Stantec

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March 16, 2012
File: 113731610

City of Winnipeg
Urban Design Division
Planning, Property and Development Department
15-30 Fort Street
Winnipeg, MB, R3C 4X5

Attention: **Kendall Thiessen, P. Eng.**
Reference: **Sturgeon Road Bridge – Response to Comments**

A Waterway Permit application was submitted by Stantec relative to the Sturgeon Road Bridge Replacement. The geotechnical report that was submitted along with the permit application is titled "Geotechnical Slope Stability Evaluation for Sturgeon Road Bridge Replacement" dated January 20, 2012. Concurrent with the Waterway Permit application, a Tender for the proposed works was issued and the Waterway Authority has been provided all Tender documentation. Stantec understands that a contract has not yet been awarded for this project.

An e-mail from your office (dated February 13, 2012) has been received requesting additional information to supplement the information already provided. For clarity, your February 13, 2012 e-mail is included in Appendix A. The questions in your e-mail have been answered in the order written.

As outlined within Addendum 2 of the Tender documentation, the configuration of the cellular concrete Light-weight fill has been altered from that shown on the slope stability analyses within our January 20, 2012 report. A summary of the revised light-weight fill geometry and implications to the slope stability already performed is provided first so that all other discussions that follow reference the most up to date stability information.

1.0 Revised Light-Weight Fill Geometry

The revised configuration of the light-weight fill geometry is as shown on Dwg's B120-12-015 and 016 as included with Addendum 2 of the project Tender. The revised geometry includes approximately the same quantity of light-weight fill as shown on the previous drawings, but the geometry has been altered to conform with construction recommendations from the material suppliers. The new geometry will consist of the light-weight fill from existing ground surface along a horizontal distance of 7.1 m and then slope up at 4H:1V.

Each stability cross section that had utilized the light-weight fill along with the proposed shear key to maintain stability conditions (i.e. Sections F, B, D and C) have been re-evaluated incorporating this revised light-weight fill geometry. The slope stability analyses for these cases are included in Appendix B. For each section, the cases of a saturated groundwater level (to original ground) and a groundwater level (GWL) at Elev. 233 m in the upper bank area have been evaluated, along with shear strength parameters within the native clay of $Ø'=16°$, $c'=5$ kPa and $Ø'=14°$, $c'=3.5$ kPa consistent with the previous analyses. The 5 m width shear key has been included as applicable. For clarity, the same figure numbers for each case as shown in our January 20, 2012 report have been used.

In all cases, the change in the light-weight fill geometry that is included on Dwg's B120-12-015 and 016 of Addendum 2 has no influence on the slope stability results already presented and therefore our recommendations as contained within the January 20, 2012 report remain valid.

2.0 Slope Movements Along North-West Abutment

As noted, there are signs of bank movements on the north-west side of the existing bridge extending from the existing abutment upstream. These observed movements are not readily visible on the historic air photos reviewed. Based upon a recent site walkover, the observed movements likely extend from immediately west of the north-west abutment corner to at least 100 m upstream. It is not exactly clear if the cause of the geometry change along this entire section is related to bank slumping or channel hydraulics, but to be conservative the former is assumed. The slumping appears to extend to approximately 7 to 10 m upslope of the winter ice edge along the approximate location shown on Drawing B120-12-007 in Appendix C. Photographs taken early March 2012 at this location are also shown in Appendix C. Photo 2, taken from approximately 30 upstream of the bridge and looking downstream shows the slump edge at 10 m± upslope of the water edge.

To account for the observed movements along this section of the Sturgeon Creek banks, the slope analyses for the Stability Sections C and D have been revised to account for the anticipated lower in-situ effective strengths within the failure zone. A summary of the revised slope stability analysis for each cross section is as outlined below. For each stability section, a review of these historical slope movements on the final recommended stability improvement works is made.

Stability Cross Section D

For the slope stability Cross Section D, the stability analysis has been revised to evaluate the effective shear strength parameters required to achieve a factor of safety of approximately unity (i.e. indicative of historical slope movements) for a critical slip surface originating at the approximate location as observed in the field. For the two groundwater level cases considered, shear strength parameters within the clay fill and native clay of Ø'=11° and c'=2 kPa are required to achieve a FS of approximately unity. The slope stability analysis output is as shown on Figures D1-1 and D1-2 in Appendix D1

The case of the previously proposed finished works for this length of creek bank (i.e. light-weight and embankment fill and 5 m width rockfill shear key) is shown on Figure D1-3 in Appendix D1. In the case shown, we have assumed that the shear strength parameters for the clay fill and native clay upslope of the rockfill shear key are as previously specified (upslope of the observed slumping area and therefore likely representative of "post peak" strengths) and downslope of the shear key residual strengths as above would apply. For the saturated and "normal" GWL cases, the critical slip surfaces have estimated FS's of 1.43 and 1.53 respectively, as shown on Figures D1-3 and D1-4 in Appendix D1. These estimated FS are approximately 0.13 lower than the modified original case as contained within Appendix B, Figures D2-4-7 and D2-4-8.

To be consistent with the original design approach, and to at least maintain the level of stability improvements already presented, we have reviewed increasing the width of the rockfill shear key within this section, and have found that a 6.5 m width shear key achieves the design intent. The slope stability analyses for the 6.5 m width shear key is as shown on Figures D1-5 and D1-6 in Appendix D1, and as shown have estimated factors of safety for the critical slip surface of 1.54 and 1.63 respectively for the saturated and "normal" GWL conditions.

This stability cross section had been taken to be representative of a 25 m± length section of creek bank extending north-west from the north west abutment. It is therefore recommended that the shear key width be increased to 6.5 m along this section, as shown schematically on Dwg. B120-12-007 in Appendix C. Note that we are in the process of updating these construction drawings and they will be provided to you office when complete.

Stability Cross Section C

For the slope stability Cross Section C, the stability analysis has been revised to evaluate the effective shear strength parameters required to achieve a factor of safety of approximately unity (i.e. indicative of historical slope movements) for a critical slip surface originating at the approximate location as observed in the field. For the two groundwater level cases considered, shear strength parameters within the clay fill and native clay of Ø'=11° and c'=2 kPa are required to achieve a FS of approximately unity. The slope stability analysis output is as shown on Figures D2-1 and D2-2 in Appendix D2

The case of the previously proposed finished works for this length of creek bank (i.e. light-weight and embankment fill) is shown on Figure D2-3 in Appendix D2. In the case shown, we have assumed that the shear strength parameters for the clay fill and native clay upslope of the location of observed movements are as previously specified (upslope of the observed slumping area and therefore likely representative of "post peak" strengths) and downslope of the observed movements residual strengths as above would apply. For the saturated and "normal" GWL cases, the critical slip surfaces have estimated FS's of 1.44 and 1.60 respectively, as shown on Figures D2-3 and D2-4 in Appendix D2. These estimated FS are approximately 0.20 lower than the modified original case as contained within Appendix B, Figures D2-5-5 and D2-5-6.

To be consistent with the original design approach, and to at least maintain the level of stability improvements already presented, we have reviewed including a rockfill shear key within this section, and have found that a 5.0 m width shear key achieves the design intent. The slope stability analyses for the 5.0 m width shear key is as shown on Figures D2-5 and D2-6 in Appendix D2, and as shown have estimated factors of safety for the critical slip surface of 1.69 and 1.88 respectively for the saturated and normal GWL conditions.

This stability cross section had been taken to be representative of a 30 m± length section of creek bank extending to the north-west limit of the retaining wall and embankment fill. It is therefore recommended that the 5.0 m width shear key be extended along the entire limit of the proposed retaining wall, as shown schematically on Dwg. B120-12-007 in Appendix C. Note that we are in the process of updating these construction drawings and will be provided to you office when complete.

3.0 Porewater Pressure Response and Settlements

3.1 Stability Analysis for Construction Conditions

The potential impact on the slope stability conditions caused by porewater pressure increases during construction have been evaluated for the slope stability cross sections E (south side) and B (north side), with the slope stability results included in Appendix E. The results of these analyses are as outlined below:

Stability Cross Section E

Assuming saturation to the original ground surface and the maximum fill placement at this location (i.e. 2.5 m), a pore pressure response of B=0.5 and B=1.0 within the clay fill and native clay have been evaluated. For B=0.5, the estimated FS of the critical overall slip surface through the rockfill shear key is 1.45, as shown on Figure E1-1 in Appendix E1. This would represent a decrease of 0.07 over the condition of no porewater pressure response. For the case of B=1.0, the estimated FS of the overall critical slip surface through the rockfill shear key is 1.37, as shown on Figure E1-3 in Appendix E1, or a decrease of 0.15 over the condition of no porewater pressure response. With the fill loading and rockfill shear key in place, a $Ø_u$=0 analysis for the clay fill and native clay materials has been reviewed. For C_u=70 kPa (clay fill) and C_u=25 kPa (native clay), the FS of the overall critical slip surface is estimated at 2.82, as shown on Figure E1-5 in Appendix E1.

Stability Cross Section B

Assuming saturation to the original ground surface and the maximum fill placement at this location (i.e. 5 m of light-weight fill and conventional fill), a pore pressure response of B=0.5 and B=1.0 within the clay fill and native clay have been evaluated. For B=0.5, the estimated FS of the critical overall slip surface through the rockfill shear key is 1.48, as shown on Figure E2-1 in Appendix E2. This would represent a decrease of 0.15 over the condition of no porewater pressure response. For the case of B=1.0, the estimated FS of the overall critical slip surface through the rockfill shear key is 1.31, as shown on Figure E2-3 in Appendix E2, or a decrease of 0.32 over the condition of no porewater pressure response. With the fill loading and rockfill shear key in place, a $Ø_u$=0 analysis for the clay fill and native clay materials has been reviewed. For C_u=70 kPa (clay fill) and C_u=25 kPa (native clay), the FS of the overall critical slip surface is estimated at 2.19, as shown on Figure E2-5 in Appendix E2.

3.2 Construction Sequencing

Based on the results of the analysis presented above in Section 3.1 and comparison with Canadian Dam Safety guidelines for end of construction conditions, the need to stage or sequence the fill placement is not considered necessary. This however will be reviewed as part of the monitoring program

3.3 Potential Settlements

As stated, the proposed fill placement above existing grade will be a maximum of approximately 2.5 m on the south abutment and 4.5 to 5 m on the north abutment. From oedometer testing performed on samples collected from TH-3, the estimated preconsolidation pressures of the native clay range from 125 kPa at 3 m depth (Elev. 231.1 m) to 175 kPa at 6 m depth (Elev. 228.1 m). Based on these preconsolidation pressures, the in-situ clay material is relatively heavily overconsolidated (OCR between 3 and 4, assuming groundwater table at surface).

For the south abutment, the maximum fill placement of 2.5 m would represent an additional applied vertical effective stress of approximately 45 kPa. For this anticipated maximum stress increase, the estimated settlement of in-situ overburden clay ranges from 100 mm to 150 mm. For this loading, the in-situ soils remain overconsolidated and settlements may be taken as "elastic".

For the north abutment, the maximum fill placement will be approximately 4.5 to 5 m, with the fill being made up of the light-weight fill and conventional fill. At the maximum height of the light-weight fill, the applied vertical effective stress increase is anticipated to be 30 kPa which would cause potential settlements of 70 to 100 mm. Beyond the limits of the Cematrix fill, the maximum conventional fill loading is estimated to be approximately 80 to 90 kPa resulting in settlements of the in-situ clay of 120 to 200 mm. In both of these cases, the in-situ clay soils remain overconsolidated with the fill loading.

The time to reach 90% consolidation of the in-situ clay soils from the fill loading are difficult to predict with some accuracy. Assuming a hydraulic conductivity within the native clay of 10^{-6} m/s, the time for 90% consolidation would be in the order of days. If a hydraulic conductivity of 10^{-8} is used, 90% consolidation might take up to a year. In all cases, the majority of the anticipated settlements of the in-situ soils from the fill loading are expected within the construction duration.

4.0 Shear Key Cut Slopes

The proposed shear key has been shown on the drawings with vertical side slopes to indicate to the Contractor that this configuration is acceptable in an effort to minimize excavation and rockfill quantities. The in-situ soils that will be removed generally consist of clay fill or silty clay, and the temporary vertical excavation is considered possible depending on the exact time of the excavation and groundwater infiltration during the excavation. During construction, if it is necessary to slope the shear key excavation because of sloughing sidewalls, there is adequate plan area to achieve a 1H:2V cut. The Contractor is compensated on a unit price basis for both excavation and rock fill placement.

5.0 Shear Strength Parameters

Your comment has been noted, and we agree that $\varnothing'=14°$ and $c'=3.5$ kPa would be on the upper limit of "residual" shear strengths for typical Winnipeg clay. A better term for these strengths is lower bound of post peak conditions.

6.0 Monitoring Program

A slope monitoring program consisting of 8 vibrating wire piezometers and 4 slope inclinometers has been included for this project, and is generally as outlined within Sec E55 of the Addendum 2 of the Tender.

As outlined within the Tender documentation, the exact location of the instrumentation will be developed in consultation with the Contractor so that locations can be provided that will maximize the information benefit and minimize the potential for damage. Once the instrumentation has been installed, the frequency and length of the monitoring program is going to be a function of the monitoring results and the impact to stability from those results. An update on the monitoring frequency will be provided to your office when the instrumentation has been installed and initial results have been obtained.

7.0 Existing Buried Utilities

It is our understanding that the surcharge loads from the proposed fill placement have been reviewed and approved by the respective utility service owners.

During excavation of the shear keys, all reasonable care will be exercised when excavating adjacent to the existing infrastructure. During excavation, the Contractor should verify the alignment and elevation of the outfalls by exposing the crown of the outfall where it intersects with the shear key. Exposing of the outfalls should be completed by hydro excavation methods. The outfalls will remain exposed during the construction of the shear key to ensure the outfall does not get damaged.

Given the significant excavation that will be performed adjacent to the existing 300 mm diameter watermain on the west side of the bridge, and considering that this watermain likely supplies the Grace Hospital, extra care and attention should be paid to protecting the service and having a contingency plan in the event service is disrupted. As this watermain is located adjacent to Phase 2 of the project, there is time to develop an adequate plan prior to any construction activities. Additional review and consultation regarding the excavation adjacent to this watermain is going to be performed with the relevant stakeholders and a appropriate plan will be supplied to your office when available.

8.0 Construction Activities

Seasonal and Scheduling Limitations

The water level in the creek may impact the shear key construction, if the water level corresponds to a significant flood event over a significant duration. The shear key installation is considered relatively routine provided the creek water level remains at elevation 231.5 m or lower. Creek levels in excess of 232.0 m will likely make excavation of the shear key difficult, and given the shear key must be installed prior to any abutment fill placement, creek levels above 232.0 m for extended periods (when the shear keys are scheduled to be installed) may impact scheduling. This may be mitigated by water control activities which can be reviewed as required.

As the majority of the slope analysis has assumed a "critical" condition of existing ground saturation, construction limitations away from the creek edge based upon seasonal groundwater level conditions appear limited.

General Construction Guidelines – Phase 1

- The proposed Contractor staging area for the Phase 1 construction is as shown on Dwg. B120-12-005. This staging area is located approximately 50 m from the creek edge and is considered suitable for the temporary stockpile of construction equipment and short term stockpile of a reasonable amount of fill (i.e. amount of fill that may be placed to its final location within 1-2 days of temporary storage).

- As outlined within the specifications, the rockfill shear key is to be installed prior to any abutment fill placement (temporary or permanent). During construction of the shear key, it has been assumed that an excavator equivalent to a Caterpillar 330 would be utilized for the trench excavation. This excavator (approximately 70,000 lbs) would have a track contact pressure of approximately 30 kPa, and the anticipated impact to stability from the temporary use of this type of equipment at the shear key location is considered minor.

- As specified, all excavated material from the shear key trench is to be removed from the site immediately upon excavation, with no stockpile of excavated material allowed on the creek bank. Rockfill is also specified to be placed directly into the excavation. For excavated material removal and rockfill delivery, it will likely be necessary for the contractor to provide an access roadway from the upper bank to the lower bank areas. We recommend that all temporary access roads consist of cut below existing grade as opposed to fill above existing grade so that there is no negative impact to the stability. The use of conventional semi-load or tandem trucks or medium sized rock trucks is considered acceptable on temporary access roads.

- Based on our understanding of the project, there is no significant in-water works planned.
- Construction related surcharge loading from fill material stockpile should be constrained to the Contractors staging area until the construction of the shear key is complete. Upon completion of the shear key, select areas outside of the staging area may be used for temporary material stockpile based upon independent analyses and approval.
- It is difficult to predict the impact to the creek bank stability from potential very large cranes or other extremely heavy equipment (say in excess of 100,000 lbs) that may be used sporadically or for very specialized tasks. For very large equipment, or for equipment that will be lifting very heavy loads (such as bridge sections), independent analysis should be performed.

General Construction Guidelines – Phase 2

For the Phase 2 construction, all general guidelines as given above for the Phase 1 construction should be followed. Given the observed bank movements along a section of this portion of the work, the following additional recommendations are made:

- The temporary stockpile of all fill materials be confined to the north-east limit of the Phase 2 staging area (staging area as shown on Dwg. B120-12-006). All stockpiled materials should be stored no closer than 40 m from the normal creek edge.
- The temporary storage of conventional construction equipment (excavators, trucks, etc) is kept within the Phase 2 staging area and no closer than 20 m from the normal water edge.
- The temporary stockpile of unusually heavy or large equipment (i.e. large cranes or equipment in excess of 100.000 lbs) be stored no closer than 40 m from the normal creek edge within the staging area.

9.0 Compaction of Rockfill within Shear Key

For the analyses that have been performed, a friction angle within the shear key rockfill of Ø'=40° has been used. For the materials specified, we feel this value is easily achievable without providing significant compactive effort.

The vibratory compaction of rockfill columns or shear keys utilizing the large lances that are locally available have typically required the mobilization of a large crane to lift the lance in place. At the time the Tender was prepared for this project, it was felt that the benefit to stability with the compaction of the rockfill (i.e. being able to utilize a higher shear strength) did not justify the anticipated costs of the equipment mobilization.

We now understand that a smaller lance able to be placed utilizing a conventional excavator may be available. When a Contractor has been awarded this work, we will obtain a cost to compact the rockfill and make a determination at that time if the potential benefit (reducing the shear key width) outweighs the costs.

10.0 Background Reports

"Final Report Geotechnical Investigation and Foundation Engineering for Sturgeon Creek Bridge Replacement" National Testing Laboratories Limited dated October 28, 2011 attached.

11.0 Drawing Details

Drawing B120-2-016 has been revised to eliminate the sub-cutting of the light-weight concrete fill below existing grade. This drawing has been submitted to your office as part of Addendum 2.

The riprap shown on Section B, Drawing B120-12-060 is to drain elsewhere along the wall. Drawing B120-12-057 shows the drainage pattern in plan. The drainage of the riprap is shown to go back under both bridges and then down and towards the river through the riprap. The purpose of the riprap is architectural in nature.

The material beneath the retaining wall footings in Sections A and C (Drawing B120-12-060) will likely consist of the in-situ clay fill or the native silty clay.

Sealed drawings will be provided when provided to the successful contractor.

Thank you for your thorough review of the documentation provided as part of the Waterway Permit application for this project. If you have any questions, or require additional information, please call the undersigned.

Respectfully,

STANTEC CONSULTING LTD.

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Senior Geotechnical Engineer
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Thomas.crilly@stantec.com

Attachement: Appendices

Appendix A – February 13, 2012 E-mail

From: Thiessen, Kendall [KThiessen@winnipeg.ca]
Sent: Monday, February 13, 2012 5:41 PM
To: Crilly, Thomas
Cc: Amy, Kevin; Klowak, Matthew
Subject: RE: Sturgeon
Attachments: RE: Sturgeon Road Bridge - Waterways Permit; Sturgeon Creek Bridge - Permit Application

Tom,

I hope you've had an enjoyable vacation. I've taken a look through the geotech report and drawings for the Sturgeon Creek Bridge Replacement. I trust that you've been updated on the conversations between Stantec, COW Public Works and Waterways The brief meeting notes, with my comments are attached.

Following is a list of items for which we're requesting clarification, additional comment, additional analysis, or documentation:

- During our site visit we identified signs of slope instabilities along the left bank, west of the proposed bridge. If your investigations have reasonably confirmed that this is likely an unstable area, then it needs to be considered with respect to the proposed development, as it may affect several design components. At the minimum, even if your analysis finds that it does not impact the current design, we want to have drawings that show the unstable area relative to the proposed works, and a comment indicating summarizing your investigation.
- As it relates to our previous discussion regarding a porewater pressure response during embankment construction,
  - Can you comment on your stability analysis for construction and post-construction conditions
  - Can you provide recommendations with respect to construction sequencing if there are any concerns.
  - While it is not directly a riverbank concern, has consolidation been considered?
- → The shear key shows vertical side slopes in the drawing. Do the ground conditions indicate that vertical cuts will be feasible? Is there a contingency or space to allow for a sloped excavation if required?
- While I am not questioning your choice of design strength parameters for the in-situ soil (phi=14 and c=3.5), I wonder whether describing them as residual strengths is accurate?
- Can you propose a construction and post-construction instrumentation and monitoring program?
- Regarding existing buried utilities:
  - Are there potential impacts of surcharge loading, and slope deformations on the existing infrastructure (including watermains, LDS pipes and outfalls) during and following construction?
  - Are there special considerations for protecting the existing infrastructure where the proposed shear key excavation is going to be in close proximity?
- Regarding construction:
  - Are there any seasonal and scheduling considerations or limitations?
  - Can you provide guidelines for the contractor for any staging, lay-down areas, temporary access, temporary works, excavations, in-water work, construction related surcharge loading etc.. This was discussed at our meeting on February 6, 2012, and it may be helpful if we spoke directly.
- Have you considered compacting the rockfill in the shear key?
- Please provide the geotechnical report for the bridge foundation design so that we have it on file. We won't be doing a detailed review of it, but want to have a copy as the foundation does "interact" with the riverbanks.
- I have received the hydraulics assessment (Unies), and letter to DFO. Thanks.

Drawing Details:

- I understand that Drawing B120-2-016 is to be revised to eliminate the sub-cutting of the light-weight concrete. Please confirm, and re-submit if required.
- How will the rip-rap shown in Section B, Drawing B120-12-060 be drained? What is the purpose of this

rip-rap (possibly a deterrent to pedestrians)?
- What is the material shown beneath the retaining wall footings in Section A and C, Drawing B120-12-060?
- We will require stamped construction drawings.

Please don't hesitate to contact me for further clarification or with additional comments. As you've suggested, it may be helpful to meet briefly to work through these comments and to follow-up on the meeting we had in your absence.

Kendall Thiessen
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Urban Design Division - Planning, Property and Development Department | 15 - 30 Fort Street | Winnipeg MB | R3C 4X5
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kthiessen@winnipeg.ca

From: Crilly, Thomas [mailto:Thomas.Crilly@stantec.com]
Sent: Monday, February 13, 2012 10:07 AM
To: Thiessen, Kendall
Cc: Amy, Kevin
Subject: Sturgeon

Kendall, good morning.

I am back from vacation and getting ready to address your comments on our permit application submission for the Sturgeon bridge. There were our discussions on site and there was an e-mail sent (by you) January 25 that informally summarized our discussions. I understand there were some additional discussions while I was away (with the City), and that there may be a formal letter forthcoming from your office.

Once this formal letter is sent, it may be a good idea to sit down and review. This week would be good if you have a chance.

FYI, I have not done much as yet to address the informal comments, but all were good comments and all need to be answered.

Cheers.

Thomas Crilly, M. Sc., P. Eng.
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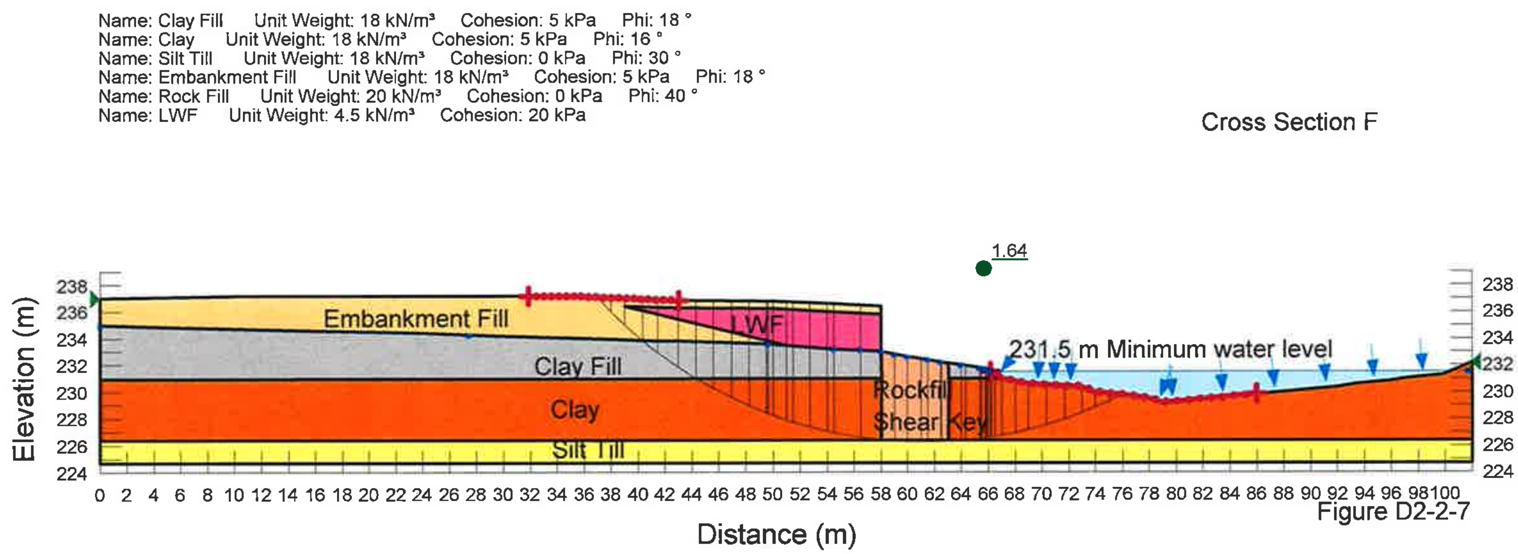
Appendix B – Revised Light-weight Fill Geometry Slope Stability Output



Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Cross Section F
1.64
Embankment Fill
LWF
Clay Fill
Clay
Silt Till
Rockfill Shear Key
231.5 m Minimum water level
Elevation (m)
Distance (m)

Figure D2-2-7

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °

Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °

Name: Embankment Fill Unit Weight: 20 kN/m³ Cohesion: 5 kPa Phi: 20 °

Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °

Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

Section F Profile

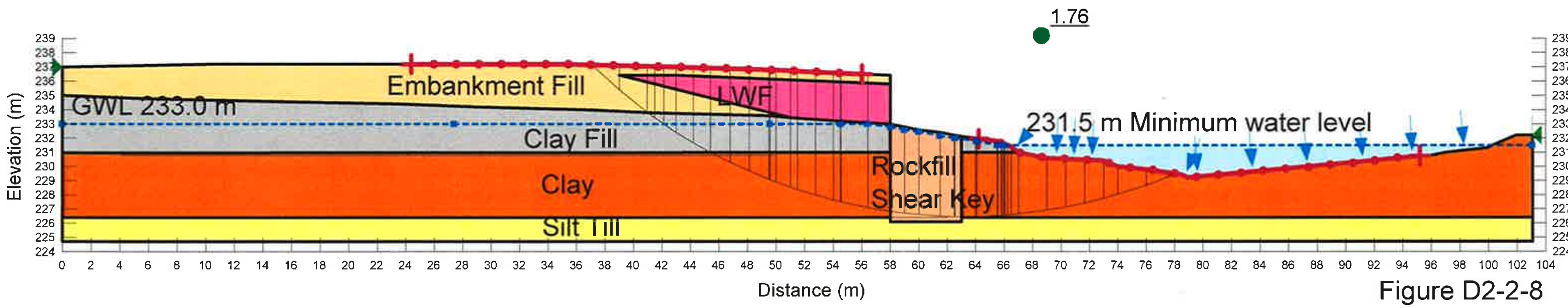


Figure D2-2-8

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

Cross Section F

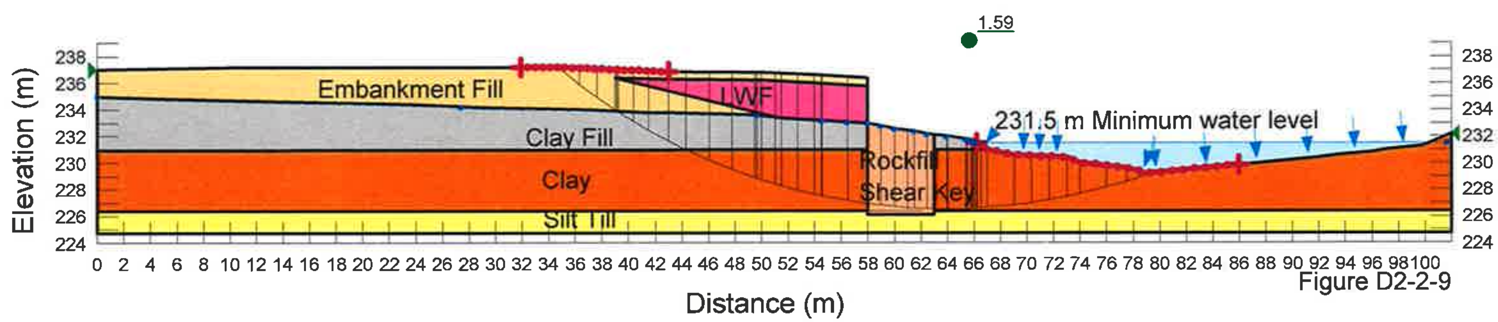


Figure D2-2-9

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 20 kN/m³ Cohesion: 5 kPa Phi: 20 °
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

Section F Profile

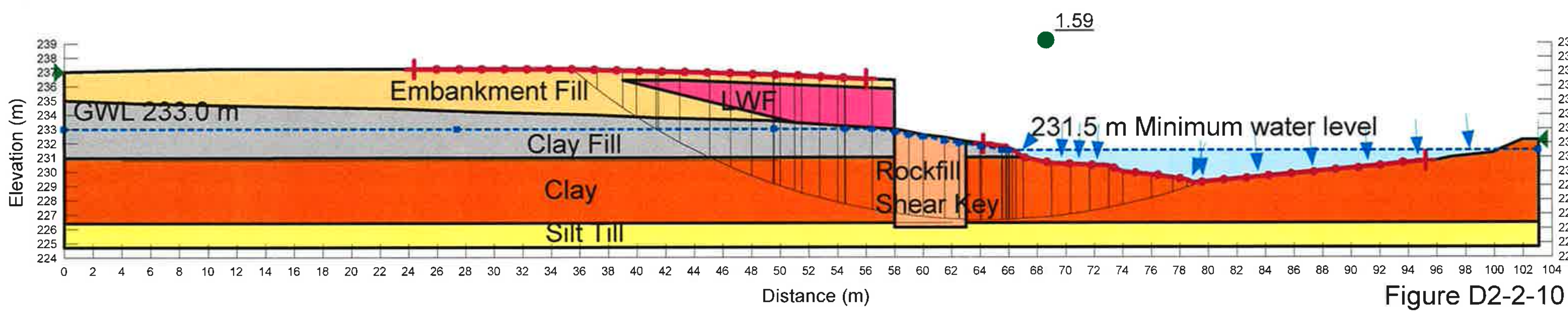


Figure D2-2-10

Cross Section B

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

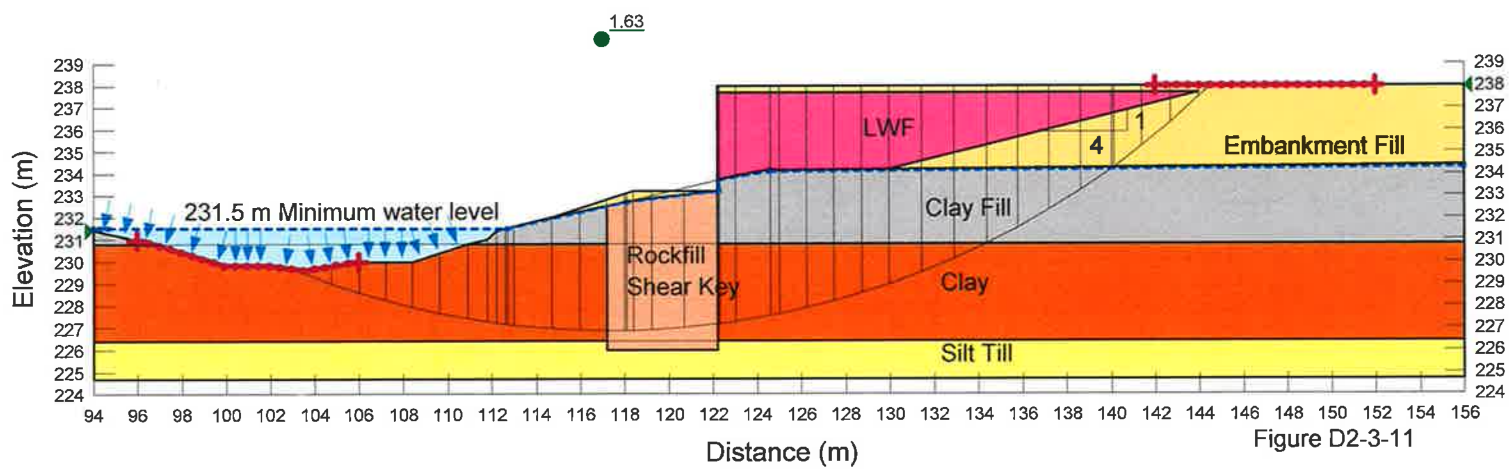


Figure D2-3-11

Cross Section B

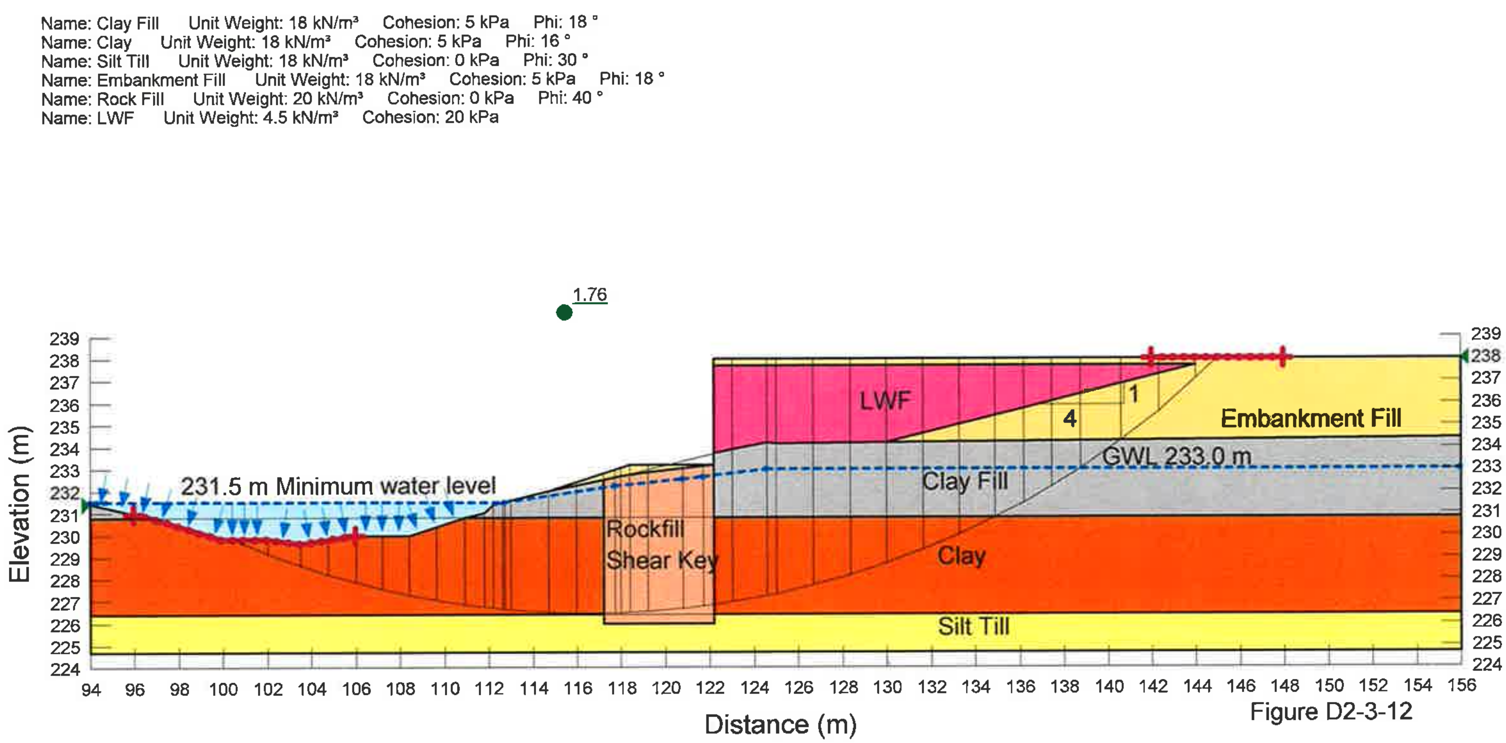


Figure D2-3-12

Cross Section B

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

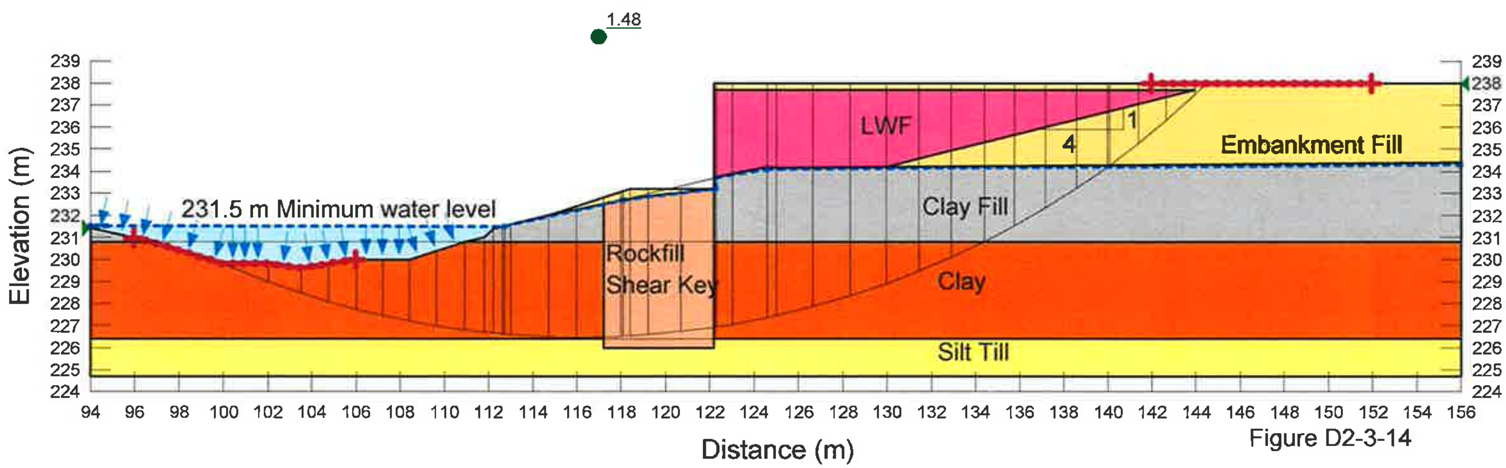


Figure D2-3-14

Cross Section B

Name: Clay Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: Clay   Unit Weight: 18 kN/m³   Cohesion: 3.5 kPa   Phi: 14 °
Name: Silt Till   Unit Weight: 18 kN/m³   Cohesion: 0 kPa   Phi: 30 °
Name: Embankment Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: Rock Fill   Unit Weight: 20 kN/m³   Cohesion: 0 kPa   Phi: 40 °
Name: LWF   Unit Weight: 4.5 kN/m³   Cohesion: 20 kPa

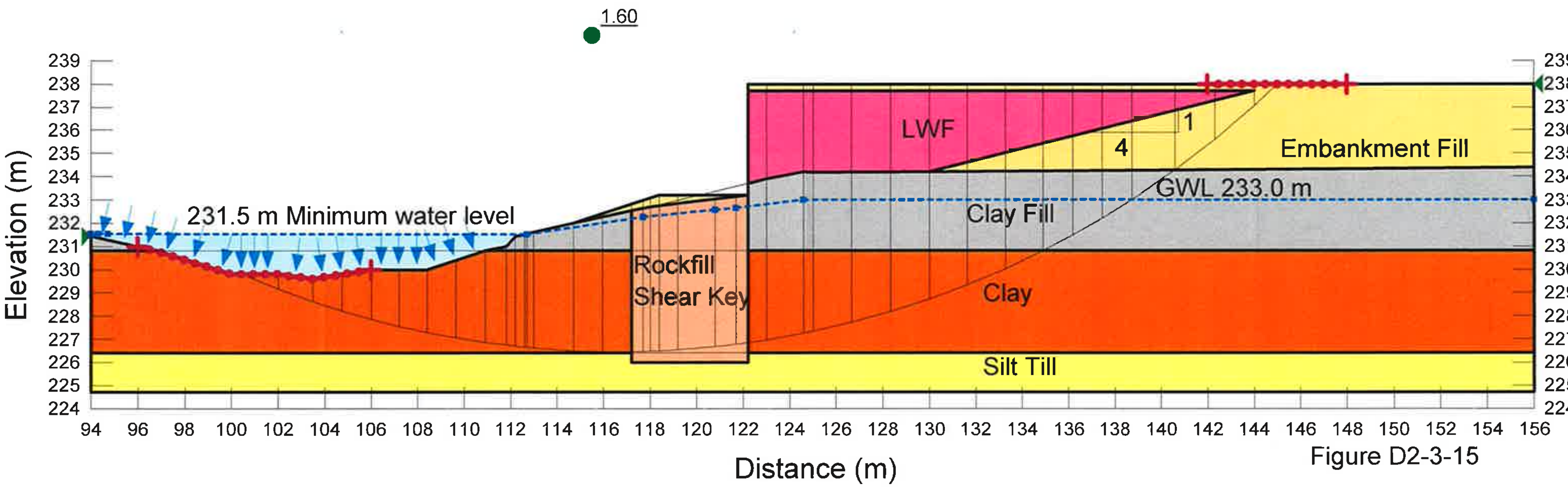


Figure D2-3-15

Cross Section D

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

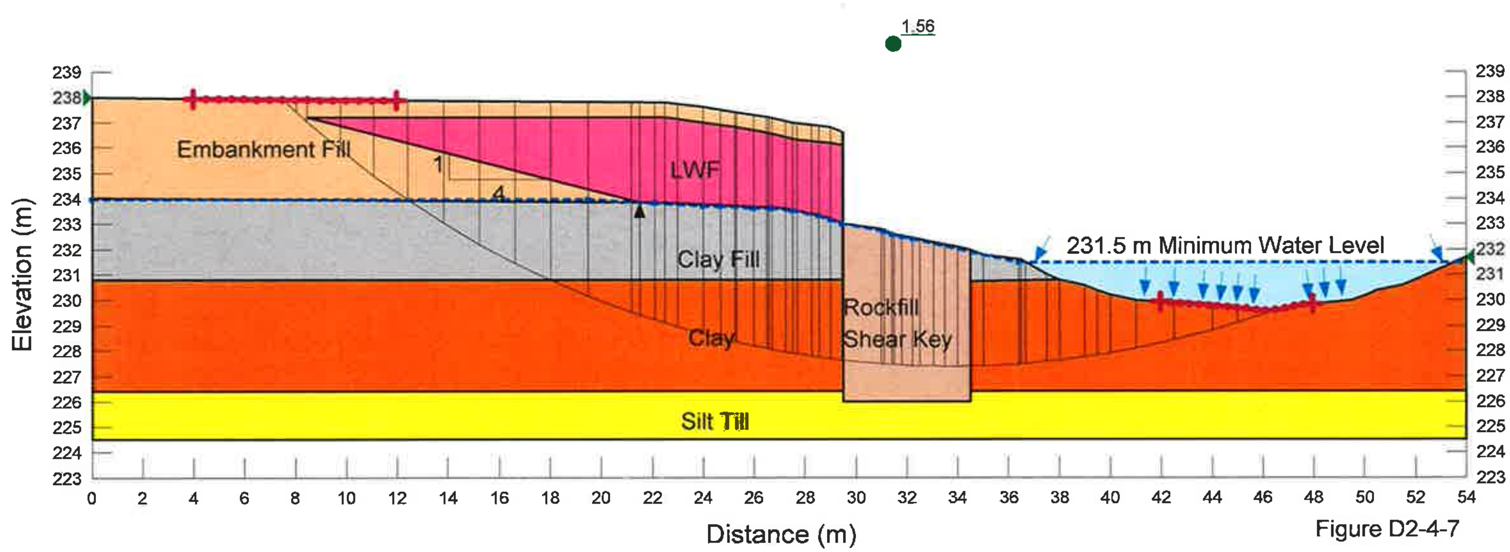


Figure D2-4-7

Cross Section D

Name: Clay Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: Clay   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 16 °
Name: Silt Till   Unit Weight: 18 kN/m³   Cohesion: 0 kPa   Phi: 30 °
Name: Embankment Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: Rockfill   Unit Weight: 20 kN/m³   Cohesion: 0 kPa   Phi: 40 °
Name: LWF   Unit Weight: 4.5 kN/m³   Cohesion: 20 kPa

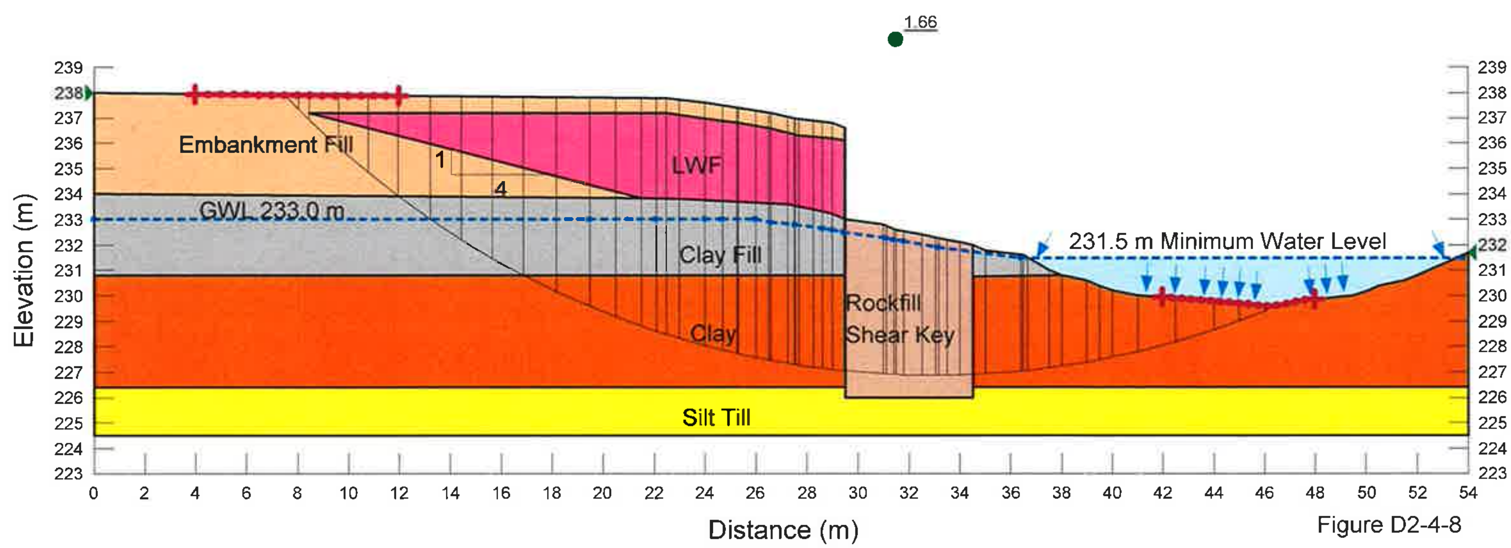


Figure D2-4-8

Cross Section D

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

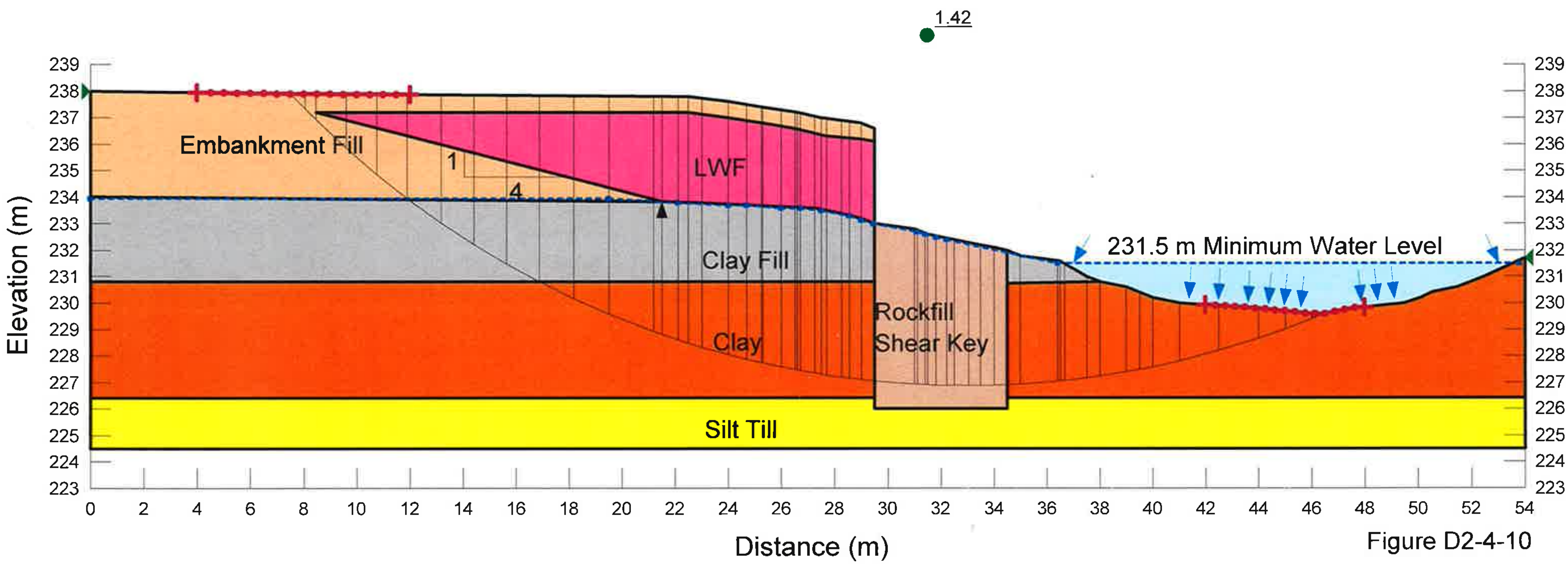


Figure D2-4-10

Cross Section D

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

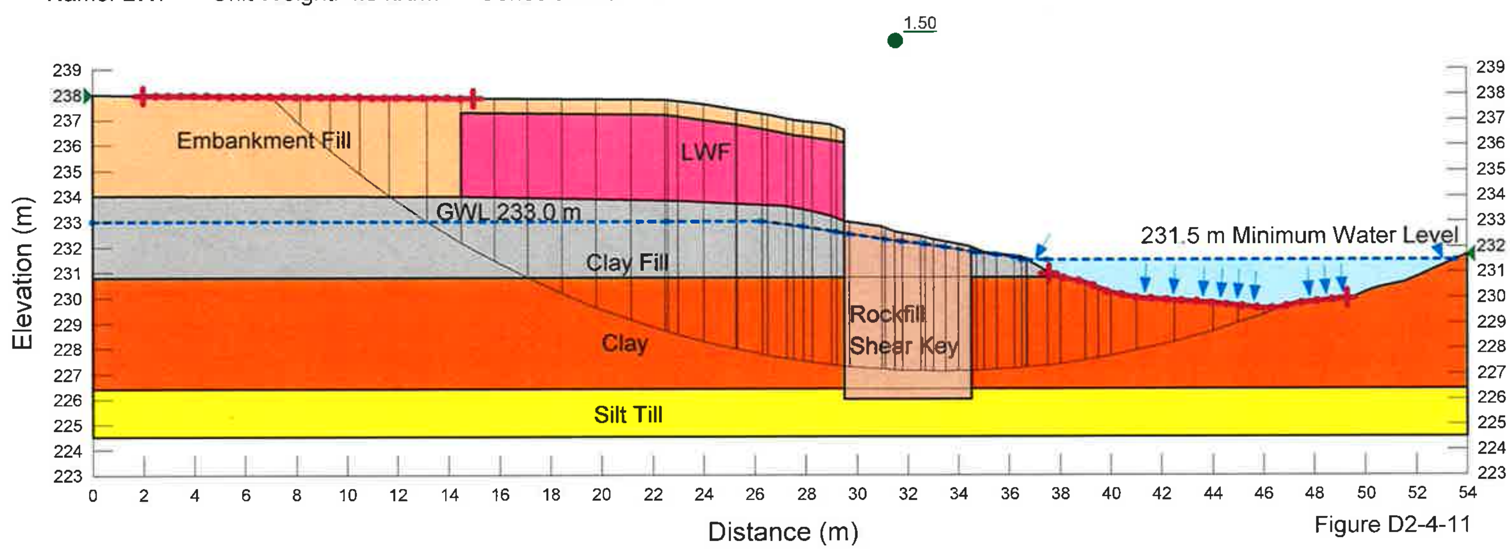


Figure D2-4-11

Name: Clay Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: Clay   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 16 °
Name: Silt Till   Unit Weight: 18 kN/m³   Cohesion: 0 kPa   Phi: 30 °
Name: Embankment Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: LWF   Unit Weight: 4.5 kN/m³   Cohesion: 20 kPa

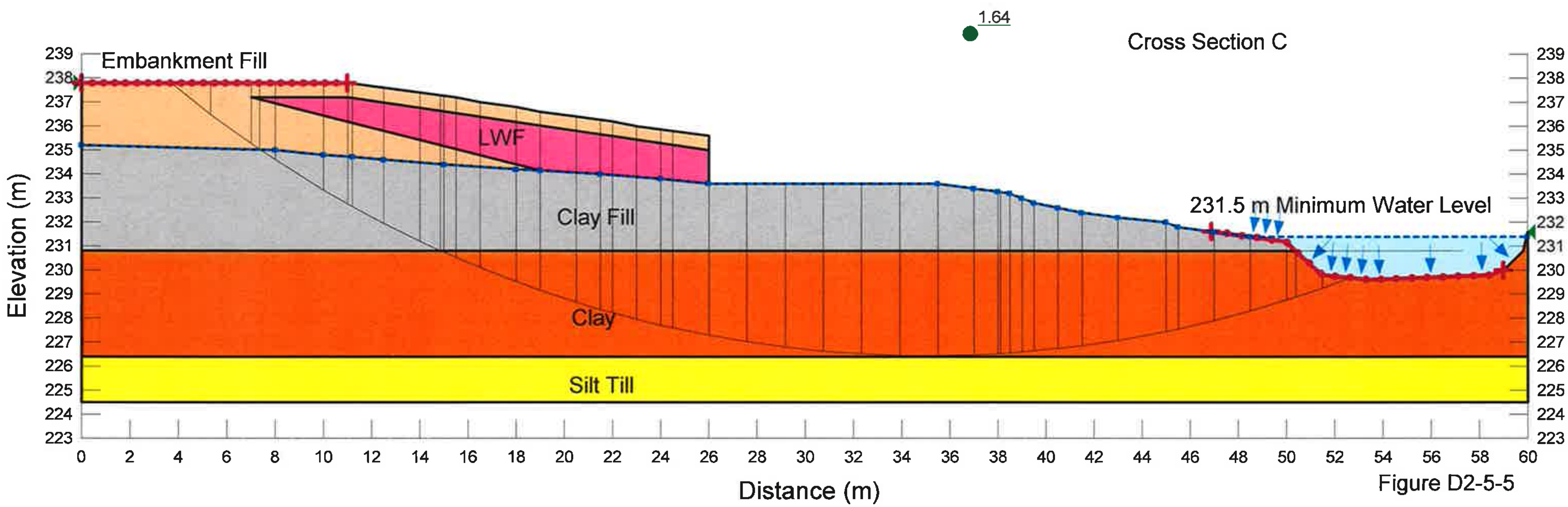


Figure D2-5-5

Name: Clay Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: Clay   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 16 °
Name: Silt Till   Unit Weight: 18 kN/m³   Cohesion: 0 kPa   Phi: 30 °
Name: Embankment Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °
Name: LWF   Unit Weight: 4.5 kN/m³   Cohesion: 20 kPa

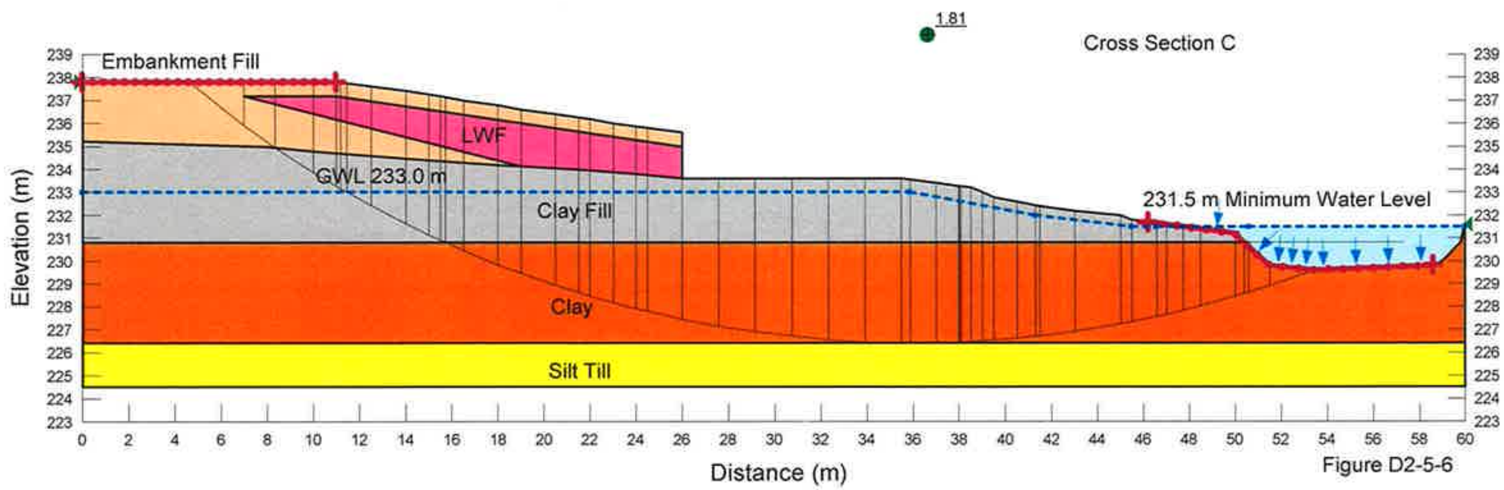


Figure D2-5-6

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

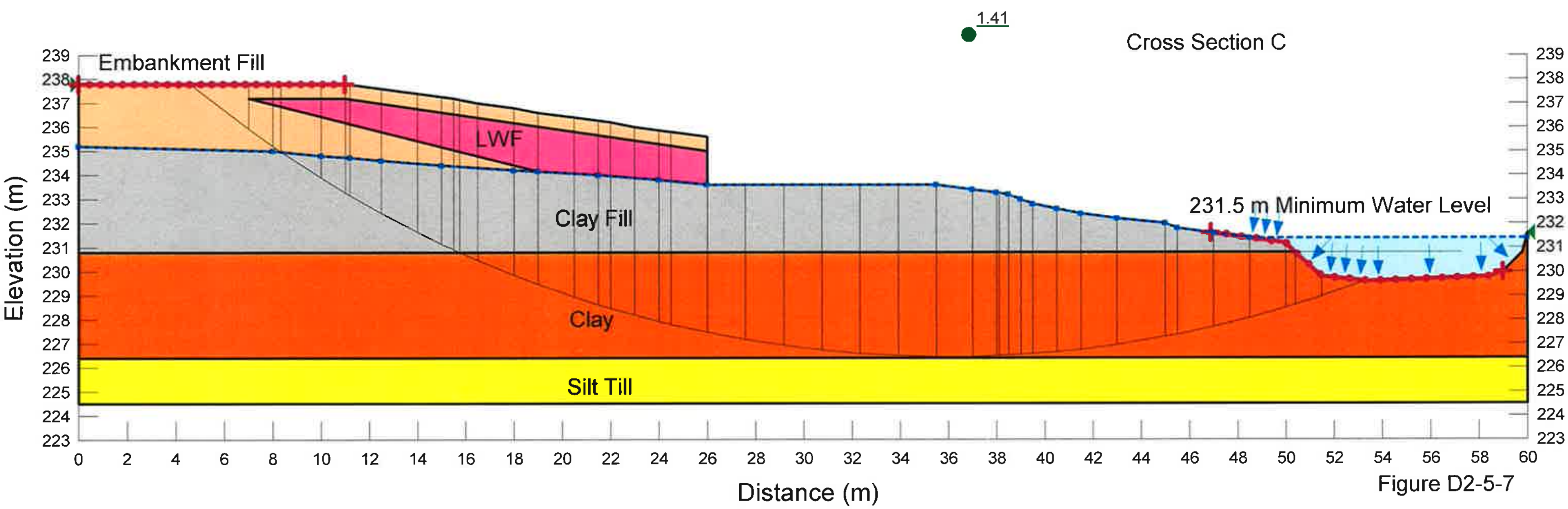


Figure D2-5-7

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 °

Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °

Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °

Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

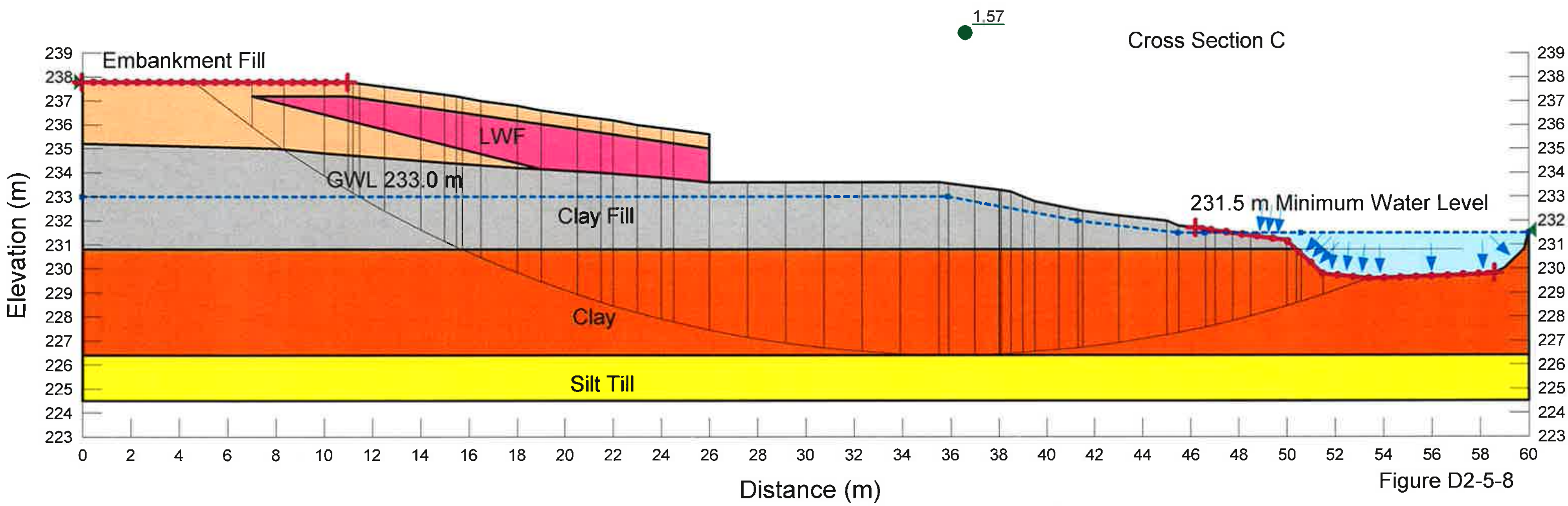


Figure D2-5-8

Appendix C – North-west Abutment Slope Movements

Photo 1: North-West Existing Bridge Abutment - Looking North

Photo 2: North-West Abutment Area - Looking South

Appendix D – Slope Stability Review

Appendix D1 – Cross Section D

Name: Clay Fill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Clay Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Silt Till Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °

Cross Section D

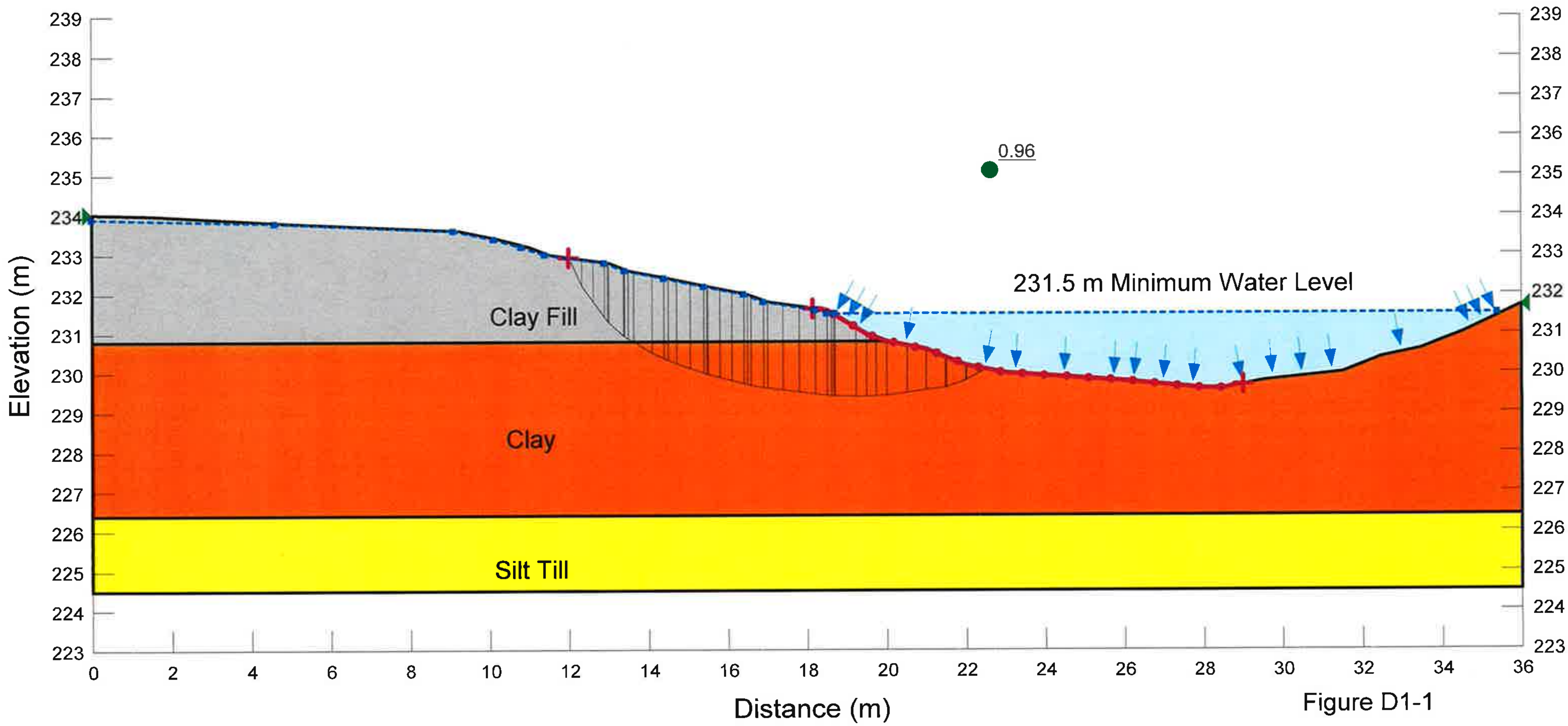
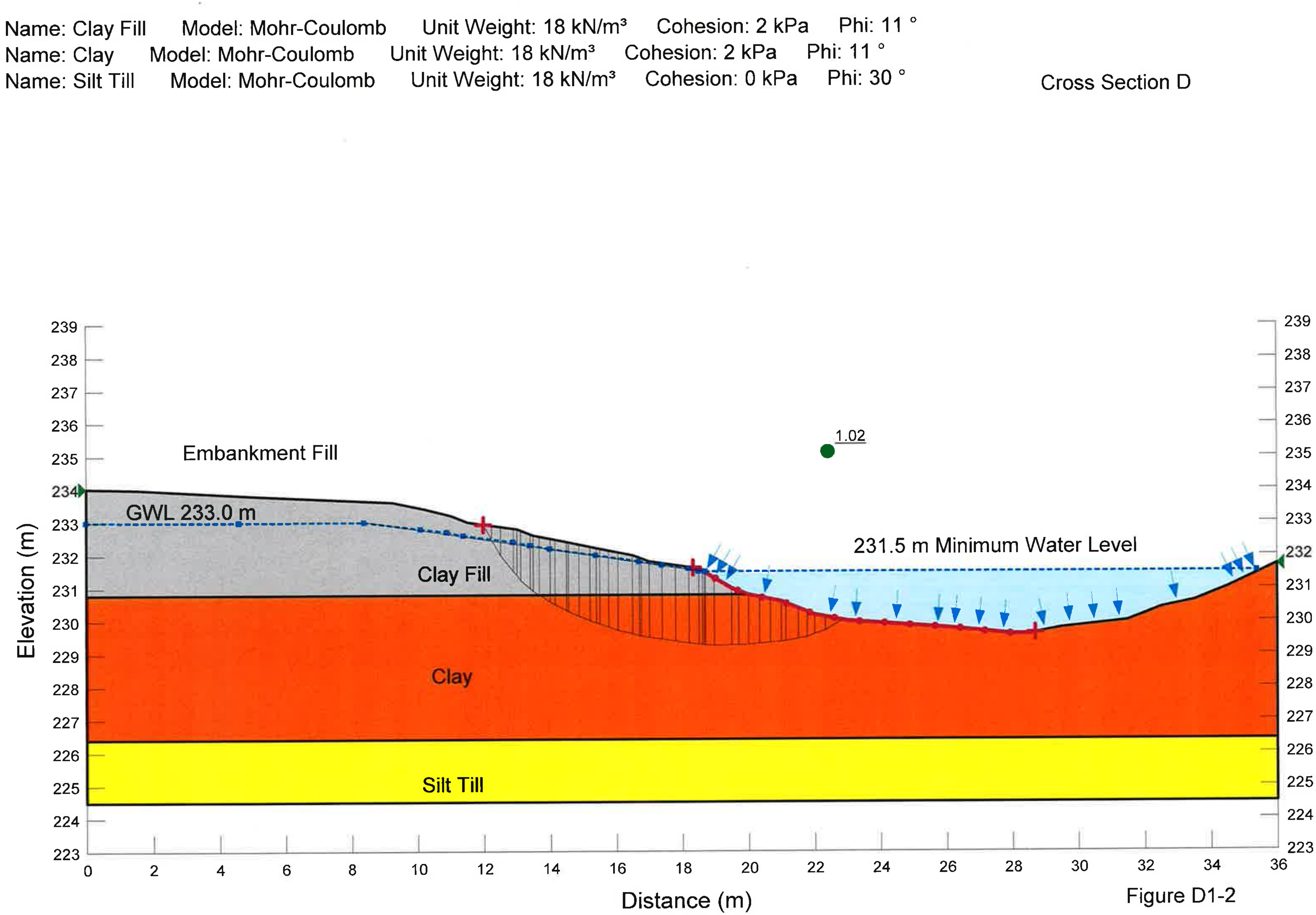


Figure D1-1


Name: Clay Fill Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Clay Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Silt Till Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Cross Section D
1.02
Embankment Fill
GWL 233.0 m
231.5 m Minimum Water Level
Clay Fill
Clay
Silt Till
Elevation (m)
Distance (m)
239
238
237
236
235
234
233
232
231
230
229
228
227
226
225
224
223
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
Figure D1-2

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Name: Residuall Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Residual Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °

Cross Section D

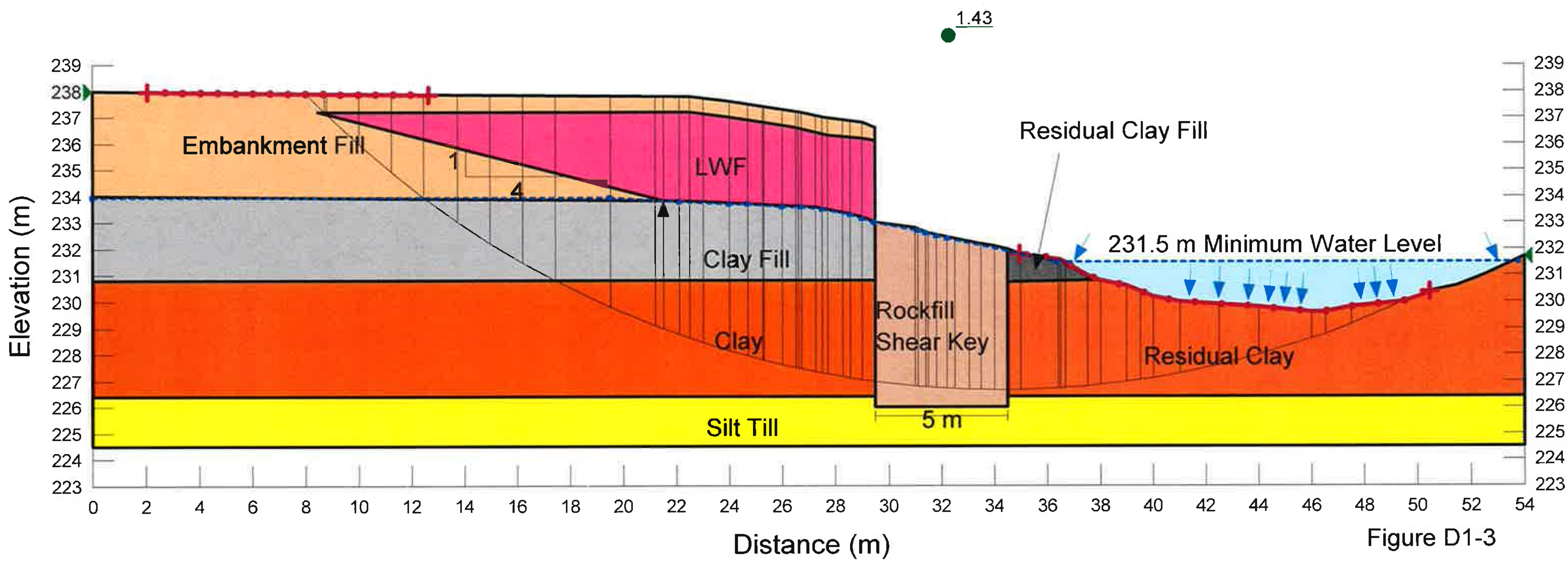


Figure D1-3

Cross Section D

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Name: Residual Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Residual Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °

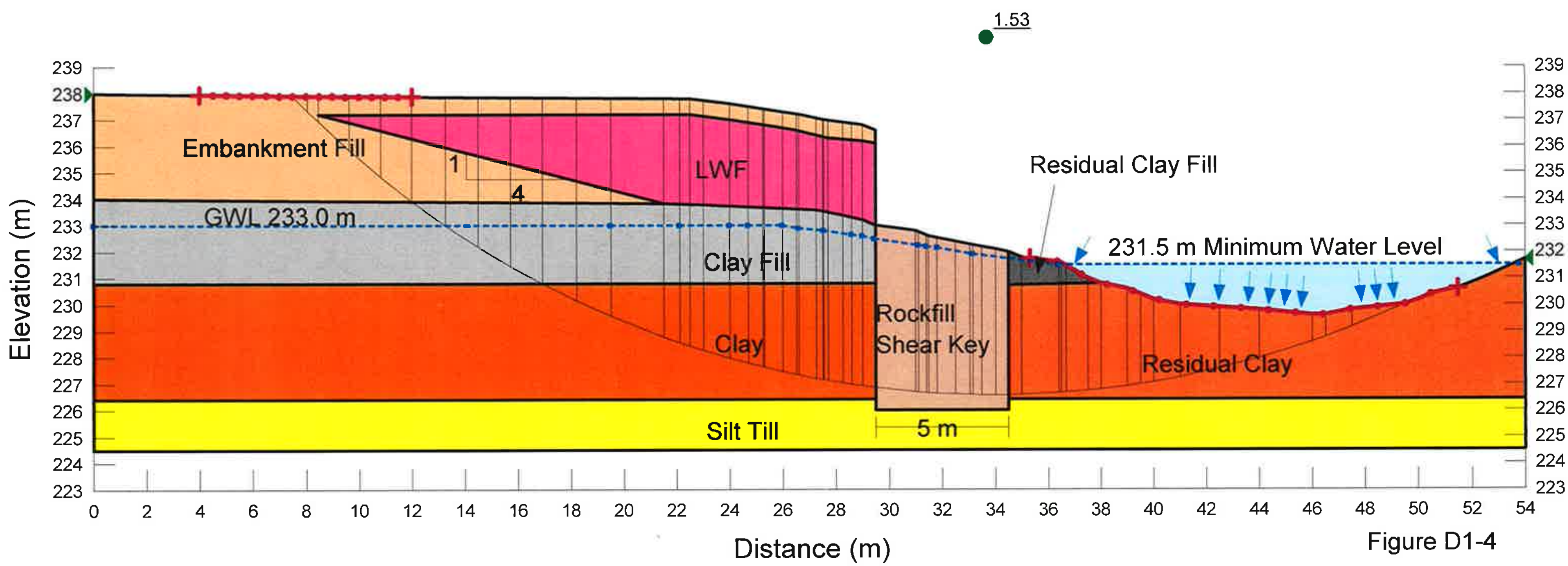


Figure D1-4

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Name: Residuall Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Residual Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °

Cross Section D

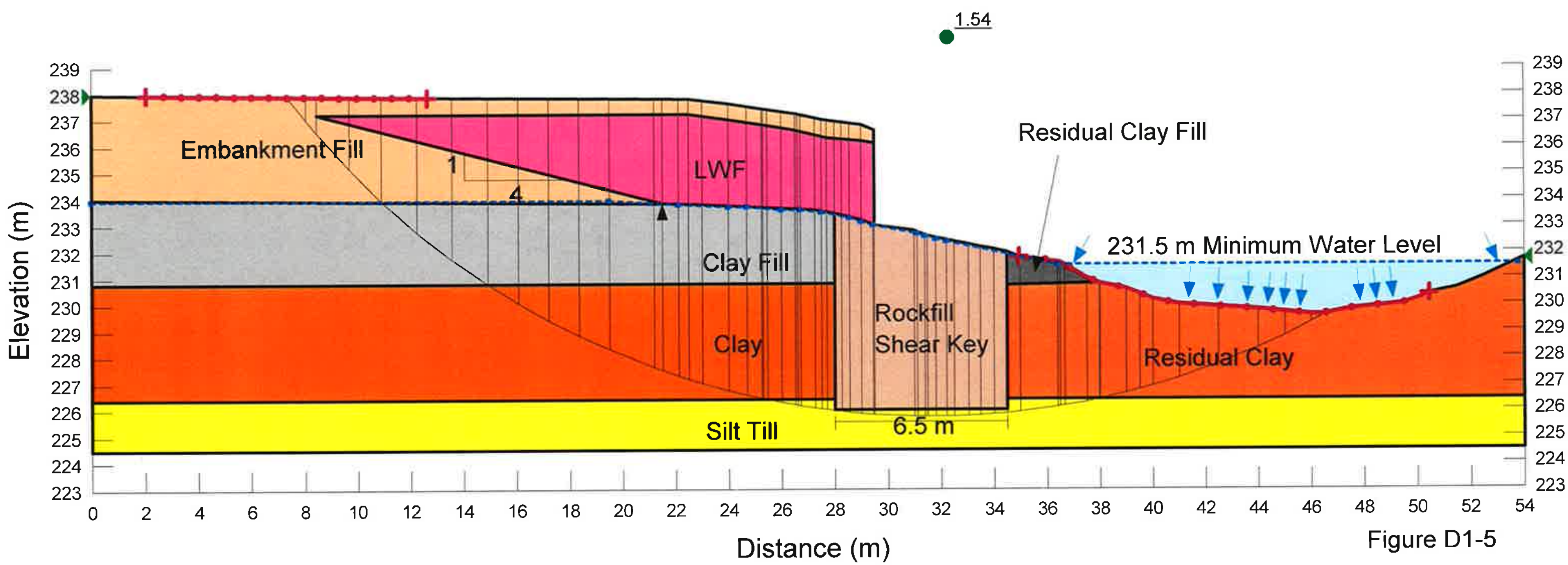


Figure D1-5

Cross Section D

Name	Unit Weight	Cohesion	Phi
Name: Clay	Unit Weight: 18 kN/m³	Cohesion: 5 kPa	Phi: 16 °
Name: Silt Till	Unit Weight: 18 kN/m³	Cohesion: 0 kPa	Phi: 30 °
Name: Embankment Fill	Unit Weight: 18 kN/m³	Cohesion: 5 kPa	Phi: 18 °
Name: Rockfill	Unit Weight: 20 kN/m³	Cohesion: 0 kPa	Phi: 40 °
Name: LWF	Unit Weight: 4.5 kN/m³	Cohesion: 20 kPa	
Name: Residual Clay	Unit Weight: 18 kN/m³	Cohesion: 2 kPa	Phi: 11 °
Name: Residual Clay Fill	Unit Weight: 18 kN/m³	Cohesion: 2 kPa	Phi: 11 °

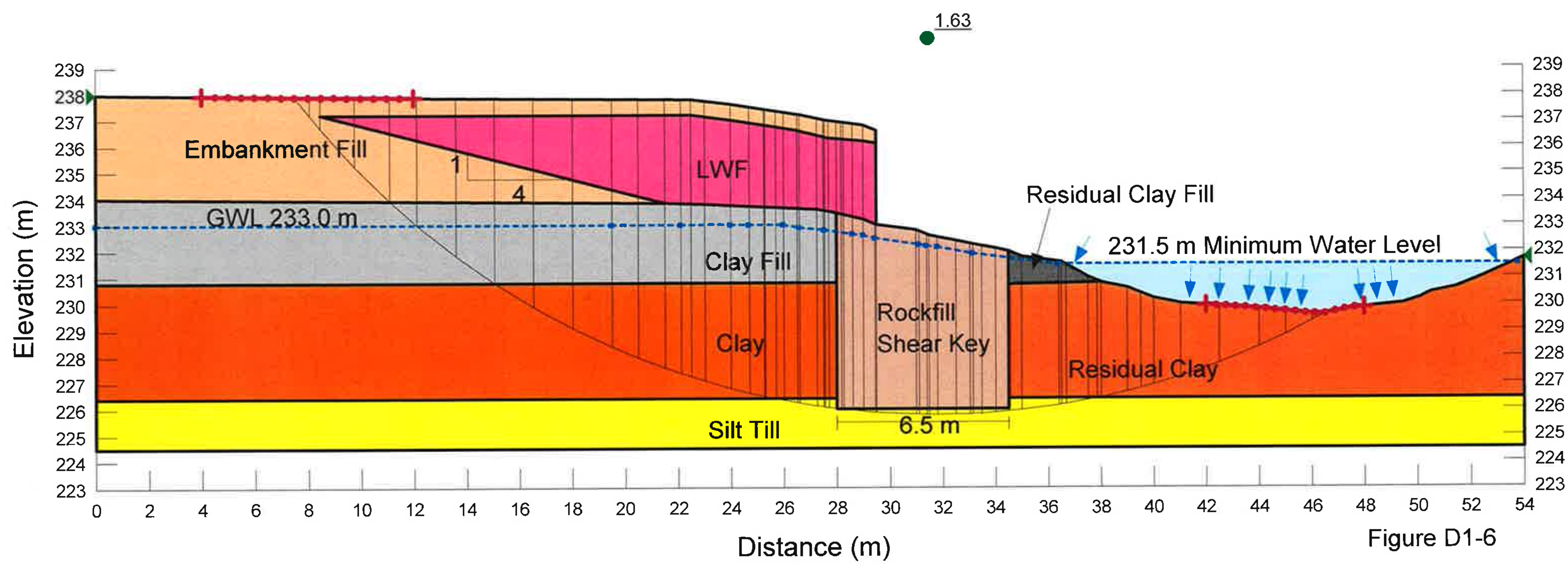


Figure D1-6

Appendix D2 – Cross Section C

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °

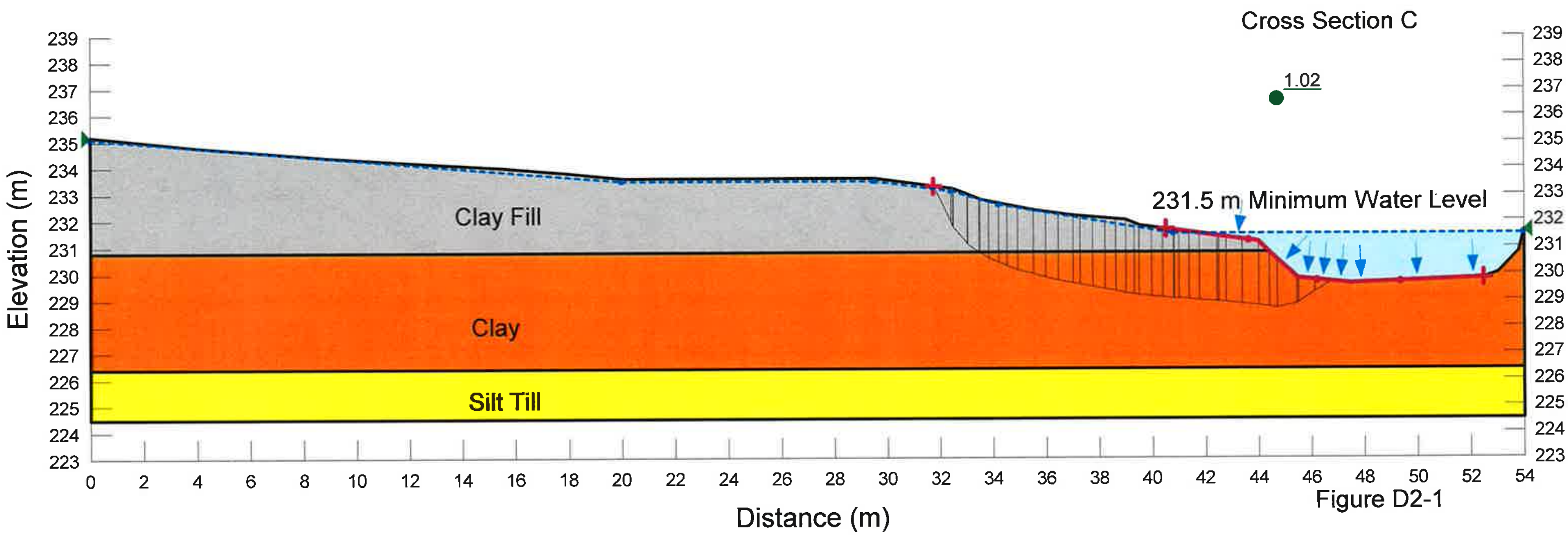


Figure D2-1

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °

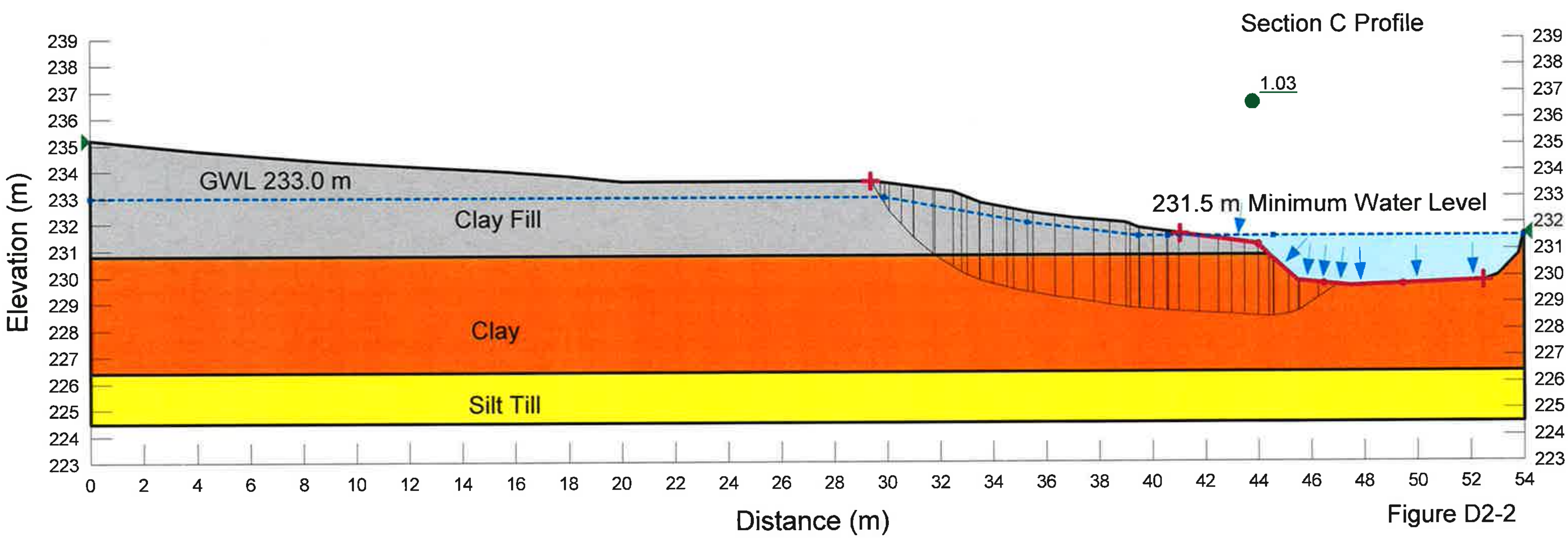


Figure D2-2

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Name: Residual Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Residual Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °

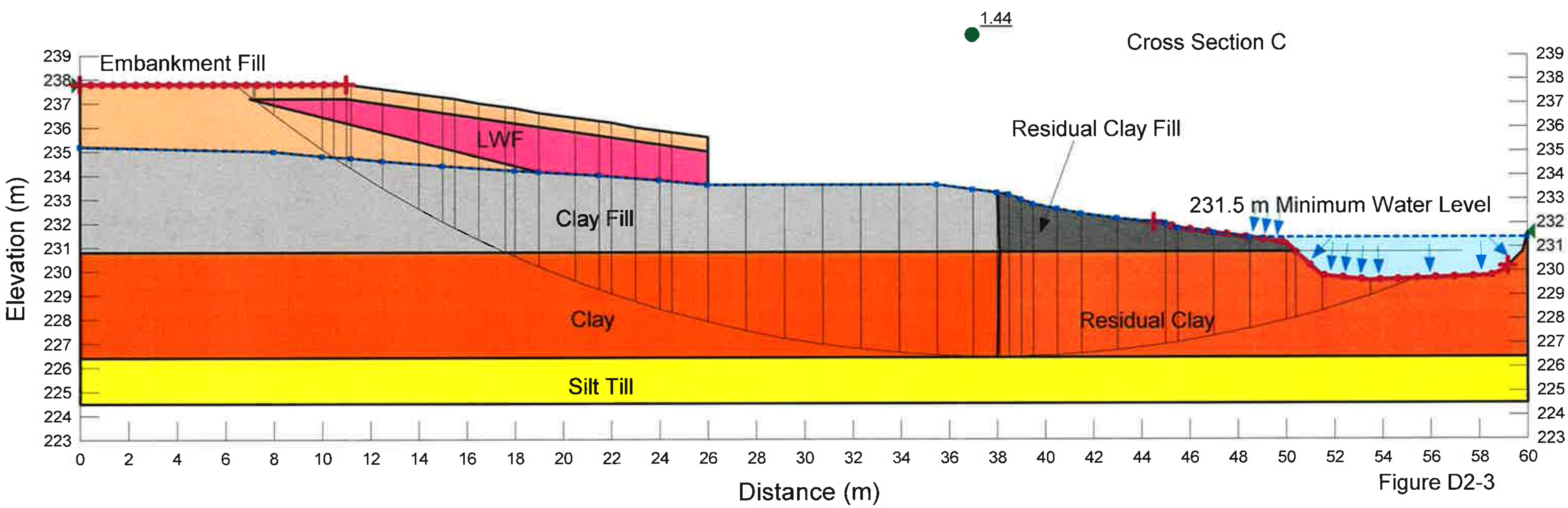


Figure D2-3

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Name: Residual Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Residual Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °

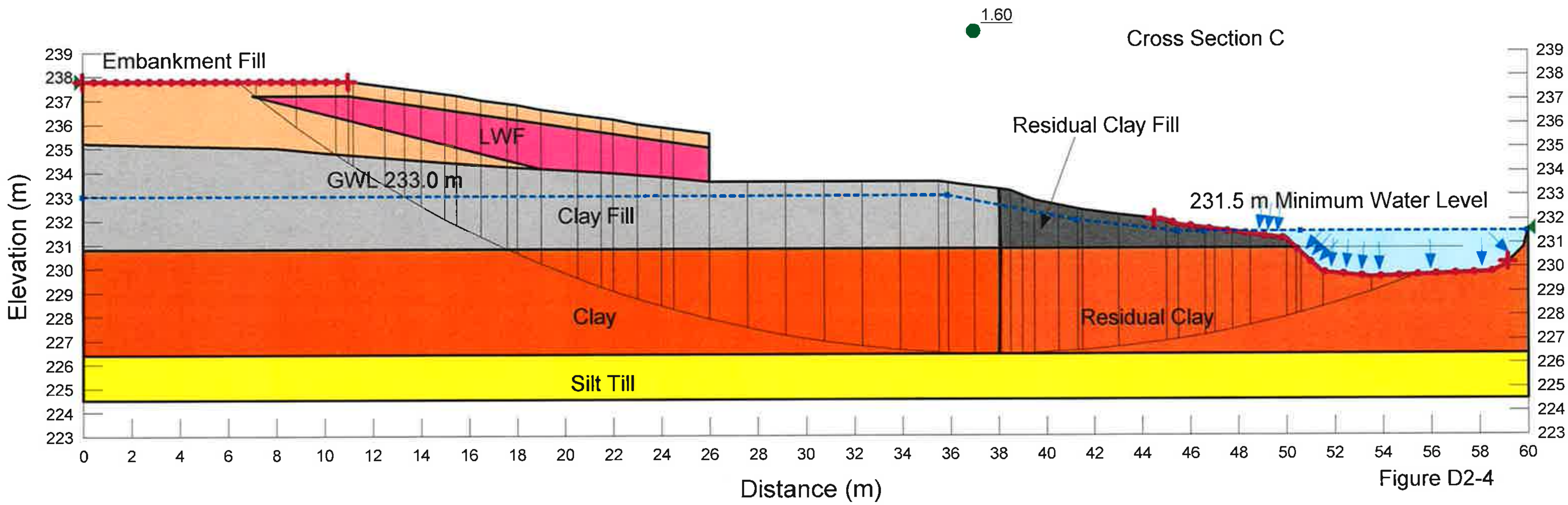


Figure D2-4

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Crushed Rock Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Name: Residual Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Residual Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °

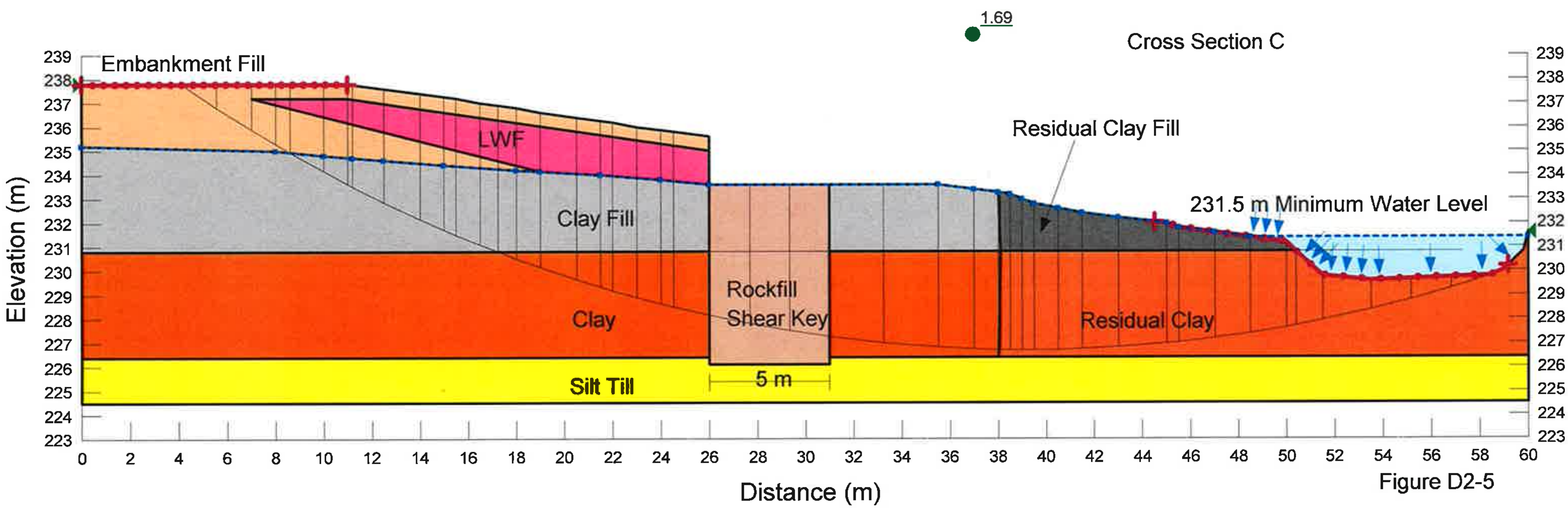


Figure D2-5

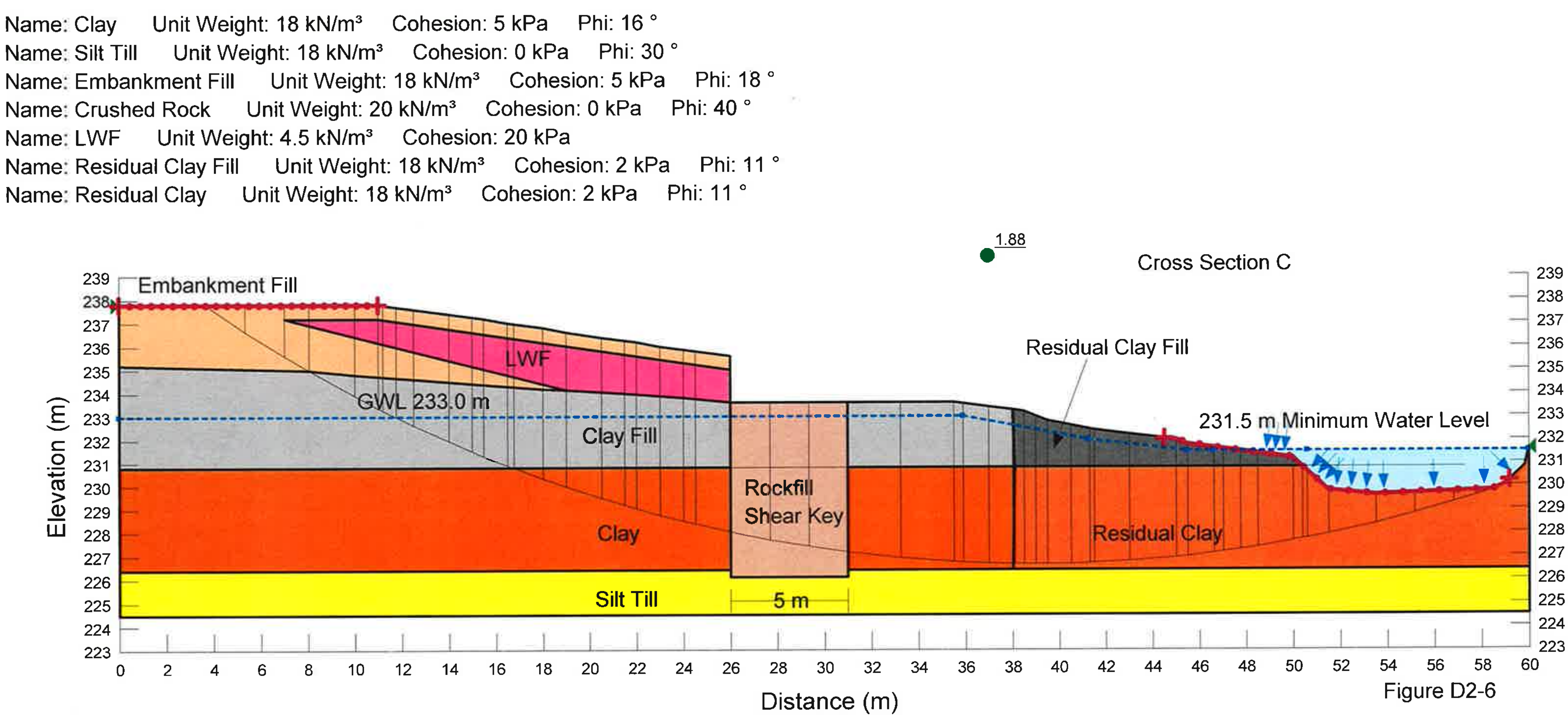

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 °
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 °
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Crushed Rock Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa
Name: Residual Clay Fill Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
Name: Residual Clay Unit Weight: 18 kN/m³ Cohesion: 2 kPa Phi: 11 °
1.88
Cross Section C
Embankment Fill
LWF
GWL 233.0 m
Clay Fill
Residual Clay Fill
231.5 m Minimum Water Level
Rockfill
Shear Key
Clay
Residual Clay
Silt Till
5 m
Elevation (m)
Distance (m)

Figure D2-6

Appendix E – Porewater Pressure Increase Review

Appendix E1 – Section E

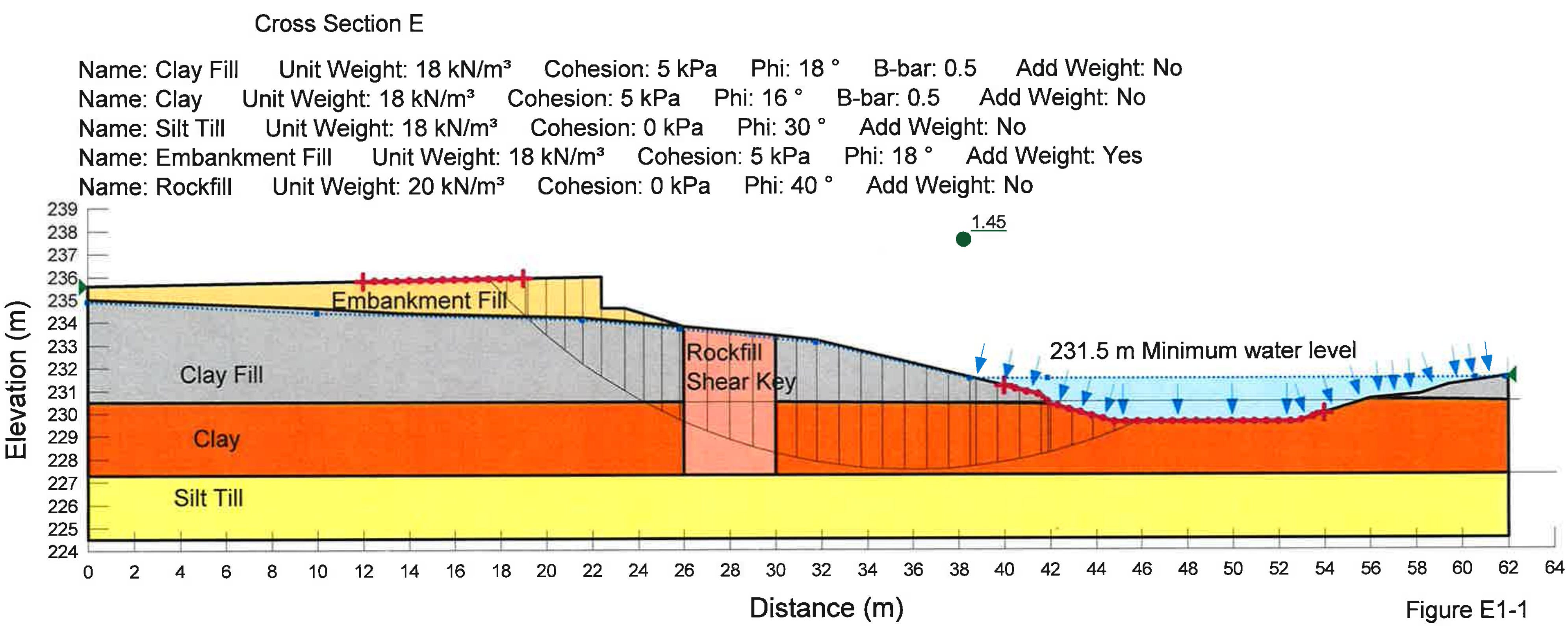

Cross Section E
Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 0.5 Add Weight: No
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 ° B-bar: 0.5 Add Weight: No
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 ° Add Weight: No
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° Add Weight: Yes
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 ° Add Weight: No
1.45
Embankment Fill
Clay Fill
Rockfill Shear Key
231.5 m Minimum water level
Clay
Silt Till
Elevation (m)
Distance (m)

Figure E1-1

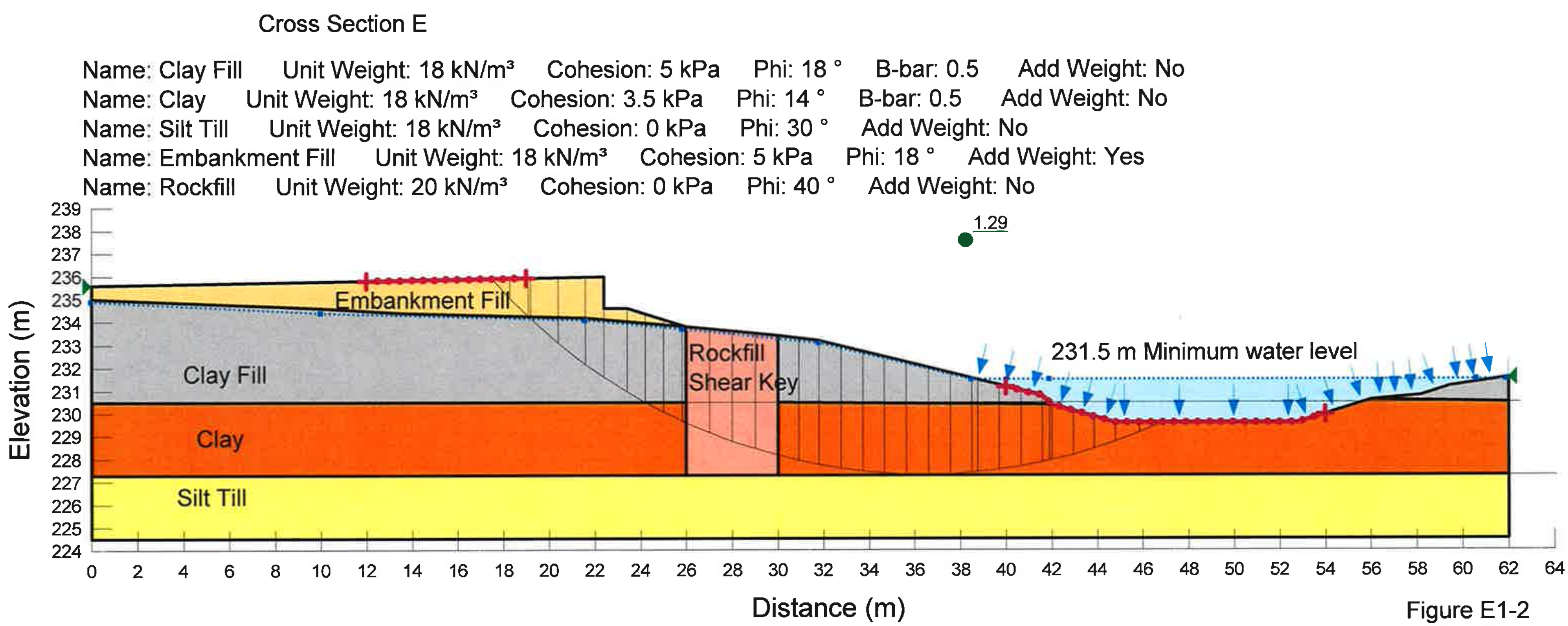
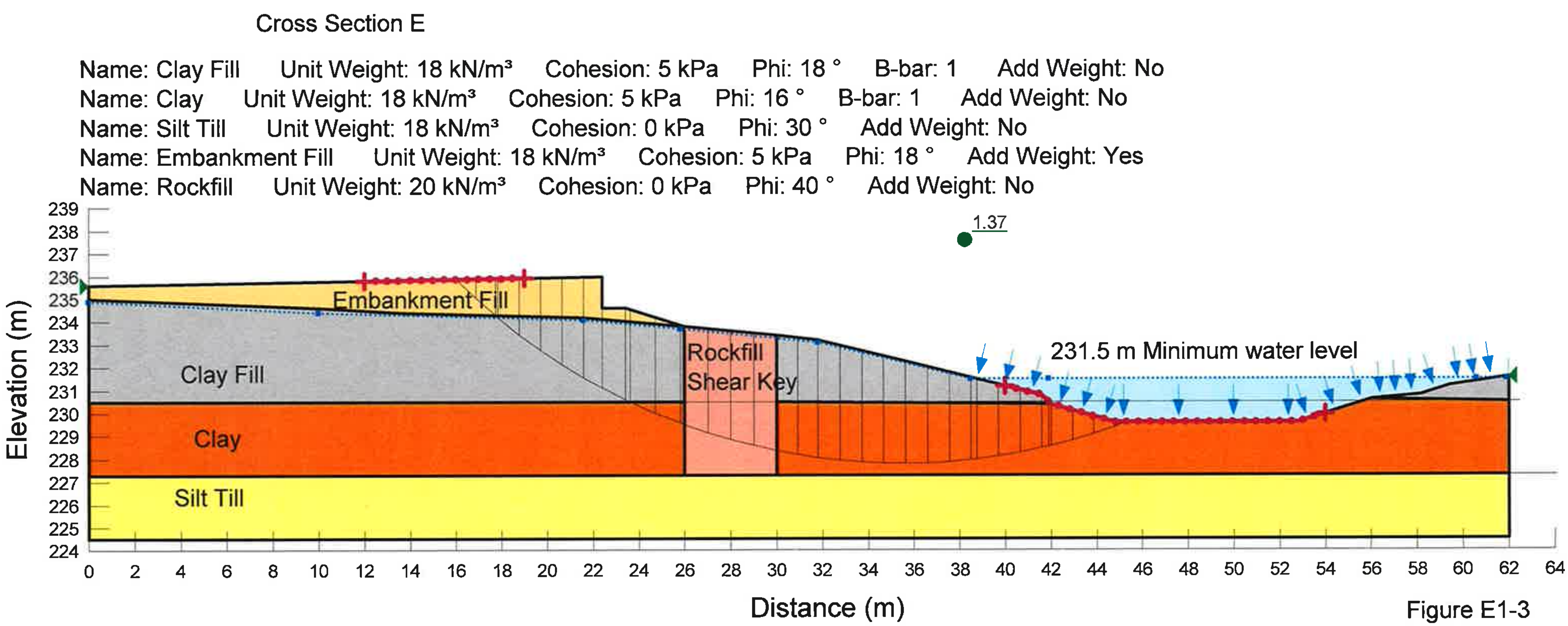

Cross Section E
Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 0.5 Add Weight: No
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 ° B-bar: 0.5 Add Weight: No
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 ° Add Weight: No
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° Add Weight: Yes
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 ° Add Weight: No
1.29
Embankment Fill
Clay Fill
Rockfill Shear Key
Clay
Silt Till
231.5 m Minimum water level
Elevation (m)
239
238
237
236
235
234
233
232
231
230
229
228
227
226
225
224
Distance (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64

Figure E1-2



Cross Section E
Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 1 Add Weight: No
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 ° B-bar: 1 Add Weight: No
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 ° Add Weight: No
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° Add Weight: Yes
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 ° Add Weight: No
1.37
Embankment Fill
Clay Fill
Rockfill
Shear Key
231.5 m Minimum water level
Clay
Silt Till
Elevation (m)
239
238
237
236
235
234
233
232
231
230
229
228
227
226
225
224
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56 58 60 62 64
Distance (m)

Figure E1-3

Cross Section E

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 1 Add Weight: No
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 ° B-bar: 1 Add Weight: No
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 ° Add Weight: No
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° Add Weight: Yes
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 ° Add Weight: No

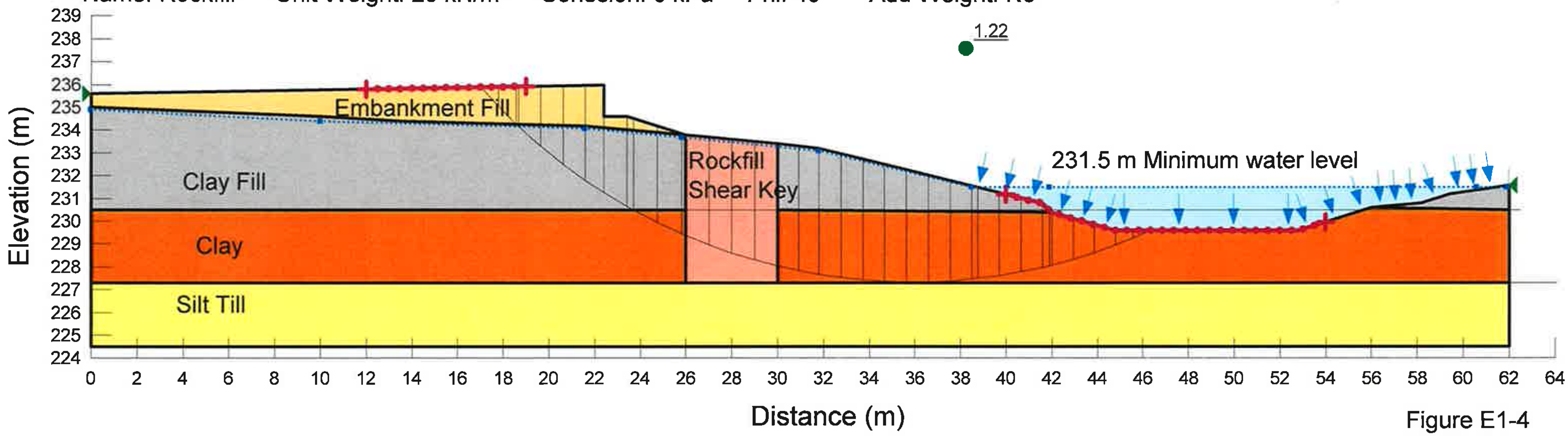
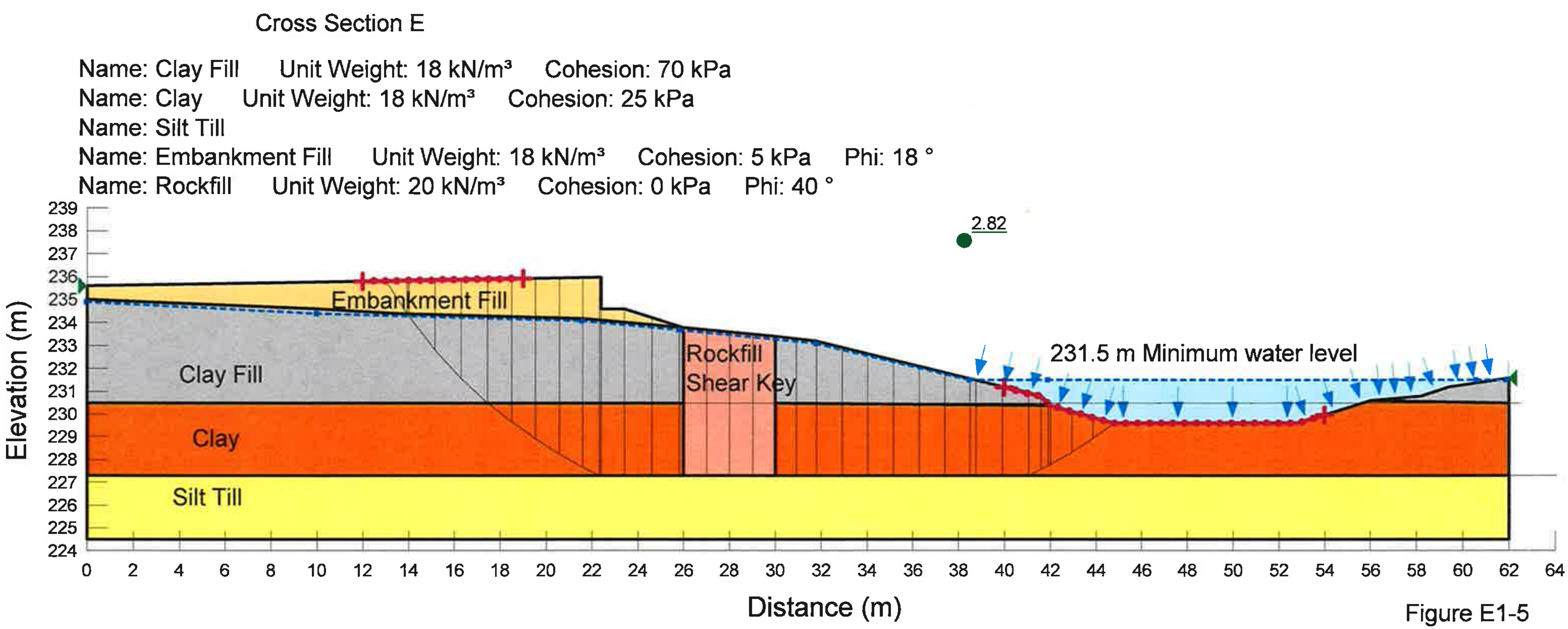


Figure E1-4



Cross Section E
Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 70 kPa
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 25 kPa
Name: Silt Till
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rockfill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
2.82
Embankment Fill
Clay Fill
Rockfill
Shear Key
231.5 m Minimum water level
Clay
Silt Till
Elevation (m)
Distance (m)

Figure E1-5

Appendix E2 – Section B

Cross Section B

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 0.5 Add Weight: No
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 16 ° B-bar: 0.5 Add Weight: No
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 ° B-bar: 0 Add Weight: No
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 0 Add Weight: Yes
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 ° Add Weight: No
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa B-bar: 0 Add Weight: Yes

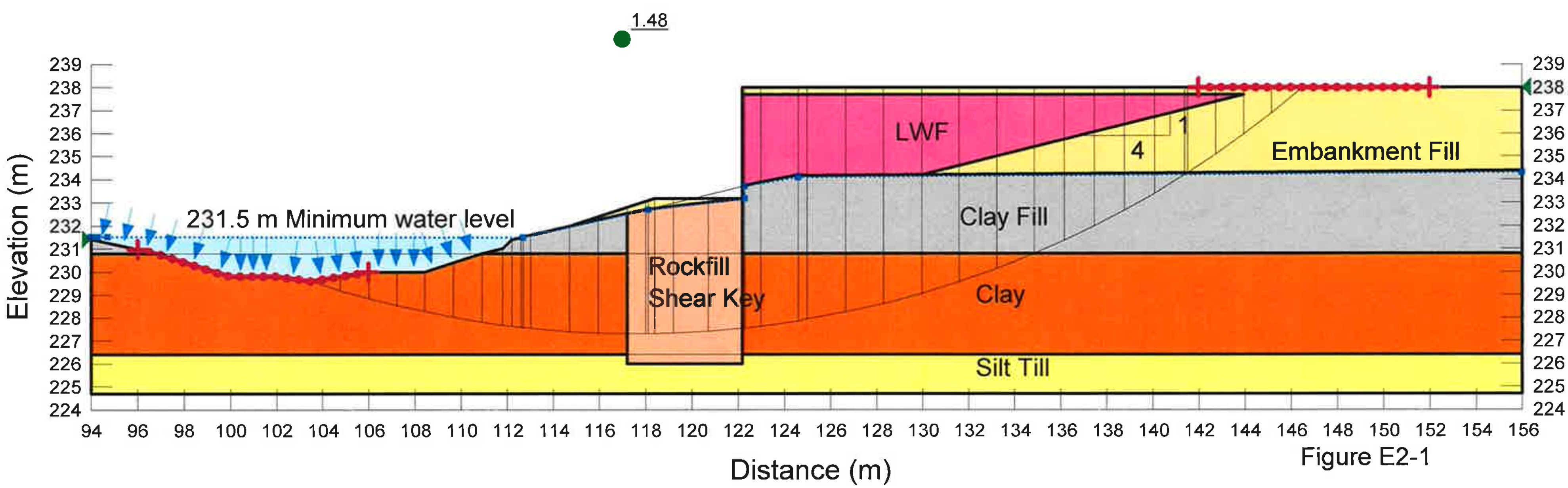


Figure E2-1

Cross Section B

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 0.5 Add Weight: No
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 ° B-bar: 0.5 Add Weight: No
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 ° B-bar: 0 Add Weight: No
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 0 Add Weight: Yes
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 ° Add Weight: No
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa B-bar: 0 Add Weight: Yes

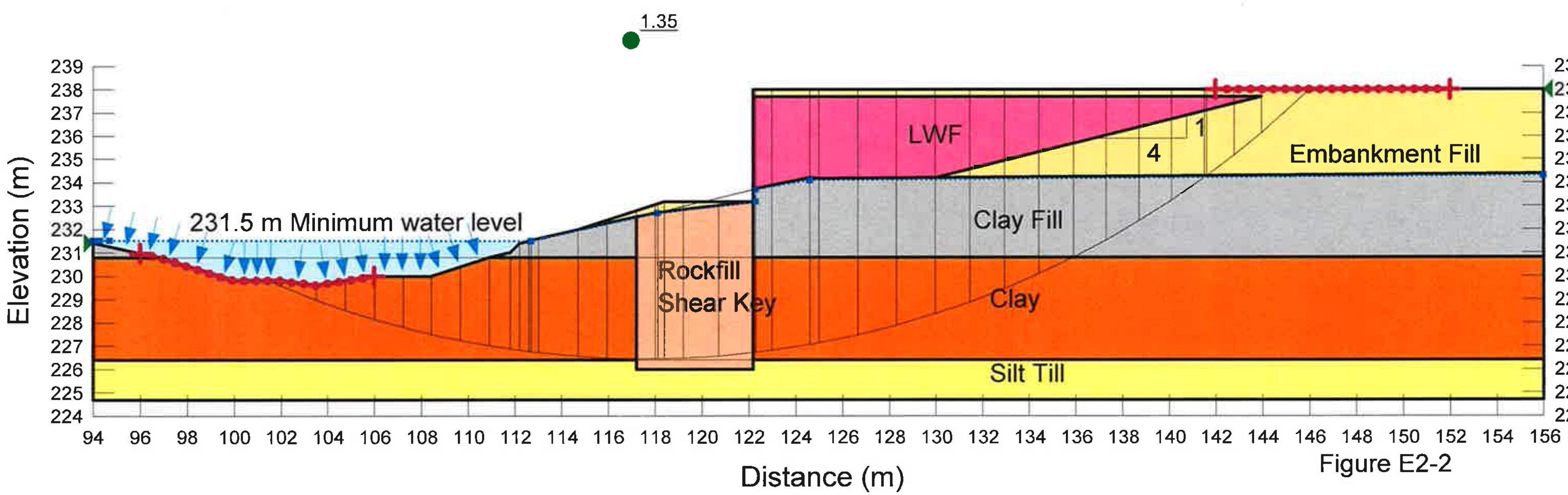


Figure E2-2

Cross Section B

Name: Clay Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °   B-bar: 1   Add Weight: No
Name: Clay   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 16 °   B-bar: 1   Add Weight: No
Name: Silt Till   Unit Weight: 18 kN/m³   Cohesion: 0 kPa   Phi: 30 °   B-bar: 0   Add Weight: No
Name: Embankment Fill   Unit Weight: 18 kN/m³   Cohesion: 5 kPa   Phi: 18 °   B-bar: 0   Add Weight: Yes
Name: Rock Fill   Unit Weight: 20 kN/m³   Cohesion: 0 kPa   Phi: 40 °   Add Weight: No
Name: LWF   Unit Weight: 4.5 kN/m³   Cohesion: 20 kPa   B-bar: 0   Add Weight: Yes

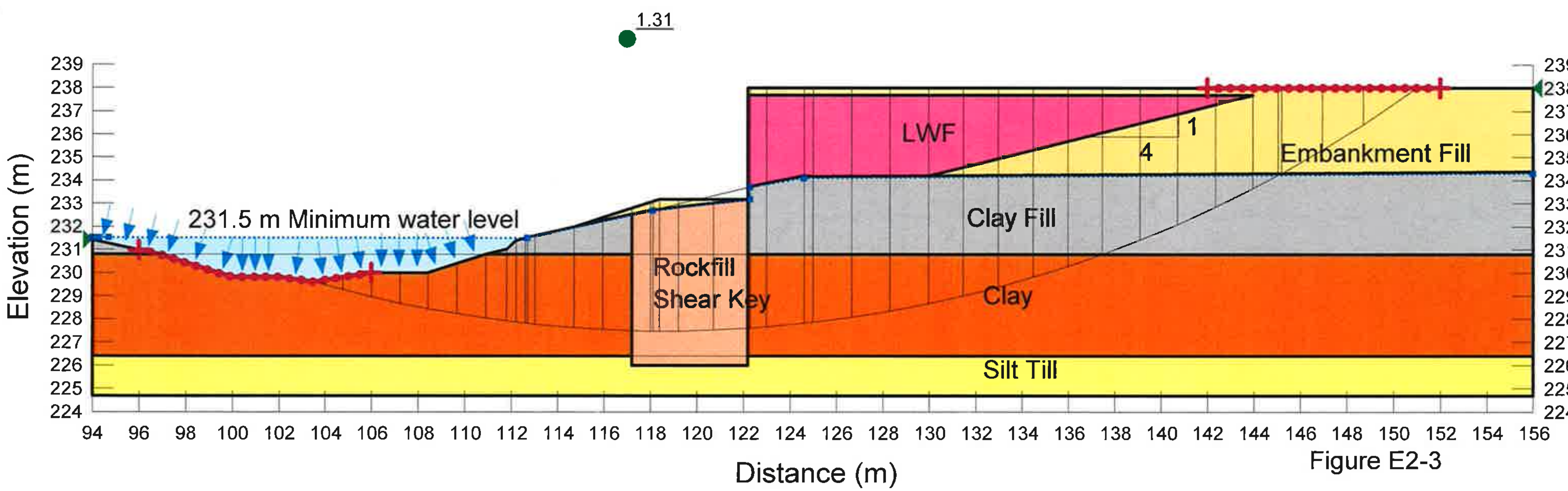


Figure E2-3

Cross Section B

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 1 Add Weight: No
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 3.5 kPa Phi: 14 ° B-bar: 1 Add Weight: No
Name: Silt Till Unit Weight: 18 kN/m³ Cohesion: 0 kPa Phi: 30 ° B-bar: 0 Add Weight: No
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 ° B-bar: 0 Add Weight: Yes
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 ° Add Weight: No
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa B-bar: 0 Add Weight: Yes

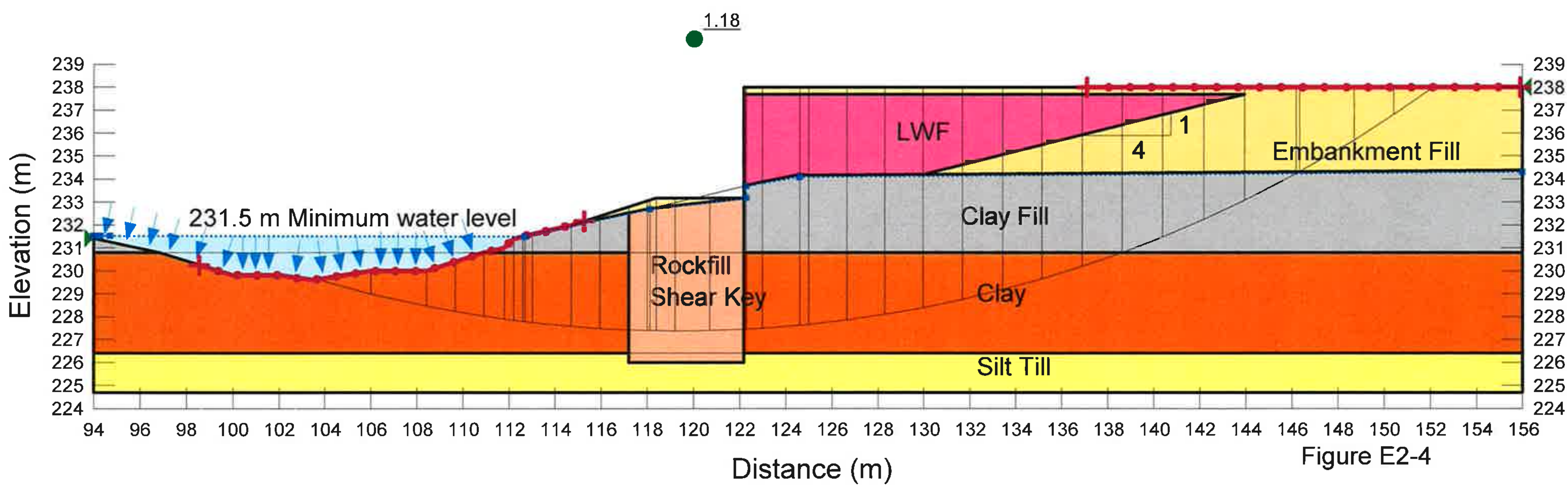


Figure E2-4

Cross Section B

Name: Clay Fill Unit Weight: 18 kN/m³ Cohesion: 70 kPa
Name: Clay Unit Weight: 18 kN/m³ Cohesion: 25 kPa
Name: Silt Till
Name: Embankment Fill Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 18 °
Name: Rock Fill Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: LWF Unit Weight: 4.5 kN/m³ Cohesion: 20 kPa

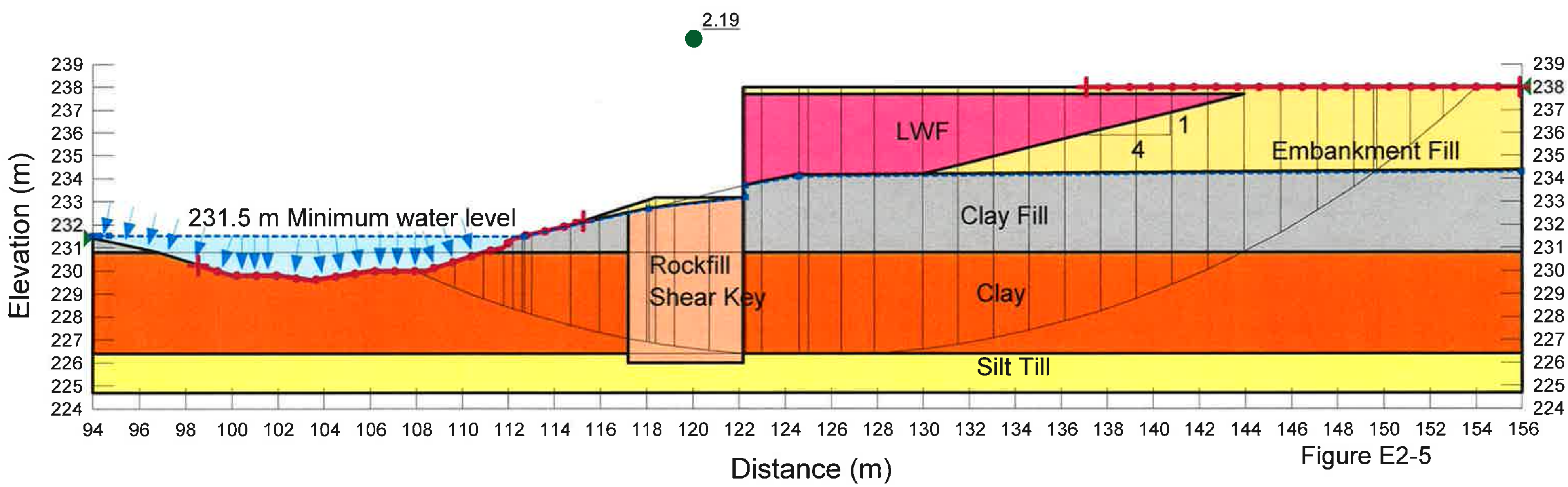


Figure E2-5