D.) using 140-lb automatic hammers falling 30 inches. A few samples were obtained by pushing 3-inch Shelby tubes. Borings B-1 through B-4 and B-8 were backfilled after drilling either at the completion of drilling or the following morning. Boring B-9 was sampled with a continuous core barrel and we did not log this boring.

The materials encountered during drilling of B-1 through B-4 and B-8 consisted of 11 to 24 feet of medium dense to dense Class I fill materials underlain by pre-2013 embankment fill or by clay, and then by weathered and relatively unweathered claystone bedrock. Presence of groundwater was indicated at about el. 5346 in B-4 during drilling. The clay and embankment fill were medium stiff to very stiff.
Embankment fill was found extending to depths of 35 to 39 feet in B-5 through B-7, underlain by 7 to 12 feet of clay and then by weathered and relatively unweathered claystone. The embankment fill and clay were medium stiff to very stiff. Groundwater was indicated at about el. 5352 during drilling (wet sample barrel) in B-5, and was measured at 5345 one day after drilling and 5349 on September 11. Water was measured in B-6 at 5343 on September 11.

Our laboratory testing focused on evaluation of strength for the embankment fill and clay. Direct shear tests were performed at a slow shearing rate to provide an indication of shear strength. Strength testing was also performed on the bedrock materials and Class 1 fill. We performed soil classification tests to allow use of estimates of fully softened and residual shear strength based on plasticity and percent clay fines (< .002 mm) using correlations available from Dr. Timothy Stark (www.tstark.net). Samples tested for soil classification were not broken down with a ball mill. Results of laboratory tests are provided in Appendix C.

The laboratory tests showed that most of the Class I fill contained slightly more fines (particles passing the No. 200 sieve, 0.75mm) than specified by CDOT. Our observations indicate the fines were micaceous (Exhibit N). We judge there could have been some crushing of materials during compaction, which led to higher fine content. Moisture content data confirmed the lower portion of the Class I fill and the portion near the abutment were more moist than other portions of the Class I fill.

**ANALYSIS**

There are four mechanisms of failure which should be considered for external stability during design of a retaining wall (FHWA-NHI-10-024) which include:

- Sliding on the base
- Limiting eccentricity (formerly known as overturning)
- Bearing resistance
- Overall stability (global slope stability)
The following image illustrates the first three mechanisms

There are also instances where local shear and punching shear should be considered, particularly when a wall is bearing on a weak cohesive (clayey soil) layer. A special case of local shear, “squeeze,” can occur when the reinforced mass is underlain by a relatively thin, softer clay layer and this clay layer is underlain by a substantially stiffer layer. We believe this condition was present or developed below Wall B1-10R.

The analyses are based on geometry of the wall and supporting soil strata, groundwater conditions and soil strength. Our observations of photos and data obtained during failure and demolition of Wall B1-10R, coupled with soil and groundwater conditions found during drilling, lead us to believe failure of the retaining wall was caused by wetting and movement within the pre-2013 embankment fill and underlying native clay, with consequent loss of strength. The wetting occurred because the drain system below the wall did not control water which penetrated behind the wall during and after construction. Water collected in the Class I materials and the drain system, and most notably in the Class I material placed below the drain after over-excavation. We believe the wall likely settled and slid initially as the foundation soils (clay or embankment fill) failed.
in local shear or squeeze, which progressed to global failure. If the movement observed during construction (settlement indicated on Exhibit H) resulted from displacement on the upper portion of a failure surface at or just below the interface between Class I fill and the pre-2013 embankment (excavated slope), it would cause a reduction in shear strength. When the wall failed, additional, progressive movement occurred on this surface, as indicated in Photo 23, the strength of the embankment fill and clay declined, and movement accelerated. Water emerged when the failure surface reached the ground surface south of the wall. The wall settled and moved south. Water which had collected in the drain system and free water in the Class I material flowed to the low point of the failed wall, near Station 2172+65, where significant water was found during demolition. We believe the failure occurred due to loss of external stability, not internal failure of the wall system.

Our retaining wall design practice typically involves use of fully softened strength. Although cohesion is theoretically zero for softened strength of cohesive soils, some minor cohesion is often assumed. With significant movement a lower, residual strength develops where no cohesion is present. “If local shear or punching shear is possible, Section 10.6.3.1b of AASHTO (2007) requires the use of reduced shear strength parameters for calculating the nominal bearing resistance. The reduced effective stress cohesion, \( c^* \) is set equal to 0.67c’. The reduced effective stress soil friction angle, \( \Phi^* \) is equal to \( \tan^{-1}(0.67\tan\Phi') \)” (FHWA-NHI-10-024).
We analyzed the stability of the wall using several methods. As-built topography was not available. We based geometry for our analyses on the 2013 Shop Drawings submittal (Exhibit A) and the 2015 wall cross-sections (AB-RD-CW-1_B1 X-Secti-tons.pdf). The section at Station 2172+50 was selected because it is near the center of the toe bulge created during failure, the location where significant water was found in the lateral drains, and the center of the wet area which developed on the slope on July 13, 2019.

**Overall Stability Analysis**

Overall (a.k.a. global) stability analysis of slopes and retaining walls is typically performed using computer models. We used the SlopeW program and the Morgenstern Price method. This analysis focuses on calculation of a factor of safety. In simple terms, the factor of safety is the ratio of forces which cause failure to the forces which resist failure. A factor of safety (FOS) less than 1.0 implies failure is likely. Some movements can occur as the FOS approaches 1.0 (i.e. less than about 1.2). Because of uncertainties in the soil profile, soil strength and density, groundwater levels, and slope geometry, design for a minimum FOS of 1.5 is normal for critical structures and 1.3 for non-critical structures.

Results of our global stability evaluation are provided in Appendix D. We began the global stability evaluation by repeating the analysis performed by the geotechnical design firm for the reported Station 2168+50 geometry to verify whether we could replicate their results. We used the reported strength parameters and calculated a FOS of 1.6 for the drained soil strength conditions, which agrees with the design firm’s results (FOS 1.6 to 1.7). We then used the design parameters and the geometry at Station 2172+50 and calculated a FOS of 1.6 for a drained condition.
We performed direct shear tests on soil samples obtained during drilling and on remolded samples of soils obtained during excavation as an indicator of shear strength. Residual strength can be estimated by failing a sample and retesting after cycling the sample back and forth along the shear plane. Our direct shear device does not allow cycling. We cut samples along the shear plane which resulted from initial testing with a wire saw and tested along the cut plane to evaluate residual strength. We also performed classification tests which were used to estimate strength using Stark’s Excel tool. The results are detailed in Table C-I (Appendix C). The estimates of fully softened strength ranged from phi=23 to 27 degrees for the embankment fill and clay and the measured values from large displacement ranged from 13 to 35 degrees. Apparent cohesion of 0 to 770 psf was measured at large displacement. Estimates of residual friction ranged from phi=14 to 21 degrees. Residual values of phi=9 to 34 degrees were measured on the cut samples, with one showing apparent cohesion of 1410 psf (with phi=9 degrees). We relied more heavily on the estimates from classification testing and Stark’s correlations in our analyses.

SlopeW allows several approaches for defining the shape of the global stability failure surface. It is possible to search numerous circular surfaces to identify the surface which results in the lowest computed factor of safety or specify a failure surface. We used the LIDAR survey data and observations during demolition to estimate a probable failure surface which was linear just below the interface (excavation slope) between Class I structural fill and the pre-2013 embankment fill and transitioned to a quasi-circular surface in the clay. Previous trial runs using the search capabilities of SlopeW indicated a critical failure circle which was very similar (very shallow within the embankment fill).

We performed parametric evaluation of slope stability by varying the drained strength of the pre-2013 embankment fill and native clay, and groundwater conditions for the section at Station 2172+50. We typically recommend fully softened design values on the order of phi=22 to 25 degrees and cohesion of 100 to 200 psf for soils similar to the embankment fill and clay. We varied phi’ from 16 to 28 degrees and used c’=0 and...
c’=200 psf. We do not believe the failure surface entered the weathered bedrock or bedrock, and did not vary strength parameters for these materials. Unit weight was also held constant for all materials. The analyses were performed assuming no groundwater and water at elevation 5347 which was the level where water emerged on the slope on July 13, 2019. Results are provided in Appendix D and summarized in Table D-1. Plots of the results are shown on Figures 11-14 to illustrate sensitivity.

The slope stability calculations show that a satisfactory FOS of 1.5 or greater could be achieved assuming phi’ greater than 24 degrees in the clay and embankment fill, with c’=200 psf and no water. A FOS of 1.5 or greater could be achieved with phi’ greater than 26 degrees and c’=200 psf with water at 5347. When wetting of the upper surface of the embankment fill and the clay layer and some displacement occurred, cohesion reduced and was eventually lost. For c’=0 psf, there were no friction combinations within the range analyzed which resulted in a FOS of 1.5 or greater. With water and no cohesion, the data indicate the wall would fail (FOS =1.0) if the fill and clay had softened strength of about phi’=22 degrees. The slope stability calculations demonstrate Retaining Wall 1B-10R was not stable once wetting and softening of the pre-2013 embankment fill and clay below the Class I fill occurred and movement was sufficient to cause fully softened strength to develop.

Analysis of Bearing, Eccentricity and Sliding

Design of MSE walls like Wall B1-10R for Bearing and Sliding is prescribed in FHWA-HI-10-024. The LRFD method is used. The traditional method has been allowable stress design. Unit weight, soil strength parameters and assumptions regarding how the reinforced soil mass performs are common to both methods. Application of factors to the variables in the LRFD approach is the principal difference. The design goal is for the factored resistance forces to equal or exceed the factored driving forces; in other words the ratio of factored resistance to factored driving forces should be 1.0 or greater. The end results should be similar in both methods. Both design methods require
evaluation of external and internal stability of the soil mass. Our analysis focused only on external stability.

We performed sliding, eccentricity and bearing resistance analyses per Chapter 4 of FHWA-NH-0-024 using hand calculations which are provided in Appendix E. These calculations were subsequently checked using a spreadsheet. The calculations were done using the cross-section shown on page 2 of the calculations. The height used in the calculations was 32.5 feet per the revised design.

The 2012 geotechnical design report includes results of global stability analyses using SlopeW. The documents indicate the global stability was evaluated using criteria of 26 degrees friction for the foundation soil without cohesion. A unit weight of 125 pcf for the reinforced fill was used for analysis of sliding, eccentricity and pull out calculations. A unit weight of 140 pcf was used for bearing capacity and rupture calculations by the wall designer. We used 140 pcf as the unit weight for the reinforced fill and retained Class I soil in our calculations based on measurements in field tests after the failure. We also used 24 degrees friction for the foundation soils based on our experience and results of our laboratory tests.

We evaluated three scenarios including the initial design, the revised design and post-failure. The design was revised because relocation of the bike path resulted in a taller wall. The field design change resulted in 2 feet of Class 1 structural fill and 2 feet of impermeable fill placed immediately below the reinforced zone. The purpose of this change was to increase the spread of load from the reinforced zone and reduce the load intensity on the clay layer. This was checked in our post-failure analysis.

For the design using the geotechnical report data, our calculations for sliding indicated a ratio of factored resisting forces to driving forces of 0.92. We used 140 vs 125 for unit weight of the Class 1 fill, however the unit weight is used to calculate both resisting and driving forces so the ratio would remain the same if the lower value was used. Using unfactored values, forces resisting sliding divided by forces driving (a.k.a. FOS)
=1.39; normally a minimum factor of 1.5 is desired for safety. These calculations indicate a design goal less than desired stability for sliding. The coefficient of friction for sliding used in the calculations was 0.25 which was stated in the geotechnical design report for preliminary design of cast in place concrete retaining walls. A friction coefficient of 0.27 would be sufficient to indicate satisfactory design stability for sliding (0.27 is equivalent to the tangent of a friction angle of about 14 degrees).

When the wall was re-designed for the additional height, the geotechnical engineer provided increased values for the factored bearing resistance based on strap length. Our bearing calculation indicated a factored value of 8.04 ksf. The calculation implies a satisfactory value for the revised factored bearing resistance.

The post-failure analysis was made using strength values based on laboratory tests and correlation using strength data published by Stark. The strength of the Class 1 fill was reduced to the maximum allowed in Chapter 4 of FHWA-NH-0-024, phi=34 degrees, which was one value measured in our laboratory. Our calculations indicate less stability in sliding is lower than the design goal; the ratio of resisting forces to driving forces is 0.78 based on a sliding coefficient of 0.25. A coefficient of 0.32 would indicate satisfactory design sliding stability (equivalent to tangent of about 18 degrees). Our evaluation of bearing indicates a value of 8.36 ksf (factored); the wall design plans indicate 8.39 ksf. Using the bearing capacity equation from Chapter 4, the bearing value with c'=100 psf is 8.40 ksf and using c'=200 the value is 9.66 ksf which indicates satisfactory bearing conditions.

We also performed spreadsheet calculations to check whether the combinations of cohesion and friction in the embankment fill and clay used by the design geotechnical engineer and wall designer satisfied the design goal using the LRFD methods. All three combinations (phi'=30 degrees, c'=500; phi'=26, c'=400; and phi'=26, c'=0) resulted in satisfactory design.
If the angle of internal friction of the foundation soils is reduced due to potential squeeze conditions, \( \phi' = 24 \) degrees is reduced to about 18 degrees. For this value, the design goal is not met for bearing failure by squeeze.

The intensity of the load imposed on the clay is reduced by increasing the effective bearing width through the added 2 feet of Class 1 materials and 2 feet of impermeable fill. Assuming the load is spread at 1:2 (horizontal to vertical) below the loaded area, the stress would be about 5.04 ksf at the bottom of the impermeable layer. It would reduce to 4.77 ksf if the surcharge load is ignored. It is not unusual to use unconfined compressive strength as an allowable bearing value. The average unconfined strength of clay samples in our tests was 4.86 ksf. For the samples with comparatively high moisture content, the average was 3.30 ksf. These values indicate loss of bearing was very likely part of the failure. Calculations indicate an unconfined strength of at least 3032 psf (3.03 ksf) is necessary to prevent squeeze.

**Finite Element**

Finite element analysis can be used to simulate deformation of a slope or retaining wall based on loading and potential changes in the stiffness of supporting materials. We used the SIGMA-W computer model to evaluate how deformation of Wall B1-10R occurred as the pre-2013 embankment fill and native clay softened due to wetting. Our intent was not to precisely depict actual movement. Rather, it was to provide a visual tool to aid in understanding of the early stages of wall deformation.

Stiffness (elastic modulus) values used in the analysis were obtained from unconfined compression tests. Figures 5 and 6 show how measured unconfined strength and modulus varied based on moisture content for samples of the embankment fill and clay. As moisture content increased, sample strength and stiffness decreased. We used modulus values for comparatively dry samples of fill and clay measured at 1% strain to simulate deformation which could have occurred during and shortly after the wall was constructed (Figures 7 and 8); about 1 inch of settlement and 3.6 inches of lateral
movement is depicted. Some (or much) of this movement would have occurred as the wall was constructed. Modulus values obtained at peak strength for comparatively wet samples were used to simulate deformation which could have occurred early in the wall failure process (Figures 9 and 10); about 2.4 inches of total settlement and 14 inches of total horizontal movement is depicted. The difference in the results compare reasonably with the deformation observed early during the wall failure (July 11-14). The FEM analysis indicates that as the foundation soils softened, a bulge would develop in the soil in front of the wall (squeezing). This occurred, as shown in Photo 24.

![Photo 24: Bulge at toe of wall (July 14, source: David Thomas, CDOT)](image)

DISCUSSION AND OPINIONS

The observations during failure and demolition of Wall B1-10R, coupled with soil and groundwater conditions found during drilling and indicated in laboratory tests, indicate failure of the retaining wall was caused by wetting, softening and movement of the pre-2013 embankment fill and clay, with consequent loss of strength. This ultimately resulted in global stability failure. We offer the following summary and opinions.

1. Wall B1-10R was subjected to flooding during September 2013 and possibly later, prior to paving.
2. The drain system designed and installed with Wall B1-10R was not sufficient to remove water behind and below the wall. The purpose of the geocomposite strip drain system was to control water which seeps below the pavement after construction. In this instance, the vertical strip drains likely transmitted flood water and subsequent precipitation to the Class I material below the wall. This could also have occurred with sloping strip drains. We found no indication the drain was lowered when the wall was redesigned with two additional feet of Class I fill. Survey data indicate the eastern-most lateral drain was about 3 feet above the base of the wall east of Station 2173+60, and 5 feet above the base of Class I fill on July 13, 2019. Lateral drains and the toe drain west of 2173+60 were also above the base of Class I fill added to increase bearing resistance. The drain could not remove water which penetrated to the base of the Class I fill.

3. Surface water on the unpaved area above the reinforced earth wall caused erosion and infiltration during heavy rainfall and flooding in September 2013. Project records indicate the fill surface was exposed prior to September 2013 and to at least December 2013 when the impermeable membrane was placed (or April 2014 when paving occurred). A portion of the surface was not paved after April 2014. Water penetration likely occurred after the September 2013 event. Surface drainage was not maintained to direct water off the fill surface and away from the bridge and vertical drains.

4. We found no indication that global stability was re-evaluated when the wall height was increased to accommodate the bike bath.

5. Water collected above the base of the Class I fill. Over time, the underlying clay and surface of the pre-2013 embankment fill softened and displaced. There is some indication some movement (settlement) occurred during construction. The soil strength when failure occurred was less than assumed in the design. Presence of water also contributed to lower frictional resistance to movement. Once sufficient soil/wall deflection occurred, the strength of the clay and surface of the embankment fill reduced to fully softened or residual values, and the wall failed globally.

6. The drain pipe bedding conditions used for the lateral drains were not consistent with manufacturer’s recommendations. Crushing of the lateral pipe could have contributed to failure of the drain system.

7. When the wall settled and slid south, the lateral drain at Station 2172+65 (+/-) became the low point in the drain system. Significant water was found at this location during demolition.
8. Failure of the storm sewer pipe likely occurred due to movement of the wall system. As the wall east of the pipe settled and slid, it pulled the wall west of the failed area, pressure on the pipe increased and the pipe broke. We do not believe the broken storm sewer pipe was a source of water which caused failure of the wall.

CREDITS

The investigation by CTL/Thompson, Inc. required access to details regarding design, construction, and subsequent failure of Wall B1-10R. HPTE and CDOT employee assistance with locating and obtaining documents regarding design and construction of the wall, survey data, and timelines of construction and wall failure was critical to our understanding and analysis.

Kraemer allowed access to the site and helped during some sampling. Their cooperation on a time-sensitive project is appreciated.

Ronald M. McOmber P.E. is the primary author of this report and directed our efforts. Robert W. Thompson, P.E. reviewed the report and supporting documents and performed analysis of external stability. John Mechling, P.E. completed global stability analysis and finite element modeling. Other employees performed laboratory testing, field observations, sampling, and compilation of data.

CLOSING

This report was prepared based on limited data. There may be documents regarding the wall design and construction which were not provide to us, and there is certainly data from forensic investigations conducted by others. The additional data and
analyses by others, including competing opinions, could result in changes to our understanding of conditions, analysis, and opinions.

Respectfully,

CTL | THOMPSON, INC.

Ronald M. McOmber, P.E., Chairman, Sr. Principal

RMM/nn

Enclosures

Via e-mail: Andrew.Gomez@coag.gov

Ronald M. McOmber

Jul 8 2021
LEGEND:

- ESTIMATED TOPOGRAPHY 7-13-19
- ESTIMATED TOPOGRAPHY 7-22-19
- B-1 LOCATION OF EXPLORATORY BORINGS

NOTE: THIS FIGURE WAS CREATED USING DATA PROVIDED BY CDOT.

COMPARISON OF 7-13-19 AND 7-22-19 TOPOGRAPHY
Unconfined Compressive Strength vs. Moisture Content
(For Embankment Fill and Clay)
Elastic Modulus (at 2% Strain) vs. Moisture Content
(For Embankment Fill and Clay)
US 36 Retaining Wall B1-10R
Sta. 2172+50 Load Deformation
Dry condition
Vertical displacement (feet)

Modulus Values

Name: Embankment fill
Model: Linear Elastic
Young's Modulus (E): 136000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Name: Clay
Model: Linear Elastic
Young's Modulus (E): 239000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Note: Modulus values were estimated from unconfined compression tests at 1% strain for comparatively dry samples.
US 36 Retaining Wall B1-10R
Sta. 2172+50 Load Deformation
Dry condition
Horizontal displacement (feet)

Modulus Values

Name: Embankment fill
Model: Linear Elastic
Young's Modulus (E): 136000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Name: Clay
Model: Linear Elastic
Young's Modulus (E): 239000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Note: Modulus values were estimated from unconfined compression tests at 1% strain for comparatively dry samples.
US 36 Retaining Wall B1-10R
Sta. 2172+50 Load Deformation
Moist condition
Vertical displacement (feet)

Modulus Values

Name: Embankment fill
Model: Linear Elastic
Young's Modulus (E): 52000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Name: Clay
Model: Linear Elastic
Young's Modulus (E): 33000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Note: Modulus values were estimated from unconfined compression tests at peak strength for comparatively moist samples

Project No. DN50165-149

Fig. 9
US 36 Retaining Wall B1-10R
Sta. 2172+50 Load Deformation
Moist condition
Horizontal displacement (feet)

Modulus Values

Name: Embankment fill
Model: Linear Elastic
Young's Modulus (E): 52000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Name: Clay
Model: Linear Elastic
Young's Modulus (E): 33000 psf
Unit Weight: 124 pcf
Poisson's Ratio: 0.49

Note: Modulus values were estimated from unconfined compression tests at peak strength for comparatively moist samples

Project No. DN50165-149

Fig. 10
Slope Stability Sensitivity - No Water
Clay and Embankment Fill c=0

Factor of Safety vs. Clay phi (degrees)

- Fill phi=16
- Fill phi=20
- Fill phi=24
- Fill phi=28
Slope Stability Sensitivity - Water
Clay and Embankment Fill c=0

Factor of Safety vs Clay phi (degrees)

- Fill phi=16
- Fill phi=20
- Fill phi=24
- Fill phi=28
Slope Stability Sensitivity - No Water
Clay and Embankment Fill c=200
Slope Stability Sensitivity - Water
Clay and Enbankment Fill c=200

Clay phi (degrees)

Factor of Safety

Fill phi=16
Fill phi=20
Fill phi=24
Fill phi=28
LEGEND:
- ESTIMATED TOPOGRAPHY 7-13-19
- ESTIMATED TOPOGRAPHY 7-22-19
- LOCATION OF EXPLORATORY BORINGS

NOTE: THIS FIGURE WAS CREATED USING DATA PROVIDED BY CDOT.

COMPARISON OF SETTLED PAVEMENT AREA TO FAILURE
APPENDIX A

TOPOGRAPHIC ESTIMATES FROM LIDAR
NOTE: THIS FIGURE WAS CREATED USING DATA PROVIDED BY COOT.

ESTIMATED TOPOGRAPHY 7-16-19
NOTE: THIS FIGURE WAS CREATED USING DATA PROVIDED BY CDOT.

ESTIMATED TOPOGRAPHY 7-27-19
APPENDIX B
LOGS OF EXPLORATORY BORINGS
### FIELD EXPLORATION

<table>
<thead>
<tr>
<th>Surveyed Elevation (Feet)</th>
<th>Graphic Log</th>
<th>Depth (Feet)</th>
<th>CLASS I STRUCTURAL FILL - SAND, SILTY, SLIGHTLY CLAYEY WITH SOME GRAVEL, MEDIUM DENSE TO DENSE, SLIGHTLY MOIST, DARK BROWN, BLACK, GRAY.</th>
<th>EMBANKMENT FILL - CLAY, SANDY, MEDIUM STIFF, SLIGHTLY MOIST, BROWN, TAN, WHITE, RUST.</th>
<th>CLAY, SANDY, STIFF TO VERY STIFF, SLIGHTLY MOIST, BROWN, TAN, WHITE (CL).</th>
<th>WEATHERED CLAYSTONE, SLIGHTLY MOIST, BROWN, GRAY, OLIVE, RUST.</th>
<th>BEDROCK, CLAYSTONE, MEDIUM HARD TO HARD, SLIGHTLY MOIST, GRAY, DARK GRAY, RUST.</th>
<th>Bottom of borehole at 41 feet.</th>
<th>INDICATES SAMPLE OBTAINED WITH MODIFIED CALIFORNIA BARREL.</th>
<th>INDICATES SAMPLE OBTAINED WITH SHELBY TUBE.</th>
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<tr>
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<td>NORTHING: 497349.59</td>
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### LABORATORY RESULTS

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<th>Sample Type</th>
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<th>UNCORR. UNCORR. BLOW SG IN.</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Plasticity Index</th>
<th>Unconfined Compressive Strength (psf)</th>
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**Boiling Date:** 08/09/19

**EQUIPMENT:** CME-55 ATV

**DRILL COMPANY:** CDOT

**LOGGED BY:** GE

**PROJECT NAME:** HWY-36

**CTL|T PROJECT NO:** DN50,165-149
## Field Exploration

<table>
<thead>
<tr>
<th>Surveyed Elevation (Feet)</th>
<th>Graphic Log</th>
<th>Depth (Feet)</th>
<th>Sample Type</th>
<th>Bow Count (BC)</th>
<th>Plasticity Index</th>
<th>Unconfined Compressive Strength (psf)</th>
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Bottom of borehole at 41 feet.

- **INDICATES SAMPLE OBTAINED WITH MODIFIED CALIFORNIA BARREL.**
- **INDICATES SAMPLE OBTAINED WITH SHELBY TUBE.**

## Laboratory Results

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Plasticity Index</th>
<th>Unconfined Compressive Strength (psf)</th>
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**BORING LOG:** B-2  
**CDOT:**  
**LOGGED BY:** GE  
**METHOD:** HOLLOW-STEM  
**EQUIPMENT:** CME-55 ATV  
**HAMMER TYPE-DROP:** 8-INCHES  
**DRILLING DATE:** 08/09/19  
**PROJECT NAME:** HWY-36  
**PROJECT NO:** DN50,165-149  
**FIG. B-2**
### Field Exploration

**Class I Structural Fill - Sand, Silty, Slightly Clayey with Some Gravel, Medium Dense to Very Dense, Slightly Moist to Moist, Dark Brown, Black, Gray, Greenish-Gray.**

**Embankment Fill - Clay, Sandy to Very Sandy, Some Gravel, Medium Stiff to Very Stiff, Slightly Moist, Olive, Brown, Gray.**

**Clay, Sandy, Some Gravel, Stiff to Very Stiff, Slightly Moist to Moist, Brown, Gray, White, Olive (Cl).**

**Weathered Claystone, Slightly Moist, Gray, Olive, Rust.**

**Bedrock, Sandstone, Silty, Slightly Clayey, Hard, Slightly Moist, Gray, Rust, White.**

**Bedrock, Claystone, Medium Hard, Slightly Moist, Olive, Gray, Rust.**

Bottom of borehole at 51 feet.

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**Laboratory Results**

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<th>Sample Type</th>
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<th>Moisture Content (%)</th>
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* Indicates blow-count not representative of soil conditions, drove through reinforcement strap.

**Indicates sample obtained with modified California barrel.**

**Indicates sample obtained with Shelby tube.**

---

**CTL/THOMPSON, INC.**

**PROJECT NAME:** HWY-36

**CTL/T PROJECT NO:** DN50,165-149

**FIG. B-3**
**FIELD EXPLORATION**

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<th>Depth (Feet)</th>
<th>Sample Type</th>
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<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Plasticity Index</th>
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</table>
**FIELD EXPLORATION**

- **EASTING:** 620145.47
- **NORTHING:** 497118.49
- **SURVEYED SURFACE:** 5387.5
- **ELEVATION (ft):** 2172+00

**LABORATORY RESULTS**

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>UNCORR. BLOWS/IN.</th>
<th>Sample No.</th>
<th>Blowing Count (BC)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Percent Passing No. 200 Sieve (%)</th>
<th>Plasticity Index</th>
<th>Unconfined Compressive Strength (psf)</th>
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<tbody>
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<tr>
<td>INDICATES SAMPLE OBTAINED WITH MODIFIED CALIFORNIA BARREL.</td>
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<tr>
<td>INDICATES SAMPLE OBTAINED WITH SHELBY TUBE.</td>
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</table>
### Field Exploration

- **9-7/8 INCHES CONCRETE.**
  - Embankment Fill - Clay, Sandy to Very Sandy, Contains Bedrock Fragments, Medium Stiff to Stiff, Slightly Moist to Moist, Brown, Olive, Gray, Rust, Tan, Dark Brown, White, Wood Debris at 35'.

- **CLAY, SANDY, VERY STIFF, SLIGHTLY MOIST, BROWN, DARK BROWN, WHITE (CL).**

- **WEATHERED CLAYSTONE, SLIGHTLY MOIST, GRAY, RUST.**

- **BEDROCK, CLAYSTONE, MEDIUM HARD TO HARD, SLIGHTLY MOIST, GRAY, DARK GRAY, RUST, BROWN, OLIVE, TAN.**

  * Indicates sampling was compromised due to lost Shelby tube.

- **Bottom of borehole at 71 feet.**

### Laboratory Results

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Blow Count (BC) UNCORR. BLOWS/6 IN.</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Peaking No. 200 Sieve (%)</th>
<th>Plasticity Index</th>
<th>Unconfined Compressive Strength (psf)</th>
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<tbody>
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<td>3/5</td>
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<td>4/4</td>
<td>5/6</td>
<td>18.5</td>
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<tr>
<td>4/7</td>
<td>5/10</td>
<td>18.5</td>
<td>50</td>
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<td>1,700</td>
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<td>5/6</td>
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<td>5/10</td>
<td>18.5</td>
<td>50</td>
<td>34</td>
<td>20</td>
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<td>1,700</td>
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</table>

* Indicates sample obtained with modified California barrel.

Bottom of borehole at 71 feet.
9-7/8 INCHES CONCRETE.

EMBANKMENT FILL - CLAY, SANDY TO VERY SANDY, CONTAINS BEDROCK FRAGMENTS, MEDIUM STIFF TO STIFF, SLIGHTLY MOIST TO MOIST, BROWN, OLIVE, GRAY, RUST, TAN, DARK BROWN, WHITE, WOOD DEBRIS AT 35'.

CLAY, SAND, VERY STIFF, SLIGHTLY MOIST TO MOIST, DARK BROWN, BROWN, OLIVE (CL).

WEATHERED CLAYSTONE, SLIGHTLY MOIST, GRAY, OLIVE, RUST.

BEDROCK, CLAYSTONE, MEDIUM HARD TO HARD, SLIGHTLY MOIST, GRAY, DARK GRAY, RUST, BROWN, OLIVE, TAN.

Bottom of borehole at 70 feet.
### Field Exploration

**Sample Type**
- EMBANKMENT FILL - CLAY, SANDY, CONTAINS GRAVEL AND BEDROCK FRAGMENTS, STIFF, SLIGHTLY MOIST TO MOIST, BROWN, REDDISH-BROWN, WHITE.
- EMBANKMENT FILL - SAND, SILTY, CLAYEY, MEDIUM DENSE, SLIGHTLY MOIST, BROWN, TAN, WHITE.

**Surveyed Elevation (Feet)**
- 5380
- 5370
- 5360
- 5350
- 5340
- 5330

**Graphic Log**

**Depth (Feet)**
- 10.5 INCHES CONCRETE.
- EMBANKMENT FILL - CLAY, SANDY, CONTAINS GRAVEL AND BEDROCK FRAGMENTS, STIFF, SLIGHTLY MOIST TO MOIST, BROWN, REDDISH-BROWN, WHITE.
- EMBANKMENT FILL - SAND, SILTY, CLAYEY, MEDIUM DENSE, SLIGHTLY MOIST, BROWN, TAN, WHITE.
- EMBANKMENT FILL - CLAY, SANDY, CONTAINS GRAVEL AND BEDROCK FRAGMENTS, STIFF, SLIGHTLY MOIST TO MOIST, BROWN, REDDISH-BROWN, WHITE, OLIVE, DARK BROWN, RUST, GRAY.

**Surveyed Surface Elevation (ft):** 2169+00

**Surveyed Elevation (Feet):**
- SURVEYED SURFACE 5386.1

**Laboratory Results**

**Laboratory Results**

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Blow Counts (BC)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Uncorr. Blows/6 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Clay, Sandy, Stiff, Moist, Brown, Olive (Cl)</strong></td>
<td>3/6</td>
<td>15.8</td>
<td>110</td>
<td>9,200</td>
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<tr>
<td><strong>Clay, Sandy, Sandy, Very Hard, Slightly Moist, Gray, Dark Gray, Rust, Brown, White</strong></td>
<td>5/9</td>
<td>15.7</td>
<td>115</td>
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<td><strong>Bottom of borehole at 61 feet.</strong></td>
<td>50/4</td>
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**Hammer Type-Drop:**
- CDOT HOLLOW-STEM

**Drill Company:**
- CDOT

**Equipment:**
- CME-55 ATV

**Hammer Type-Drop:**
- 8-INCHES 140LB-AUTOMATIC-30"
### FIELD EXPLORATION

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Surveyed Elevation (Feet)</th>
<th>Graphic Log</th>
<th>Depth (Feet)</th>
<th>Sample</th>
<th>BLOW COUNTS (BC)</th>
<th>UNCORR. BLOWS/6 IN.</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Plasticity Index</th>
<th>Unconfined Compressive Strength (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLASS I STRUCTURAL FILL - SAND, SILTY, SLIGHTLY CLAYEY WITH SOME GRAVEL, MEDIUM DENSE, SLIGHTLY MOIST, DARK BROWN, BLACK, GRAY, GREENISH-GRAY.</td>
<td>360</td>
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<tr>
<td>EMBANKMENT FILL - CLAY, SANDY, CONTAINS BEDROCK FRAGMENTS, MEDIUM STIFF TO STIFF, SLIGHTLY MOIST TO MOIST, BROWN, DARK BROWN, GRAY, RUST, TAN, WHITE.</td>
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<td>CLAY, SANDY, STIFF TO VERY STIFF, SLIGHTLY MOIST TO MOIST, BROWN, OLIVE, DARK BROWN, RUST (CL).</td>
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<tr>
<td>WEATHERED CLAYSTONE, SLIGHTLY MOIST, BROWN, GRAY, OLIVE, RUST.</td>
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<td>BEDROCK, CLAYSTONE, MEDIUM HARD TO VERY HARD, SLIGHTLY MOIST, BROWN, GRAY, DARK GRAY, RUST.</td>
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Bottom of borehole at 46 feet.

- INDICATES SAMPLE OBTAINED WITH MODIFIED CALIFORNIA BARREL.
- INDICATES SAMPLE OBTAINED WITH SHELBY TUBE.
**FIELD EXPLORATION**

<table>
<thead>
<tr>
<th>Surveyed Elevation (Feet)</th>
<th>Graphic Log</th>
<th>Depth (Feet)</th>
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<tbody>
<tr>
<td>5350</td>
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<tr>
<td>5330</td>
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<td>30</td>
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</table>

TOE BUTTRESS FILL - CLAY, SANDY TO VERY SANDY, WITH SOME GRAVEL, STIFF TO VERY STIFF, SLIGHTLY MOIST TO MOIST, BROWN, TAN, WHITE.

CLAY, SANDY, STIFF TO VERY STIFF, SLIGHTLY MOIST TO MOIST, OLIVE, BROWN, WHITE, RUST, GRAY (CL).

BEDROCK, SANDSTONE, SLIGHTLY MOIST, BROWN, RUST, GRAY.
WEATHERED CLAYSTONE, SLIGHTLY MOIST, GRAY, RUST, TAN.

BEDROCK, CLAYSTONE, SILTY, HARD, SLIGHTLY MOIST, RUST, TAN, OLIVE, GRAY, BLACK.

Bottom of borehole at 35 feet.

---

**LABORATORY RESULTS**

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>BLOW COUNTS (BC)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>UNCORR. BLOWS/6 IN.</td>
</tr>
</tbody>
</table>

- **Dry Density (pcf):**
  - 22.9
  - 23.1
  - 14.2

- **Moisture Content (%):**
  - 103
  - 103
  - 120

- **Liquid Limit:**
  - 53
  - 60
  - 42

- **Plasticity Index:**
  - 36
  - 42
  - 60

- **Unconfined Compressive Strength (psf):**
  - 3,400
  - 3,600
  - 6,300

- **INDICATES SAMPLE OBTAINED WITH MODIFIED CALIFORNIA BARREL.**
- **INDICATES SAMPLE OBTAINED WITH SHELBY TUBE.**
APPENDIX C
LAB DATA
Sample of CLASS I STRUCTURAL FILL
From SAMPLE 1

GRANULOMETRY

Sample of CLASS I STRUCTURAL FILL
From SAMPLE 2

Gradation FIG. C-1

CDOT
US HWY-36
CTL|T PROJECT NO. DN50,165.001
Sample of CLASS I STRUCTURAL FILL
From SAMPLE 3

Hydrometer Analysis
Sieve Analysis

U.S. Standard Series Clear Square Openings

<table>
<thead>
<tr>
<th>Diameter of Particle in Millimeters</th>
<th>Percent Passing</th>
<th>Percent Retained</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
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<tr>
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<td>100</td>
</tr>
<tr>
<td>200</td>
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<td>25 HR. 7 HR. TIME READINGS</td>
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</tr>
<tr>
<td>45 MIN. 15 MIN. 60 MIN. 19 MIN. 4 MIN. 1 MIN.</td>
<td>*200</td>
<td>*100</td>
</tr>
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<td>3/8&quot; 3/4&quot; 1 1/2&quot; 3&quot; 5 1/2&quot; 8&quot;</td>
<td>0</td>
<td>100</td>
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</tbody>
</table>

Sample of CLASS I STRUCTURAL FILL
From SAMPLE 4

Hydrometer Analysis
Sieve Analysis

U.S. Standard Series Clear Square Openings

<table>
<thead>
<tr>
<th>Diameter of Particle in Millimeters</th>
<th>Percent Passing</th>
<th>Percent Retained</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
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<tr>
<td>0.002</td>
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<td>0.005</td>
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<tr>
<td>0.149</td>
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<td>100</td>
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<tr>
<td>0.297</td>
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<td>100</td>
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<td>1.19</td>
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<td>100</td>
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<td>1.9</td>
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<td>2.0</td>
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<td>4.76</td>
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<td>19.1</td>
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<td>76.2</td>
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<td>127</td>
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<td>100</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>25 HR. 7 HR. TIME READINGS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>45 MIN. 15 MIN. 60 MIN. 19 MIN. 4 MIN. 1 MIN.</td>
<td>*200</td>
<td>*100</td>
</tr>
<tr>
<td>3/8&quot; 3/4&quot; 1 1/2&quot; 3&quot; 5 1/2&quot; 8&quot;</td>
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</table>

Gradation

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CTRL|T PROJECT NO. DN50,165.001

FIG. C-2
Sample of Embankment Fill From Sample 5

Sample of Embankment Fill From Sample 6

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US HWY-36
Ctl|T Project No. DN50,165.001
Sample of EMBANKMENT FILL From SAMPLE 7

Sample of EMBANKMENT FILL From SAMPLE 8

CDOT
US HWY-36
CTL|T PROJECT NO. DN50,165.001
Sample of EMBANKMENT FILL
From SAMPLE 9

Sample of EMBANKMENT FILL
From SAMPLE 10

Gradation
Fig. C-5
Sample of EMBANKMENT FILL
From SAMPLE 11

Sample of CLASS I STRUCTURAL FILL
From SAMPLE 12
Sample of **EMBANKMENT FILL**
From **SAMPLE 13**

- **Silt & Clay**: 92%
- **Liquid Limit**: 61%
- **Plasticity Index**: 39%

---

Sample of **EMBANKMENT FILL**
From **SAMPLE 14**

- **Silt & Clay**: 99%
- **Liquid Limit**: 70%
- **Plasticity Index**: 47%

---

**Gradation**

FIG. C-7
Sample of **EMBANKMENT FILL**
From **SAMPLE 15**

- **SANDS**
  - Fine
  - Medium
  - Coarse

- **GRAVEL**
  - Fine
  - Coarse
  - Cobble

**CLAY (PLASTIC) TO SILT (NON-PLASTIC)**

- **GRAVEL** 0%
- **SAND** 16%
- **SILT & CLAY** 84%
- **LIQUID LIMIT** 57%
- **PLASTICITY INDEX** 40%

**Gradation**

FIG. C-8
Sample of CLASS I STRUCTURAL FILL
From SAMPLE COMBINED 1,2,3,4

Clay (Plastic) to Silt (Non-Plastic)
Sands Fine Medium Coarse
Gravel Fine Coarse Cobble

Gradation

Sample of EMBANKMENT FILL
From SAMPLE COMBINED 9,15,16

CDOT
US HWY-36
CTL|T PROJECT NO. DN50,165.001
Sample of CLASS I STRUCTURAL FILL
From B - 1 AT 5 FEET

Sample of CLAY, SANDY (CL)
From B - 1 AT 20 FEET

Gradation

FIG. C-10
Sample of CLASS I STRUCTURAL FILL
From B - 2 AT 5-10 FEET

Sample of CLAYSTONE
From B - 2 AT 35 FEET

Gradation

FIG. C-11
Sample of **CLASS I STRUCTURAL FILL**
From **B - 3 AT 2.5-17.5 FEET**

**Gravel** 12% **Sand** 68%
**Silt & Clay** 20% **Liquid Limit** 25%
**Plasticity Index** 2%

---

Sample of **WEATHERED CLAYSTONE**
From **B - 3 AT 32.5 FEET**

**Gravel** 0% **Sand** 1%
**Silt & Clay** 99% **Liquid Limit** 65%
**Plasticity Index** 43%

Gradation FIG. C-12
Sample of CLAY, SANDY (CL) From B - 4 AT 32.5 FEET

Sample of WEATHERED CLAYSTONE From B - 4 AT 37.5 FEET

Gradation FIG. C-13
Sample of CLAYSTONE
From B - 4 AT 45 FEET

Sample of CLASS I STRUCTURAL FILL
From B-4 COMBINED

Gradation

FIG. C-14
Sample of CLAY, SANDY (CL)
From B - 5 AT 45 FEET

Sample of EMBANKMENT FILL
From B - 6 AT 43595 FEET

**Gradation**

FIG. C-15
Sample of EMBANKMENT FILL
From B - 6 AT 30 FEET

Sample of EMBANKMENT FILL
From B - 6 AT 35 FEET

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Gradation

FIG. C-16
Sample of **CLAYSTONE**
From **B - 7 AT 55 FEET**

- GRAVEL 0%
- SAND 3%
- SILT & CLAY 97%
- LIQUID LIMIT 50%
- PLASTICITY INDEX 30%

**Gradation**

Sample of **EMBANKMENT FILL**
From **B - 8 AT 12.5 FEET**

- GRAVEL 7%
- SAND 31%
- SILT & CLAY 31%
- LIQUID LIMIT 38%
- PLASTICITY INDEX 25%

**Gradation**

*FIG. C-17*
Sample of CLAY, SANDY (CL) From B - 8 AT 17.5 FEET

Sample of CLAYSTONE From B - 8 AT 45 FEET

Gradation FIG. C-18

CDOT
US HWY-36
CTL|T PROJECT NO. DN50,165.001
Sample of CLASS I STRUCTURAL FILL FROM B-8 COMBINED:

- **Gravel**: 9%  
- **Sand**: 69%  
- **Silt & Clay**: 22%  
- **Liquid Limit**: 25%  
- **Plasticity Index**: 3%

Sample of CLAY, SANDY (CL) FROM B - 10 AT 11 FEET:

- **Gravel**: 0%  
- **Sand**: 27%  
- **Silt & Clay**: 73%  
- **Liquid Limit**: 53%  
- **Plasticity Index**: 36%

**Gradation**

FIG. C-19
Sample of CLAY, SANDY (CL) From B - 10 AT 12.5 FEET

Sample of TOE DRAIN BEDDING From STA. 12+35

CDOT
US HWY-36
CTL|T PROJECT NO. DN50,165.001

Gradation FIG. C-20
Sample Description: Clay, Sandy
Sample Type: California
Remarks:

Direct Shear Test Results
Sample Description: Clay, Sandy
Sample Type: California
Remarks: Shear Plane cut with wire prior to test

Direct Shear Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%) Before</th>
<th>Moisture Content (%) After</th>
<th>Clay Content, %</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>69</td>
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<td>2</td>
<td>B-1</td>
<td>20’</td>
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<td>20.3</td>
<td></td>
<td>112.5</td>
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<tr>
<td>3</td>
<td>B-1</td>
<td>20’</td>
<td>15.0</td>
<td>20.4</td>
<td></td>
<td>112.6</td>
</tr>
</tbody>
</table>

LL, %: 40  PI, %: 25  -200: 69  Clay Content, %: 54
Thickness (in): 1.0  Diameter (in): 1.935
Shearing Rate (in/min): 0.0016

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Residual Stress (KSF)</th>
<th>Large Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1</td>
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<td>0.36</td>
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<td>2</td>
<td>2</td>
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<td>3</td>
<td>4</td>
<td>2.47</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Residual ϕ (DEG): 32
Residual C (PSF): 0
Sample Description: Claystone
Sample Type: California
Remarks:

Direct Shear Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B-2</td>
<td>35'</td>
<td>13.8</td>
<td>21.1</td>
</tr>
<tr>
<td>2</td>
<td>B-2</td>
<td>35'</td>
<td>13.8</td>
<td>20.7</td>
</tr>
<tr>
<td>3</td>
<td>B-2</td>
<td>35'</td>
<td>13.8</td>
<td>19.9</td>
</tr>
</tbody>
</table>

LL, %: 53  PI, %: 35  -200: 98  Clay Content, %: 49
Thickness (in): 1.0  Diameter (in): 1.935
Shearing Rate (in/min): 0.0016

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Peak Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>2.59</td>
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<td>3</td>
<td>8</td>
<td>5.79</td>
<td>3.86</td>
<td>0.38</td>
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</tbody>
</table>

Peak $\phi$ (DEG): 27
Large Displacement $\phi$ (DEG): 24
Peak C (PSF): 1990
Large Displacement C (PSF): 490
Direct Shear Test Results

Sample Description: Claystone
Sample Type: California
Remarks: Shear Plane cut with wire prior to test

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
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<tbody>
<tr>
<td>1</td>
<td>B-2</td>
<td>35'</td>
<td>13.8</td>
<td>21.1</td>
</tr>
<tr>
<td>2</td>
<td>B-2</td>
<td>35'</td>
<td>13.8</td>
<td>20.7</td>
</tr>
<tr>
<td>3</td>
<td>B-2</td>
<td>35'</td>
<td>13.8</td>
<td>19.9</td>
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</table>

Residual Shear

<table>
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<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Residual Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement (IN.)</th>
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</thead>
<tbody>
<tr>
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<td>1.04</td>
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<tr>
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<td>0.36</td>
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<tr>
<td>3</td>
<td>8</td>
<td>4.1</td>
<td>4.05</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Residual $\phi$ (DEG): 27
Residual C (PSF): 0

LL, %: 53  PI, %: 35  -200: 98  Clay Content, %: 49
Thickness (in): 1.0  Diameter (in): 1.935
Shearing Rate (in/min): 0.0016

Residual $\phi$ (DEG): 27
Residual C (PSF): 0
Direct Shear Test Results

Sample Description: Weathered Claystone
Sample Type: California
Remarks: 

**Direct Shear Test Results**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>B-3</td>
<td>32.5'</td>
<td>20.3 before 25.9 after</td>
<td>105.9</td>
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<tr>
<td>2</td>
<td>B-3</td>
<td>32.5'</td>
<td>20.3 before 25.2 after</td>
<td>105.8</td>
</tr>
<tr>
<td>3</td>
<td>B-3</td>
<td>32.5'</td>
<td>20.3 before 23.7 after</td>
<td>104.6</td>
</tr>
</tbody>
</table>

**Shear Stress (KSF)**

**Normal Stress (KSF)**

**Horizontal Deformation (IN.)**

**Clay Content, %**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Peak Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>2.58</td>
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<tr>
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<td>0.38</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>3.79</td>
<td>3.5</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Peak $\phi$ (DEG): 12
Large Displacement $\phi$ (DEG): 23
Peak C (PSF): 1910
Large Displacement C (PSF): 70
Weathered Claystone

Sample Description: Weathered Claystone
Sample Type: California
Remarks: Shear Plane cut with wire prior to test

Direct Shear Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B-3</td>
<td>32.5'</td>
<td>20.3</td>
<td>105.9</td>
</tr>
<tr>
<td>2</td>
<td>B-3</td>
<td>32.5'</td>
<td>20.3</td>
<td>105.8</td>
</tr>
<tr>
<td>3</td>
<td>B-3</td>
<td>32.5'</td>
<td>20.3</td>
<td>104.6</td>
</tr>
</tbody>
</table>

Shearing Rate (in/min): 0.0016

Clay Content, %: 66

Residual φ (DEG): 15

Residual C (PSF): 170
**Sample Description:** Weathered Claystone

**Sample Type:** California

**Remarks:**

**Direct Shear Test Results**
**Direct Shear Test Results**

Sample Description: Weathered Claystone

Sample Type: California

Remarks: Shear Plane cut with wire prior to test
Direct Shear Test Results

Sample Description: Embankment Fill
Sample Type: California
Remarks: 

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B-6</td>
<td>5'</td>
<td>20.3</td>
<td>29.0</td>
</tr>
<tr>
<td>2</td>
<td>B-6</td>
<td>5'</td>
<td>20.3</td>
<td>31.1</td>
</tr>
<tr>
<td>3</td>
<td>B-6</td>
<td>5'</td>
<td>20.3</td>
<td>29.3</td>
</tr>
</tbody>
</table>

LL, %: 57  PI, %: 38  -200: 79  Clay Content, %: 54
Thickness (in): 1.0  Diameter (in): 1.935
Shearing Rate (in/min): 0.0016

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Peak Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>0.92</td>
<td>0.35</td>
<td>0.38</td>
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<td>2</td>
<td>1</td>
<td>1.32</td>
<td>0.71</td>
<td>0.38</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>1.86</td>
<td>1.41</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Peak $\phi$ (DEG): 32
Large Displacement $\phi$ (DEG): 35
Peak C (PSF): 650
Large Displacement C (PSF): 0
Sample Description: Embankment Fill
Sample Type: California
Remarks: Shear Plane cut with wire prior to test

Direct Shear Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%) Before</th>
<th>Moisture Content (%) After</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B-6</td>
<td>5'</td>
<td>20.3</td>
<td>29.0</td>
<td>99.8</td>
</tr>
<tr>
<td>2</td>
<td>B-6</td>
<td>5'</td>
<td>20.3</td>
<td>31.1</td>
<td>98.4</td>
</tr>
<tr>
<td>3</td>
<td>B-6</td>
<td>5'</td>
<td>20.3</td>
<td>29.3</td>
<td>97.4</td>
</tr>
</tbody>
</table>

LL, %: 57  PI, %: 38  -200: 79

Clay Content, %: 54

Thickness (in): 1.0  Diameter (in): 1.935

Shearing Rate (in/min): 0.0016

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Residual Shear Stress (KSF)</th>
<th>Residual Large Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5</td>
<td>0.32</td>
<td>0.36</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.66</td>
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<tr>
<td>3</td>
<td>2</td>
<td>1.21</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Residual $\phi$ (DEG): 32
Residual C (PSF): 0
Sample Description: Embankment Fill
Sample Type: California
Remarks:

Direct Shear Test Results
Embankment Fill

California

Shear Plane cut with wire prior to test

Direct Shear Test Results
**Claystone Sample Description:**

**Sample Type:** California

**Remarks:**

---

### Direct Shear Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B-7</td>
<td>55'</td>
<td>15.2</td>
<td>20.5</td>
</tr>
<tr>
<td>2</td>
<td>B-7</td>
<td>55'</td>
<td>15.2</td>
<td>18.2</td>
</tr>
<tr>
<td>3</td>
<td>B-7</td>
<td>55'</td>
<td>15.2</td>
<td>20.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LL, %:</th>
<th>50</th>
<th>PI, %:</th>
<th>30</th>
<th>-200: 97</th>
<th>Clay Content, %: 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (in):</td>
<td>1.935</td>
<td>Shearing Rate (in/min): 0.0016</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Peak Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.5</td>
<td>5.8</td>
<td>2.11</td>
<td>0.37</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>4.58</td>
<td>4.16</td>
<td>0.38</td>
</tr>
<tr>
<td>3</td>
<td>14</td>
<td>6.6</td>
<td>4.89</td>
<td>0.38</td>
</tr>
</tbody>
</table>

**Peak ϕ (DEG):** 6

**Large Displacement. ϕ (DEG):** 14

**Peak C (PSF):** 4790

**Large Displacement C (PSF):** 1740

---

**Figure C-33**
Direct Shear Test Results

Sample Description: Claystone
Sample Type: California
Remarks: Shear Plane cut with wire prior to test

LL, %: 50  PI, %: 30 -200: 97  Clay Content, %: 6
Thickness (in): 1.0  Diameter (in): 1.935
Shearing Rate (in/min): 0.0016

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B-7</td>
<td>55'</td>
<td>15.2</td>
<td>20.5</td>
</tr>
<tr>
<td>2</td>
<td>B-7</td>
<td>55'</td>
<td>15.2</td>
<td>18.2</td>
</tr>
<tr>
<td>3</td>
<td>B-7</td>
<td>55'</td>
<td>15.2</td>
<td>20.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Residual Stress (KSF)</th>
<th>Large Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.5</td>
<td>2.03</td>
<td>0.36</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>3.94</td>
<td>0.36</td>
</tr>
<tr>
<td>3</td>
<td>14</td>
<td>6.34</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Residual \( \phi \) (DEG): 22
Residual C (PSF): 830
### Direct Shear Test Results

#### Sample Description
- **Embankment Fill**

#### Sample Type
- **California**

#### Remarks

### Direct Shear Test Results Table

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B-8</td>
<td>12.5'</td>
<td>15.3</td>
<td>107.4</td>
</tr>
<tr>
<td>2</td>
<td>B-8</td>
<td>12.5'</td>
<td>15.3</td>
<td>108.9</td>
</tr>
<tr>
<td>3</td>
<td>B-8</td>
<td>12.5'</td>
<td>15.3</td>
<td>109.6</td>
</tr>
</tbody>
</table>

**Clay Content (%)**
- LL, %: 38
- PI, %: 25
- -200: %

**Thickness (in):** 1.0
**Diameter (in):** 1.935

**Shearing Rate (in/min):** 0.0016

### Graph 1
- **Shear Stress (KSF) vs. Horizontal Deformation (IN.)**

### Graph 2
- **Shear Stress (KSF) vs. Normal Stress (KSF)**

### Additional Information
- **Peak φ (DEG):** 30
- **Large Displacement φ (DEG):** 33
- **Peak C (PSF):** 370
- **Large Displacement C (PSF):** 0

---

DN50165-149
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Fig. C-35
**Sample Description:** Embankment Fill

**Sample Type:** California

**Remarks:** Shear Plane cut with wire prior to test

---

**Direct Shear Test Results**

- **Residual Shear Plane:**
  - Sample No. 1: B-8, Depth 12.5', Moisture Content 15.3, Shearing Rate 0.36
  - Sample No. 2: B-8, Depth 12.5', Moisture Content 15.3, Shearing Rate 0.38
  - Sample No. 3: B-8, Depth 12.5', Moisture Content 15.3, Shearing Rate 0.36

- **Shear Stress and Normal Stress Diagram:**
  - Sample No. 1: Normal Stress 0.55 KSF, Residual Shear Stress 0.55 KSF
  - Sample No. 2: Normal Stress 0.97 KSF, Residual Shear Stress 0.97 KSF
  - Sample No. 3: Normal Stress 2.99 KSF, Residual Shear Stress 2.9 KSF

- **Clay Content:**
  - LL, %: 38
  - PI, %: 25
  - 2-200: — 
  - Thickness (in): 1.0
  - Diameter (in): 1.935
  - Shearing Rate (in/min): 0.0016

---

**Fig. C-36**
Sample Description: Clay, Sandy
Sample Type: Shelby
Remarks: 

Direct Shear Test Results
Sample Description:  Clay, Sandy
Sample Type:  Shelby
Remarks:  

Direct Shear Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>B-10</td>
<td>12.5'</td>
<td>22.9</td>
<td>27.6</td>
</tr>
<tr>
<td>2</td>
<td>B-10</td>
<td>12.5'</td>
<td>22.9</td>
<td>25.9</td>
</tr>
<tr>
<td>3</td>
<td>B-10</td>
<td>12.5'</td>
<td>22.9</td>
<td>25.1</td>
</tr>
</tbody>
</table>

Shear Displacement Stress (IN.)

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Peak Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1.4</td>
<td>1.23</td>
<td>0.58</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>1.6</td>
<td>0.86</td>
<td>0.57</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>2.01</td>
<td>1.78</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Peak $\phi$ (DEG):  12
Large Displacement $\phi$ (DEG):  13
Peak C (PSF):  1190
Large Displacement C (PSF):  770
Sample Description: Clay, Sandy

Sample Type: Shelby

Remarks: Shear Plane cut with wire prior to test

Direct Shear Test Results

- Shearing Rate (in/min): 0.0024
- Clay Content, %: 59
- Diameter (in): 2.86
- Thickness (in): 1.0
- LL, %: 60
- PI, %: 42
- -200: 81

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Residual Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0.55</td>
<td>0.55</td>
<td>0.56</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>0.79</td>
<td>0.79</td>
<td>0.56</td>
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<tr>
<td>3</td>
<td>4</td>
<td>1.76</td>
<td>1.41</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Residual $\phi$ (DEG): 20
Residual C (PSF): 0
Direct Shear Test Results

Sample Description:  Class 1 Structural Fill
Sample Type:  Bulk (-10 Material)
Remarks:  Remolded to 120 @ 7.0%

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S-1,2,3,4</td>
<td>--</td>
<td>6.9</td>
<td>17.8</td>
</tr>
<tr>
<td>2</td>
<td>S-1,2,3,4</td>
<td>--</td>
<td>6.9</td>
<td>17.0</td>
</tr>
<tr>
<td>3</td>
<td>S-1,2,3,4</td>
<td>--</td>
<td>6.9</td>
<td>16.8</td>
</tr>
</tbody>
</table>

**Direct Shear Stress**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Peak Shear Stress (KSF)</th>
<th>Large Displacement Shear Stress (KSF)</th>
<th>Large Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>1.46</td>
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<td>0.37</td>
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<td>2</td>
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<td>2.26</td>
<td>2.26</td>
<td>0.38</td>
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<tr>
<td>3</td>
<td>6</td>
<td>4.46</td>
<td>4.27</td>
<td>0.37</td>
</tr>
</tbody>
</table>

**Peaks and Displacements**

- Peak $\phi$ (DEG): 34
- Large Displacement $\phi$ (DEG): 34
- Peak C (PSF): 370
- Large Displacement C (PSF): 250

**Additional Data**

- LL, %: 25
- PI, %: 2
- -200: 17
- Clay Content, %: 
- Thickness (in): 1.0
- Diameter (in): 1.935
- Shearing Rate (in/min): 0.0063

**Graphs**

1. Shear stress vs. horizontal deformation
2. Normal stress vs. shear stress

---

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Fig. C-40
Direct Shear Test Results

Sample Description: Class 1 Structural Fill
Sample Type: Bulk (-10 Material)
Remarks: Shear Plane cut with wire prior to test

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Boring No.</th>
<th>Depth (FT)</th>
<th>Moisture Content (%) Before</th>
<th>Moisture Content (%) After</th>
<th>Dry Density (PCF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S-1,2,3,4</td>
<td>--</td>
<td>6.9</td>
<td>17.8</td>
<td>120.1</td>
</tr>
<tr>
<td>2</td>
<td>S-1,2,3,4</td>
<td>--</td>
<td>6.9</td>
<td>17.0</td>
<td>120.1</td>
</tr>
<tr>
<td>3</td>
<td>S-1,2,3,4</td>
<td>--</td>
<td>6.9</td>
<td>16.8</td>
<td>120.1</td>
</tr>
</tbody>
</table>

LL, %: 25  PI, %: 2  -200: 17
Thickness (in): 1.0  Diameter (in): 1.935
Shearing Rate (in/min): 0.0063

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal Stress (KSF)</th>
<th>Residual Shear Stress (KSF)</th>
<th>Residual Large Displacement (IN.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5</td>
<td>1.25</td>
<td>0.36</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>2.3</td>
<td>0.38</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>4.56</td>
<td>0.36</td>
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</tbody>
</table>

Residual \(\phi\) (DEG): 37
Residual C (PSF): 0
Sample Description: Class I Material

Location: Combined Samples from S-1 through S-4

Compaction Test Procedure: ASTM D 698

METHOD "A"

Compaclion Test Results

FIG. C-42

MOISTURE CONTENT - %

DRAIN DENSITY - PCF

CURVE NUMBER  I

MAXIMUM DRY DENSITY  138.5  PCF

OPTIMUM MOISTURE CONTENT  6.5  %

ZERO AIR VOIDS SPECIFIC GRAVITY = 2.70

LIQUID LIMIT  25

PLASTICITY INDEX  2

GRAVEL  17  %

SAND  66  %

SILT AND CLAY  17  %

DN50165-149
US-36
Sample Description: Embankment Fill

Location: Combined Samples from S-9, S-15 and S-16

Compaction Test Procedure: ASTM D 698

COMPACTATION TEST RESULTS

<table>
<thead>
<tr>
<th>Sample Description</th>
<th>Liquid Limit</th>
<th>Plasticity Index</th>
<th>Gravel</th>
<th>Sand</th>
<th>Silts and Clays</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment Fill</td>
<td>55</td>
<td>34</td>
<td>0</td>
<td>4</td>
<td>96</td>
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</tbody>
</table>

**FIG. C-43**
<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>LOCATION</th>
<th>DEPTH</th>
<th>FIELD MOISTURE CONTENT (%)</th>
<th>SOIL DRY DENSITY (g/cm³)</th>
<th>LAB MOISTURE CONTENT (%)</th>
<th>Saturated Compaction (%)</th>
<th>PLASTICITY INDEX</th>
<th>PASSING NO. 300 SCREEN (%)</th>
<th>PASSING NO. 200 SCREEN (%)</th>
<th>PEAK STRESS (MPa)</th>
<th>PEAK STRESS ANGLE DEGREES</th>
<th>DIRECT BIAS</th>
<th>COMPRESSION (%P)</th>
<th>COMPRESSION ANGLE DEGREES</th>
<th>DILATION ANGLE DEGREES</th>
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<tbody>
<tr>
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<td>18 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
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<td>3.0</td>
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<td>14 FT FROM WALL</td>
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<td>3.5</td>
<td>3.5</td>
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</tr>
<tr>
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<td>215-93</td>
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<td>CURRENT EXCAVATION</td>
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<td>3.0</td>
<td>2.2</td>
<td>111.2</td>
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<td>17.0</td>
<td>17.0</td>
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<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
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<tr>
<td>5</td>
<td>219-99</td>
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<td>3.0</td>
<td>2.2</td>
<td>110.2</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
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<tr>
<td>6</td>
<td>220-102</td>
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<td>3.0</td>
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<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
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<tr>
<td>7</td>
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<td>CURRENT EXCAVATION</td>
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<td>3.0</td>
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<td>17.0</td>
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<tr>
<td>8</td>
<td>222-104</td>
<td>8 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
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<td>3.0</td>
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<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
</tr>
<tr>
<td>9</td>
<td>223-105</td>
<td>6 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
<td>3.2</td>
<td>3.0</td>
<td>2.2</td>
<td>110.2</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
</tr>
<tr>
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<td>224-106</td>
<td>4 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
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<td>3.0</td>
<td>2.2</td>
<td>110.2</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
<td>17.0</td>
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<td>CURRENT EXCAVATION</td>
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<td>2.2</td>
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<td>2.2</td>
<td>110.2</td>
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<td>2 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
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<td>2.2</td>
<td>110.2</td>
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<td>2 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
<td>3.2</td>
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<td>232-114</td>
<td>2 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
<td>3.2</td>
<td>3.0</td>
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<tr>
<td>19</td>
<td>233-115</td>
<td>2 FT FROM WALL</td>
<td>CURRENT EXCAVATION</td>
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</tbody>
</table>

**NOTES:**
- All samples were taken from the same location.
- The soil type is indicated for each sample.
- The table includes additional columns for compression, dilation, and other laboratory test results.
APPENDIX D

GLOBAL SLOPE STABILITY CALCULATIONS
TABLE D-1
SUMMARY OF GLOBAL STABILITY CALCULATIONS

Description
Reported Station 2168+50 - Replicate Kleinfelder Analysis
Station 2172+50- Kleinfelder Assumptions - No Water
Station 2172+50- Kleinfelder Assumptions - Water at 5347
2172+50 Specified Surface – No Cohesion – No Water
2172+50 Specified Surface – No Cohesion – No Water
2172+50 Specified Surface – No Cohesion – No Water
2172+50 Specified Surface – No Cohesion – No Water
2172+50 Specified Surface – No Cohesion – No Water
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2172+50 Specified Surface – No Cohesion – No Water
2172+50 Specified Surface – No Cohesion – Water at 5347
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2172+50 Specified Surface – Cohesion – No Water
2172+50 Specified Surface – Cohesion – Water at 5347
2172+50 Specified Surface – Cohesion – Water at 5347
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2172+50 Specified Surface – Cohesion – Water at 5347

Shear Strength Parameters
Embankment Fill
Clay
φ' (deg)
c' (psf)
φ' (deg)
c' (psf)
26
400
26
200
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400
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200
26
400
26
200
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16
200

Factor
of
Safety
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1.6
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0.9
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0.8
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1.0
0.9
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0.8
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1.1

Figure
Number
D-1
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D-2B
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D-66


US 36 Retaining Wall B1-10R
Sta 2168+50
Replicate Kleinfelder Analyses (Appendix D-8)

Name: Native clay
Unit Weight: 126 pcf
Cohesion: 200 psf
Phi: 26 °

Name: Claystone
Unit Weight: 128 pcf
Cohesion: 5000 psf
Phi: 0 °

Name: Retained fill
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 38 °

Name: MSE wall
Unit Weight: 140 pcf
Cohesion: 10000 psf
Phi: 50 °

Name: Clay fill
Unit Weight: 122 pcf
Cohesion: 400 psf
Phi: 26 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Clay fill
Model: Mohr-Coulomb
Unit Weight: 122 pcf
Cohesion: 400 psf
Phi: 26 °

Name: Retained fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 38 °

Name: MSE wall
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 10000 psf
Phi: 50 °

Name: Native clay
Model: Mohr-Coulomb
Unit Weight: 126 pcf
Cohesion: 200 psf
Phi: 26 °

Note: Kleinfelder values from Appendix D-8
### Specified Failure Surface

<table>
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<th>Unit Weight</th>
<th>Cohesion</th>
<th>Cohesion Value</th>
<th>Phi</th>
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<td>Clay fill</td>
<td>Mohr-Coulomb</td>
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<td>400 psf</td>
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<td>26 °</td>
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<tr>
<td>Retained fill</td>
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<td>140 pcf</td>
<td>0 psf</td>
<td></td>
<td>38 °</td>
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<tr>
<td>MSE wall</td>
<td>Mohr-Coulomb</td>
<td>140 pcf</td>
<td>10000 psf</td>
<td></td>
<td>50 °</td>
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<tr>
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<td>200 psf</td>
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<td>26 °</td>
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Note: Kleinfelder values from Appendix D-8

---

**US 36 Retaining Wall B1-10R**  
**Station 2172+50**  
Westminster, Colorado
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °

Figure D-3
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Embankment fill
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 16 °

- **Name:** Retained fill and Reinforced fill
  - Model: Mohr-Coulomb
  - Unit Weight: 140 pcf
  - Cohesion: 0 psf
  - Phi: 34 °

- **Name:** Clay
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Embankment fill
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 24°

- **Name:** Retained fill and Reinforced fill
  - Model: Mohr-Coulomb
  - Unit Weight: 140 pcf
  - Cohesion: 0 psf
  - Phi: 34°

- **Name:** Clay
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 20°

CTL\T Project No. DN50,165-149  Figure D-6
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20°

Figure D-7
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16°

Figure D-9
Specified Failure Surface

- **Name**: Embankment fill  
  **Model**: Mohr-Coulomb  
  **Unit Weight**: 124 pcf  
  **Cohesion**: 0 psf  
  **Phi**: 20 °

- **Name**: Retained fill and Reinforced fill  
  **Model**: Mohr-Coulomb  
  **Unit Weight**: 140 pcf  
  **Cohesion**: 0 psf  
  **Phi**: 34 °

- **Name**: Clay  
  **Model**: Mohr-Coulomb  
  **Unit Weight**: 124 pcf  
  **Cohesion**: 0 psf  
  **Phi**: 16 °

**US 36 Retaining Wall B1-10R**  
**Station 2172+50**  
**Westminster, Colorado**

**Figure D-10**
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16°
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name**: Embankment fill
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 16 °

- **Name**: Retained fill and Reinforced fill
  - Model: Mohr-Coulomb
  - Unit Weight: 140 pcf
  - Cohesion: 0 psf
  - Phi: 34 °

- **Name**: Clay
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Retained fill and Reinforced fill
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 140 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 34 °

- **Name:** Clay
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 124 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 16 °

- **Name:** Embankment fill
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 124 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 24 °

- **Name:** Retained fill
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 140 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 34 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Retained fill and Reinforced fill**
  - Model: Mohr-Coulomb
  - Unit Weight: 140 pcf
  - Cohesion: 0 psf
  - Phi: 34 °

- **Clay**
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 16 °

- **Embankment fill**
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 0 psf
  - Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminister, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16°
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16°
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Embankment fill
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 200 psf
  - Phi: 16 °

- **Name:** Retained fill and Reinforced fill
  - Model: Mohr-Coulomb
  - Unit Weight: 140 pcf
  - Cohesion: 0 psf
  - Phi: 34 °

- **Name:** Clay
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 200 psf
  - Phi: 16 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24°

Elevation (x 1000)

Distance

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180

5.32 5.33 5.34 5.35 5.36 5.37 5.38 5.39

Embankment fill
Retained fill
Reinforced fill
Clay
WBR
Bedrock

Figure D-31

CTL\T Project No. DN50,165-149
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24°

CTL\T Project No. DN50,165-149
Figure D-32
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Embankment fill
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 124 pcf
  - **Cohesion:** 200 psf
  - **Phi:** 24 °

- **Name:** Retained fill and Reinforced fill
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 140 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 34 °

- **Name:** Clay
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 124 pcf
  - **Cohesion:** 200 psf
  - **Phi:** 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °

CTL\T Project No. DN50,165-149
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name**: Retained fill and Reinforced fill
  - Model: Mohr-Coulomb
  - Unit Weight: 140 psf
  - Cohesion: 0 psf
  - Phi: 34 °

- **Name**: Clay
  - Model: Mohr-Coulomb
  - Unit Weight: 124 psf
  - Cohesion: 200 psf
  - Phi: 16 °

- **Name**: Embankment fill
  - Model: Mohr-Coulomb
  - Unit Weight: 124 psf
  - Cohesion: 200 psf
  - Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16°
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16 °

CTLT Project No. DN50,165-149

Figure D-38
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °

CTL\T Project No. DN50,165-149

Figure D-43
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Retained fill and Reinforced fill
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 140 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 34°

- **Name:** Clay
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 124 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 16°

- **Name:** Embankment fill
  - **Model:** Mohr-Coulomb
  - **Unit Weight:** 124 pcf
  - **Cohesion:** 0 psf
  - **Phi:** 28°
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28°
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 24 °

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 20 °

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °

Distance

Elevation (x 1000)

5.39
5.38
5.37
5.36
5.35
5.34
5.33
5.32
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180

Figure D-51
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 0 psf
Phi: 16 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Embankment fill  
  - **Model:** Mohr-Coulomb  
  - **Unit Weight:** 124 pcf  
  - **Cohesion:** 200 psf  
  - **Phi:** 24°

- **Name:** Retained fill and Reinforced fill  
  - **Model:** Mohr-Coulomb  
  - **Unit Weight:** 140 pcf  
  - **Cohesion:** 0 psf  
  - **Phi:** 34°

- **Name:** Clay  
  - **Model:** Mohr-Coulomb  
  - **Unit Weight:** 124 pcf  
  - **Cohesion:** 200 psf  
  - **Phi:** 28°

Distance (0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180)

Elevation (x 1000) (5.32 5.33 5.34 5.35 5.36 5.37 5.38 5.39)

- Embankment fill
- Retained fill
- Reinforced fill
- Clay
- WBR
- Bedrock

Figure D-54
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °

Elevation (x 1000)

Distance

5.39
5.38
5.37
5.36
5.35
5.34
5.33
5.32
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180

CTL\T Project No. DN50,165-149
Figure D-56
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °
US 36 Retaining Wall B1-10R  
Station 2172+50  
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill  
Model: Mohr-Coulomb  
Unit Weight: 140 pcf  
Cohesion: 0 psf  
Phi: 34 °

Name: Clay  
Model: Mohr-Coulomb  
Unit Weight: 124 pcf  
Cohesion: 200 psf  
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 16°

Figure D-59
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Embankment fill**
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 200 psf
  - Phi: 28 °

- **Retained fill and Reinforced fill**
  - Model: Mohr-Coulomb
  - Unit Weight: 140 pcf
  - Cohesion: 0 psf
  - Phi: 34 °

- **Clay**
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 200 psf
  - Phi: 28 °

Distance

Elevation (x 1000)

CTLT Project No. DN50,165-149

Figure D-60
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24°

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34°

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28°
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 24 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

Name: Embankment fill
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 28 °

Name: Retained fill and Reinforced fill
Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion: 0 psf
Phi: 34 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 124 pcf
Cohesion: 200 psf
Phi: 20 °
US 36 Retaining Wall B1-10R
Station 2172+50
Westminster, Colorado

Specified Failure Surface

- **Name:** Embankment fill
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 200 psf
  - Phi: 28 °

- **Name:** Retained fill and Reinforced fill
  - Model: Mohr-Coulomb
  - Unit Weight: 140 pcf
  - Cohesion: 0 psf
  - Phi: 34 °

- **Name:** Clay
  - Model: Mohr-Coulomb
  - Unit Weight: 124 pcf
  - Cohesion: 200 psf
  - Phi: 16 °
APPENDIX E
WALL STABILITY CALCULATIONS
GEOMETRY FOR ALL STRENGTH COMBINATIONS

DETAIL AS BUILT SHEET NO. 210

2' OF CLASS 1 STRUCTURAL FILL BELOW REINFORCED FILL
2' OF IMPERMEABLE FILL BELOW STRUCTURAL CLASS 1

THIS IS A DESIGN CHANGE REGARDING BEARING CAPACITY
TOTAL HEIGHT H = 32.15' FROM PANEL DRAWINGS
STRAP LENGTH L = 24'

RETAINED FILL TO ELEV 5354
CLASS 1 BELOW REINFORCED FILL TO ELEV. 5352
IMPERMEABLE FILL BELOW CLASS 1 TO ELEV 5350
NATURAL CLAY TO ELEV 5336
WEATHERED BEDROCK TO ELEV. 5336 TO 5330
CLAYSTONE BEDROCK TO ELEV 5330

GROUND WATER AT ELEV 5341 IN CTL BORING 5

THICKNESS & ELEVATIONS OF NATURAL CLAY, WEATHERED
BEDROCK & CLAYSTONE ESTIMATED USING 4
CTL/ T BORING B-5 & B-8

NOTE
The 2' layer of class 1 fill, combined with
the compacted 2' layer of impermeable fill
were placed to spread the weight of the reinforced
fill and decrease risk of slump, sliding, overturning
or spreading are assumed not to occur in this
4 feet thick zone in the analysis that follows.
GEOMETRY Q 2172+50

5390

5380

5370

5360

5350

5340

5330

Existing Embankment

(Old Fill)

Retained Fill

1.5

1.0

Approx

Reinforced Fill

Class 1

2'

Impermeable

2'

Old Fill

Natural Clay

Weathered Bedrock

Claystone

Water

CTT/ B-5

EL 96

EL 30
<table>
<thead>
<tr>
<th>Source</th>
<th>Layer</th>
<th>Unit Wt (pcf)</th>
<th>Cohesion (lbf/ft²)</th>
<th>Psi (Degrees)</th>
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<td>RETAIN FILL</td>
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<td>38</td>
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<tr>
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<td>OLD FILL</td>
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<td>126</td>
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<td>128</td>
<td>5,000</td>
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<td>WALLS</td>
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<tr>
<td>OLD EMB</td>
<td>124</td>
<td>200</td>
<td>24</td>
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</tbody>
</table>
Subject: Pogemetric
Station 2172+50
Per: Kleinfield Rep #19

\[ H = 32.5' \quad L = 24' \quad \gamma_a = 140 \quad \gamma_b = 140 \quad q = 250 \text{ psf} \]

\[ \gamma_A = \gamma_a H L = 140 (32.5)(24) = 109.2 \text{ k} \]

\[ k_{AB} = 0.24 \]

\[ F_1 = \frac{1}{2} \gamma_A H^2 k_{AB} \]
\[ = \frac{1}{2} (140)(32.5)^2 (0.24) = 17.74 \text{ k} \]

\[ F_2 = 250(32.5)(0.24) = 19.55 \text{ k} \]

\[ T = F_1 + F_2 = 19.69 \]

Factors:

Sliding: \[ u_e = 1.00 \quad \gamma_{EH} = 1.50 \]

Bearing: \[ \gamma_{VMA} = 1.25 \quad \gamma_{EHMA} = 1.50 \]

LRFD Sliding:

\[ \sigma_{(vertical)}(H) = 109.2(0.25) = 27.3 \]

\[ 27.3(\gamma_e) = 27.3 \]

\[ P_d = 19.69(\gamma_{EH}) = 19.69(1.50) = 29.58 \]

\[ R_r = 27.3 \]

\[ \frac{R_r}{P_d} = \frac{27.3}{29.58} = 0.92 \]

Indicates sliding.
UNFACTORED - NOMINAL

FORCES DRIVING = 19.69

FORCES RESISTING = 33.3

FORCES RESISTING = 27.30 NO SURCHARGE

\[
\frac{27.30}{19.69} = 1.39 \quad \text{NO SURCHARGE}
\]

\[
\frac{33.3}{19.69} = 1.69 \quad \text{WITH SURCHARGE}
\]

CHAPTER 4 SAYS NEGLECT SURCHARGE ON REINFORCE ZONE

CHECK ECCENTRICITY

\[
\varepsilon = \frac{\sum M_d - \sum M_e}{\sum V}
\]

\[
\varepsilon = \frac{Y_{EH MAX} F_1 (H/3) + Y_{LV} F_{2.5} (H/2)}{Y_{EH MAX} V_1}
\]

\[
Y_{EH MAX} = 1.5 \quad Y_{LV} = 1.75
\]

\[
\varepsilon = \frac{1.5 (17.74) (32.5) + 1.75 (1.95) (32.5)}{1.00 (109.2)}
\]

\[
\varepsilon = \frac{2,883 + 55.4}{109.2} = 3.15 \quad \text{WITH OUT SURCHARGE ON ENSURE ZONE}
\]

\[
\varepsilon = \frac{2,883 + 55.4}{109.2 + 6.0} = 2.98 \quad \text{WITH SURCHARGE}
\]

USE \( \varepsilon \) WITH SURCHARGE = 2.98
\[ L - 2q = 24 - 2(2.98) = 13.04 \]

Calc. \( c_b = \frac{\text{VEMAX} E_1 \left( \frac{h_1}{2} \right) + Y_{1b} F_{2b} \left( \frac{h_2}{2} \right)}{Y_{\text{VEMAX}} V_1 + Y_{1b} q_b} \)

\[ \begin{align*} Y_{\text{VEMAX}} &= 135 \\ Y_{\text{VEMAX}} &= 150 \\ V_1 &= 1.75 \\ 1.75 &= 1.75 \end{align*} \]

\[ \left( 135 \right) \left( 109.2 \right) + 1.75 \left( 106 \right) = 147.4 + 106 \]

\[ = 157.9 \]

\[ c_b = \frac{343.7}{157.9} = 2.18 \]

**CHECK BEARING**

\[ q_y = \frac{\sum V}{L - 2c_b} \]

\[ \Sigma V = 109.2 + 6 = 115.2 \text{ k}\]

\[ L - 2c_b = 24 - 2(2.98) = 19.64 \]

**Factor**

\[ \Sigma V = 157.9 \text{ k} \]

\[ \frac{157.9}{19.64} = 8.04 \text{ ksf} \]

**Nominal**

\[ q_y = \frac{115.2}{19.64} = 5.86 \text{ ksf} \]  (Material not considered)

**Uniform**

\[ q_y = \frac{115.2}{24} = 4.80 \text{ ksf} \]
\[ Q_{N} = C_{1} N_{C} + 0.5 \gamma_{r} \gamma_{f} N_{Y} \]

PER PAGE 19 \( \phi_{f} = 30^\circ \) \( c = 500 \)

\[ N_{C} = 30.1 \quad \gamma_{f} = 1.22 \]
\[ N_{Y} = 22.4 \]

\[ Q_{bN} = 500(30.1) + 0.5(19.64)(12.2)(22.4) \]
\[ = 15,050 \text{ kN} + 26,830 = 41,880 \text{ kN} \]

\[ t_{f} = 0.065(41.88) = 2.722 \]

27.22 > 8.04 \( \text{indicates bearing OK} \)

CHECK SQUEEZE

\[ \gamma_{f} H \leq 3 \gamma_{u} \]

\[ 140(32.6) = 4,550 \]

\[ \frac{4,550}{29} = \gamma_{u} = 151.6 \text{ PkF} \]

\( \text{indicates 3032 PkF unconfined c Zea} \)
REVISED DESIGN

RETAINED FILL = 140° 38°
REINFORCED FILL = 140° 38°
FOUNDATION = 120° 26°

\[ h = 32.5' \]
\[ l = 24' \]
\[ \theta = 250 \text{ psf} \]

\[ \tan 38° = 0.24 \]
\[ \tan 26° = 0.48 \]

\[ F_1 = \frac{1}{2} \cdot \tan \theta \cdot L \cdot P = \frac{1}{2} \cdot (32.5) \cdot (32.5) \cdot (24) = 17.74 \text{ k} \]

\[ F_2 = \theta \cdot H \cdot \tan \theta = 250 \cdot 32.5 \cdot 0.24 = 195 \text{ k} \]

FACTORS

SLIDING

\[ Y_{E1} = 1.0 \]
\[ Y_{E2} = 1.50 \]
\[ u = 0.25 \text{ (KleinFels)} \]

REINFORCEMENT

\[ Y_{E1, \text{max}} = 1.95 \]
\[ Y_{E2, \text{max}} = 1.50 \]

LRFD SLIDING

VERTICAL NO. 1

\[ 109.2 \cdot (0.25) = 27.3 \]

VERTICAL WITH \[ \theta \]

\[ 109.2 + 6.0 = 115.2 \]
\[ 115.2 \cdot (0.25) = 28.8 \]

RESISTING WITH \[ \theta \]

\[ R_\text{E1} = 23.04 \]

RESISTING NO. 2

\[ R_\text{E2} = 21.84 \]

DRIVING

\[ P_\text{d} = 19.49 \cdot (1.50) = 29.53 \]

\[ R_\text{E2} = 27.3 \]
\[ P_\text{d} = 29.53 \text{ indicates sliding} \]
Because design parameters for reinforced fill are the same values as call for original design parameters, 

- $\phi$ without surcharge = 3.15
- $\phi$ with surcharge = 2.98

For bearing check use $\phi$ with surcharge.

$C_0$ for original design criteria = 2.18

Bearing remain same.

$S_Y$ factor $= 8.04$ ksf

Nominal $S_Y = 5.86$ ksf

Uniform $S_Y = 4.80$ ksf

New values:

- Unit wt foundation = 120
- $C = 0$
- $\phi = 26^\circ$

For $26^\circ$, $N_e = 22.3$, $N_y = 12.5$

$N = C N_e + 0.151 N_y$

$= 0(22.3) + 0.151(10.64)(120)(12.5)$

$= 14.73$ ksf

$C_{bN} = 14.73$ ksf
4_n (0.65)(14.73) = 9.57

9.57 > 3.04  Indicates Bearing OK

No C' Value Cannot Check Squeezes
H = 32.5  L = 24  \gamma_r = 140  \gamma_b = 140  \phi = 250  \psi = F

\phi_1 = 34^\circ  \phi_\sigma = 34^\circ

FOUNDATION 1  CLAY = 124  C = 100  \phi = 24^\circ

FOUNDATION 2  CLAY = 124  C = 200  \phi = 24^\circ

\kappa_{ab} 34^\circ = 0.283

\gamma = 32.5(24)(140) = 169.2 k

\gamma_\frac{1}{2} = 24(250) = 6.0 k

F_1 = \gamma_\frac{1}{2} \gamma H^2 \kappa_{ab} = 6.5(140)(32.5)^2 (0.283) = 20.92 k

F_2 = 250(32.5)(0.283) = 2.30 k

FACTORS

SLIDING  \gamma_{EV} = 1.00  \gamma_{EH} = 1.50

BEARING  \gamma_{EA} = 1.50  \gamma_{EA} = 1.50

LRFD SLIDING  CHART A NO SURCHARGE SLIDING ON BEARING ZONE

RESCISTING = \gamma_1(0.25) = 109.2(0.75) = 27.3 k

Pd = (20.92 + 2.30)(1.50) = 34.83

\frac{Rc}{Pd} = \frac{27.3}{34.83} = 0.78  INDICATES SLIDING

NOMINAL 20.92 + 2.30 = 23.22  DRIVING

27.3 / 23.22 = 1.17, 5.5 6 SHOWN BE 1.5

INDICATES SLIDING RISK
Eccentricity

\[ e = \frac{Y_{eun} \cdot e_1 (\frac{h}{2}) + Y_{e1} \cdot F_{L1} (\frac{h}{2})}{Y_{eun}} \]

\[ e = 1.5 \cdot \frac{Y_{eun}}{1.75} \]

\[ Y_{eun} = 1.5 \quad Y_{e1} = 1.75 \]

\[ e = 1.5 \left( 10.32 \right) \left( 32.5/2 \right) + 1.75 \left( 2.30 \right) \left( 32.5/2 \right) \]

\[ 1.10 \left( 109.2 \right) \]

\[ 339.95 + 65.41 = \frac{405}{100} = 3.71 \]

\[ e \text{ without surcharge on rein. zone} = 3.71 \]

\[ e \text{ with surcharge on rein. zone} \]

\[ e = \frac{405}{\left( 1.10 \right) \left( 109 + 6.0 \right)} \]

\[ e = 3.52 \]

\[ \text{Use } e \text{ with surcharge } = 3.52 \]

\[ L - 2e = 24 - 2 \left( 3.52 \right) = 16.96 \]

\[ \text{Calc. } \phi_0 = \frac{Y_{eun} \cdot e_1 (h/2) + Y_{e1} \cdot F_{L1} (h/2)}{Y_{eun} \cdot e_1 + Y_{e1} \cdot 4L} \]

\[ Y_{eun} = 1.5 \quad Y_{e1} = 1.75 \quad Y_{eun} = 1.35 \]

\[ 1.35 \left( 109.2 \right) + 1.75 \left( 6 \right) = 157.9 \]

\[ e_0 = \frac{405}{157.9} = 2.56 \]
L' = 24 - 2(2.56) = 18.88

CHECK BEARING

Z' V = 109.2 + 6.0 = 115.2

Nominal = \frac{115.2}{18.88} = 6.10 \text{ KSF} \text{ INCLUDE SURCHARGE}

Factored = (109.2 \times 1.35) = 147.4

C \times (1.75) = \frac{105}{157.9}

\sigma = \frac{157.9}{18.88} = 8.36 \text{ KSF}

Nominal no surcharge = \frac{109.2}{18.88} = 5.78

\phi N = C C (N_{c}) + 0.5 \phi_{b} N_{b}

2 ALTERNATIVES  

\phi = 24^\circ  \quad C = 100  \quad \phi_{b} = 124

\phi = 24^\circ  \quad C = 200

\phi = 24^\circ  \quad N_{c} = 19.3

N_{b} = 9.4
For $\phi = 24 \ deg$  $c = 100$

$Q_n = 100(19.3) + 0.5(18.88)(124)(9.4)$

$= 1.924 \ text{kF} + 11.0 \ text{kF} = 12.924 \ text{kF}$

For $\phi = 24 \ deg$  $c = 200$

$Q_n = 200(19.3) + 0.5(18.88)(124)(9.4)$

$= 3.96 \ text{kF} + 11.0 \ text{kF} = 14.96 \ text{kF}$

bearing factor = 0.65

$Q_{100} = 0.65(19.3) = 12.4 \ text{kF}$

$Q_{200} = 0.65(14.86) = 9.46 \ text{kF}$

For $c =$ Required  $3.36 \ text{kF} < 3.40 \ text{kF}$

$3.36 \ text{kF} < 3.66 \ text{kF}$

B-5 clay $UC = 3500 \ text{psi}$

B-3 clay $UC = 3000 \ text{psi}$

Allow $1.5(UC) = 5.25$

$1.5(UC) = 4.50$

For Service Condition allow bearing @ 4.9 $UC = 5.3 \ text{kF}$

Dead Load + Q = 6.10 $\text{kF}$

D only = $\frac{109.2}{18.88} = 5.76 \ text{kF}$
EXHIBIT A

RETAINING WALL GENERAL NOTES (AS BUILT WD-109) 4 4 13
Dear Mr. Cordts,

Please find attached revised shop drawings for the referenced project. The drawings have been revised by the change made to the wall design for CDOT.


The design contained herein is based on information supplied to TBSS by others. The scope of our services did not include verification, analysis, or design of the foundation, native material, or backfill material. Furthermore, TBSS did not visit the site, nor did TBSS determine the required elevations and heights of the incorporated MSE structures. It is the responsibility of others to verify that the elevations and stations that are given in the shop drawings are true and accurate before beginning erection of the wall system.

The structure has been designed assuming a 75 year life. Steel degradation rates are in accordance with AASHTO requirements. Corrosion loss rates are based on the backfill material meeting or exceeding the electrochemical requirement specified in AASHTO Article 11.10.6.4.2 – *Design Life Considerations*. The soil reinforcing shall be fabricated in accordance with ASTM
A1064 and shall be hot dip galvanized in accordance with ASTM A123. TBSS has made general assumptions concerning the strength parameters of the foundation; select backfill material; and retained fill material as defined in the contract documents and as shown in the following table. Two design values for the unit weight of the reinforced soil backfill have been used. These varying unit weights will produce a conservative design. A light unit weight is used in the analysis of sliding, overturning (limiting eccentricity) and pullout calculations. A heavy unit weight has been used for the analysis of bearing capacity and rupture calculations.

<table>
<thead>
<tr>
<th>Location</th>
<th>Unit Weight (pcf)</th>
<th>Friction Angle (deg)</th>
<th>Cohesion (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Fill</td>
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<tr>
<td>Retained Fill</td>
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<tr>
<td>Foundation</td>
<td>120</td>
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</table>

The Contractor must verify that these design parameters are appropriate before building the MSE structure. Based on the above parameters, TBSS is certifying the internal stability of the structure only. This certification is contingent on the fill material strength parameters meeting, or exceeding, the design assumptions. Furthermore, certification is contingent on the contractor meeting or exceeding the recommended installation procedures as set forth by the contract documents, and instructions as provided by TBSS. The backfill and foundation shall be placed and compacted in accordance with the contract documents.

The shop drawings that are provided are extremely important. TBSS requires that these documents be read by the Contractor and approving agency. These general guidelines will greatly aid the reviewer and Contractor in the approval and installation of this structure. The coping shown in our submittal is for information only. The installer shall refer to the contract documents for the steel layout and the coping dimensions.
Please review the submittal as required. TBSS requires that an approved stamped calculations and shop drawings be received before fabrication of the MSE wall components or installation of the MSE wall begins.

If you require further information, clarification, or assistance, please contact this office.

Sincerely,

Christopher M. Staud  
Engineering Manager  
Licensed PE (CO, FL, GA, KS, NH, OH, TX, VA)

cc:  file 12019
CONSTRUCTION DRAWINGS

TBSS SEGMENTAL CONCRETE PANEL MSE RETAINING WALL SYSTEM
US-36 MANAGED LANES / DESIGN BUILD
COLORADO DEPARTMENT OF TRANSPORTATION
DENVER, COLORADO
PROJECT NO. 0361-093; CODE: 17516

PREPARED FOR:
AMES/GRANITE JOINT VENTURE

<table>
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<th>DRAWING INDEX</th>
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<tr>
<td>1 TITLE SHEET</td>
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<tr>
<td>2 GENERAL NOTES</td>
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<tr>
<td>3-7 MSE RETAINING WALL A2, 1R LAYOUT</td>
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<td>8-13 MSE RETAINING WALL A2, 2R LAYOUT</td>
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<td>14-19 MSE RETAINING WALL A3, 1R LAYOUT</td>
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<td>20-25 MSE RETAINING WALL A3, 2R LAYOUT</td>
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<td>26-31 MSE RETAINING WALL A3, 1R PLAN VIEW</td>
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<td>32-37 MSE RETAINING WALL A1, 1R LAYOUT</td>
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<td>38-43 MSE RETAINING WALL A1, 2R LAYOUT</td>
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<td>44-49 MSE RETAINING WALL A1, 1R PLAN VIEW</td>
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<td>50-55 STANDARD COMPONENT DETAILS</td>
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<td>56-61 INSTALLATION GUIDELINES</td>
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<td>62-67 TYPICAL WALL SECTION DETAIL</td>
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<td>68-73 PANEL BLOCKOUT FOR FIRE DETAIL</td>
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</tbody>
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* INCLUDED IN THIS SUBMITTAL
EXHIBIT B
LOCATION OF DESIGN GEOTECHNICAL BORINGS, LOGS AND FACTORED BEARING RESISTANCE
**Date Begin - End:** 5/14/12  
**Logged By:** Darrell (Vine)  
**Hor.-Vert. Datum:** UTM 13N - UTM 13N  
**Angle from Vert.:** 0 degrees  
**Weather:**  

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<tr>
<th>Elevation (feet)</th>
<th>Graphical Log</th>
<th>Northing: 497183</th>
<th>Easting: 619974</th>
<th>Surveyed Surface Elevation (ft): 5344.8</th>
<th>Surface Condition: Grass &amp; Woods</th>
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**Drill Company:** Vine Laboratories  
**Drill Crew:** Chris, Eric  
**Drill Equipment:** CME-55  
**Exploration Method:** Solid Stem Auger  
**Auger Diameter:** 4 inches  
**Hammer Type - Drop:** 140 lb. Automatic - 30"

**FIELD EXPLORATION**

**LABORATORY RESULTS**

- **Sample Type:** Claystone
- **Recovery:** BC=34 50/4
- **Moisture Content (%):** 18.0
- **Un. Compress. Strength (kpsi):** 102
- **Plasticity Index (IP):** 18

**CLAYSTONE:** gray to olive, moist, firm to very hard, iron-oxide staining, trace of lignite and calcareous deposits

The boring was terminated at approximately 44 feet below ground surface. Boring was backfilled with auger cuttings on May 14, 2012.

**GROUNDWATER LEVEL INFORMATION:**
Groundwater was not encountered during drilling or after completion.

**GENERAL NOTES:**
The boring location and elevation were surveyed by HCL.

**PROJECT NO.:** 126130-1  
**DRAWN BY:** KL  
**CHECKED BY:** BTM  
**DATE:** 5/2/2012  
**REVISED:**
**Date Begin - End:** 5/14/12

**Logged By:** Darrell (Vine)

**Hor.-Vert. Datum:** UTM 13N - UTM 13N

**Angle from Vert.:** 0 degrees

**Weather:**

---

### FIELD EXPLORATION

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5340</td>
<td></td>
<td>Topsoil</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sandy LEAN CLAY (GL): brown, moist, firm, some calcareous deposits</td>
</tr>
<tr>
<td>5335</td>
<td>BC=3</td>
<td>Clayey SAND with gravel (GC): brown, moist, medium dense</td>
</tr>
<tr>
<td></td>
<td>4</td>
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<tr>
<td></td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>5330</td>
<td>BC=8</td>
<td>Weathered CLAYSTONE: gray, moist, weathered, iron-oxide staining</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>5325</td>
<td>BC=11</td>
<td>CLAYSTONE: gray, moist, moderately hard to very hard, iron-oxide staining</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
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<td>18</td>
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### LABORATORY RESULTS

<table>
<thead>
<tr>
<th>Sample</th>
<th>AASHTO Symbol</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Un. Compress. (kips/ft²)</th>
<th>Un. Strength (ksi)</th>
<th>Plasticity Index (NP)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>18.5</td>
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</tr>
<tr>
<td>BC=8</td>
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<td>9.1</td>
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</tr>
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<tr>
<td>BC=11</td>
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<td>18.6</td>
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<td></td>
</tr>
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<td>BC=15</td>
<td>A-7.6</td>
<td>11.6</td>
<td>111</td>
<td>97</td>
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<td>37</td>
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<td></td>
<td>A-7.6</td>
<td>16.1</td>
<td>105</td>
<td>99</td>
<td>55</td>
<td>40</td>
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**PROJECT NO.:** 126130-1  
**DRAWN BY:** KL  
**CHECKED BY:** BTTM  
**DATE:** 5/21/2012  
**REVISION:** B-27

---

**BOATING LOG RE-KA-24**

---

**Boring Log RE-KA-24**

---

**JS 36 Managed Lane / BRT Design-Build Project**

---

**Segment B1**

---

**Sta. 2135+00 to Sta. 2275+00**

---

**Between Boulder and Denver, Colorado**

---

KLEINFELDER - 4815 List Drive, Unit 115 | Colorado Springs, CO 80919 | PH: 719-632-3593 | FAX: 719-632-2648 | www.kleinfelder.com
The boring was terminated at approximately 44.5 feet below ground surface. Boring was backfilled with auger cuttings on May 14, 2012.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

GENERAL NOTES:
The boring location and elevation were surveyed by HCL.
**Date Begin - End:** 5/9/12

**Logged By:** Darrel (Vine)

**Hor.-Vert. Datum:** UTM 13N - UTM 13N

**Angle from Vert.:** 0 degrees

**Weather:**

**Field Exploration**

<table>
<thead>
<tr>
<th>Surveyed Elevation (ft)</th>
<th>Graphical Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>5340</td>
<td>Topsoil</td>
</tr>
<tr>
<td>5335</td>
<td></td>
</tr>
<tr>
<td>5330</td>
<td>Weathered CLAYSTONE: olive to gray, moist, weathered, some calcareous deposits</td>
</tr>
<tr>
<td>5325</td>
<td></td>
</tr>
<tr>
<td>5320</td>
<td>CLAYSTONE: olive to gray, moist, moderately hard to very hard, some calcareous deposits</td>
</tr>
<tr>
<td>5316</td>
<td></td>
</tr>
<tr>
<td>5310</td>
<td>SANDSTONE: light brown, moist to wet, very hard, iron-oxide staining, some calcareous deposits</td>
</tr>
</tbody>
</table>

**Laboratory Results**

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Recovery</th>
<th>AASHTO</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
<th>Un Comp. Srength (lbf/In)</th>
<th>Passing Test (%)</th>
<th>Liquid Limit (lub)</th>
<th>Plasticity Index (IP=PL)</th>
<th>Cohesion (K)</th>
<th>Plasticity (IP=PL)</th>
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<tbody>
<tr>
<td>BC=6</td>
<td>7</td>
<td>14.5</td>
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<td>BC=12</td>
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<td>BC=7</td>
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<td>19.9</td>
<td>94</td>
<td>100</td>
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<tr>
<td>BC=10</td>
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</tr>
<tr>
<td>BC=50/6&quot;</td>
<td>BC=21</td>
<td>32</td>
<td>12.6</td>
<td>50/6&quot;</td>
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<tr>
<td>BC=50/4&quot;</td>
<td>BC=50/3&quot;</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

**Remarks:**

- Expansion: 0.1% under 1 ksf when wetted.
- Expansion: 2.7% under 1 ksf when wetted.

**BORING LOG RE-KA-25**

**US 36 Managed Lane / BRT Design-Build Project**
**Segment B1**
**Sta. 2135+00 to Sta. 2275+00**
**Between Boulder and Denver, Colorado**

**KLEINFELDER - 4815 List Drive, Unit 115 | Colorado Springs, CO 80919 | PH: 719-632-3593 | FAX: 719-632-2648 | www.kleinfelder.com**

B-29
### FIELD EXPLORATION

- **Surveyed Elevation (feet):** 5306.0
- **Graphical Log:**
  - **Northing:** 496648.911
  - **Easting:** 205097.242
  - **Surveyed Surface Elevation:** 5340.9
  - **Surface Condition:** Grass, caulis

#### Sample Type
- **Recovery:** BC=50/1
- **AASHTO Symbol:**
- **Moisture Content (%):**
- **Dry Density (pcf):**
- **Un-Compress. Strength (kN/m2):**
- **Passing 250 sieve (%):**
- **Liquid Limit (%):**
- **Plasticity Index:**
- **Other Tests:**

#### Description
- **SANDSTONE:** Light brown, moist to wet, very hard, iron-oxide staining, some calcareous deposits

---

**GROUNDWATER LEVEL INFORMATION:**
- Groundwater was observed at approximately 33 ft below ground surface during testing.

**GENERAL NOTES:**
- The boring location and elevation were surveyed by HCL.

---

**BORING LOG RE-KA-25**

**PROJECT NO.:** 126130-1

**DRAWN BY:** KL

**CHECKED BY:** BTM

**DATE:** 5/21/2012

**REVISED:**

**US 36 Managed Lane / BRT Design-Build Project**

**Segment B1**

**Sta. 2135+00 to Sta. 2275+00**

**Between Boulder and Denver, Colorado**

**KLEINFELDER - 4815 List Drive, Unit 115 | Colorado Springs, CO 80919 | PH: 719-632-3593 | FAX: 719-632-2648 | www.kleinfelder.com**
Date Begin - End: 4/26/12
Logged By: Dan (Vine)
Hor.-Vert. Datum: UTM 13N - UTM 13N
Angle from Vert.: 0 degrees
Weather: Clear, 60's

Drill Company: Vine Laboratories
Drill Crew: Darrell, Jeff
Drill Equipment: CME-55
Exploration Method: Solid Stem Auger
Auger Diameter: 4 inches

Hammer Type - Drop: 140 lb. Automatic - 30°

FIELD EXPLORATION

Surveyed Elevation (ft): 5380
Graphical Log

Depth (ft):

Asphalt: 6 inches
Base Course: 4 inches
Fill
Lean CLAY (CL): light brown, moist, soft to firm

Surficial sand lenses up to 3-inches

Becomes Fat (CH)

Fat CLAY with sand (CH): brown, moist, firm to hard

LABORATORY RESULTS

Sample Type: Bolt Core-5'-Jumbo, 12' increment
Recovery: 18.1

AASHTO Moisture Content (%)
Dry Density (pcf)
Passing No. 4 Sieve (%)
Passing No. 8 sieve (%)
Passing No. 12 sieve (%)
Passing No. 16 sieve (%)
Unified Soil Group (Gradation)
Plasticity Index (IPk)
Consistency (NP, %)

57 35

Expansion= 1.9% under 1 ksf when wetted.

US 36 Managed Lane / BRT Design-Build Project
Sta. 1388+50 to Sta. 2416+60 (Bridges)
Between Boulder and Denver, Colorado

KLEINFELDER - 4815 List Drive, Unit 115 | Colorado Springs, CO 80919 | PH: 719-632-3593 | FAX: 719-632-2648 | www.kleinfelder.com
**BOARING LOG BNSF-B7-KA**

**DATE:** 4/26/12

**Drill Company:** Vine Laboratories

**Hor.-Vert. Datum:** UTM 13N - UTM 13N

**Drill Crew:** Darrell, Jeff

**Angle from Vert.:** 0 degrees

**Drill Equipment:** CME-55

**Exploration Method:** Solid Stem Auger

**Hammer Type - Drop:** 140 lb. Automatic - 30°

**Auger Diameter:** 4 inches

**Surveyed Surface Elevation (ft):** 5383.7

**Surface Condition:** Asphalt

**FIELD EXPLORATION**

Fat CLAY with sand (CH): brown, moist, firm to hard

Becomes lean (CL)

Denver/rapahoe Formation

CLAYSTONE: brown to gray, moist, hard

**LABORATORY RESULTS**

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Recovery</th>
<th>AASHTO</th>
<th>Moisture (Silts)</th>
<th>Dry Density (pcf)</th>
<th>Passing No.-4</th>
<th>Sieve (%)</th>
<th>Plasticity Index</th>
<th>Plasticity (NP)</th>
<th>Expansion (%)</th>
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</thead>
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<tr>
<td>BC=7</td>
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<td>16.0</td>
<td>111</td>
<td>72</td>
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</tr>
<tr>
<td>BC=17</td>
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<td>A-7-6</td>
<td>21.8</td>
<td>101</td>
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<td>BC=30</td>
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</tr>
<tr>
<td>BC=32</td>
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</tr>
</tbody>
</table>

**GROUNDWATER LEVEL INFORMATION:**

Groundwater was observed at approximately 26 ft. below ground surface during drilling.

**GENERAL NOTES:**

The boring location and elevation were surveyed by HCL.

The boring was terminated at approximately 60 feet below ground surface. Boring was backfilled with auger cuttings on April 26, 2012.
**Date Begin - End:** 9/13/12

**Logged By:** Andrew (Vine)

**Hor.-Vert. Datum:** UTM 13N - UTM 13N

**Angle from Vert.:** 0 degrees

**Weather:**

<table>
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<tr>
<th>Approximate Elevation (feet)</th>
<th>Depth (feet)</th>
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<tbody>
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<td>5375</td>
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<td>5370</td>
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<td>5365</td>
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</tr>
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<td>5360</td>
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<td>5355</td>
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</tr>
<tr>
<td>5350</td>
<td>6</td>
</tr>
</tbody>
</table>

**Field Exploration**
- Nothing: 497390.6904 Easting: 620203.4044
- Approximate Surface Elevation (ft): 5384.3
- Surface Condition: Asphaltic concrete

- Boring drilled with no sampling down to 60 ft below ground surface.

**Laboratory Results**

**Sample Type:**
- Sample Count
- U.S. Gauge
- Samples Taken

**AASHO Symbol:**
- Recovery
- Moisture Content (%)
- Liquid Limit (%)
- Plastic Limit (%)
- Plasticity Index
- Other Test Remains

**Hammer Type - Drop:** 140 lb. Automatic - 30°

**Drill Company:** Vine Laboratories

**Drill Crew:** Matt, Barry

**Drill Equipment:** CME-55

**Exploration Method:** Solid Stem Auger

**Auger Diameter:** 4 inches
**Date Begin - End:** 9/13/12

**Logged By:** Andrew (Vine)

**Hor.-Vert. Datum:** UTM 13N - UTM 13N

**Angle from Vert.:** 0 degrees

**Drill Company:** Vine Laboratories

**Drill Crew:** Matt, Barry

**Drill Equipment:** CME-55

**Exploration Method:** Solid Stem Auger

**Auger Diameter:** 4 inches

**Hammer Type - Drop:** 140 lb. Automatic - 30°

<table>
<thead>
<tr>
<th>Approximate Elevation (feet)</th>
<th>Depth (feet)</th>
<th>Graphical Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>-5345</td>
<td>60</td>
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<td>-5340</td>
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<td>-5320</td>
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</tr>
<tr>
<td>-5315</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

Boring drilled with no sampling down to 60 ft below ground surface.

**CLAYSTONE:** olive gray, dry, very hard

**Northing:** 467090.6904

**Easting:** 820203.4044

**Approximate Surface Elevation (ft):** 5384.3

**Surface Condition:** Asphaltic concrete

---

**FIELD EXPLORATION**

**LABORATORY RESULTS**

**Sample Type**

**Recovery**

**ASTM**

**Moisture Content (%)**

**Dry Density (pcf)**

**Plasticity Index (%)**

**Other Tests/Remarks**

---

**PROJECT NO.:** 120130-1

**DRAWN BY:** JM

**CHECKED BY:** BL

**DATE:** 9/14/2012

**REVISED:**

---

**KLEINFELDER**

Bright People. Right Solutions.

---

**US 36 Managed Lane / BRT Design-Build Project**

**Sta. 1389+50 to Sta. 2416+60 (Bridges)**

**Between Boulder and Denver, Colorado**

**PLATE**

**B-72**

---

**KLEINFELDER - 4815 List Drive, Unit 115 | Colorado Springs, CO 80919 | PH: 719-632-3593 | FAX: 719-632-2549 | www.kleinfelder.com**
The boring was terminated at approximately 75 feet below ground surface. Boring was backfilled with auger cuttings on September 13, 2012.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

GENERAL NOTES:
Boring is situated approximately 25 ft south of BNSF-B7-KA. The boring location and elevation are approximate and were estimated by H2R.
The boring was terminated at approximately 65 feet below ground surface. Boring was backfilled with auger cuttings on May 02, 2012.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.

GENERAL NOTES:
The boring location and elevation were surveyed by HCL.
Notes:
Factored Bearing Resistance is for MSE Wall segments of 10 feet in length and at least 2 feet embedment. The factored bearing resistance should not be extrapolated beyond the provided data points.
EXHIBIT C

FDC #177 WALL RE-DESIGN 8 2 13
**FDC FORM**

<table>
<thead>
<tr>
<th>Field Design Change No:</th>
<th>FDC #177</th>
<th>Date</th>
<th>8/2/13</th>
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<tbody>
<tr>
<td>Project Segment / Work Element:</td>
<td>Segment B1 - BNSF Abutment Drainage</td>
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<tr>
<td>Requested By:</td>
<td>Steve Cordts</td>
<td></td>
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<td>Subject Document Type:</td>
<td>Shop Drawings</td>
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<td>Affected Sheets / Pages / Section:</td>
<td>BNSF-8637</td>
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</tbody>
</table>

**Subject:** West Side of BNSF Abutment Drain with Wall

**DESCRIPTION OF ISSUE / PROPOSED RESOLUTION**

MSE Wall B1-10R was redesigned to achieve a bikeway over BNSF Bridge coupled with ROW concerns; hence the wall is significantly taller than previously expected. Due to the redesign it is no longer possible to connect the BNSF Bridge Abutment drainage system to the wall system (as the wall is roughly 20 feet tall at this location), as required per “Section Perpendicular to Abutment (BNSF Plan Sheet B637).”

AGJV recommends bringing the pipe through an opening between the wall and the abutment and extending the pipe under the slope paving, extending to the bottom of the slope paving and daylighting.

---

**RFC**

**AUG 05 2013**

**Release for Construction**
PAY LIMITS FOR STRUCTURAL BACKFILL (CLASS 1) AND MECHANICAL REINFORCEMENT @ 50.

SECTION PERPENDICULAR TO ABUTMENT

SUBSURFACE DRAIN PIPE (6" PERFORATED PIPE) INTO MSE WALL DRAIN SYSTEM.

NOTE:
1. GEOTEXTILE REINFORCEMENT SHALL BE MOSS FABRIC WITH A MINIMUM AVERAGE ROLL VALUE OF 600 LBS/FT2 FOR INSTALLATIONS WITH A GAP AND 2400 LBS/FT2 WITHOUT A GAP BASED ON ASTM D4595.
2. GEOTEXTILE REINFORCEMENT SHALL BE INSTALLED IN A MANUFACTURER SPECIFIED DIRECTION.
3. THE GEOTEXTILE WRAP AT BACK FACES OF ABUTMENTS SHALL BE BOWED BACK AND PINNED WITH ITS END ANCHORED TO THE SOIL UNDERNEATH WITH STAPLES OR PINS.
4. MINIMUM SPLICING OF GEOTEXTILE SHAL CONSIST OF 9" OF OVERLAP.
In other words, it needs to daylight at the bottom of the slope and not midway up it.

Bob Hays, P.E.
US 36 Express Lanes Design Manager
500 Eldorado Blvd. Building 2 Suite 2301
Broomfield, CO 80021
Phone: (303) 404-7033
Cell: (303) 913 3085
Fax: (303) 404-7039
Project Website:
http://www.colorado.gov/projects/US36ExpressLanes

On Tue, Jul 30, 2013 at 11:18 AM, Emmons, Doug <Doug.Emmons@hdrinc.com> wrote:

Steve,

I talked with Bob and he is ok with day lighting the abutment under drain in place of tying into the wall drain system, but we need to take the day lighted pipe and carry it to the bottom of and under the slope paving where we can daylight it then. Proceed with the FDC as needed.

DOUG EMMONS  PE  HDR Engineering, Inc.
Professional Associate Deputy Design Manager
500 Eldorado Blvd, Suite 2301 [Broomfield, CO 80021
303.404.6967]
doug.emmons@hdrinc.com  hdrinc.com
EXHIBIT D
FDC #075 WALL OVER-EXCAVATION
DESCRIPTION OF ISSUE / PROPOSED RESOLUTION:

US 36 current Retaining Wall Plans requires 2ft of overexcavation for impermeable material for Wall B2-10R. During the shop drawing review process, it was noted that the bearing pressures provided by Vist-A-Walls was higher than the allowable bearing pressures provided by Kleinfelder for this wall. Kleinfelder was asked to review the wall and propose additional ground improvements to increase the allowable bearing pressure capacity at this wall to meet the design bearing pressures. Kleinfelder analyzed the wall and determined 2 additional feet of Class 1 overexcavation would be required between the bottom of MSE wall and the impermeable material. This Class 1 requirement modifies the overexcavation requirements shown on WD-210.
Dear Carlaufes, Jason,

Attached is a new bearing graph for Wall B1-10R. It looks like 2 feet of overexc and replace with Class 1 beneath the wall should be sufficient to get bearing resistance to an acceptable level. We will require that the same swell mitigation depth be performed below the bottom of the structural till. The Class 1 is permeable enough that it won’t act as an effective cap to water infiltration. Let me know if you have any further questions or requests.

Spencer
Notes:
Factored Bearing Resistance is for MSE Wall segments of 10 feet in length and at least 2 feet embedment.
Factored Bearing Resistance is dependant on the installation of at least 24-inches of compacted Class I Structural Fill beneath the wall strap zone.
The factored bearing resistance should not be extrapolated beyond the provided data points.
EXHIBIT E
WALL DRAIN DETAILS
EXHIBIT F
WALL B1-10R CONSTRUCTION REPAIR TIMELINE
**Wall B1-10R Construction Timeline**

03/08/13 – FDC #075 – Revised over excavation requirements

3/20/13 – Excavation for wall B1-10R begins per Inspector Daily Diary Week Ending 03.23.13

3/28/13 – FDC #090 – B1-10R lengthened to alleviate ROW issues, FDC #092 – Wall B1-10R changes to Shop Drawings.

4/22/13 – Begin Wall Construction per As-Built Schedule (PE 46 Rev 2 Full Schedule)

5/23/13 – Cut and embankment operations continued. The bottom of the wall reached for the majority of the wall’s length. Per Inspectors Daily Diary week ending 05.25.13

5/24/13 – Over excavation begins per FDC 075 – Over excavation placed as embankment fill from STA 2166+50, 80’RT to 2176+00, 80’ RT. Per Inspectors Daily Diary week ending 05.25.13

5/31/13 – Class 1 being placed, graded and compacted for top 2’ of treatment section of MSE wall B1-10R. Per Inspectors Daily Diary week ending 06.01.13

6/6/13 – Concrete pour for the leveling pas at MSE wall b1-10R per Inspectors Daily Diary week ending 06.08.13

6/10/13 – Setting panels at MSE Wall B1-10R per Inspectors Daily Diary week ending 06.08.13

6/15/13 – Placing underdrain for MSE wall B1-10R per Inspector Daily Diary week ending 06.15.13

6/17/13 – Continuing placing underdrain for B1-10R per Inspector Daily Diary week ending 06.22.13.

On 6/22/13 (pdf page 110) – there’s a description of 4” non-perforated pipe install

6/25/13 – Inspector Daily Diary week ending 06.29.13 – pdf page 24 – description of strip drain installation – “The new section of strip drain is overlapped by 6” and a 3” wide section of duct tape is used to attach a new section to old section.”

7/8/13 – Panel installation continues, strip drain extensions continue

8/2/13 – FDC #177 – B1-10R was redesigned to achieve a bikeway over BNSF Bridge couple with ROW concerns; hence the wall is significantly taller than previously expected.

8/15/13 – FDC #192 – Coping-Fence Detail at Bikeway

8/19/13 – Begin setting coping and level up on Wall B1-10R

12/02/13 – Inspector Daily Diary week ending 12.07.13 – **geomembrane installation STA 8+08 to STA 14+34**

12/12/13 – Inspector daily Diary week ending 12.14.13 – AG removed snow and frozen class 1 from behind wall B1-10R in preparation for placing class 1 structural backfill.

12/18/13 & 12/19/13 – Inspector Daily Diary week ending 12.21.13 – **Repaired 30 mil membrane where it was torn for the fence post foundations behind MSE wall B1-10R.**

1/15/14 – begin chain link fence installation
4/08/14 to 4/16/14 – PCCP EB Mainline: 2150+00-2183+50 (38'W)

4/17/14 to 4/21/14 – PCCP EB Mainline: 2167+00-2186+00 (12'W)

9/8/14 – Inspector Daily Diary week ending 09.13.14 – Observed severe erosion along MSE wall B1-10R beneath BNSF bridge running adjacent to proposed slope paving (pdf page 14)

9/10/14 – RFI #316 – Proposed repair for erosion

9/18/14 – Process Audit PA_creich~309 –NCR 2 for failure to protect work


11/10/14 – Inspector Daily Diary week ending 11.15.14 – observed geomembrane installation from STA 15+08 to 15+72.67

7/10/15 – RFI #432A – erosion unknown and not referenced in RFI 316 is found

7/17/15 – Inspector Daily Diary week ending 07.18.15 - Flow fill for B1-10R in accordance with RFI 432A – approximately 40 CY.

7/11/19 – Wall failure begins
EXHIBIT G
PRECIPITATION RECORDS
Record of Climatological Observations
These data are quality controlled and may not be identical to the original observations.
Generated on 09/11/2019

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Summary: 7.64

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Generated on 08/29/2019

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**Summary:** 5.46 | 0.0

Empty, or blank, cells indicate that a data observation was not reported.

*Ground Cover: 1=Grass; 2=Fallow; 3=Bare Ground; 4=Bronze grass; 5=Sod; 6=Straw much; 7=Grass much; 8=Bare much; 9=Unknown

*s" This data value failed one of NCDC's quality control tests.

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# Record of Climatological Observations

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Generated on 09/29/2019

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Empty, or blank, cells indicate that a data observation was not reported.

*Ground Cover: 1=Grass; 2=Fallow; 3=Bare Ground; 4=Brine grass; 5=Sod; 6=Straw mulch; 7=Grass mulch; 8=Bare mulch; 9=Unknown

"a" This data value failed one of NCDC's quality control tests.

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**Record of Climatological Observations**

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Generated on 09/10/2019

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**Summary**

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Empty, or blank, cells indicate that a data observation was not reported.

*Ground Cover: 1=Grass; 2=Fallow; 3=Bare Ground; 4=Brome grass; 5=Sod; 6=Straw mulch; 7=Grass muck; 8=Bare muck; 0=Unknown

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# Record of Climatological Observations

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Generated on 09/10/2019

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Empty, or blank, cells indicate that a data observation was not reported.

*Ground Cover: 1=Grass; 2=Fallow; 3=Bare Ground; 4=Brome grass; 5=Sod; 6=Straw mulch; 7=Grass muck; 8=Bare muck; 0=Unknown

*s* This data value failed one of NCDC's quality control tests.

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# Record of Climatological Observations

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Generated on 09/10/2019

Observation Time Temperature: Unknown  Observation Time Precipitation: Unknown

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**Summary**

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*Ground Cover: 1=Grass; 2=Fallow; 3=Bare Ground; 4=Brome grass; 5=Sod; 6=Straw mulch; 7=Grass muck; 8=Bare muck; 0=Unknown

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## Record of Climatological Observations

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Generated on 09/11/2019

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Generated on 09/11/2019

Observation Time Temperature: Unknown Observation Time Precipitation: Unknown

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Empty, or blank, cells indicate that a data observation was not reported.

*Ground Cover: 1=Grass; 2=Fallow; 3=Bare Ground; 4=Brome grass; 5=Sod; 6=Straw mulch; 7=Grass mulch; 8=Bare mulch; 0=Unknown

*"T" values in the Precipitation or Snow category above indicate a "trace" value was recorded.

*A" values in the Precipitation Flag or the Snow Flag column indicate a multiday total, accumulated since last measurement, is being used.

Data value inconsistency may be present due to rounding calculations during the conversion process from SI metric units to standard imperial units.
EXHIBIT H

CDOT US 36 PAVEMENT SETTLEMENT AREA MAP
REQUEST FOR INFORMATION

Date Requested: 7/10/2015

Date Response Requested By:
7/14/2015

Discipline: Structures/Walls

Data Requested

Ames/Granite (AG) has recently discovered that MSE Wall B1-10R experienced erosion which is believed to be from the heavy rains in 2013 that was unknown and not referenced in HDR RFI 316. The attached plan sheet shows the approximate limits of the visible erosion. It is difficult to see the extents of the erosion as there is only an approximate 1.5' gap between the sidewalk and the wall. At this time the wall does not show any signs of negative batter from the erosion. AG is requesting both Big R Bridge and HDR review RFI 316 (attached) and state if the same procedures for repair are acceptable.

RFI 432A revision:

On 7/10/15 Ames/Granite, HDR, CDOT, and GROUND Engineering met onsite to review the erosion at the above referenced location. After all parties had the opportunity to investigate it was determined that it was difficult to determine the exact extent of the erosion but to the best of our ability we were able to determine that the depth from top of coping to bottom of wall varies from 2' to 10' and the width was approximately 0'-14' from the wall towards the roadway.

The method of repair that was discussed during this meeting was filling the void with flow fill in a maximum of 3' lifts as described in RFI 316 response. The flowfill will be placed in the opening of the membrane on closest to the abutment in efforts to maintain the intact membrane. To assure that the flowfill fills the entire void Ames/Granite would drill ½” to 1” diameter holes in the approach slab side walk to serve as witness holes allowing observation confirming that the void is being filled. Can you please state if this is an acceptable method of backfilling and observation to assure the cavity is filled.

Response

Following site visit, observed minor deflection of sleeper relative to approach is within acceptable limits (~1/4’). Therefore, repair procedures are still acceptable (with concurrence from Chris Staud at Big R/Vista agreeing repair is acceptable) with the following modification:

Flow-fill shall be stopped at a level 2”-3” below bottom of approach slab. Approach slab is designed to span from abutment to sleeper slab so full bearing is not necessary. Stopping fill below approach is necessary to not allow fill between approach and sleeper slab which could effect bridge movement.

Response
By: Taylor Johnson (HDR)
Chris,

This will be taken care on this wall once the flowfill is placed. The 1' gap gets 4" flatwork between the coping and the bikepath to protect the backfill and the eliminate this problem. I believe this had not been done previously as they were waiting for fence to be installed prior to putting the flatwork.

AMay

From: Chris Staud [mailto:cstaud@bigrbridge.com]
Sent: Friday, June 26, 2015 1:30 PM
To: Johnson, Taylor; May, Andrew
Subject: RE: US 36 Express Lanes Project: HDR RFI 432: MSE Wall B1-10R - Erosion Repairs

Taylor,

The proposed method of repair is acceptable considering the extent of erosion. I am concerned that this continues to be an issue during heavy rainfall events. Steps need to be taken to limit the potential for exposure of backfill do concentrated runoff.

Yours,

Chris

Christopher M. Staud, PE
Senior Engineer
Big R Bridge I bigrbridge.com
T 817-507-0454 | C 817-807-5034
Licensed PE (CO, FL, GA, KS, NH, OH, TX, VA)

From: Johnson, Taylor [mailto:Taylor.Johnson@hdrinc.com]
Sent: Friday, June 26, 2015 2:23 PM
To: May, Andrew
Cc: Chris Staud
Subject: RE: US 36 Express Lanes Project: HDR RFI 432: MSE Wall B1-10R - Erosion Repairs

Chris,

This appears to be the same issue as before as far as the wall is concerned. Anything else jump out at you?

Andy,

Can you confirm that the sleeper slab is not settling or deflecting excessively due to the loss of material underneath it?

Taylor Johnson, PE
D 303.318.6270 – please note new number

From: May, Andrew [mailto:Andrew.May@gcinc.com]
Sent: Thursday, June 25, 2015 8:33 PM
To: 'cmlapostolides@amesco.com'
Cc: Johnson, Taylor; 'cstaud@bigbridge.com'
Subject: US 36 Express Lanes Project: HDR RFI 432: MSE Wall B1-10R - Erosion Repairs

Will you please submit the attached RFI to HDR for response.

Andy May
US 36 Express Lanes Project
Direct 303.404.6988 | Cell 812.223.4665 | Fax 303.404.6890
Andrew.May@goinc.com

Ames Construction, Inc.
Due to heavy rains in 2013, Ames/Granite JV (AGJV) has experienced significant erosion at MSE Wall B1-10R; please reference attached pictures. We have visited the site and the MSE Panels do not show batter issues.

2 Questions Need Answered

(#1) Big R Bridge – Can the eroded areas be backfilled back to finished grade with approved flow fill in 3 ft lifts? AGJV will repair any damaged staps and/or drainage prior to placing flowfill.

(#2) HDR Structures – Can the eroded portions of the bridge backfill be filled with flow fill to fill any voids? It will be very difficult to replace the damaged fabric; however, flowfill would ensure voids will be filled.

Is this acceptable?

<table>
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<tbody>
<tr>
<td>Due to heavy rains in 2013, Ames/Granite JV (AGJV) has experienced significant erosion at MSE Wall B1-10R; please reference attached pictures. We have visited the site and the MSE Panels do not show batter issues.</td>
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<tr>
<td>(#1) Big R Bridge – Can the eroded areas be backfilled back to finished grade with approved flow fill in 3 ft lifts? AGJV will repair any damaged staps and/or drainage prior to placing flowfill.</td>
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<tr>
<td>(#1) The proposed repair is acceptable. Care should be taken to restore panel alignment should panel have been displaced and to maintain alignment during placement of flowable fill. Maximum depth of fill per lift is 3.0’. Additional bracing to be used if required.</td>
</tr>
<tr>
<td>(#2) The proposed repair is acceptable. The contractor shall reset the bridge underdrain pipe such that positive flow is maintained. The bridge underdrain pipe shall be daylighted out of the proposed slope paving above the wall.</td>
</tr>
</tbody>
</table>

Response #1 Provided By: Chris Staud, Vist-A-Wall Systems
9/10/14

Response #2 Provided By: Darin Freeman, HDR
9/10/14
# Process Audit Assessment

**Date Approved** 9/18/2014

<table>
<thead>
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<th>Management Plan</th>
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**Scope**

Erosion BNSF Abt 1 RT

**Lead Responder Organization**

Ames

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<th>Lead Responder</th>
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<tr>
<td>Randy Pahlke</td>
<td>Chad Reich</td>
<td>Kurt Legerski</td>
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This report contains 3 observations

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### Part I - Process Audit

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<th>Result</th>
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<th>Response</th>
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<tbody>
<tr>
<td>1</td>
<td>US 36 Managed Lanes RFP, 12.0 - Drainage, 12.0 - Drainage,</td>
<td>The Contractor shall also design and construct the storm drainage facilities to limit drainage-related hazards within and outside the ROW, while minimizing future operation and maintenance costs, public inconvenience, flood damages, and water quality impacts during construction</td>
<td>NC-2</td>
<td>At the BNSF bridge, Abutment 1, RT location erosion has occurred. The erosion has created a hazard inside the ROW.</td>
<td></td>
<td>The erosion will be flow filled per RFI 316 and the field meet held with CDOT, AG, HDR and GROUND on 9/30/14. A QA hold point inspection will be performed prior to placing flow fill and between lifts of flow fill to ensure panel batter and strap integrity is within the specified requirements.</td>
<td>Jared Pittman</td>
<td>AG, Ground Engineering, and CDOT met in the field and the proposed repair method is acceptable.</td>
</tr>
<tr>
<td>2</td>
<td>US 36 Managed Lanes RFP, 12.0 - Drainage, 12.3 Construction Requirements</td>
<td>Drainage facilities shall be designed to accommodate the construction phasing of the Project</td>
<td>NC-2</td>
<td>The temporary drainage design does not accommodate the ability to move water from the newly constructed bridge to a inlet before causing damage to existing work.</td>
<td></td>
<td>The bridge drain pipe was daylighted in a location which caused erosion with the heavy rains in 2013. This bridge drain will be daylighted and protected to prevent additional erosion.</td>
<td>Jared Pittman</td>
<td>AG, Ground Engineering, and CDOT met in the field and the proposed repair method is acceptable.</td>
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<td>2011 CDOT Standard Specifications for Road and Bridge Construction</td>
<td>NC-2 The contractor failed to maintain the previously constructed work during all construction operations.</td>
<td>NC-2</td>
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<td>Requirement</td>
<td>Response</td>
<td>Jared Pittman</td>
<td>QA and CDOT met in the field and the repair method his acceptable.</td>
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105.19 Maintenance During Construction. The Contractor shall maintain all work that is included in the Contract during construction and until final written acceptance, except as otherwise specified in subsection 107.17. This maintenance shall constitute continuous and effective work prosecuted with adequate equipment and forces so the roadway or structures are kept in satisfactory condition at all times. In the case of a Contract involving the placement of material on or utilization of, a previously constructed subgrade, pavement structure or structure, the Contractor shall maintain the previously constructed work during all construction operations. All cost of maintaining the contract work during construction and before final written acceptance will not be paid for separately, but shall be included in the work, except as otherwise specified in subsection 107.17. 

Heavy rains in 2013 were classified as a 100 year storm and caused damage to construction areas. The bridge drain will be daylighted and protected to prevent additional erosion.
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<td>Randy Pahlke</td>
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<tr>
<td>1</td>
<td>007.JPG</td>
<td>Damage at wall B1-10R</td>
<td>Randy Pahlke</td>
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<td>005.JPG</td>
<td>Damage at wall B1-10R</td>
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Attachment 1

Wall B1-10R

003.JPG

PA_creich~309
Attachment 2

Damage at wall B1-10R

005.JPG

PA_creich~309
Title: Damage at wall B1-10R
Attachment 3

Damage at wall B1-10R

007.JPG

PA_creich~309
Title: Damage at wall B1-10R
EXHIBIT J
EVENT TIMELINE FOR FAILURE OF WALL B1-10R
Event Timeline for Failure of Wall B1-10R under US 36 EB

7/11/19, Thursday

While inspecting the 20th St Express Lane bridge with CDOT Staff Bridge, Justin Doles with Ferrovial Services brought up a crack that they had just noticed on GP lane #1, EB US 36 just east of the BNSF bridge. I drove up to the sight with Mohammed Zaina and Greg Marcuson to insect. We observed:

An approximate 2-inch-deep, 30-foot-long crack that ran along the GP #1 lane. We observed separation between the bike path and the adjacent barrier wall of 2-4 inches. The bikeway had a steel cover over the bridge expansion joint and it had bowed up several inches. We walked along the base of wall B1-10R and did not notice any cracking or buckling in the soil. We thought this might be a few panels cracking due to the high temps (96 deg F). Staff Bridge recommended that we continue to monitor the crack.
7/12/19, Friday

I received a call from Justin Doles in the A.M. that the crack had grown overnight and I drove up to the sight, arriving at 10:30 am. The crack had dropped another inch or more and had spread from GP 1 to GP 2 (further to the west). There was a lip along the longitudinal joint that could have been an issue for motorcycles. We discussed scabbing HMA over the crack, but agreed that it would only be a temporary fix and could still be a safety hazard.

I asked Ferrovial to set a closure of GP1 and GP 2, pushing all traffic into the EB Express Lane. After the closure had been set, we were able to get a closer look at the cracked area. We noticed that there was also cracking in the Express Lane and along the expansion joint between the Express Lane and GP1. This led us to close the express lane and move all traffic to the outside shoulder. As the day progressed, we continued to see movement in the gap between the bike path and the barrier wall. Our geotechnical engineer onsite began to discuss the possibility of a sliding failure. With the growing gap between the bike path and the barrier wall continuing to widen and a growing bump at the approach to the bridge, we decided to move traffic to the median shoulder.

The working theory at this point was that there was a void under the pavement and it may be caused by a sliding failure. However, there were no signs of cracking or buckling at the toe of the slope. We planned to core the concrete and then remove several panels to see what was going on underneath. We had Ground Penetrating Radar out to define the area of the void. At 9:15 we closed all of eastbound traffic, completed several cores of the concrete to reveal there was a void of several inches under the concrete. CDOT Maintenance proceeded to remove 2 concrete panels. We also cut longitudinal tie bars to the express lane and GP1. High temp for the day was 94 deg F.
On returning to the site on Saturday morning, the roadway had continued to move. The area where panels had been removed showed a fresh crack in the soil. We began to see cracks in the soil at the base of the wall and buckling of the soil at the toe of the slope. This began to make the cause of the movement clearer and point to a sliding failure, bearing capacity failure and/or a global stability failure. A strategy was developed to stabilize the wall. CDOT arranged to have embankment material hauled to the sight to build a buttress at the toe of slope in an effort to prevent additional movement and stabilize the wall. This work continued from approximately 12:00p - 6:00p. We were also planning to start removing concrete panels to reduce loading on the wall. High temp for the day was 97 deg F.

The bikepath was closed around 10:00 am and we began working on setting up a detour. We began the process of setting up an Emergency Incident Command.

Movement of the pavement and the wall continued throughout the day. The crack in the soil in the area where panels had been removed appeared to accelerate. Cracking and popping sounds were audible. A visual bowing of the MSE wall was now forming. Measuring from the fault scarpe (where panels had been removed) from 12:00 pm to 6:00 pm, the pavement had dropped approximately 12 inches or 2 inches per hour. The CDOT night crew was preparing to start removing concrete panels and work throughout the night. I had a conversation with the CDOT Foreman onsite about the safety of completing the work. Looking at the accelerated rate of movement and the unknowns surrounding the condition of the site, we cancelled the planned panel removal work for the evening.
7/14/19, Sunday

Upon returning to the site on Sunday morning, there had been a considerable amount of additional movement. New cracks were forming in the concrete hourly. Our strategy shifted to mitigating damage to the bridge and the EB median shoulder. CDOT began cutting the steel and asphalt connections between the approach slab and the bridge abutment and longitudinal joints and well as used the excavator to break up the sleeper slab.
7/15/19, Monday

On Monday morning, CDOT completed cutting all steel connections between the approach slab and the abutment. Shortly after the first major failure of the MSE wall panels occurred. Our focus then shifted to designing and implementing the detour of EB traffic into a head to head configuration on the WB side.
7/16/19, Tuesday

Tuesday is the official end of the Emergency Response period. We opened EB traffic at 5:30 am. Degradation of the wall continued.
Additional photos can be found in the shared Google Drive and are organized by day.
EXHIBIT K
GPRS REPORT – US 36 at LOWER CHURCH LAKE WESTMINSTER
Subsurface Investigation to Locate Underground Voids

Prepared For: Yeh and Associates, Inc.

Prepared By:
Ryan Shannon
Senior Project Manager-Colorado/Wyoming
7/15/2019
July 15, 2019

Yeh and Associates, Inc.
Attn: Todd Schlittenhart
Site: US 36 Eastbound at Lower Church Lake, Westminster, CO

We appreciate the opportunity to provide this report for our work completed on 7/12/2019 at the above address in Westminster, CO.

PURPOSE
The purpose of this project was to search for subsurface voids within the project boundaries provided by the client. The client was looking to determine if subsurface voids existed after a crack formed on the highway.

EQUIPMENT

- **350 MHz GPR Antenna** The antenna rolls over the surface which needs to be reasonably smooth and unobstructed in order to obtain readable scans. Obstructions such as curbs, landscaping, and vegetation will limit the feasibility of GPR. The data is displayed on a screen and marked in the field in real time. GPR works by sending pulses of energy into a material and recording the strength and the time required for the return of the reflected signal. Reflections are produced when the energy pulses enter into a material with different electrical properties from the material it left. The strength of the reflection is determined by the contrast in signal speed between the two materials. The total depth achieved can be as much as 8’ or more with this antenna but can vary widely depending on the conductivity of the materials. Depths provided should always be treated as estimates as their accuracy can be affected by multiple factors.

- **2600 MHz GPR Antenna.** The antenna is only approximately 9”x6” and rolls over the surface. The antenna needs a reasonably smooth, unobstructed surface for scanning so we would not be able to scan within 3” of obstructions such as walls and metal tracks unless they are removed prior to our work. The data is displayed on a screen during the scanning and marked on the surface in real time. GPR works by sending pulses of energy into a material and recording the strength and the time required for the return of the reflected signal. Reflections are produced when the energy pulses enter into a material with different electrical properties from the material it left. The strength of the reflection is determined by the contrast in signal speed between the two materials. The total depth achieved can be as much as 18” or more with this antenna but can vary widely depending on the conductivity of the materials and other factors such as the spacing of the reinforcing.

PROCESS
Our process began with using GPR scans in order to evaluate the data and calibrate the equipment. Based on these findings, a scanning strategy is formed, typically consisting of scanning the entire area in a grid with 5’ scan spacing in order to locate any unknown anomalies or potential voids. The GPR data is interpreted in real time and anomalies in the data are located and marked on the surface using spray paint, pin flags, etc.

LIMITATIONS
Please keep in mind that there are limitations to any subsurface investigation. The equipment may not achieve maximum effectiveness due to soil conditions, above ground obstructions, reinforced concrete, and a variety of other factors. No subsurface investigation or equipment can provide a complete image of what lies below. Our results should always be used in conjunction with as many methods as possible including consulting existing plans.
and drawings, exploratory excavation or potholing, visual inspection of above ground features, and utilization of services such as One Call/811.

Void mapping is not a definitive process. A void will be the highest amplitude negative response in the GPR signal but there are materials other than air that may cause strong negative responses. Therefore, there may be false positives in our findings. There also may be voids that are not able to be detected for a number of reasons. GPR can determine the approximate boundaries/edges of voids that are detected along with the approximate depth to the top of the void but cannot determine the depth to the bottom of the void (volume).

**FINDINGS**

We found that the soil allowed for maximum GPR depth penetration of approximately 3’ in most areas. This GPR Investigation revealed an area potentially consistent with void data signatures found within the scan area. Although GPR detected potential void data signatures in the area discussed in this report, due to varying subsurface conditions, it is possible that voids could form after our scan took place. All findings are based on interpretation of data acquired on site.

Sample void data signatures can be found on page 4 in this report. These data screenshots were not captured on this site and are only for reference of what typical void data signatures would look like.

The following pages will provide photos and further explanation of our findings.
US 36 near Lower Church Lake
Westminster, CO

Yellow indicates area found with potential void data signatures

Legend
- Approximate Scan Area
- Observed area

Aerial Photo
US 36

Google Earth
Sample GPR data screenshot. Typical data signature of a subsurface air pocket or void; identified by a bright black then white anomaly. This data screenshot was not taken from this project.

Sample GPR data screenshot. This is a larger example of a subsurface air pocket or void. This data screenshot was not taken from this project.

Sample GPR data screenshot. Typical data signature of severe separation between grade and concrete slab; identified by bright black line where grade meets concrete slab.
US 36 near Lower Church Lake
Westminster, CO

<table>
<thead>
<tr>
<th>GPR Data Screenshots</th>
<th>US 36 near Lower Church Lake Westminster, CO</th>
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350 MHz GPR data screenshot taken on site. The depth scale is on the left and the distance of the scan is across the top, forming a cross section view of the subsurface. This GPR data screenshot revealed data consistent with void data signatures (Circled in Red).

350 MHz GPR data screenshot taken on site. This GPR data screenshot revealed data consistent with void data signatures (Circled in Red).

2600 MHz GPR data screenshot taken on site. This GPR data screenshot revealed data consistent with void data signatures (Circled in Red).

2600 MHz GPR data screenshot taken on site. This GPR data screenshot revealed data consistent with void data signatures (Circled in Red).
Site Photo of crack formed on highway that the client wished to determine if subsurface voids existed below concrete slab.

Aerial Photo of area found to have data consistent with void data signatures (Highlighted Yellow).
CLOSING
GPRS, LLC has been in business since 2001, specializing in underground storage tank location, concrete scanning, utility locating, and shallow void detection for projects throughout the United States. I encourage you to visit our website (www.gprsinc.com) and contact any of the numerous references listed.

GPRS appreciates the opportunity to offer our services, and we look forward to continuing to work with you on future projects. Please feel free to contact us for additional information or with any questions you may have regarding this report.

Regards,

Ryan Shannon
Senior Project Manager – Colorado/Wyoming
GPRS, LLC
Direct: 303.913.8630
Ryan.shannon@gprsinc.com
www.gprsinc.com
EXHIBIT L
DRAIN PIPE DOCUMENTS
COLORADO DEPARTMENT OF TRANSPORTATION

FIELD REPORT FOR SAMPLE IDENTIFICATION OR MATERIALS DOCUMENTATION

Metric Units □ yes ☑ no

Sample submitted:
4" Corrugated (FE) Pipe Perforated 12,750ft
4" Corrugated (FE) Pipe Solid 2,250ft

Field Sheet No. 624-0001  Date 9/27/2012
Project No. NH 0361-093  Location US 36 Express Lanes
Project code 17516  Function  Part.

Sample submitted:
4" Corrugated (FE) Pipe Perforated 12,750ft
4" Corrugated (FE) Pipe Solid 2,250ft

Item 624  Class  Grading  Special provisions applicable: □ yes ☑ no
Previously used on project CDOT Form #157 No.  CDOT Form #633 (sack) □
NA 624-0001  CDOT Form #634 (can) □

Describe tests required, use to be made of material, and/or documentation details

Attached are the COC's for 15,000ft of solid and perforated corrugated (PE) pipe for use as MSE Wall drain pipe.

COC references ASTM F405, CDOT specifications reference AASHTO M252. The pipe was accepted for use on the project by Jason Carleafes (HDR) and Scott Rees (CDOT).

Preliminary Construction Maintenance Emergency Date needed
☑ ☑ ☐ ☐

Contractor: AMES/GRANITE  Supplier: ADS, Inc.
Sampled from: N/A  Pit name or owner: N/A

Quantity represented
Project total Solid Pipe 2,250ft
Perf Pipe 12,750ft

Previous quantity
Solid Pipe 0
Perf Pipe 0

Total quantity to date
Project total Solid Pipe 2,250ft
Perf Pipe 12,750ft

Sample submitted:
☐ Yes ☑ No
Shipped to: Central Lab ☐ Region Lab ☐

Via
Date

Sampled or inspected by (Name)
Erik K. Campbell

Title Lead QA Inspec.

Lab phone number

Supervisor (Pro./Res./Matls. Engr./Maint. Supt.)
Jarred Pittman

Title QC Manager

Address
US 36 Design Build

CDOT Form #157
Material Receiving INSPECTION REPORT

Date: 9/17/12  Shift: DAY  Test/Inspec. VISUAL  Primary Spec: 712.12  Name: JARED PITTMAN  Report No.  

Structure/Alignment  Location1: 112TH ST ADO  Location2:  

Element Name or Location  Shop Drawings/Standard Plates  Schedule ID  

Work Name  Supplier  

Weather  ADVANCED DRAINAGE SYSTEMS, INC.  

Activity Summary  Inspected permanent materials delivered to site  

Material Inspection Parameters  
(check if NOT acceptable)  

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<tr>
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<tbody>
<tr>
<td>1</td>
<td>IDENTIFICATION</td>
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<td>6</td>
<td>MATERIAL CLASSIFICATION</td>
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<tr>
<td>2</td>
<td>QUANTITY</td>
<td></td>
<td>7</td>
<td>COC's or CTR's</td>
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<tr>
<td>3</td>
<td>DAMAGE</td>
<td></td>
<td>8</td>
<td>DIMENSIONS VERIFIED</td>
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<tr>
<td>4</td>
<td>REQUIRED MARKINGS</td>
<td></td>
<td>9</td>
<td>CLEANLINESS/GOOD CONDITION</td>
</tr>
<tr>
<td>5</td>
<td>CONFORMANCE TO SPECS</td>
<td></td>
<td>10</td>
<td>PROPERLY STORED &amp; PROTECTED</td>
</tr>
</tbody>
</table>

*All material marked on hold will be marked with red ribbon*

*List below deficient Item and Qty. If all is deficient just write "All" Also mark as hold or reject*

<table>
<thead>
<tr>
<th>Item</th>
<th>Material/Equipment</th>
<th>Quantity</th>
<th>Hold / Reject</th>
</tr>
</thead>
</table>

Remarks

Deficient Element  Proposed Resolution  

Acceptable  Unacceptable  

*If all items are acceptable attach COC or CTR and forward to the CQAM for approval. Mark the adjacent check box indicating that the material is acceptable.*

Title: CQAM  Signature: 

File No.  QCP No.7.6-A  Rev. 0  

Revision: 5  

US36 Design-Build Project
Aug 27, 2012

To: Denver Industrial
850 S Lipan St
Denver Co 80223

Re: Advanced Drainage Systems, Inc.
Certification of Compliance

This will certify that the 100mm (4") I.D. corrugated polyethylene pipe and fittings manufactured by Advanced Drainage Systems, Inc. comply with the requirements for "heavy duty" pipe as specified in the latest revision of ASTM F 405 Standard Specification for Corrugated Polyethylene (PE) Pipe and Fittings.

Project: US 36
Project #: 5014 PERF PIPE
County: Jefferson
Quantity: 12750ft 4" Single Wall Reg. Perf
2250ft 4" Single Wall Reg. Solid

Sincerely,

Mike Austin
Freight Manager

I hereby certify under penalty of perjury that the material listed in this Certificate of Compliance represents All (quantity not units) of the above.

Nov 398

9/17/12

ADVANCED DRAINAGE SYSTEMS, INC., 4640 TRUeman BLVD., HILLIARD, OH 43026 PHONE: 800/733-7473
E-mail: info@ads-pipe.com  Web site: www.ads-pipe.com
ADS SINGLE WALL HEAVY DUTY PIPE SPECIFICATION

Scope
This specification describes 3- through 24-inch (75 to 600 mm) ADS single wall heavy duty polyethylene pipe, for use in gravity-flow drainage applications.

Pipe Requirements
ADS single wall corrugated heavy duty pipe shall have annular interior and exterior corrugations.
- 3- through 6-inch (75 to 150 mm) shall meet ASTM F405
- 8- through 24-inch (200 to 600 mm) shall meet ASTM F667.

Joint Performance
Joints for 3- to 24- inch (75 – 600 mm) shall be made with split or snap couplings. Standard connections shall meet the requirements of the ASTM F405 or ASTM F667. Gasketed connections shall incorporate a closed-cell synthetic expanded rubber gasket meeting the requirements of ASTM D1056 Grade 2A2. Gaskets, when applicable, shall be installed by the pipe manufacturer.

Fittings
Fittings shall conform to ASTM F405 or ASTM F667.

Material Properties
Pipe and fitting material shall be high density polyethylene conforming with the minimum requirements of cell classification 423410C as defined and described in the latest version of ASTM D3350; or ASTM D1248 Type III, Class C, Category 4, Grade P33.

Installation
Installation shall be in accordance with ASTM D2321 and ADS installation guidelines, with the exception that minimum cover in trafficked areas for 3- through 24-inch (75 to 600 mm) diameters shall be one foot (0.3 m). Contact your local ADS representative or visit our website at www.ads-pipe.com for a copy of the installation guidelines.

Pipe Dimensions

<table>
<thead>
<tr>
<th>Nominal Diameter, in (mm)</th>
<th>Pipe I.D. (in)</th>
<th>Pipe O.D. (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3 (75)</td>
<td>5 (125)</td>
</tr>
<tr>
<td></td>
<td>4 (100)</td>
<td>6 (150)</td>
</tr>
<tr>
<td></td>
<td>6 (150)</td>
<td>8 (200)</td>
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<tr>
<td></td>
<td>8 (200)</td>
<td>10 (250)</td>
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<tr>
<td></td>
<td>10 (250)</td>
<td>12 (300)</td>
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<tr>
<td></td>
<td>12 (300)</td>
<td>15 (375)</td>
</tr>
<tr>
<td></td>
<td>15 (375)</td>
<td>18 (450)</td>
</tr>
<tr>
<td></td>
<td>18 (450)</td>
<td>24 (600)</td>
</tr>
</tbody>
</table>

Perforations: All diameters available with or without perforations.

*Pipe O.D. values are provided for reference purposes only, values stated for 3- through 8-inch are ± 0.5 inch. Contact a sales representative for exact values.
As discussed, I have not verified that the pipe in the field is the pipe specified. The pipe in the documentation is adequate if the designers agree.

~~~~~~~~~~~~~~

Scott

I have spoken to Scott Rees again, and he has notified me that the pipe we have onsite is acceptable.

Thank you,

Steve
FyI....let's print everything out and go talk with cdot!

From: Carlaftes, Jason
To: Cordts, Stephen
Cc: Johnson, Bradford ; Allard, Daniel
Sent: Thu Sep 06 11:23:47 2012
Subject: ADS Single Wall Heavy Duty Pipe for MSE Wall Drainage

Steve,

I have looked at the Heavy Duty Pipe Specification from ADS. The cut sheet for the Heavy Duty Pipe references ASTM F405. The CDOT specifications for HDPE references AASHTO M252. From what I have researched, both specifications cover HDPE pipes with slightly different testing and material requirements. For the intended use of the Heavy Duty Pipe, which is the drainage system in the MSE retained zone (the longitudinal pipe and transverse daylight pipes), I believe the Heavy Duty Pipe meets the design intent for the MSE drainage and can be considered equivalent to what is required in the CDOT specifications for the intended use. Please let me know if you need anything else. Thanks.
Introduction

The information in this document is designed to provide answers to general cover height questions; the data provided is not intended to be used for project design. The design procedure described in the Structures section (Section 2) of the Drainage Handbook provides detailed information for analyzing most common installation conditions. This procedure should be utilized for project specific designs.

The two common cover height concerns are minimum cover in areas exposed to vehicular traffic and maximum cover heights. Either may be considered “worst case” scenario from a loading perspective, depending on the project conditions.

Minimum Cover in Traffic Applications

Pipe diameters from 3- through 24-inch (75-600 mm) installed in traffic areas (AASHTO H-25 or HS-25 loads) must have at least one foot (0.3m) of cover over the pipe crown. The backfill envelope must be constructed in accordance with the Installation section (Section 5) of the Drainage Handbook and the requirements of ASTM D2321. The backfill envelope must be of the type and compaction listed in Table 2-3 of the Drainage Handbook. In Table 1 below, this condition is represented by a Class III material compacted to 90% standard Proctor density, although other material can provide similar strength at slightly lower levels of compaction. Structural backfill material should extend six inches (0.15m) over the crown of the pipe; the remaining cover should be appropriate for the installation and as specified by the design engineer. If settlement or rutting is a concern, it may be appropriate to extend the structural backfill to grade. Where pavement is involved, sub-base material can be considered in the minimum burial depth. While rigid pavements can be included in the minimum cover, the thickness of flexible pavements should not be included in the minimum cover.

Additional information that may affect the cover requirements is included in the Installation section (Section 5) of the Drainage Handbook. Some examples of what may need to be considered are temporary heavy equipment, construction loading, paving equipment and similar loads that are less than the design load, the potential of pipe flotation, and the type of surface treatment which will be installed over the pipe zone.

Table 1
Minimum Cover Requirements for ADS Single Wall Highway and Heavy Duty Pipe with AASHTO H-25 or HS-25 Load

<table>
<thead>
<tr>
<th>Inside Diameter, ID, in.(mm)</th>
<th>Minimum Cover ft. (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 (75)</td>
<td>1 (0.3)</td>
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<tr>
<td>4 (100)</td>
<td>1 (0.3)</td>
</tr>
<tr>
<td>6 (150)</td>
<td>1 (0.3)</td>
</tr>
<tr>
<td>8 (200)</td>
<td>1 (0.3)</td>
</tr>
<tr>
<td>10 (250)</td>
<td>1 (0.3)</td>
</tr>
<tr>
<td>Inside Diameter, ID, in.(mm)</td>
<td>Minimum Cover ft. (m)</td>
</tr>
<tr>
<td>24 (600)</td>
<td>1 (0.3)</td>
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<tr>
<td>12 (300)</td>
<td>1 (0.3)</td>
</tr>
<tr>
<td>15 (375)</td>
<td>1 (0.3)</td>
</tr>
<tr>
<td>18 (450)</td>
<td>1 (0.3)</td>
</tr>
</tbody>
</table>

Note: Minimum covers presented here were calculated assuming Class III backfill material compacted to 90% standard Proctor density around the pipe and a minimum of 6-inches (0.15m) structural backfill over the pipe crown, as recommended in Section 5 of the Drainage Handbook, with an additional layer of compacted traffic lane sub-base for a total cover as required. In shallow traffic installations, especially where pavement is involved, a good quality compacted material to grade is required to prevent surface settlement and rutting.
Maximum Cover

Wall thrust generally governs the maximum cover a pipe can withstand and conservative maximum cover heights will result when using the information presented in the Structures section (Section 2) of the Drainage Handbook.

The maximum burial depth is highly influenced by the type of backfill and level of compaction around the pipe. General maximum cover limits for ADS Single Wall Highway and Heavy Duty pipes are shown in Table 2 for a variety of backfill conditions.

Table 2 was developed assuming pipe is installed in accordance with ASTM D2321 and the Installation section (Section 5) of the Drainage Handbook. Additionally, the calculations; assume zero hydrostatic load, incorporate the maximum safety factors represented in structures section of the Drainage Handbook, and assume the native soil is of adequate strength and is suitable for installation. For applications requiring fill heights greater than those shown in Table 2, contact the ADS Regional Engineering or Application Engineering departments.

Table 2

<table>
<thead>
<tr>
<th>Diameter in (mm)</th>
<th>Class 1 Compacted (ft)</th>
<th>Class 1 Dumped (ft)</th>
<th>Class 2 95%</th>
<th>Class 2 90%</th>
<th>Class 2 85%</th>
<th>Class 3 95%</th>
<th>Class 3 90%</th>
<th>Class 3 85%</th>
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<tbody>
<tr>
<td>4 (100)</td>
<td>41 (12.5)</td>
<td>13 (4.0)</td>
<td>27 (8.2)</td>
<td>18 (5.5)</td>
<td>13 (4.0)</td>
<td>19 (5.8)</td>
<td>13 (4.0)</td>
<td>11 (3.9)</td>
</tr>
<tr>
<td>6 (150)</td>
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<td></td>
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<td>8 (200)</td>
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<tr>
<td>10 (250)</td>
<td>38 (11.6)</td>
<td>12 (3.7)</td>
<td>25 (7.6)</td>
<td>17 (5.2)</td>
<td>12 (3.7)</td>
<td>18 (5.5)</td>
<td>12 (3.7)</td>
<td>10 (3.0)</td>
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<td>12 (300)</td>
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<td>15 (375)</td>
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<tr>
<td>24 (600)</td>
<td>32 (9.8)</td>
<td>11 (3.4)</td>
<td>21 (6.4)</td>
<td>15 (4.6)</td>
<td>11 (3.4)</td>
<td>16 (4.9)</td>
<td>11 (3.4)</td>
<td>9 (2.7)</td>
</tr>
</tbody>
</table>

Notes:
1. Results based on calculations shown in the Structures section of the ADS Drainage Handbook. Calculations assume no hydrostatic pressure and a density of 120 pcf (1926 kg/m³) for overburden material.
2. Installation assumed to be in accordance with ASTM D2321 and the Installation section of the Drainage Handbook.
3. Backfill materials and compaction levels not shown in the table may also be acceptable. Contact ADS for further detail.
4. Material must be adequately “knifed” into haunch and in between corrugations. Compaction and backfill material is assumed uniform throughout entire backfill zone.
5. Compaction levels shown are for standard Proctor density.
6. For projects where cover exceeds the maximum values listed above, contact ADS for specific design considerations.
7. Calculations assume no hydrostatic pressure. Hydrostatic pressure will result in a reduction in allowable fill height. Reduction in allowable fill height must be assessed by the design engineer for the specific field conditions.
8. Fill height for dumped Class I material incorporate an additional degree of conservatism that is difficult to assess due to the large degree of variation in the consolidation of this material as it is dumped. There is limited analytical data on its performance. For this reason, values as shown are estimated to be conservatively equivalent to Class 2, 85% SPD.
### Classes of Embedment and Backfill Materials

<table>
<thead>
<tr>
<th>CLASSES OF EMBEDMENT AND BACKFILL MATERIALS</th>
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<tbody>
<tr>
<td><strong>ASTM D2321</strong>&lt;sup&gt;1&lt;/sup&gt; (CSA B182.11)</td>
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<td>NOTATION</td>
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<td>_______</td>
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<tr>
<td><strong>CLASSED ROCK, ANGULAR</strong>&lt;sup&gt;1&lt;/sup&gt;</td>
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<tr>
<td><strong>CLEAN COURSE-GRAINED SOILS</strong></td>
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<tr>
<td><strong>COURSE-GRAINED SOILS; BODERING, CLEAN TO SW FRACTIONS</strong></td>
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**NOTES:**
1. REFER TO ASTM D2321 / CSA B182.11 / BHG 2560 FOR MORE COMPLETE SOIL DESCRIPTIONS.
2. CLASS MATERIALS ALLOW A BROAD RANGE OF FINES THAN PREVIOUS VERSIONS OF D2321 / B182.11. WHEN SPECIFYING CLASS MATERIAL FOR INFILTRATION SYSTEMS, THE ENGINEERING SHALL INCLUDE A REQUIREMENT FOR AN ACCEPTABLE LIMIT OF FINES.
3. ALL PARTICLE SIZE SIZES SHALL BE FRACHTED.
4. ASSUMES LESS THAN 5% PASSING THE SIEVE.
5. CLASS IV MATERIALS REQUIRE A GEOLOGICAL EVALUATION PRIOR TO USE AND SHOULD ONLY BE USED AS BACKFILL UNDER THE GUIDANCE OF A QUALIFIED ENGINEER.
6. THE VARIOUS FRACTIONS SHEET IS INTENDED TO PROPERLY SHOW THE劃S NATURE OF THE MATERIALS. THE INSTALLATION DETAILS PROVIDED HEREIN ARE GENERAL RECOMMENDATIONS AND ARE NOT SPECIFIC FOR THIS PROJECT. THE DESIGN ENGINEER SHALL REVIEW THESE DETAILS PRIOR TO CONSTRUCTION. IT IS THE DESIGN ENGINEER'S RESPONSIBILITY TO ENSURE THE DETAILS PROVIDED HEREIN MEET OR EXCEEDS THE APPLICABLE NATIONAL, STATE, OR LOCAL REQUIREMENTS AND TO ENSURE THAT THE DETAILS PROVIDED HEREIN ARE ACCEPTABLE FOR THIS PROJECT.
EXHIBIT M
EMAIL RE: WATERLINE PRESSURE TEST
Ron,

CDOT coordinated with the City of Westminster to perform the pressure test for the 16 inch reclaimed water line running under US36. The test was coordinated with Bret Eastberg the Reclaimed Water System Coordinator for the City of Westminster. The 24 hour pressure test was started at 1:00 P.M. on Thursday August 8th, and ended at 1:00 P.M. on Friday August 9th. The location of the test was the valve on the north side of US 36 at the SYNC 36 apartments 6963 W 109th Ave, Westminster, CO 80020. The test was witnessed by Kurt Kionka with CDOT.

Process:
The line was isolated and de-pressurized to confirm that there were no leaking valves. The line was then pressurized to 100 PSI by opening the valve on the north side of US 36 in the parking area of the SYNC 36 apartments. The valve was shut off and the section under US 36 was isolated. The section was isolated for 24 hours and the pressure gauge was read. I observed a reading of 100 PSI after 24 hours, which was the same reading at the start of the test.

Thanks,

*Kurt Kionka, P.E.*
*Region 1 North Resident Engineer*

P 303.398.6738 | F 303.398.6781 | C 720.390.8701
4670 Holly Street, Denver, CO 80216
*kurt.kionka@state.co.us* | *www.codot.gov*
EXHIBIT N

CLASS I FILL MICACEOUS OBSERVATION
Ron, At your request, this morning I looked at a composites sample of the Class 1 Structure Backfill that was available in our laboratory. In its natural state, it appeared to be crushed metamorphic rock typical of the Front Range quarries. The fines are goldish-grey-black when wetted. I took a few grams of the finer fraction without gravel, stirred with water, poured them on a paper towel and then decanted the water to leave the mineral matter. On observation with a 10x lens, I estimate these sands, silts and clays are largely comprised of muscovite and biotite mica, with minor hornblende or amphibolite, quartz, feldspar and clay. The attached photo shows the goldish glint of the muscovite mica on the right side, grading to clay-silt fines on the left. The black platy minerals are biotite mica. I consider the overall sample to be significantly micaceous.

David A. Glater, P.E., C.P.G. | Principal Geological Engineer

CTL | Thompson, Inc.
1971 W 12th Avenue
Denver, CO 80204
Office: 303-825-0777
Direct: 303-628-5022
Cell: 303-944-5021
dglater@ctltthompson.com
www.ctlt.com