

**FOUNDATION DESIGN REPORT
EMBANKMENT GRADE RAISES AND WIDENING
PROPOSED RECONSTRUCTION OF HIGHWAY 400 NBL
FROM 1.2 KM SOUTH OF MUSKOKA ROAD INTERCHANGE
NORTHERLY 22.7 KM, HUNTSVILLE AREA
G.W.P. 152-98-00**

GEOCRES NO. 31D-407

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**Project: SPT1117
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5. DISCUSSION & RECOMMENDATIONS

5.1 SITE 1 (STA. 13+050 TO 13+400)

A total of fourteen boreholes and nine dynamic cone penetration tests was put down in this 350 m long section to depths ranging between 2 and 15 m below the ground surface. Three of the boreholes were advanced from the surface of the existing highway embankment while the rest were advanced from the adjacent ground near the toe of the embankment. Boreholes were also drilled by MTO for the then proposed southbound lanes, adjacent to the site and this information is presented in Appendix D.

The boreholes drilled for this investigation revealed the presence of an organic deposit consisting of peat and organic clayey silt to a depth of 0.5 to 2.1 m below the ground surface underlain by a major clay to silty clay deposit. The site appears to present a bowl-shaped depression and the thickness of the clay to silty clay ranges from zero near the rock outcrop areas, increasing to a maximum of 10 m towards the middle of the swampy area. The clay to silty clay deposit is underlain by granular deposits under excess hydrostatic pressure with groundwater level at the ground surface level.

The clay to silty clay deposit encountered at the site is generally very soft and highly compressible.

The presently proposed design profile is given in Appendix G1 along with a typical cross-section (at Station 13+183). From this, it appears that up to 200 mm of grade raise is contemplated between about Stations 13+130 and 13+200 and up to 100 mm between Stations 13+250 and 13+330.

5.1.1 EMBANKMENT STABILITY

In this section, the height of the embankments generally range from about 1.0 m at Station 13+370 to 2.1 m at about Station 13+100 m but is generally between 1.2 and 1.5 m above the adjacent (original) grades. While the proposed embankment loading (i.e. up to 200 mm of asphalt) is very unlikely to cause any instability, this aspect was checked for completeness, as well as to obtain an opinion on the previous conditions (when the

embankments were first built) to see if lateral yielding may have taken place (i.e. low safety factors).

The foundation embankment stability was analyzed by the limit equilibrium method, utilizing Bishop's Simplified method of analysis. For this purpose, the computer program Slope/W was utilized. Bishop's Simplified method is known to be slightly on the conservative side because of the fact that side forces on the slices are ignored, as opposed to more rigorous methods. Some analyses by Janbu's method were also conducted.

For the undrained (short-term) stability analyses, undrained shear strengths (c-values) of the clay were based on the field vane tests results at each borehole location and assuming a ϕ -value of zero. The c-values used in our analysis ranged from 4 to 44. No correction factor to the field vane test results was applied (e.g. Bjerrum, Aas, etc. correction).

Long-term (drained) analyses were also performed at selected borehole locations.

The soil parameters used in the slope stability analyses are presented in Table 5.1.1.1

Table 5.1.1.1
Soil Parameters Used in Slope Stability Analyses

Material	Short-Term Analyses			Long-Term Analyses		
	ϕ (degrees)	c (kPa)	γ (kN/m ³)	ϕ' (degrees)	c' (kPa)	γ (kN/m ³)
Embankment Fill (Sand)	31	0	21.0	31	0	21.0
Embankment Fill (Silty Sand)	29	0	20.0	29	0	20.0
Peat/Organic Clayey Silt	0	15	13.0	20	0	13.0
Clay	0	4-44	16.5	20	2	16.5
Lower Granular Soils	32-35	0	20.0-21.5	32-35	0	20.0-21.5

In two of the three boreholes drilled from the top of highway embankment, organic soils had been stripped from the base of the embankment while in the third borehole a 0.8 m thick layer was found to have been left in place. Since this latter borehole is located in the extreme north end of the area where the grade will be raised only marginally (i.e. at this particular borehole location the grade raise is expected to be only about 50 to 100 mm), in our analysis we have assumed that the organic soils have been removed from beneath the footprint of the entire embankment width but not beyond it.

Typical slope stability analysis results are given in Appendix H1.

These results indicate that the proposed 200 mm grade raise is not expected to cause slope instability. As well, large scale lateral yields of the weak clay are unlikely to have occurred.

5.1.2 SETTLEMENT OF EMBANKMENTS

We do not know exactly when the northbound lanes of the Highway were built but in the Pavement Performance Records (PPR) made available to us, the age of the pavement in 1988 is shown as 12 years old. From this, the embankment appears to have been constructed on or about 1976. Our analyses, based on consolidation test results, indicate that the primary settlement during the 28 years since 1976 would be very substantially completed throughout the site but a secondary settlement of 40 mm can be expected to occur in the next 20-year period. Our analyses also shows that the embankments would have settled between 100 mm where the thickness of the clay is minimal to about 400 mm to 450 mm at Station 13+150 where the clay is 9.4 m thick [see Record of Borehole 13+150 (6 m Rt)] and about 450 mm at Station 13+260 (26 m Rt) where the clay is also 9.4 m thick but appears to be slightly more compressible. In our analysis, we assumed a pre-consolidation pressure in excess of existing overburden (i.e. $P_c - P_o$) of the order of 15 kPa and embankment heights of about 0.3 to 0.4 m in excess of existing (i.e. embankments have settled since 1976 by at least this amount).

This assessment is further complicated by the fact that Contract No. 89-71 drawings indicate to us that in the general area, a 120 to 140 mm thick asphalt layer was probably placed to rehabilitate the highway. It is possible that other adjustments may have been implemented during the contract and also during the construction of southbound lanes of the highway, however, we have no evidence of these.

Assuming that an average of 130 mm thick asphalt layer was placed for the aforementioned re-surfacing (and no other modifications were made), this is likely to cause a settlement of about 30 mm in areas where the clay thickness is in excess of 9 m. Some of the settlement would still be on-going (which is in addition to secondary consolidation), while in areas where the clay is less than 6 m thick the settlement due to this re-surfacing would be completed.

Another complication in analysis is the fact that embankment loads may have caused some lateral yielding of the weak clay and some settlements may have taken place due to lateral plastic yielding.

Our analysis shows that based on available field and laboratory test data, the proposed 200 mm re-surfacing between Stations 13+130 and 13+200 is likely to cause a settlement of about 40 mm which should take place within the next ten years. In addition, a combination of secondary settlement and residual settlement (i.e. due to 1989 re-surfacing) can be expected to be of the order of 30 mm, bringing the total to 70 mm. While this

settlement is not expected to cause any structural problems, nevertheless, it is recommended that any grade raise be kept to a minimum and should not exceed this amount (i.e. 200 mm).

Between Stations 13+250 and 13+330, the anticipated settlement due to the proposed 100 mm grade raise (i.e. re-surfacing) would be about 25 mm in the next ten years. To this a combination of residual and secondary settlements (due to previous activities) of 30 mm should be added, bringing the total to about 55 mm. Again, while settlements of this magnitude are not expected to cause any structural (pavement) distress, it is recommended that the proposed grade raise should not exceed the presently proposed value (i.e. 100 mm).

Appendix G1

Proposed Profiles and Typical Cross-Sections

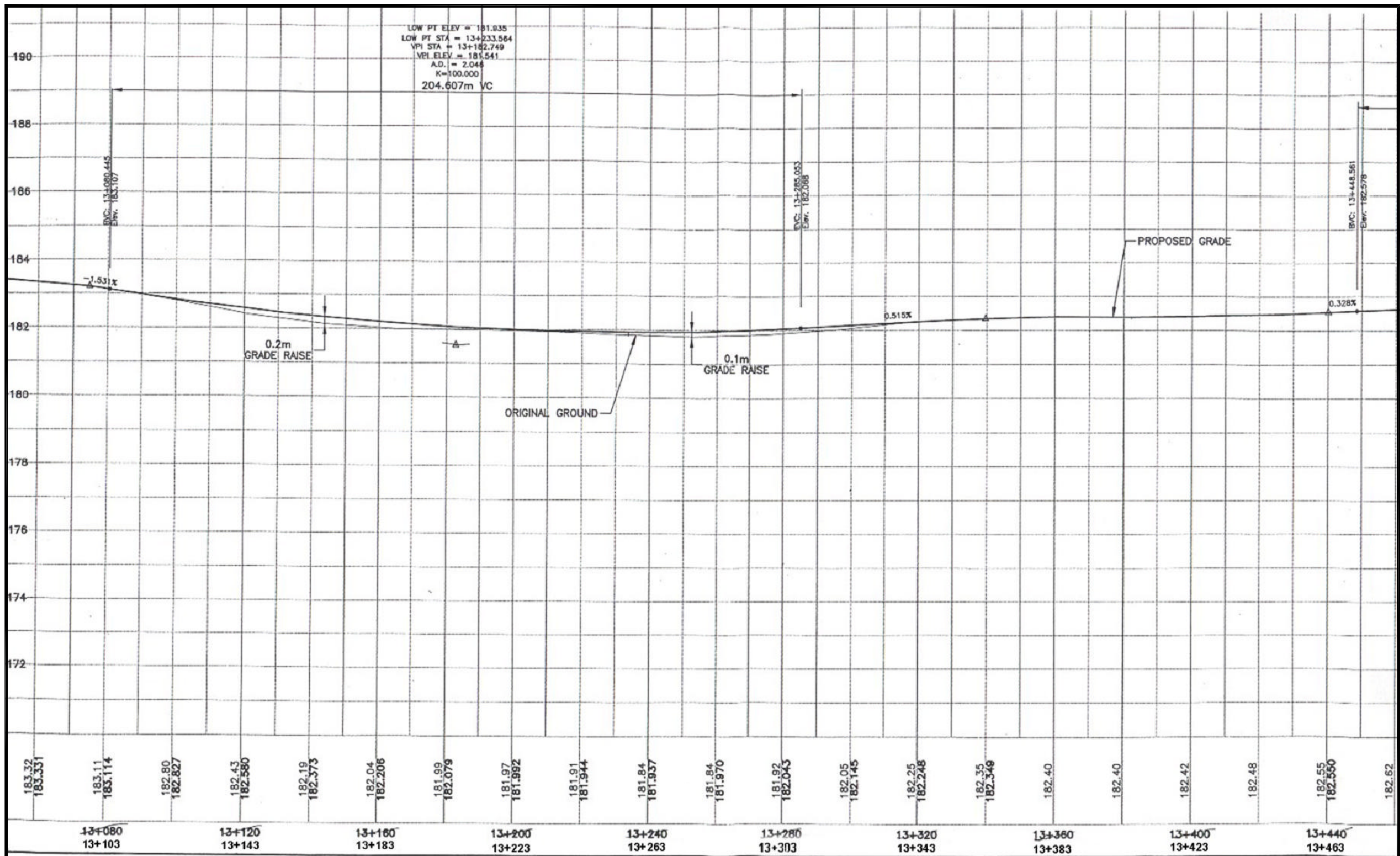
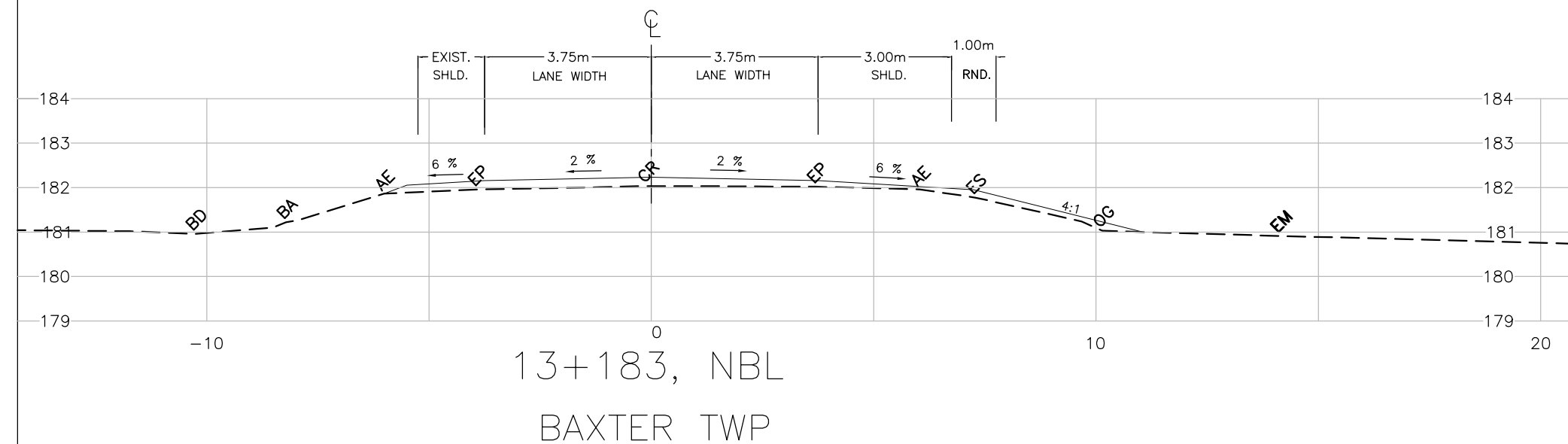


FIGURE G1-1

STATION BASED ON MEDIAN CL CHAINAGE

0.20 m GRADE RAISE



Appendix H1

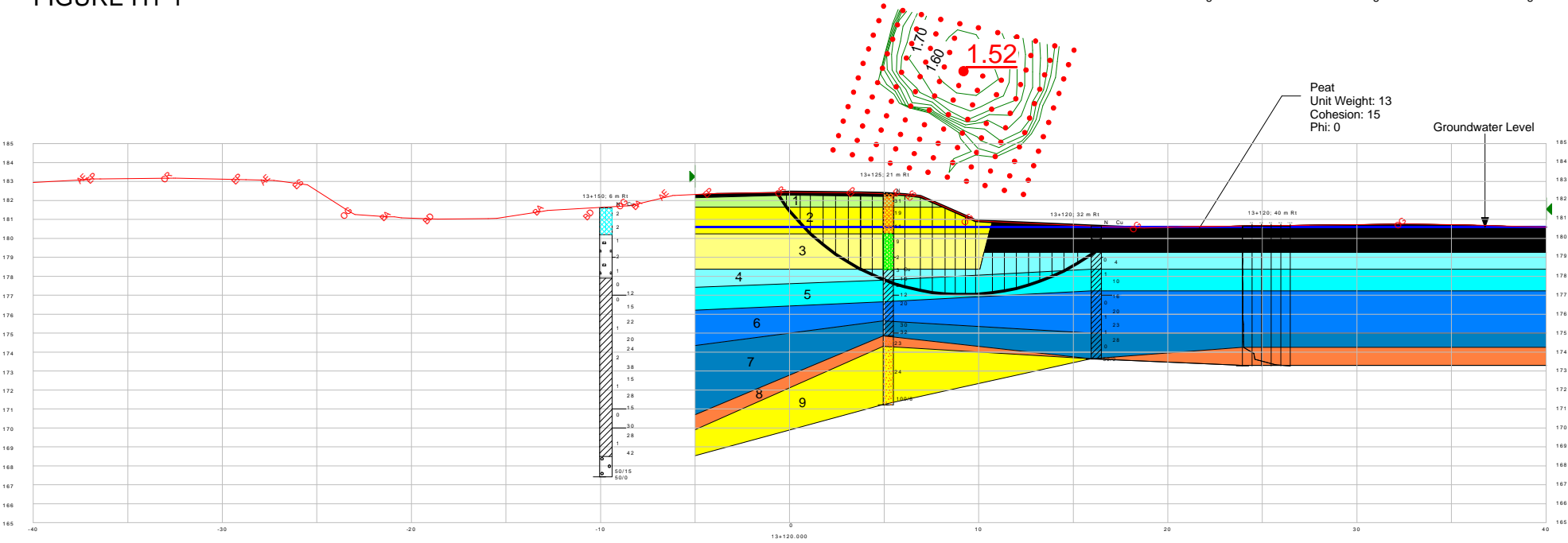
Slope Stability Analysis Results

Highway 400 NBL, Twp of Baxter
Station 13+143 (Existing Embankment Height)

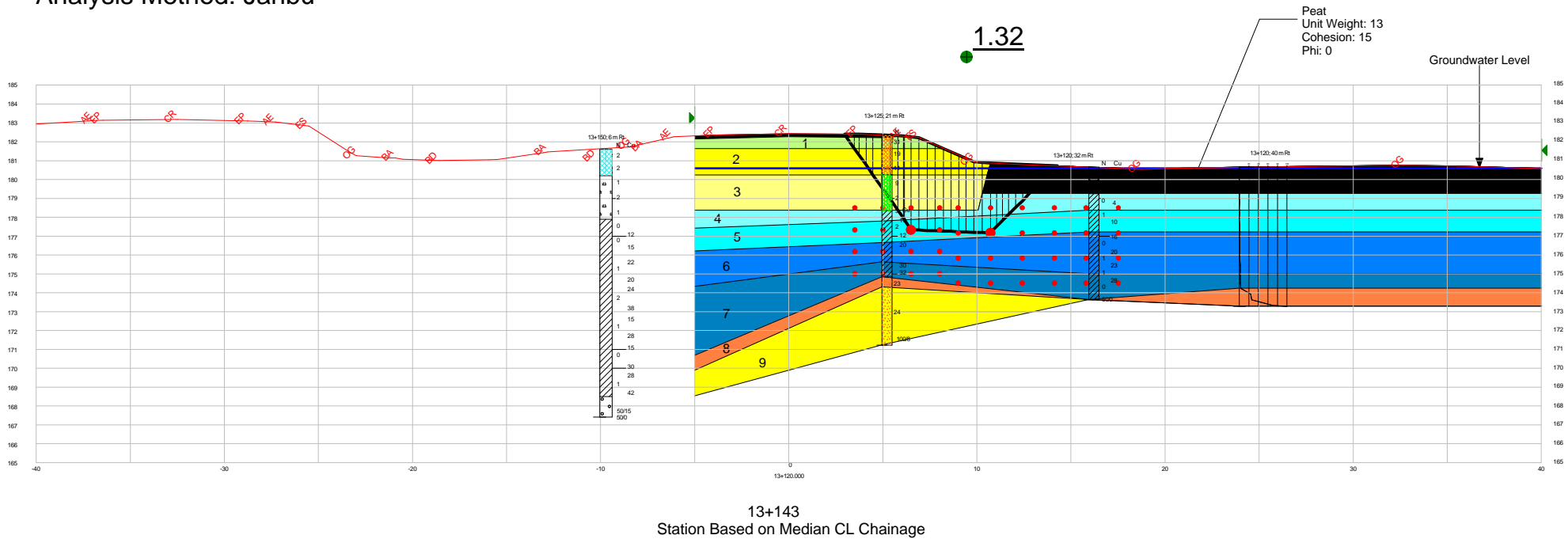
Undrained Case (Short-Term)

FIGURE H1-1

- | | | |
|---|--|--|
| 1 - Sand and Gravel
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 4 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 28 kPa
Phi: 0 degree |
| 2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 10 kPa
Phi: 0 degree | 8 - Sand and Gravel
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 34 degree |
| 3 - Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 29 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree | 9 - Sand (some gravel)
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 33 degree |



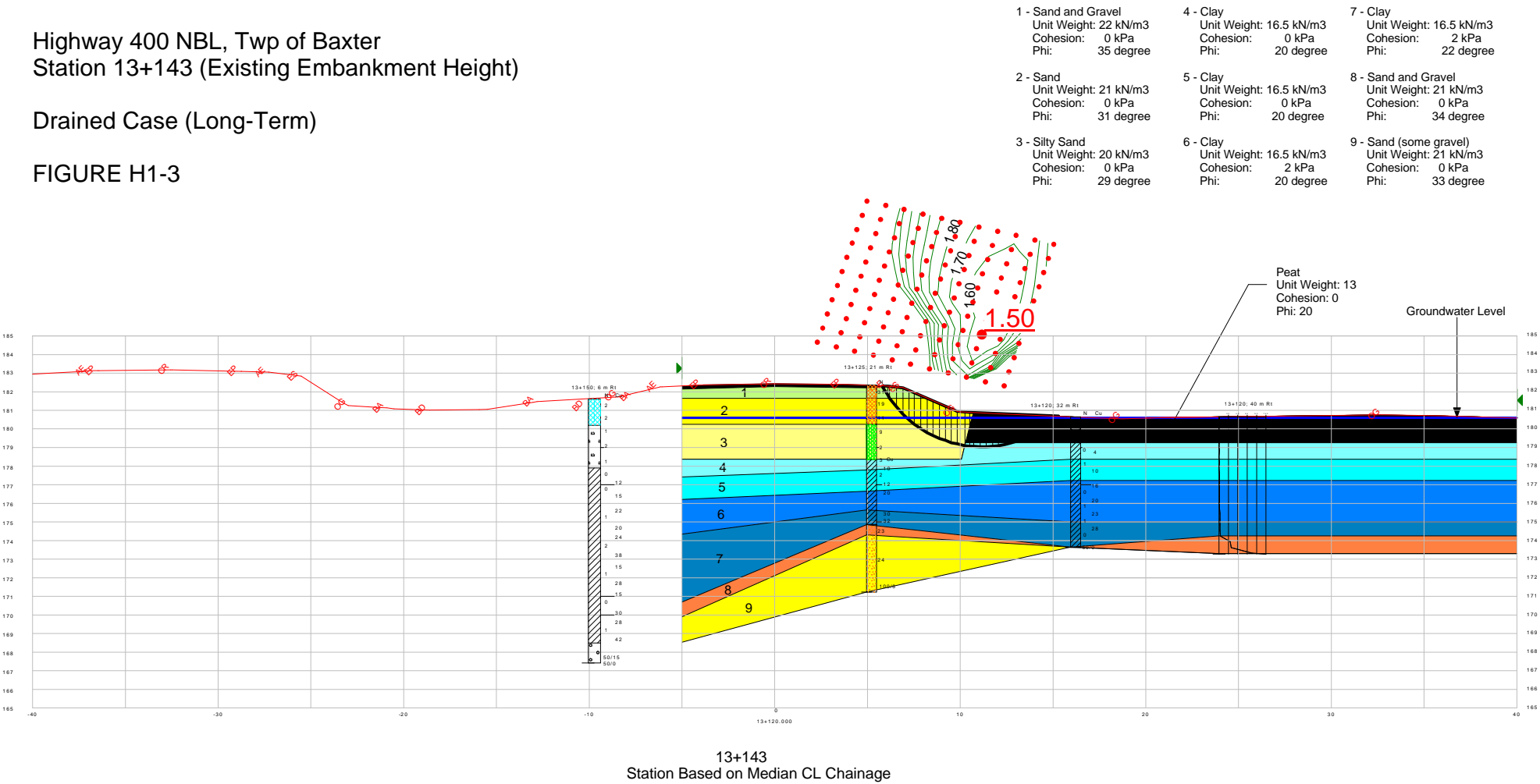
13+143
Station Based on Median CL Chainage



Highway 400 NBL, Twp of Baxter
Station 13+143 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H1-3

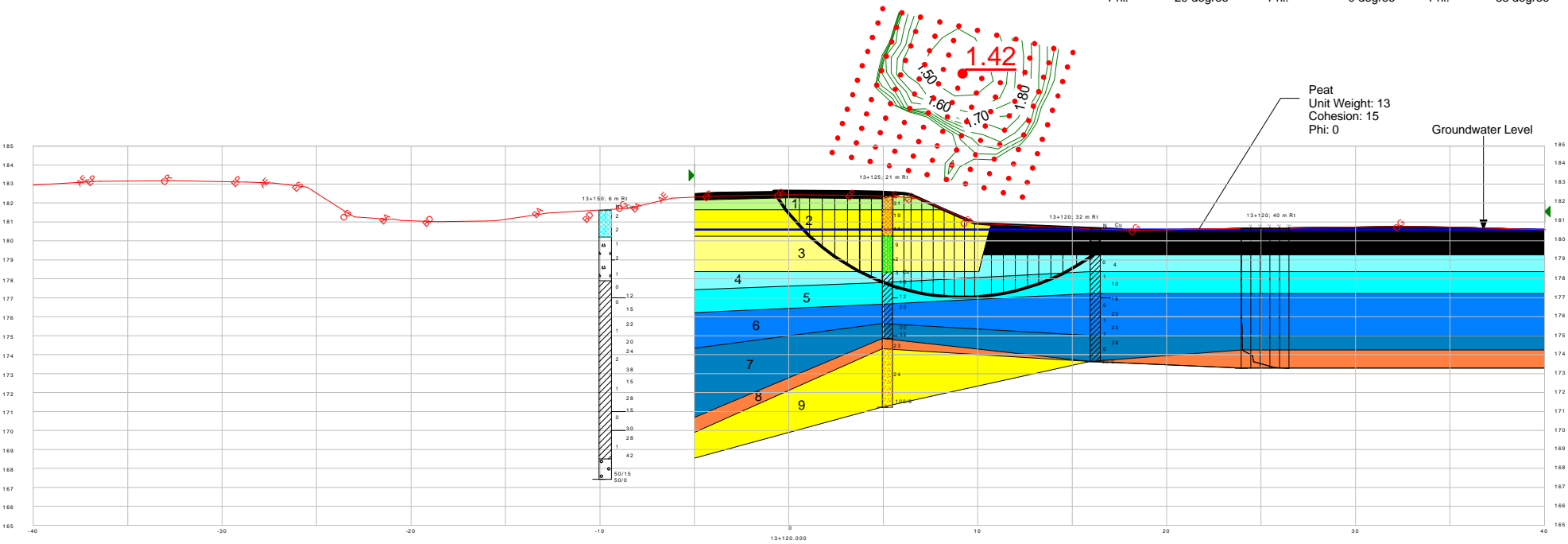


Highway 400 NBL, Twp of Baxter
Station 13+143 (Existing Embankment Height + 0.2 m Grade Raise)

Undrained Case (Short-Term)

FIGURE H1-4

- | | | |
|---|--|--|
| 1 - Sand and Gravel
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 4 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 28 kPa
Phi: 0 degree |
| 2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 10 kPa
Phi: 0 degree | 8 - Sand and Gravel
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 34 degree |
| 3 - Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 29 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree | 9 - Sand (some gravel)
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 33 degree |

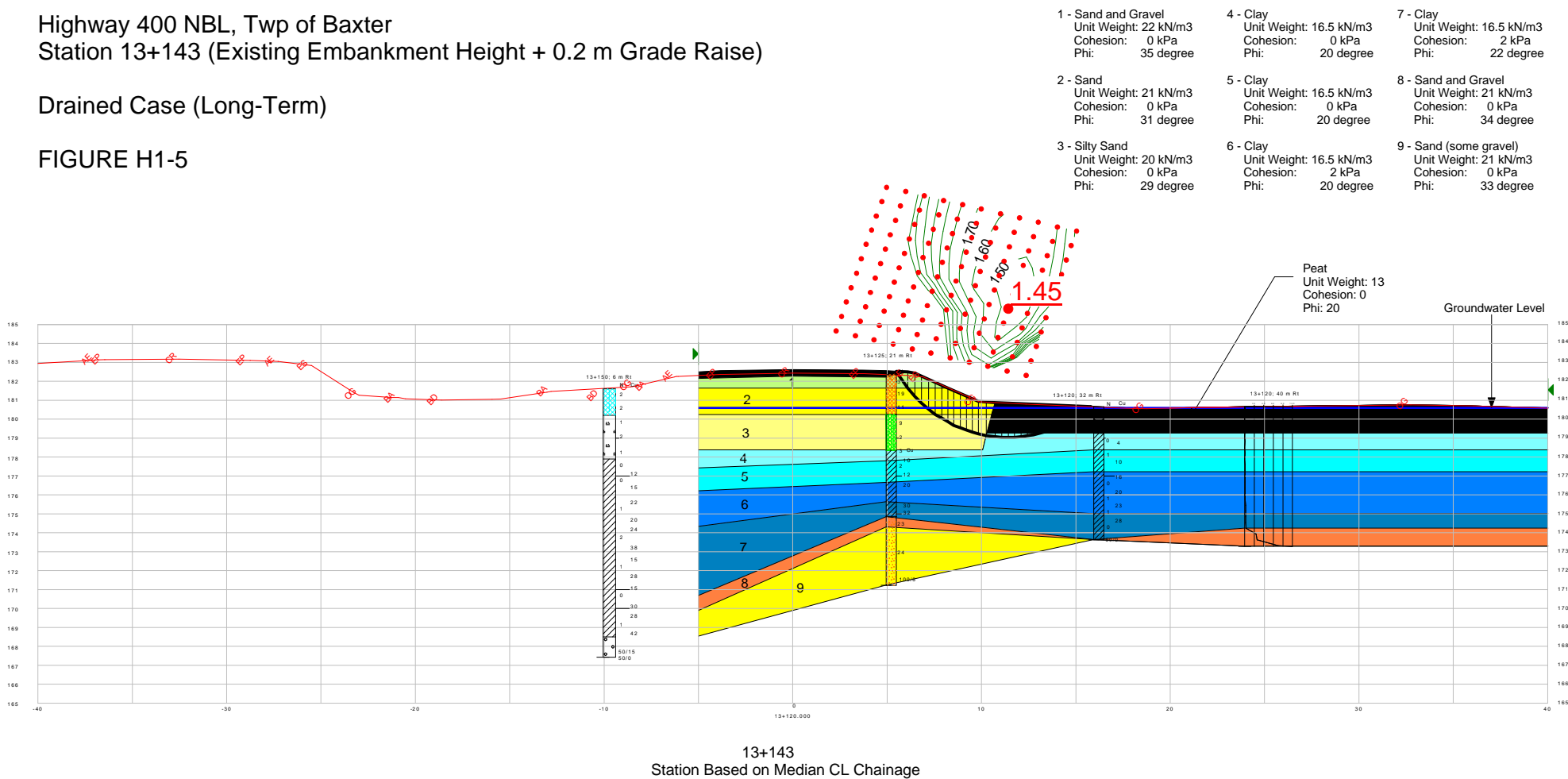


13+143
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Baxter
Station 13+143 (Existing Embankment Height + 0.2 m Grade Raise)

Drained Case (Long-Term)

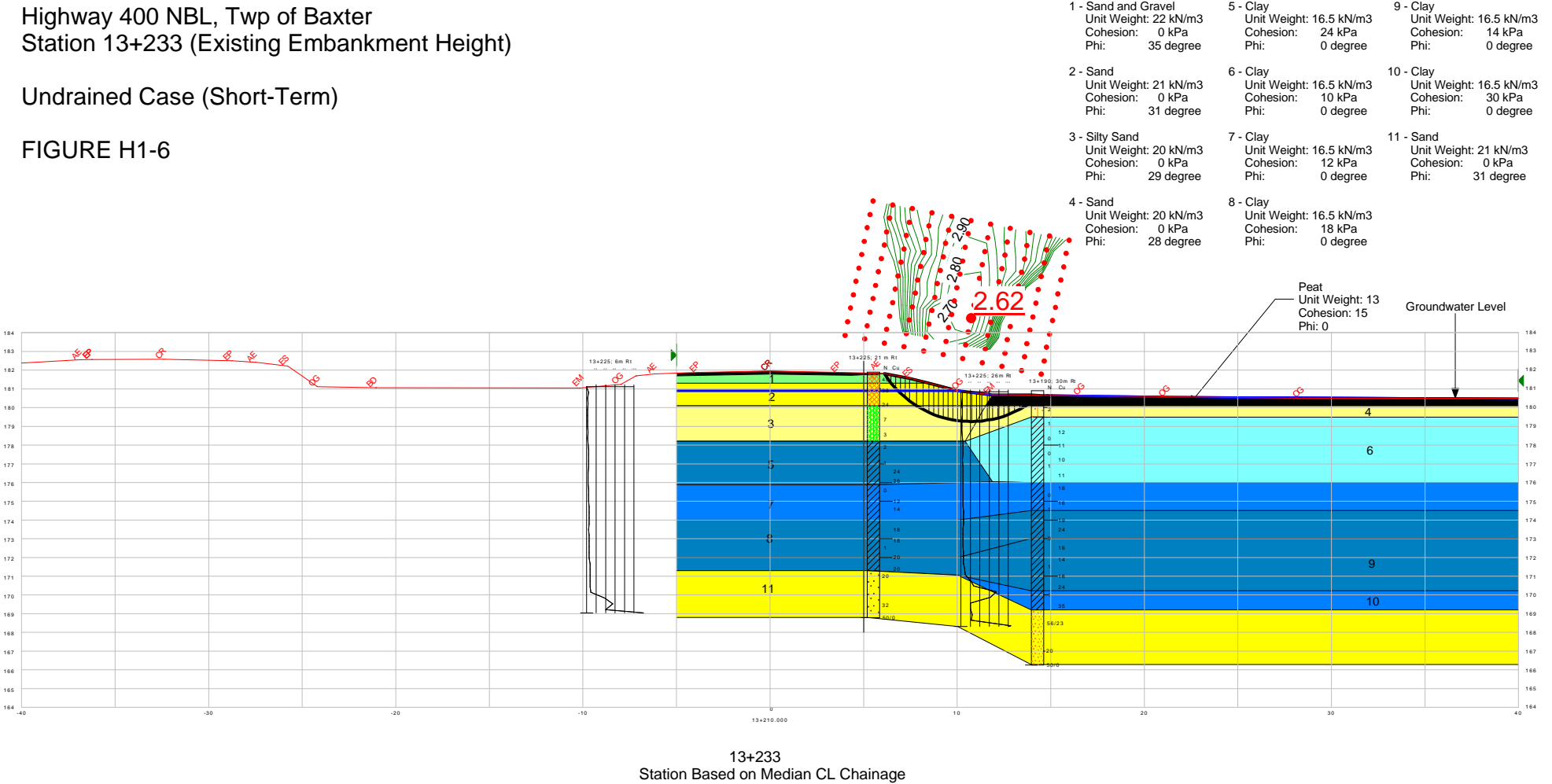
FIGURE H1-5



Highway 400 NBL, Twp of Baxter
Station 13+233 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H1-6

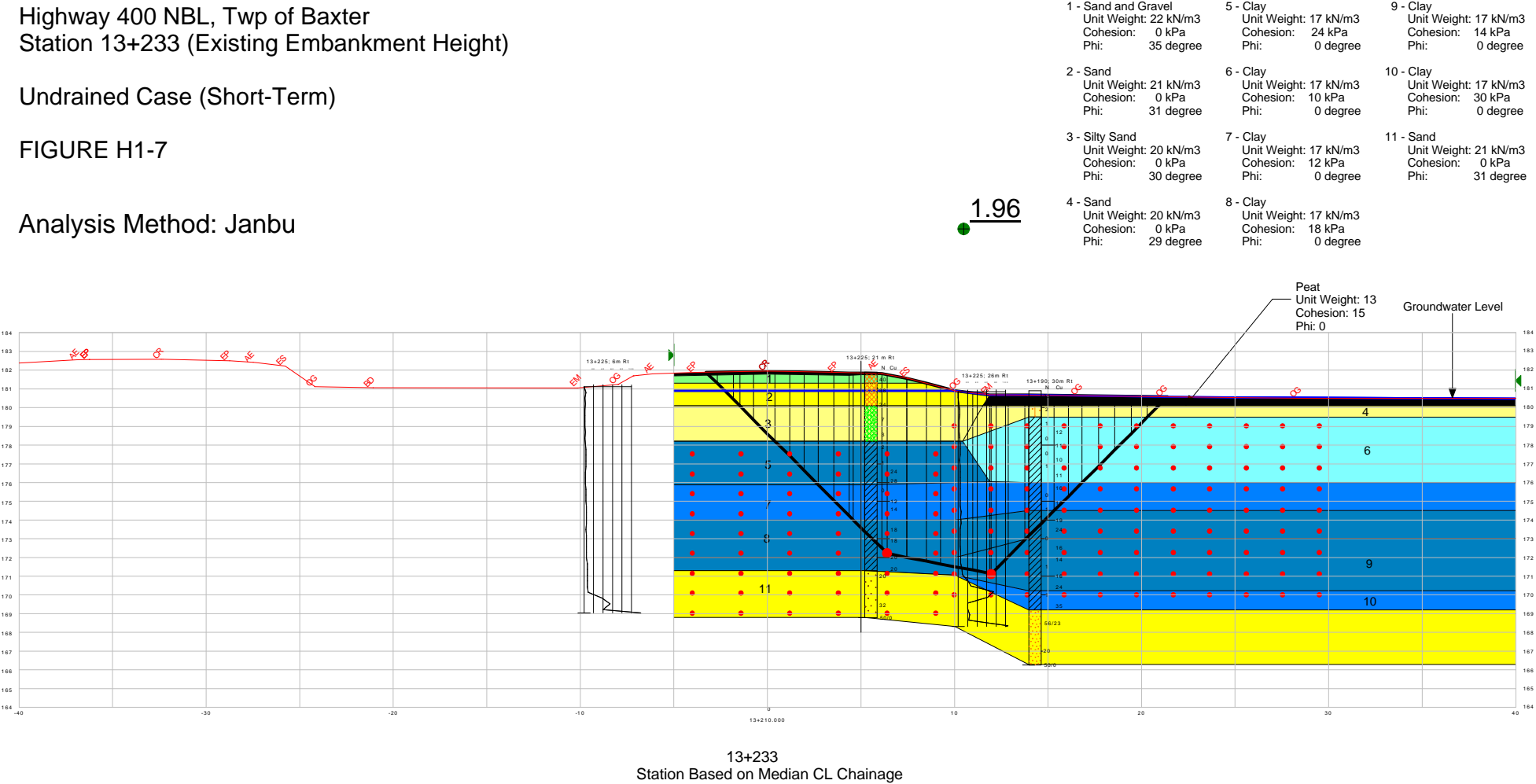


Highway 400 NBL, Twp of Baxter
Station 13+233 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H1-7

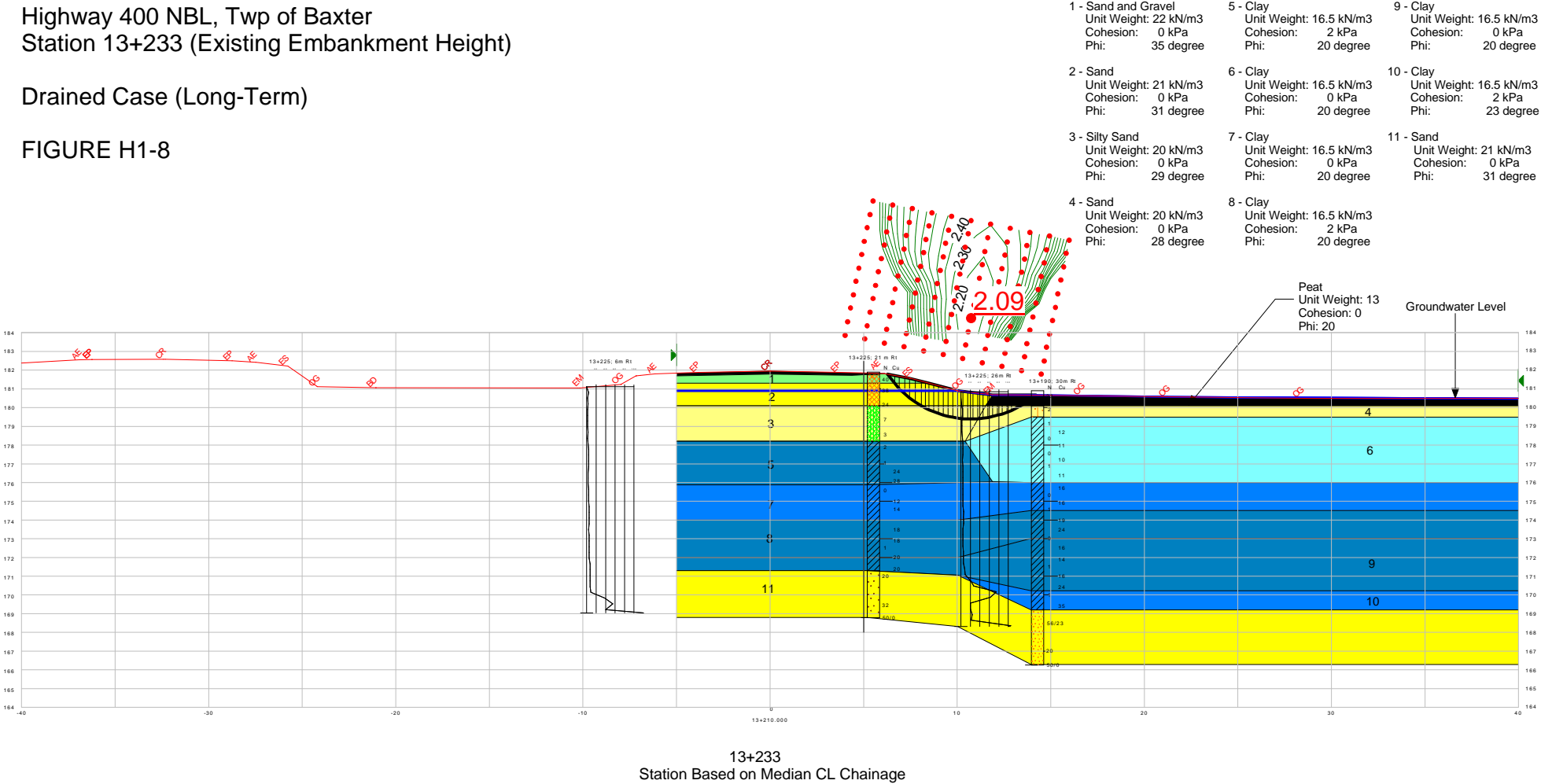
Analysis Method: Janbu



Highway 400 NBL, Twp of Baxter
Station 13+233 (Existing Embankment Height)

Drained Case (Long-Term)

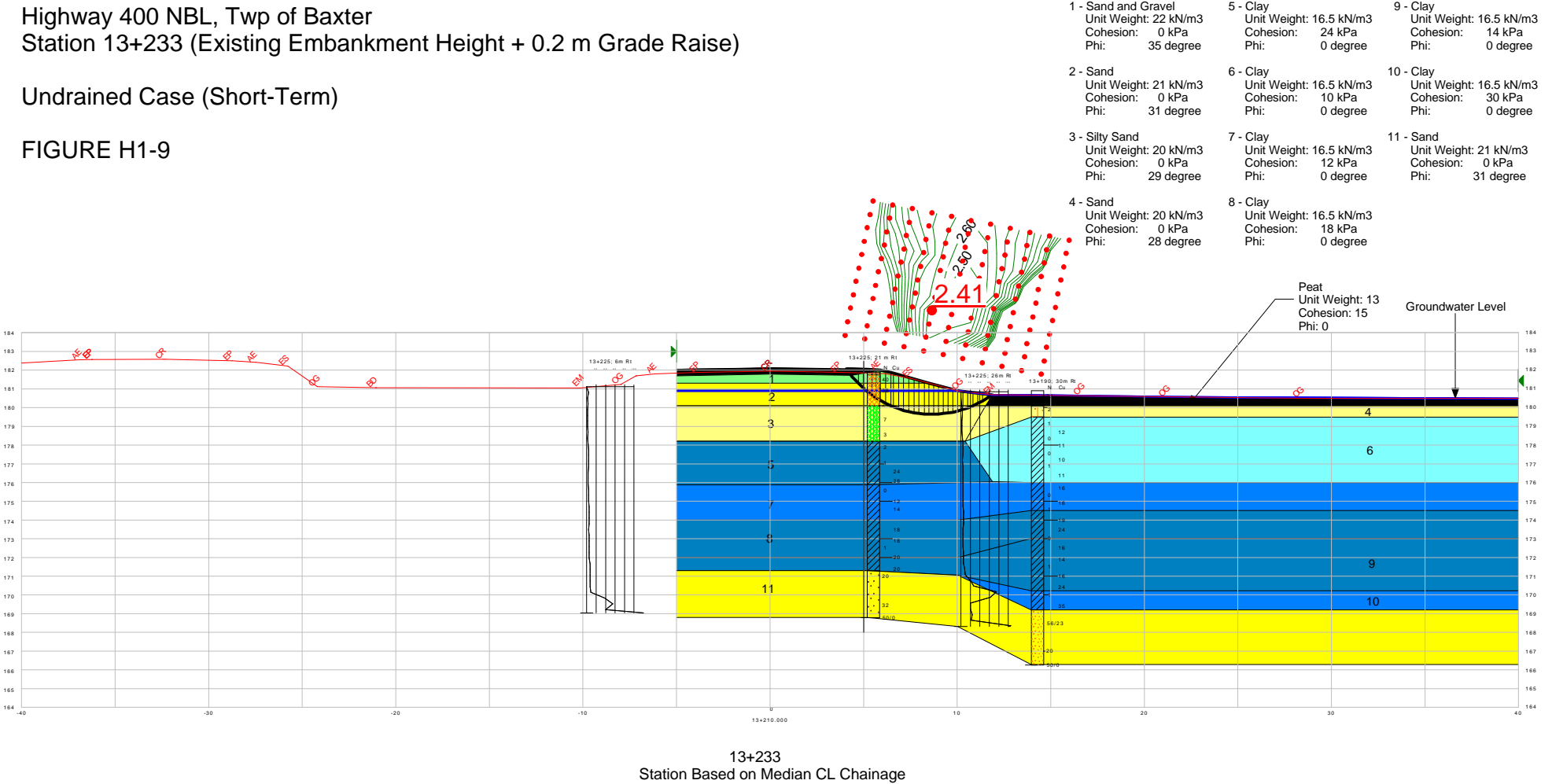
FIGURE H1-8



Highway 400 NBL, Twp of Baxter
Station 13+233 (Existing Embankment Height + 0.2 m Grade Raise)

Undrained Case (Short-Term)

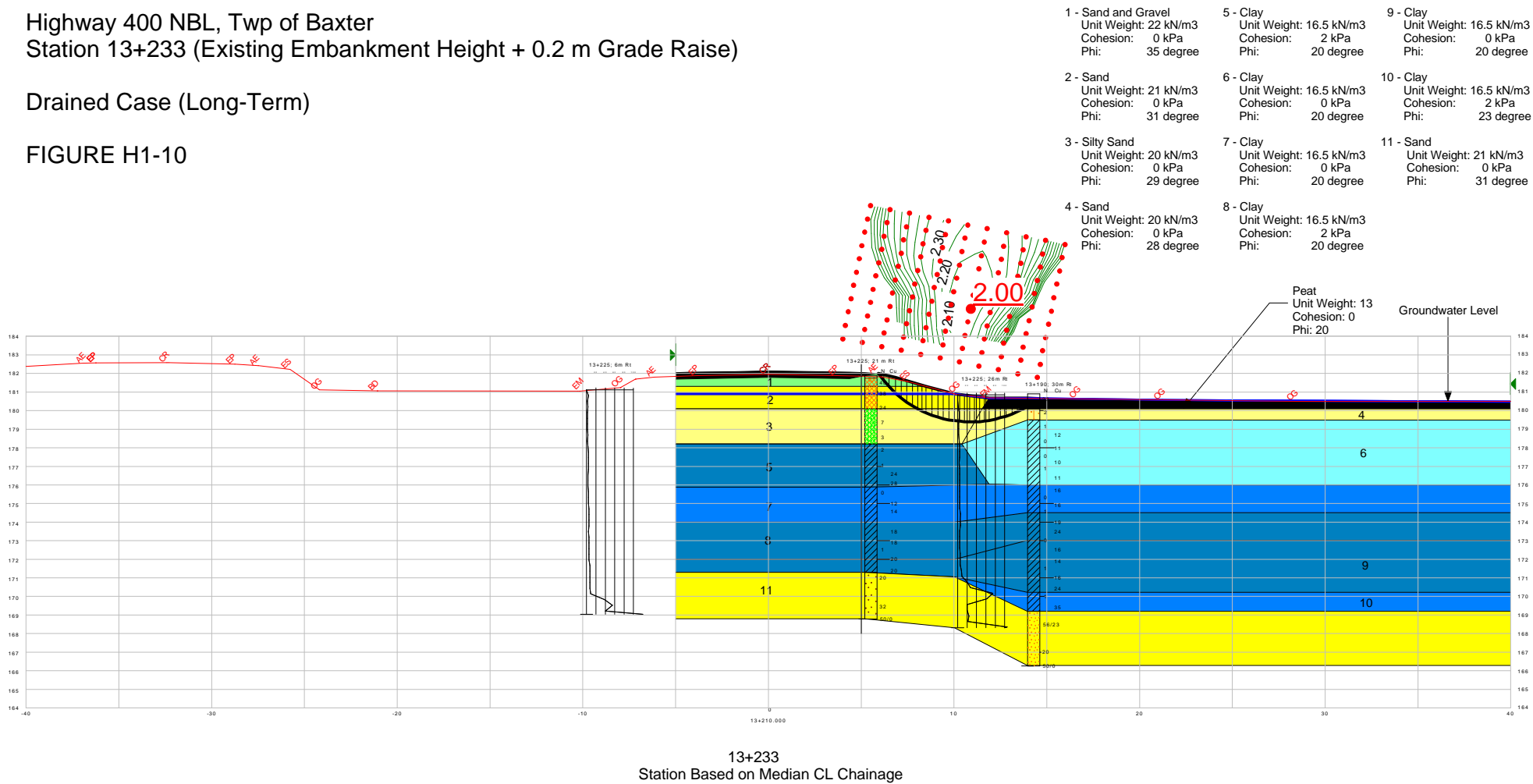
FIGURE H1-9



Highway 400 NBL, Twp of Baxter
Station 13+233 (Existing Embankment Height + 0.2 m Grade Raise)

Drained Case (Long-Term)

FIGURE H1-10



5.2 SITE 2 (STA. 13+500 TO 13+800)

Boreholes drilled for this 300 m long section between Stations 13+500 and 13+800, designated as Site 2 for the purposes of this report, show up to about 9.3 m overburden (El. 171.8 m) based on refusal depths. In general, the boreholes drilled in the swampy areas show the presence of up to about 2 m organic soils underlain by up to about 4 m thick clay to silty clay. At most borehole locations, the clay to silty clay deposit are in turn underlain by basal granular soils.

The organic soils and the underlying clay to silty clay deposits are generally soft and highly compressible.

In this section, the existing embankment heights are generally between 1 and 2 m, except between Stations 13+710 and 13+760 where the embankment heights reach to between about 3.0 and 3.7 m. The presently proposed profile is given in Appendix G2, along with several typical cross sections. The proposed grades show that up to 100 mm grade raise contemplated. We understand that widening of the right shoulder is contemplated between Stations 13+690 and 13+750. A typical section depicting the proposed widening is given in Figure G2-2 (Station 13+732).

5.2.1 EMBANKMENT STABILITY

The proposed grade raises of 0.05 to 0.11 m (asphalt layer) are unlikely to cause any instability of the existing embankments (see Figure G2-2 in Appendix G2). This aspect was, however looked into (as was explained in Section 5.1.1 of this report). The results of our analyses are presented in Appendix H2. The soil parameters used in the analyses are shown on the individual computer print-out sheets given in Appendix H2. The soil parameters used in the analyses are shown on the individual computer print-out sheets given in Appendix H2.

Stability analyses were performed on typical section (i.e. Station 13+510) where the embankment height is about 1.8 m and where the grade will be raised by 0.1 m (by asphalt placement). The existing conditions are given in Figures H2-1, H2-2 and H2-3 of Appendix H2. These indicate a minimum factor of safety of about 1.5. Based on this, it is our opinion that when the embankment was first built, large scale horizontal (plastic) yield of the soil is unlikely to have taken place. Figures H2-4, H2-5 and H2-6 show stability analysis results with the proposed 0.1 m grade raise. In this case, the safety factors drop only very marginally and are still about 1.5, which is acceptable. Based on this, we do not anticipate slope instability problems with the proposed grade raises in this area.

A section was presented to us at Station 13+732 (Figure G2-2). At this section, the embankment is relatively high (i.e. 3.6 m high) while the subsurface conditions as

represented by the findings of Boreholes 13+725 (28 m Rt) and 13+733 (21.5 m Rt) show representative conditions for the general swampy areas in this section. An analysis based on the findings of these boreholes indicates a factor of safety of about 1.3 for the original short (undrained) and long-term (drained) conditions. In this section, a 0.05 m grade raise is contemplated and this is not likely to change the stability conditions. In addition to this, however, a widening of the right (east) shoulder is planned. This is planned to be 3H:1V in the upper granular pavement section and 1.25H:1V side slopes for the lower part, which will consist of rock fill. The analysis with this configuration yielded a safety factor of about 1.1 for both the short and the long-term for shallow failures and about 1.2 for relatively deeper seated failures. The figures for the shallow seated failures are considered unacceptable and to remedy this situation it is suggested that a 1.0 m high (over the o.g. level) and a minimum 3.0 m long rock fill berm (beyond the toe of the final configuration) be placed (see Figure G2-3). Rock fill berm should be placed before widening. From the stability point of view, the existing peat/organic silt need not be removed provided rock fill is pushed into it. However, MTO will likely require that all organic soils be removed from beneath the toe berm as well as the widening. The removal of the organic soils must be carried out in a manner so as not to cause a failure of the existing embankment. For this purpose, the excavation should be carried out in sufficiently narrow sections (i.e. no wider than 3 m sections) perpendicular to the roadway (i.e. perpendicular to the longitudinal direction). Excavation and backfilling of each section should be carried out concurrently and under water. Rock fill placed below groundwater table may be end dumped. The rock fill should be placed according to OPSS 206.07.08 and current MTO practice. This operation should be carried along the entire length of proposed widening prior to its implementation. With this approach, the safety factors reach to about 1.5 or more, a figure which is considered acceptable (see Appendix H2).

5.2.2 SETTLEMENT OF EMBANKMENTS

It is understood that along this section of the highway the existing grades will be raised by up to about 0.1 m in between Stations 13+500 and 13+540 and between 13+580 and 13+660 and by up to about 0.07 m between 13+700 and 13+740. In addition, the widening of the right shoulder is planned for in the latter section (i.e. in between about Stations 13+700 and 13+740).

For the purposes of our settlement analysis, it was assumed that the organic soils underlying the existing embankments were removed during the construction of the highway (as evidenced by the findings of Boreholes 13+511 (21 m Rt) and 13+733 (21.5 m Rt)). The thickness of the underlying clay to silty clay deposit at the borehole locations reaches a maximum of about 4.0 m.

Our settlement analysis indicates that a consolidation settlement of the order of 400 to 500 mm is likely to take place in areas where the clay to silty clay deposit is relatively thick.

As was mentioned in Section 5.1.2 of this report, the embankments appear to have been constructed on or about 1976. If this is the case, the primary consolidation settlements would have been by now completed. A secondary settlement of the order of 5 to 10 mm can be expected within the next ten years or so, due to existing embankment stresses. Also as was discussed before, some rehabilitation and minor regrading may have taken place since the original construction (e.g. during the construction of the SBL).

These settlement figures were based on the consolidation test results carried out for this investigation and also by MTO in 1991 for the construction of the SBL.

Based on these data, the anticipated settlements under the additional 0.05 to 0.1 m pavement raise using asphaltic concrete would be of the order of 10 to 20 mm. This would be completed within the next five years.

The situation is somewhat more complicated in the section where shoulder widening is anticipated (see Figure G2-2 in Appendix G2). Here the widening is expected to cause a settlement of the order of 70 mm at the outer edge of the widened shoulder. In anticipation of this, consideration may be given to raising the grade at the outer edge of the widened shoulder by about this amount, decreasing to zero at a point about 4 m to the west of this point.

Appendix G2

Proposed Profiles and Typical Cross-Section

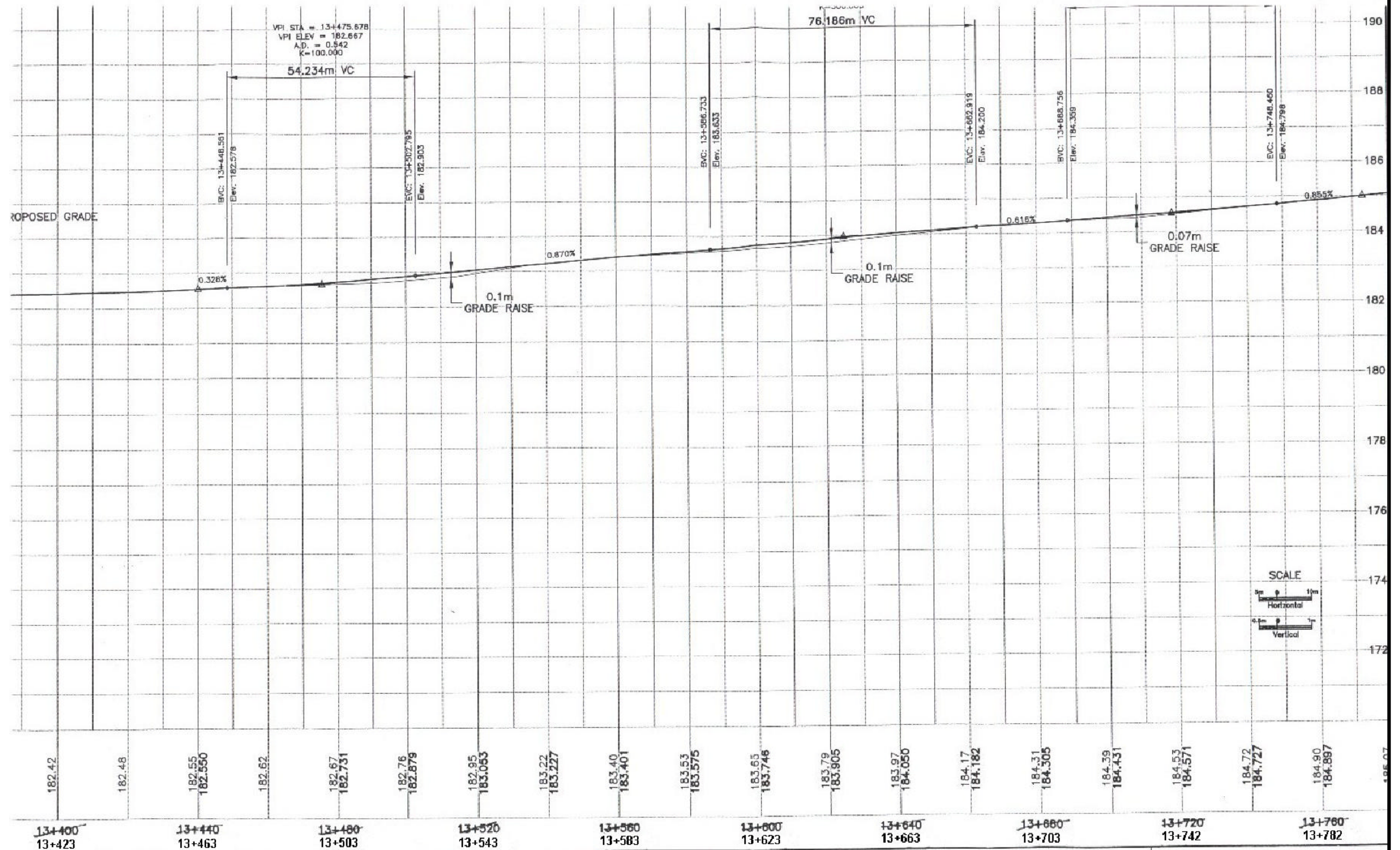
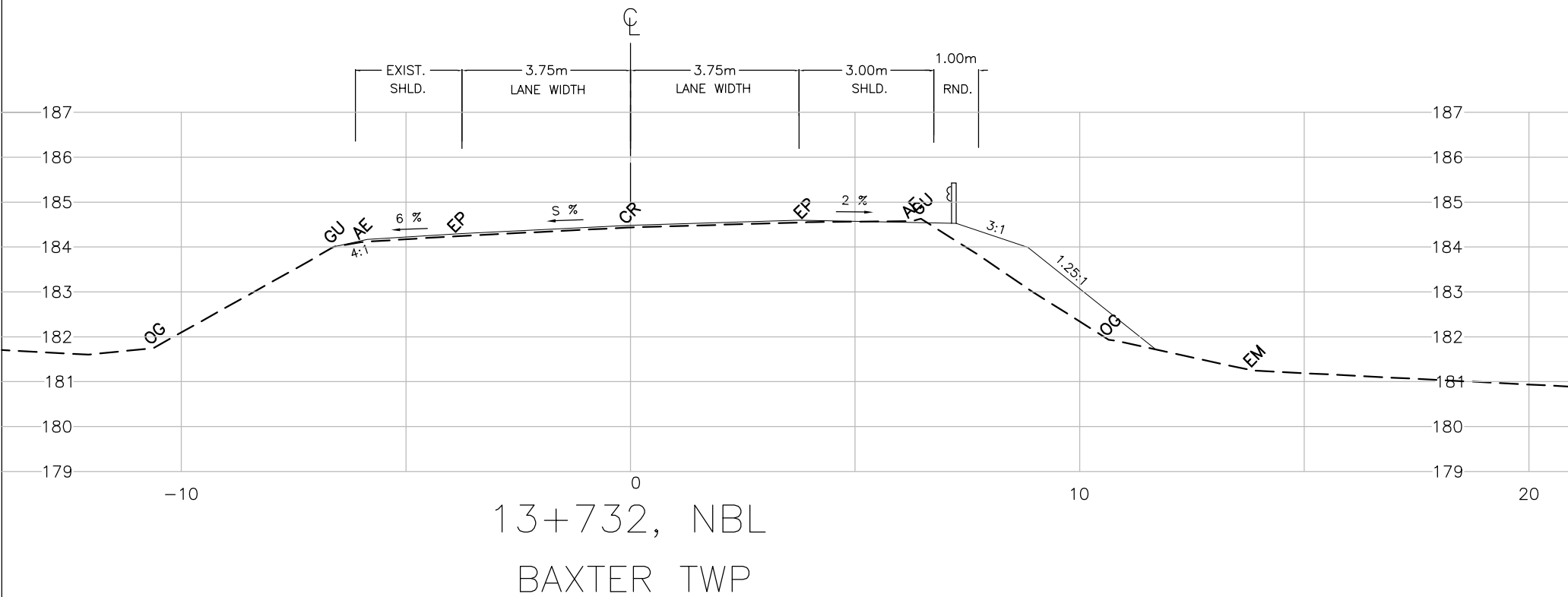


FIGURE G2-1

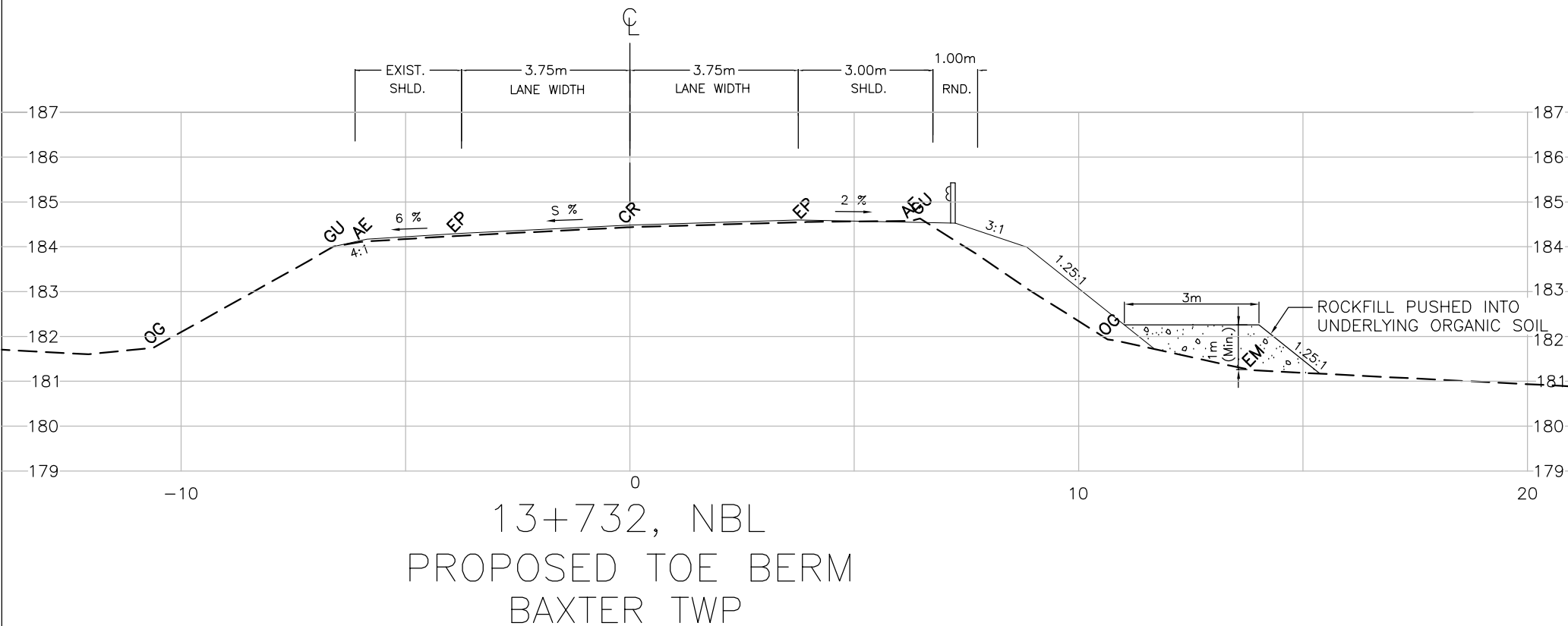
STATION BASED ON MEDIAN CL CHAINAGE

0.05 m GRADE RAISE



STATION BASED ON MEDIAN CL CHAINAGE

0.05 m GRADE RAISE



Appendix H2

Slope Stability Analysis Results

Highway 400 NBL, Twp of Baxter
Station 13+510 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H2-1

- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 30 degree

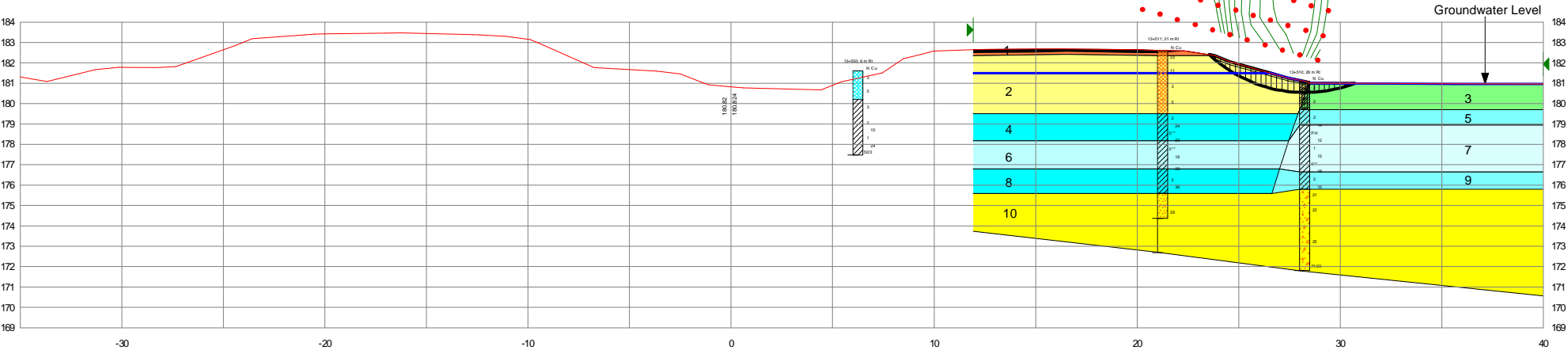
3 - Organic Clayey Silt / Silty Clay
Unit Weight: 15 kN/m3
Cohesion: 0 kPa
Phi: 28 degree
- 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree

5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 14 kPa
Phi: 0 degree

6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 16 kPa
Phi: 0 degree
- 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 12 kPa
Phi: 0 degree

8 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 25 kPa
Phi: 0 degree

9 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 15 kPa
Phi: 0 degree
- 10 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree



13+510, NBL
BAXTER TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Baxter
Station 13+510 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H2-2

- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 30 degree

3 - Organic Clayey Silt / Silty Clay
Unit Weight: 15 kN/m3
Cohesion: 0 kPa
Phi: 28 degree
- 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree

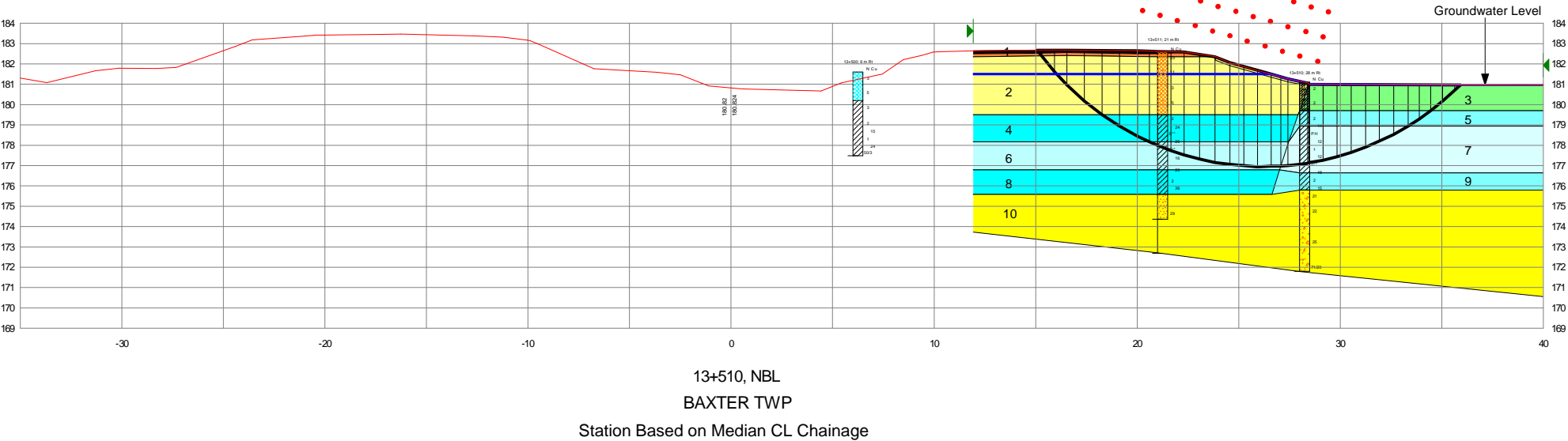
5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 14 kPa
Phi: 0 degree

6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 16 kPa
Phi: 0 degree
- 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 12 kPa
Phi: 0 degree

8 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 25 kPa
Phi: 0 degree

9 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 15 kPa
Phi: 0 degree

10 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree



Highway 400 NBL, Twp of Baxter
Station 13+510 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H2-3

- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 30 degree

3 - Organic Clayey Silt / Silty Clay
Unit Weight: 15 kN/m3
Cohesion: 0 kPa
Phi: 28 degree
- 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree

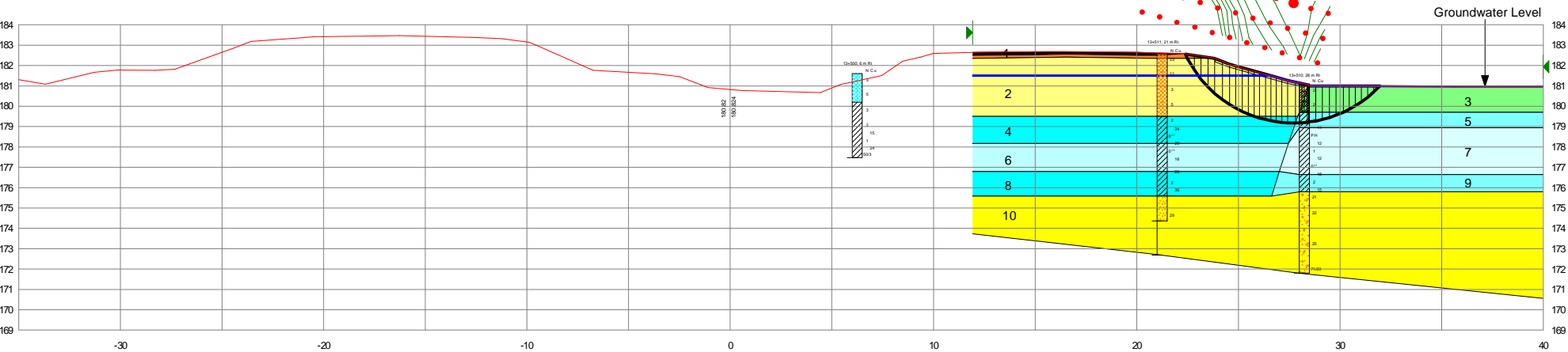
5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree

6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 21 degree
- 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree

8 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree

9 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 21 degree

10 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree



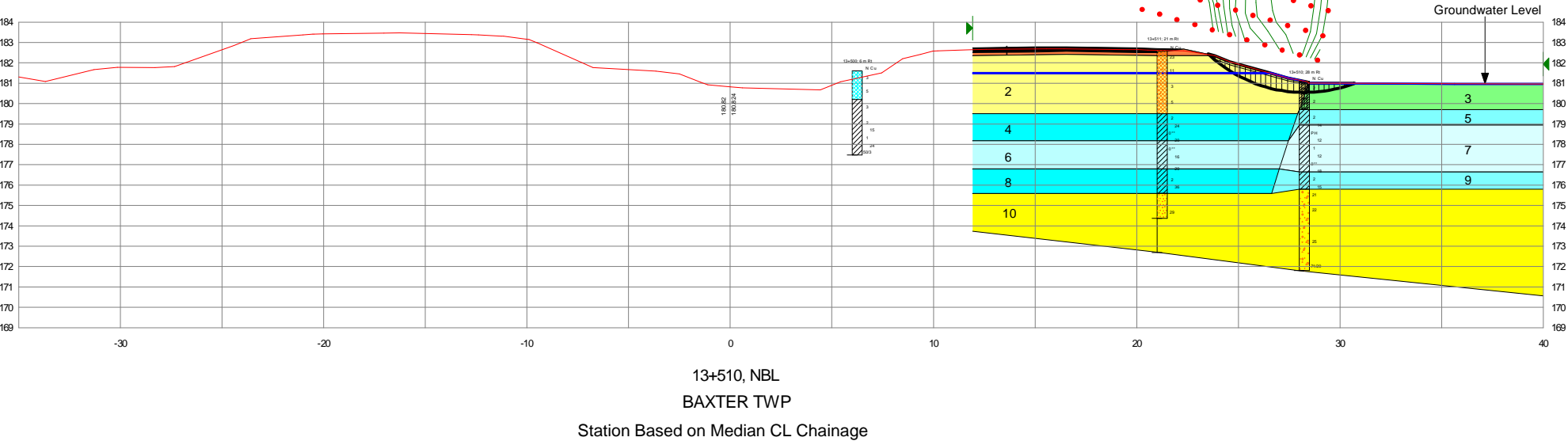
13+510, NBL
BAXTER TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Baxter
Station 13+510 (Existing Embankment Height)
[with 0.1 m Grade Raise]

Undrained Case (Short-Term)

FIGURE H2-4

- | | | |
|--|--|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 12 kPa
Phi: 0 degree |
| 2 - Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 14 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 25 kPa
Phi: 0 degree |
| 3 - Organic Clayey Silt / Silty Clay
Unit Weight: 15 kN/m3
Cohesion: 0 kPa
Phi: 28 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 16 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 15 kPa
Phi: 0 degree |
| | | 10 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree |

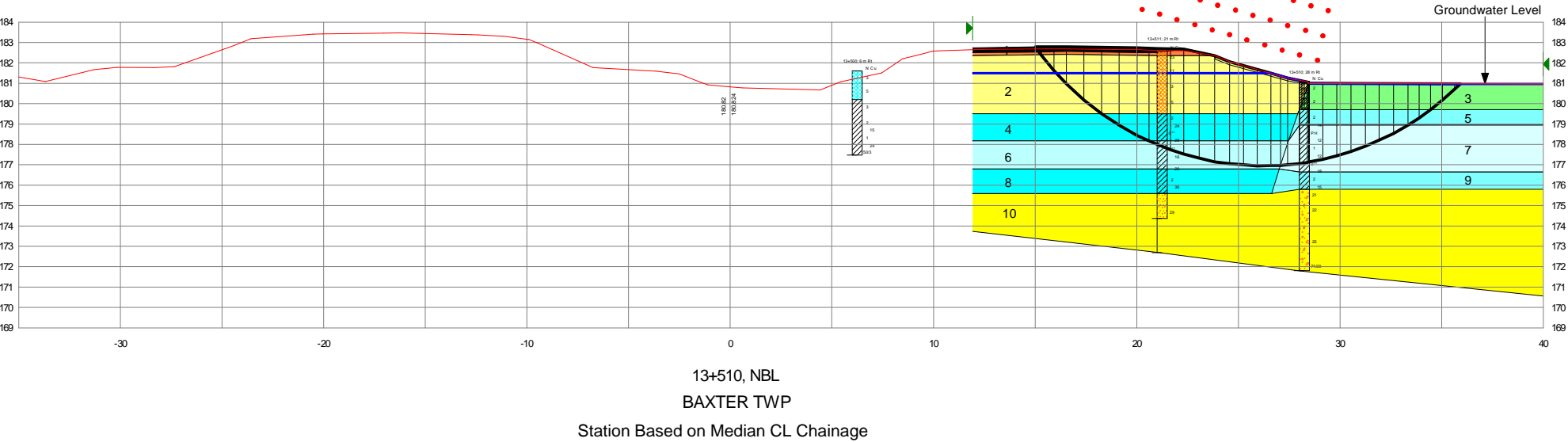


Highway 400 NBL, Twp of Baxter
Station 13+510 (Existing Embankment Height)
[with 0.1 m Grade Raise]

Undrained Case (Short-Term)

FIGURE H2-5

- | | | |
|--|--|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 12 kPa
Phi: 0 degree |
| 2 - Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 14 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 25 kPa
Phi: 0 degree |
| 3 - Organic Clayey Silt / Silty Clay
Unit Weight: 15 kN/m3
Cohesion: 0 kPa
Phi: 28 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 16 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 15 kPa
Phi: 0 degree |
| | | 10 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree |



Highway 400 NBL, Twp of Baxter
Station 13+510 (Existing Embankment Height)
[with 0.1 m Grade Raise]

Drained Case (Long-Term)

FIGURE H2-6

- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 30 degree

3 - Organic Clayey Silt / Silty Clay
Unit Weight: 15 kN/m3
Cohesion: 0 kPa
Phi: 28 degree
- 4 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree

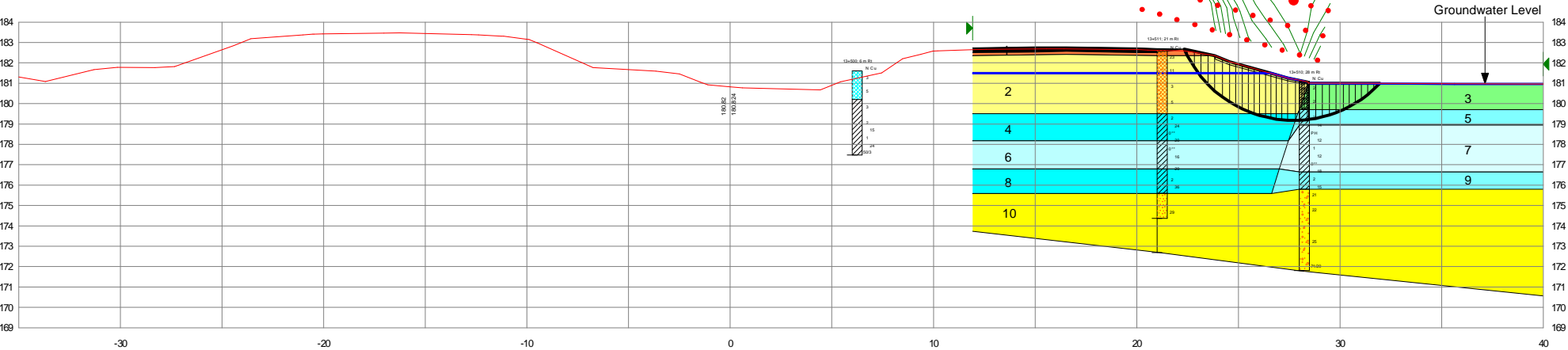
5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree

6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 21 degree
- 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree

8 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree

9 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 21 degree

10 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree

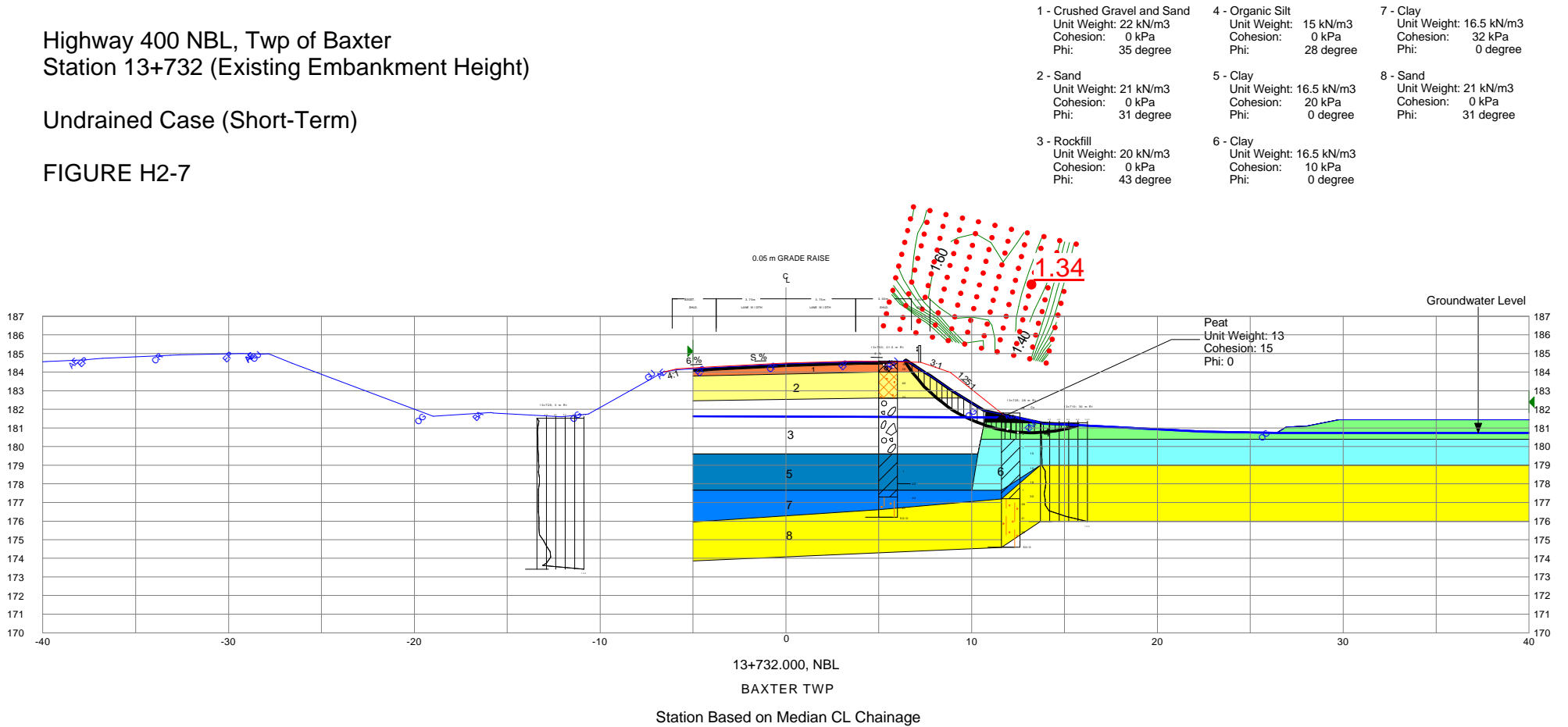


13+510, NBL
BAXTER TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height)

Undrained Case (Short-Term)

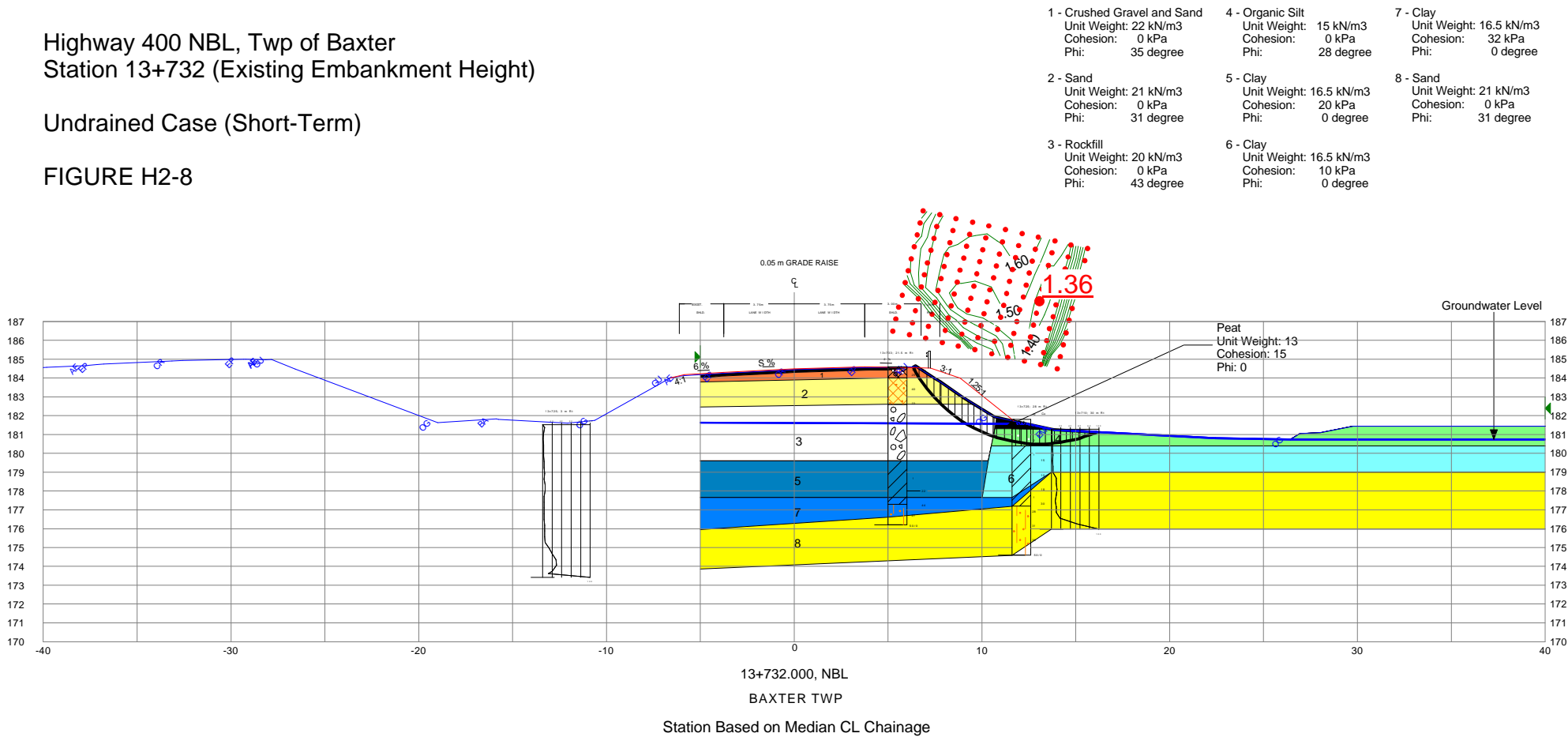
FIGURE H2-7



Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height)

Undrained Case (Short-Term)

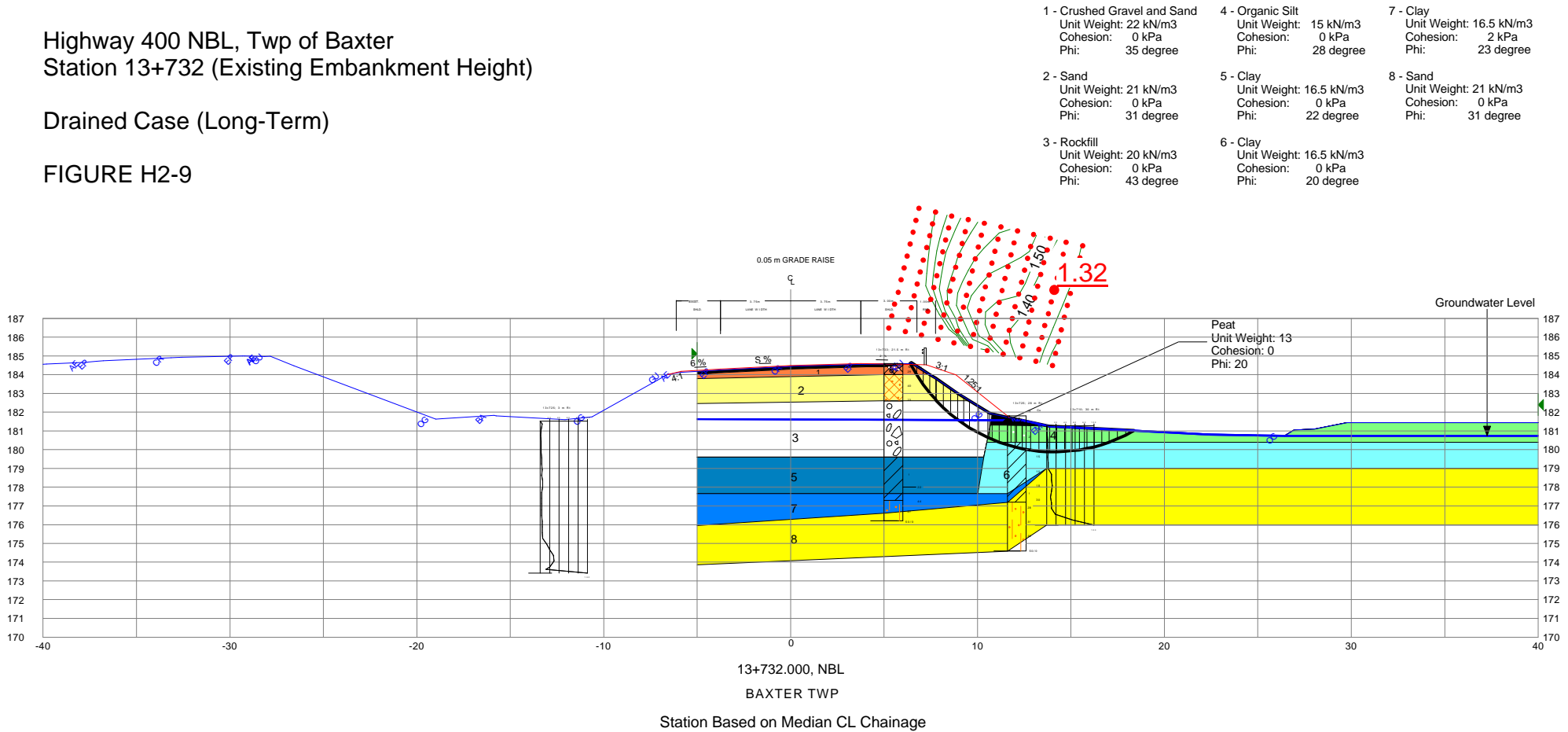
FIGURE H2-8



Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H2-9

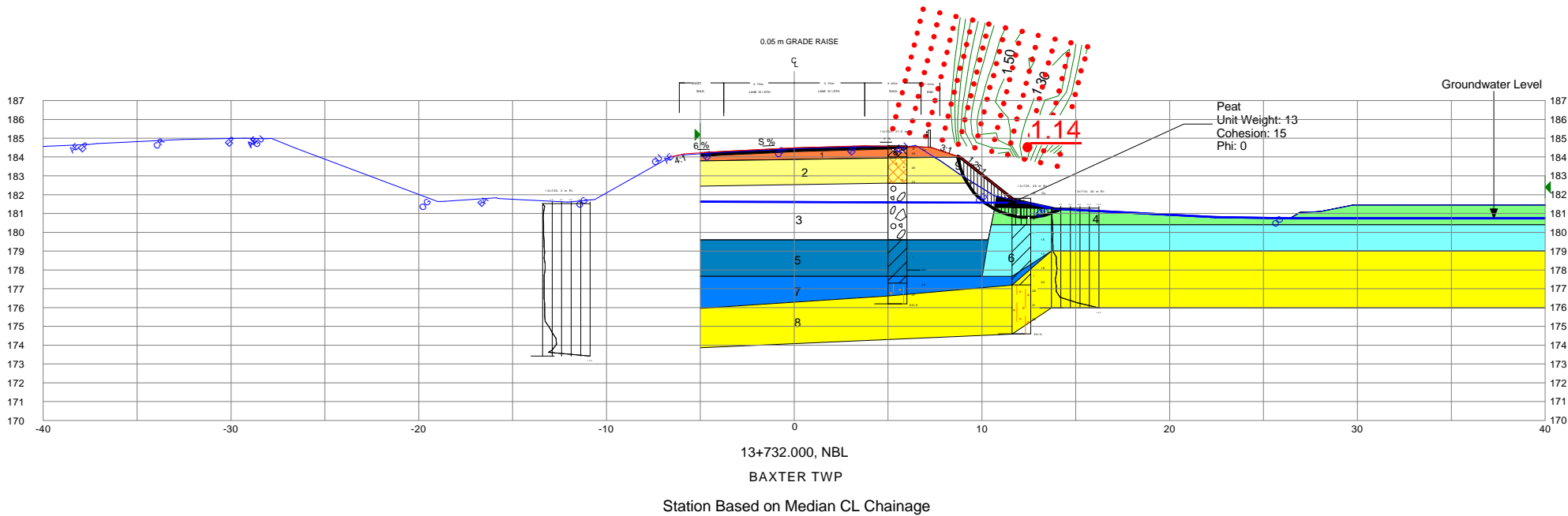


Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height + Widening + 0.05 m Grade Raise)

Undrained Case (Short-Term)

FIGURE H2-10

- | | | |
|---|--|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Organic Silt
Unit Weight: 15.0 kN/m3
Cohesion: 0 kPa
Phi: 28 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 32 kPa
Phi: 0 degree |
| 2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree | 8 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 10 kPa
Phi: 0 degree | 9 - Widening Material (Rockfill)
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree |

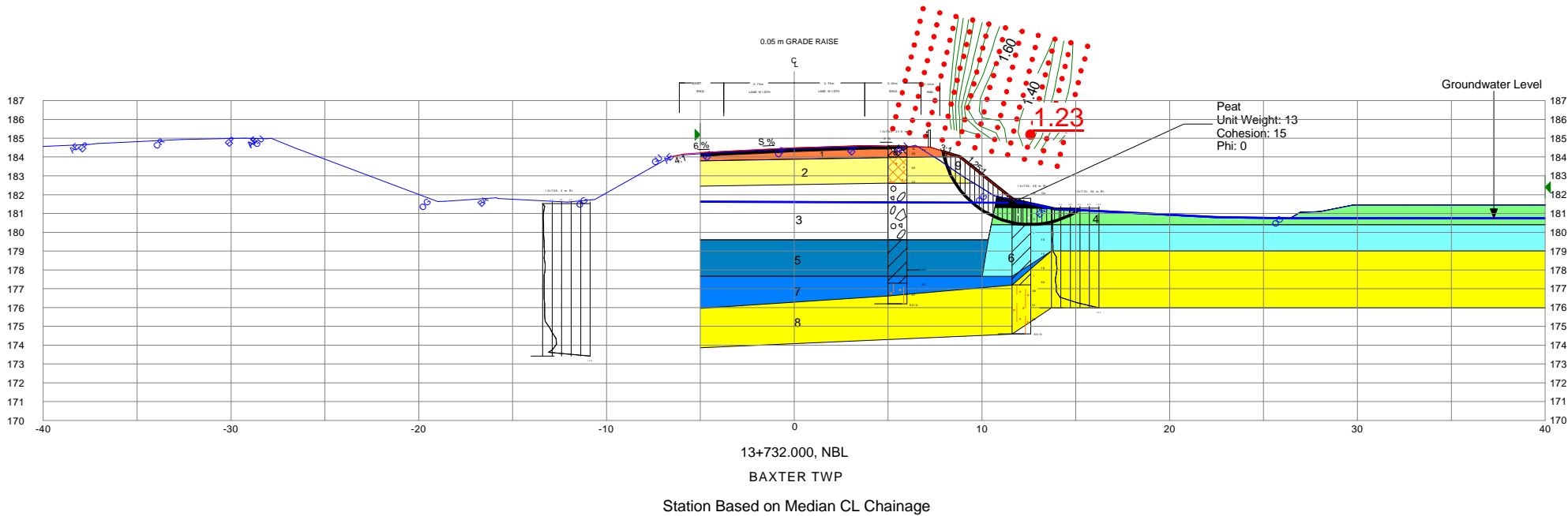


Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height + Widening + 0.05 m Grade Raise)

Undrained Case (Short-Term)

FIGURE H2-11

- | | | |
|---|--|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Organic Silt
Unit Weight: 15.0 kN/m3
Cohesion: 0 kPa
Phi: 28 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 32 kPa
Phi: 0 degree |
| 2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 20 kPa
Phi: 0 degree | 8 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 10 kPa
Phi: 0 degree | 9 - Widening Material (Rockfill)
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree |

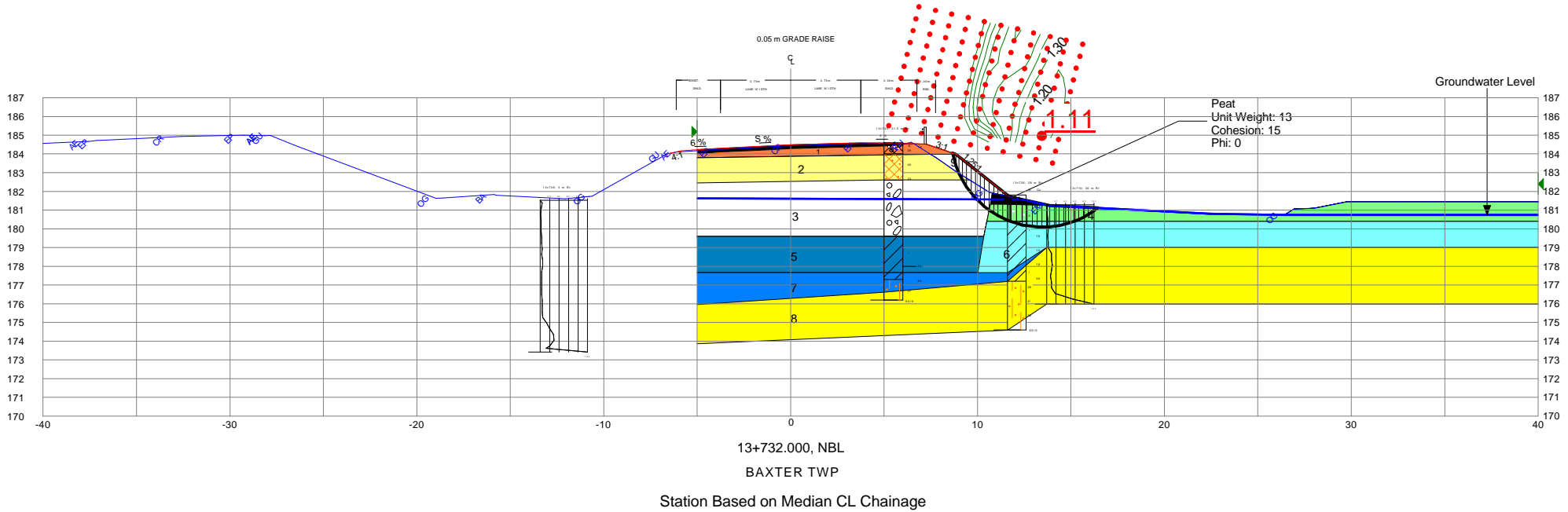


Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height + Widening + 0.05 m Grade Raise)

Drained Case (Long-Term)

FIGURE H2-12

- | | | |
|---|--|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Organic Silt
Unit Weight: 15.0 kN/m3
Cohesion: 0 kPa
Phi: 28 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 23 degree |
| 2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree | 8 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree | 9 - Widening Material (Rockfill)
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree |

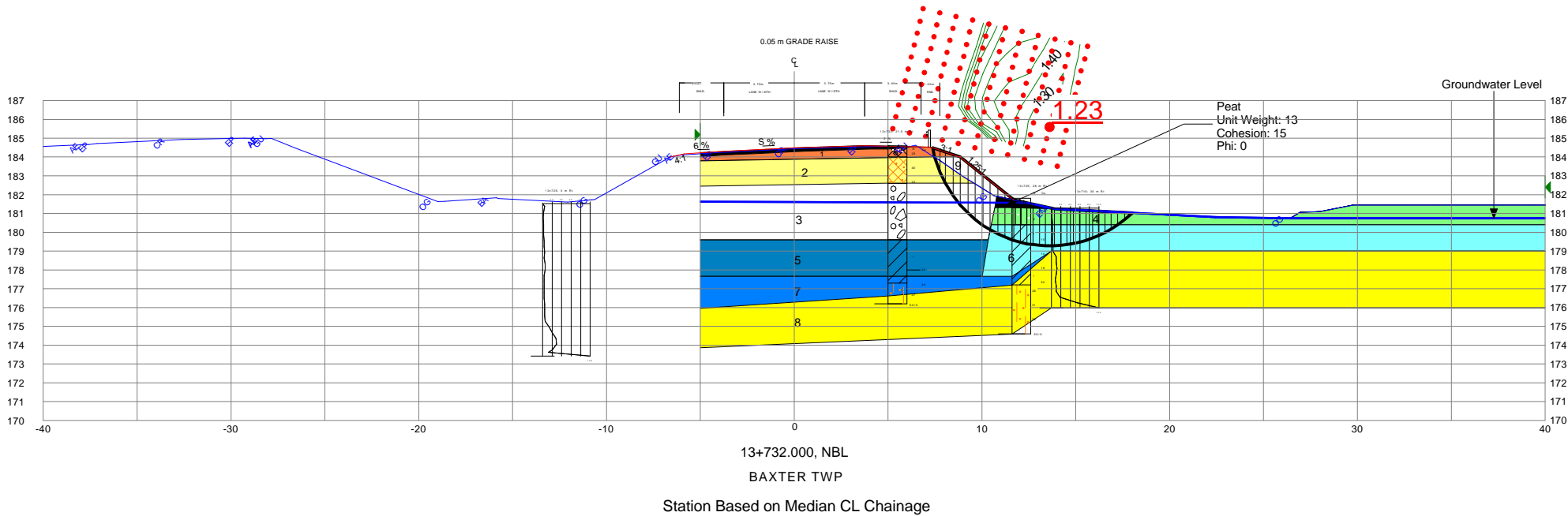


Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height + Widening + 0.05 m Grade Raise)

Drained Case (Long-Term)

FIGURE H2-13

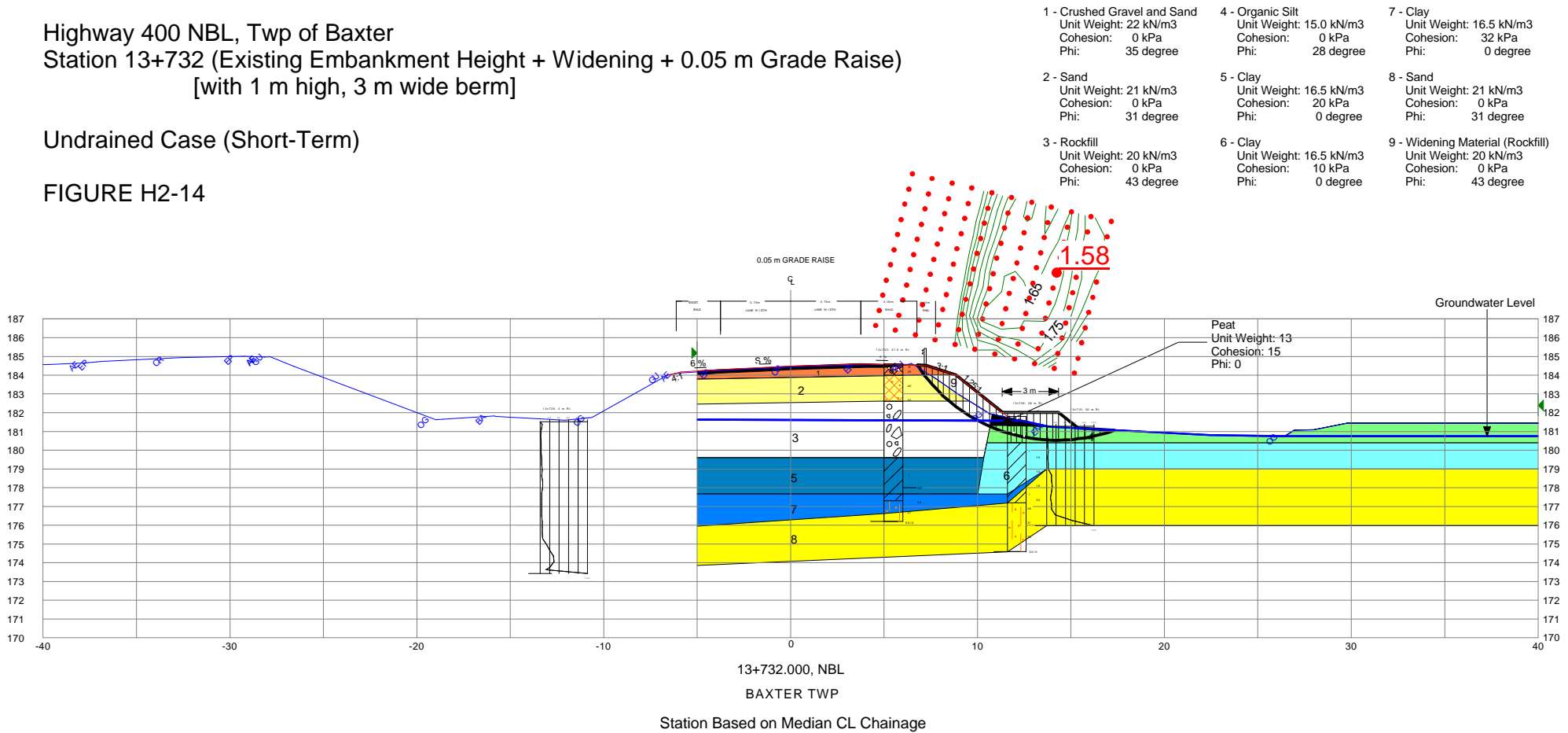
- | | | |
|---|--|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Organic Silt
Unit Weight: 15.0 kN/m3
Cohesion: 0 kPa
Phi: 28 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 2 kPa
Phi: 23 degree |
| 2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree | 8 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree | 9 - Widening Material (Rockfill)
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree |



Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height + Widening + 0.05 m Grade Raise)
[with 1 m high, 3 m wide berm]

Undrained Case (Short-Term)

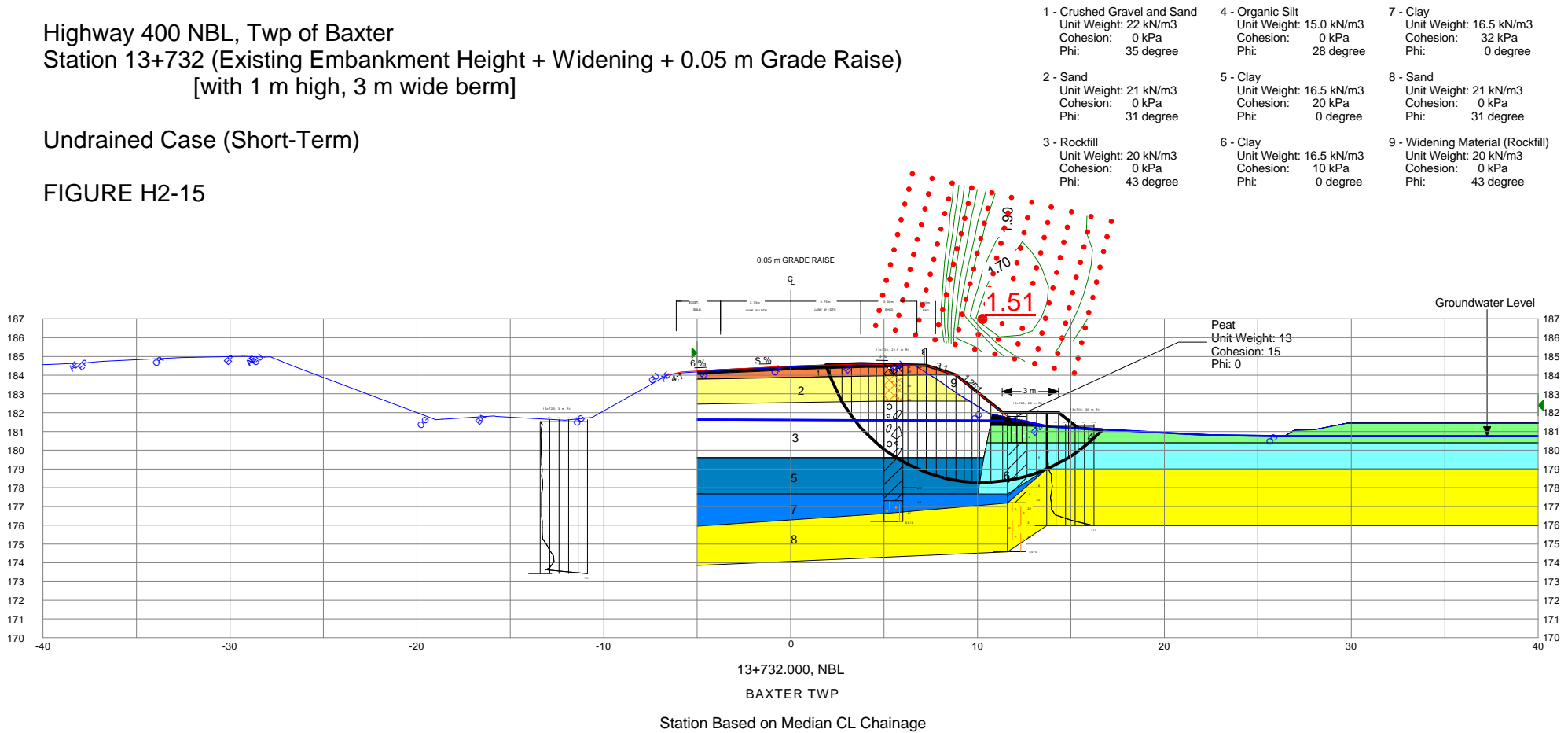
FIGURE H2-14



Highway 400 NBL, Twp of Baxter
Station 13+732 (Existing Embankment Height + Widening + 0.05 m Grade Raise)
[with 1 m high, 3 m wide berm]

Undrained Case (Short-Term)

FIGURE H2-15



5.3 SITE 3 (STA. 14+550 TO 14+750)

The section of Hwy 400 NBL between Sta. 14+550 and 14+750 is characterized by rock outcrops near both south and north extremities (i.e. near Stations 14+540 and 14+750), as shown in Drawing No. 3. In between the rock outcrops is a low-lying swampy section where the embankment is typically 1.2 to 1.5 m high.

In this 200 m long section, the recorded overburden thicknesses generally increase towards the middle of the swampy section indicating a bowl-shaped bottom, similar to previous sections, with some variation also in the east to west direction. In general, the boreholes show in this middle section, the presence of peat and organic clayey silt to a maximum depth of 2.1 m at the borehole locations. Below this, typically, a clay to silty clay deposit was contacted which is up to 8 m thick. It in turn is underlain by a thin layer of basal silty sand or sand extending to refusal depths which is likely to represent the surface of the bedrock or elevations close to it.

MTO previously drilled boreholes for the construction of the southbound lanes of the highway to the immediate west of this site. This information is presented in Appendices A3-2, B3-2 and D. From this, it can be inferred that in general the thickness of the overburden appears to increase from east to west.

The proposed design profile is given in Appendix G3 (see Figure G3-1). This shows that up to about 0.1 m grade raise is planned (probably by adding asphaltic concrete topping), primarily between Stations 14+620 and 14+660. A typical cross-section provided to us at Station 14+647 is given in Figure G3-2.

5.3.1 EMBANKMENT STABILITY

As was mentioned before in this section, the height of the embankment in the middle swampy sections (i.e. typically between Stations 14+550 and 14+700) is 1.2 to 1.5 m high. A foundation stability analysis was conducted to gauge available approximate safety factors when the embankments were first built (probably some 28 years ago). Slope stability analysis by the computer program Slope/W using Bishop's simplified method, using assumed and/or measured soil properties yielded safety factors in the order of between 1.7 and 3.5. Large scale horizontal deformations of the foundation soils are unlikely to have occurred with safety factors of these magnitudes when the embankments were first built. For this purpose, the section provided to us (i.e. at Station 14+647) was utilized and it was assumed that all organic soils under the footprint of the embankment were removed, as evidenced by the findings of Borehole 14+660 (24 m Rt) and that the embankment fill consists of mainly rockfill as indicated by the same borehole. The results of the analyses are given in Figures H3-1, H3-2, H3-3 and H3-4 in Appendix H3.

Figures H3-5, H3-6, H3-7 and H3-8 in Appendix H3 show the results of the stability analysis carried out with the anticipated 0.1 m grade raise (by asphalt topping). In this case, the lowest factor of safety obtained for drained or long-term condition drops to about 1.4 which is also acceptable. Based on this, we do not anticipate foundation instability problems due to the proposed grade raise of 0.10 m.

5.3.2 SETTLEMENT OF EMBANKMENTS

Our analysis, based on available borehole information, field and laboratory data, indicate that the proposed 0.10 m grade raise (using asphaltic concrete) will cause a settlement of about 15 to 18 mm. This figure includes both primary and secondary consolidation settlements. The primary settlement which is about 90% of this settlement should be completed within the next seven years. In addition, about 5 mm of settlement may occur within the next 10 years due to secondary consolidation caused by previous loadings (e.g. original embankment construction in or about 1976). Such settlements are not expected to cause pavement distress. It should be pointed out, as with the previous sections, in our analyses we have assumed that all the organic soils were stripped from beneath the entire width of the existing embankments in accordance with established MTO procedures, during their construction. If this is not the case, settlements would be greater.

Appendix G3

Proposed Profiles and Typical Cross-Section

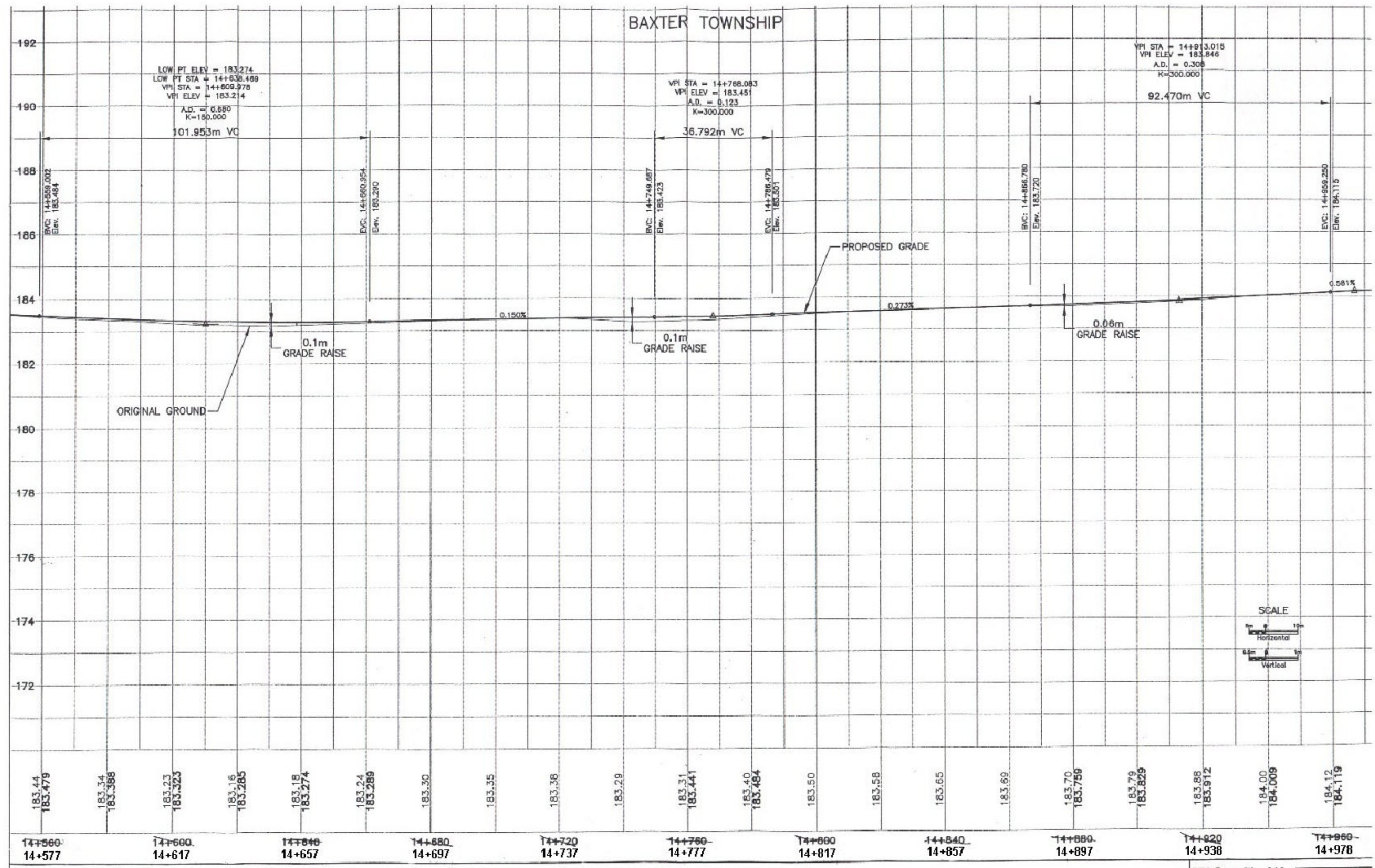


FIGURE G3-1

STATION BASED ON MEDIAN CL CHAINAGE

0.10 m GRADE RAISE

CL

2.50m
SHLD.

3.75m
LANE WIDTH

3.75m
LANE WIDTH

3.00m
SHLD.

1.00m
RND.

185

184

183

182

181

180

-10

0

10

185

184

183

182

181

180

14+647, NBL

BAXTER TWP

ES

AE

6 %

EP

2 %

2 %

EP

6 %

AE

ES

4:1

WIDENING/BENCHING ?

OG

OG

N.T.S.

Figure G3-2

Appendix H3

Slope Stability Analysis Results

Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H3-1

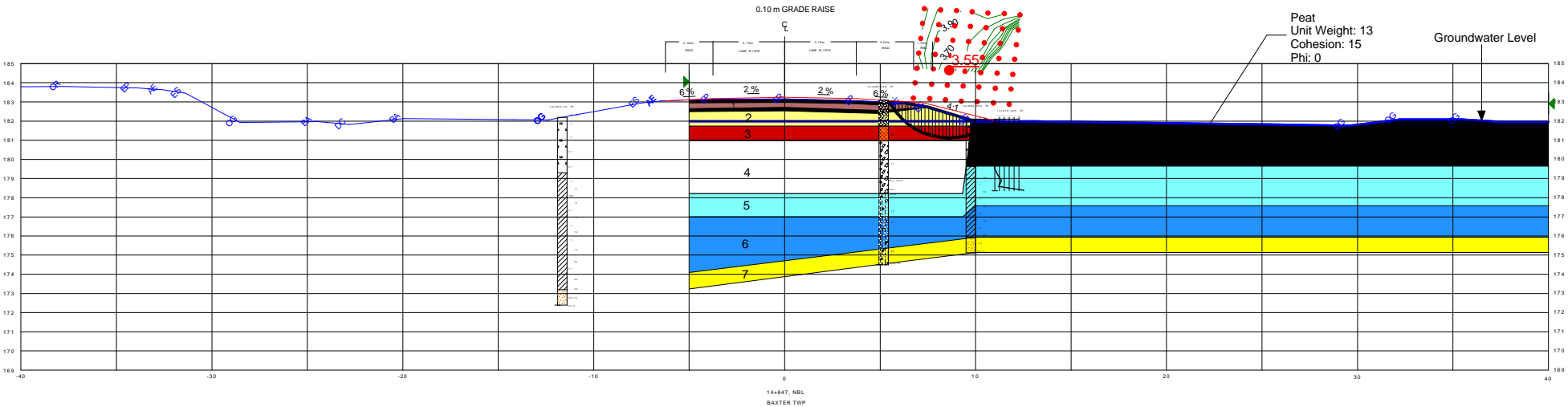
- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree

3 - Sandy Gravel
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 33 degree
- 4 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree

5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 8 kPa
Phi: 0 degree

6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 14 kPa
Phi: 0 degree
- 7 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree



Station Based on Median CL Chainage

Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H3-2

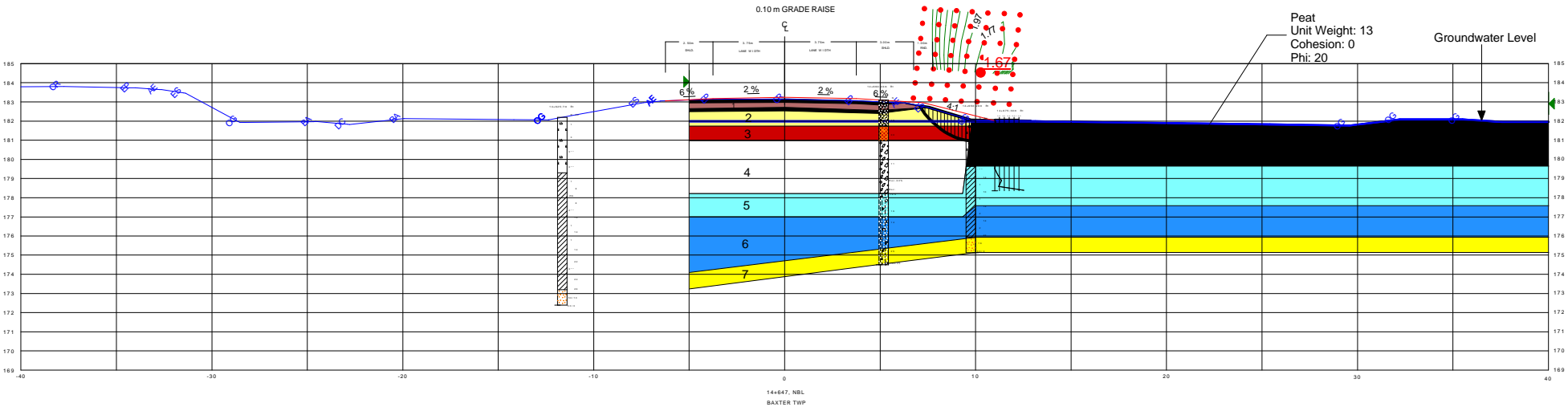
- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree

3 - Sandy Gravel
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 33 degree
- 4 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree

5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree

6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree
- 7 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree

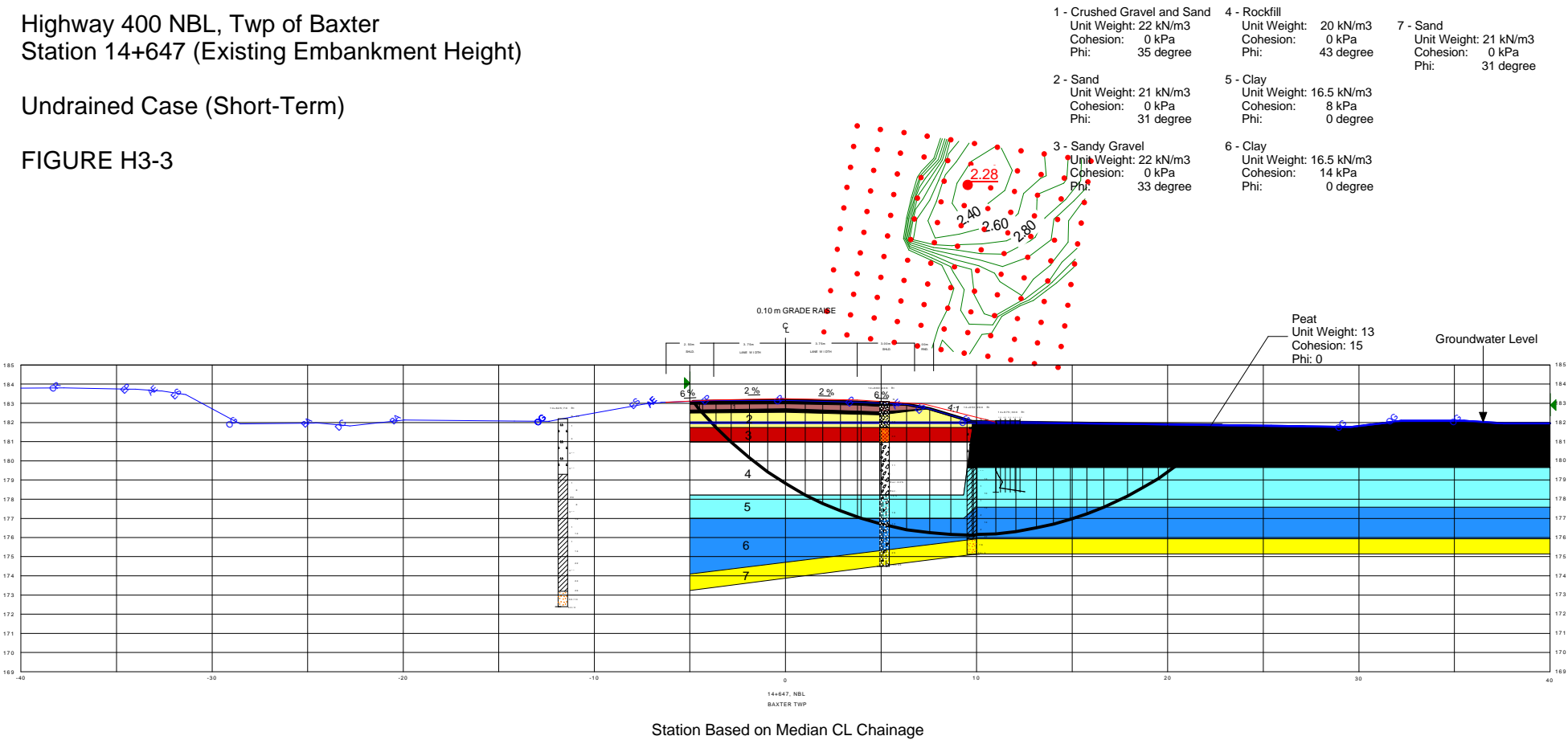


Station Based on Median CL Chainage

Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height)

Undrained Case (Short-Term)

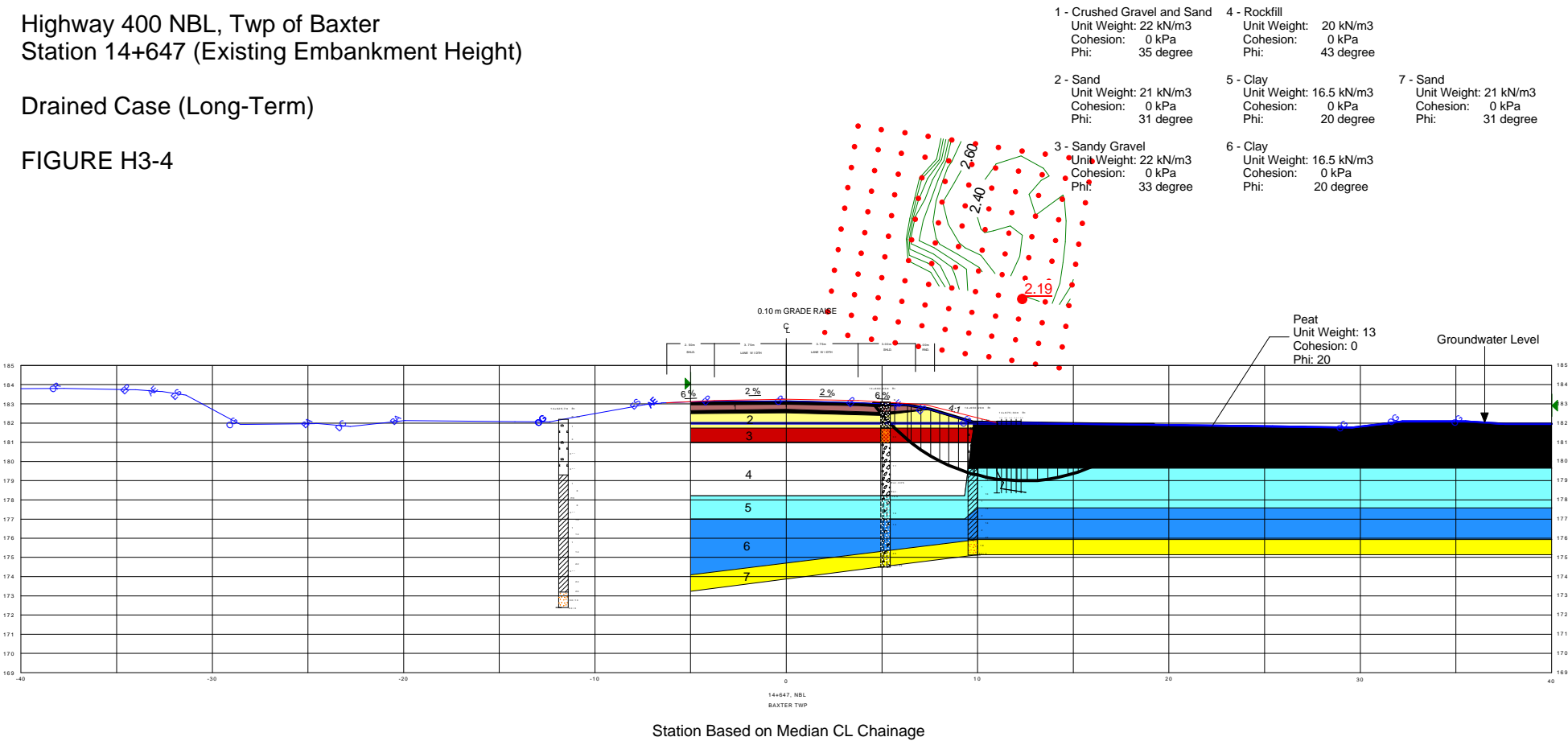
FIGURE H3-3



Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H3-4



Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height + Widening + 0.1 m Grade Raise)

Undrained Case (Short-Term)

FIGURE H3-5

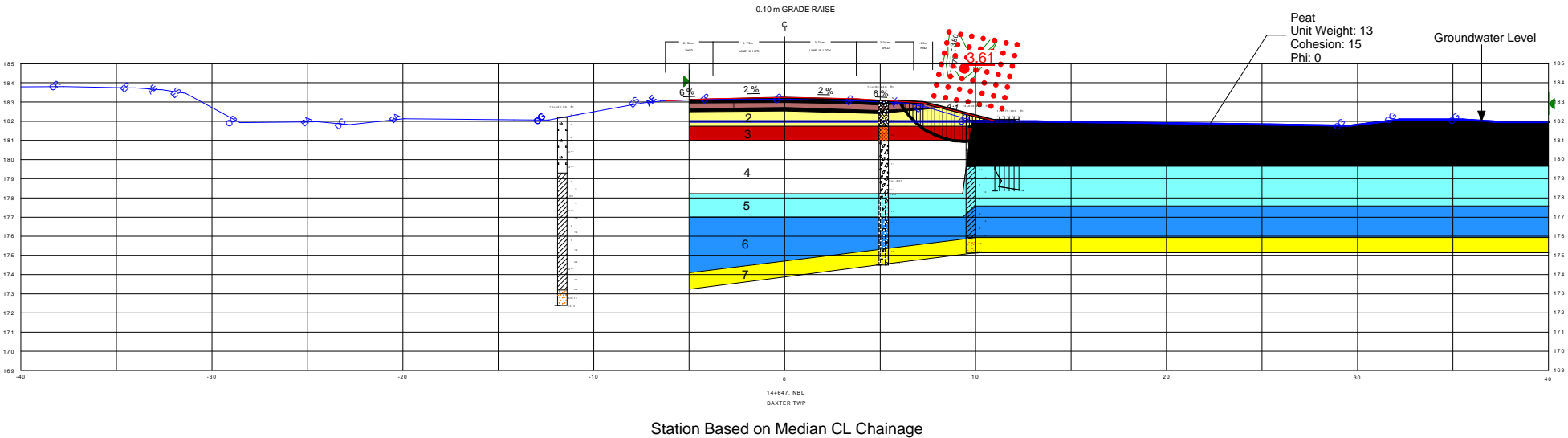
- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree

3 - Sandy Gravel
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 33 degree
- 4 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree

5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 8 kPa
Phi: 0 degree

6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 14 kPa
Phi: 0 degree
- 7 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree



Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height + Widening + 0.1 m Grade Raise)

Drained Case (Long-Term)

FIGURE H3-6

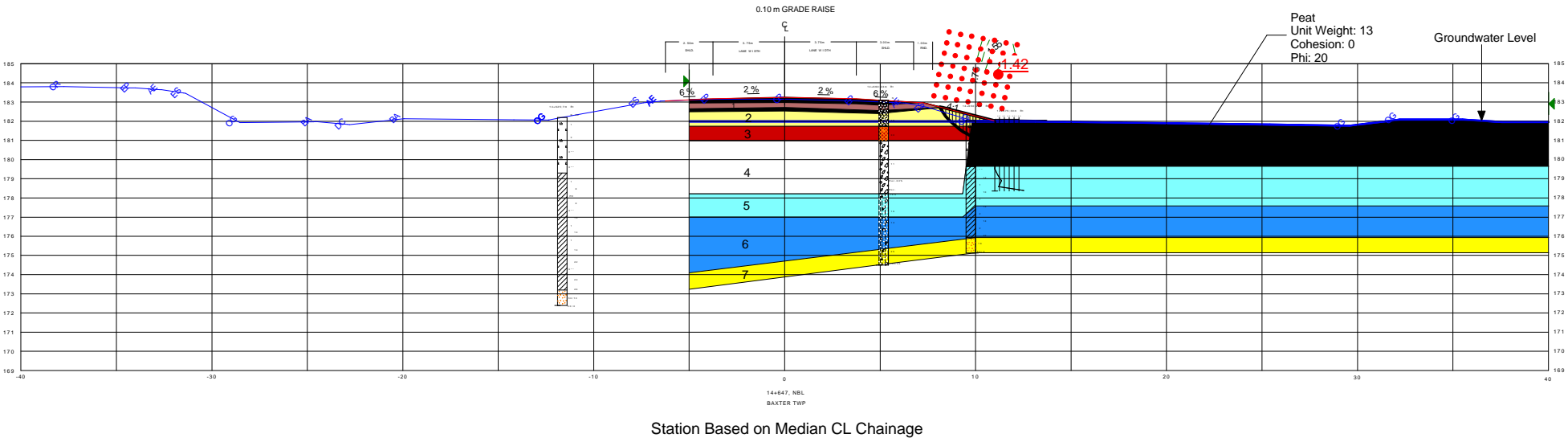
- 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree

2 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree

3 - Sandy Gravel
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 33 degree
- 4 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree

5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree

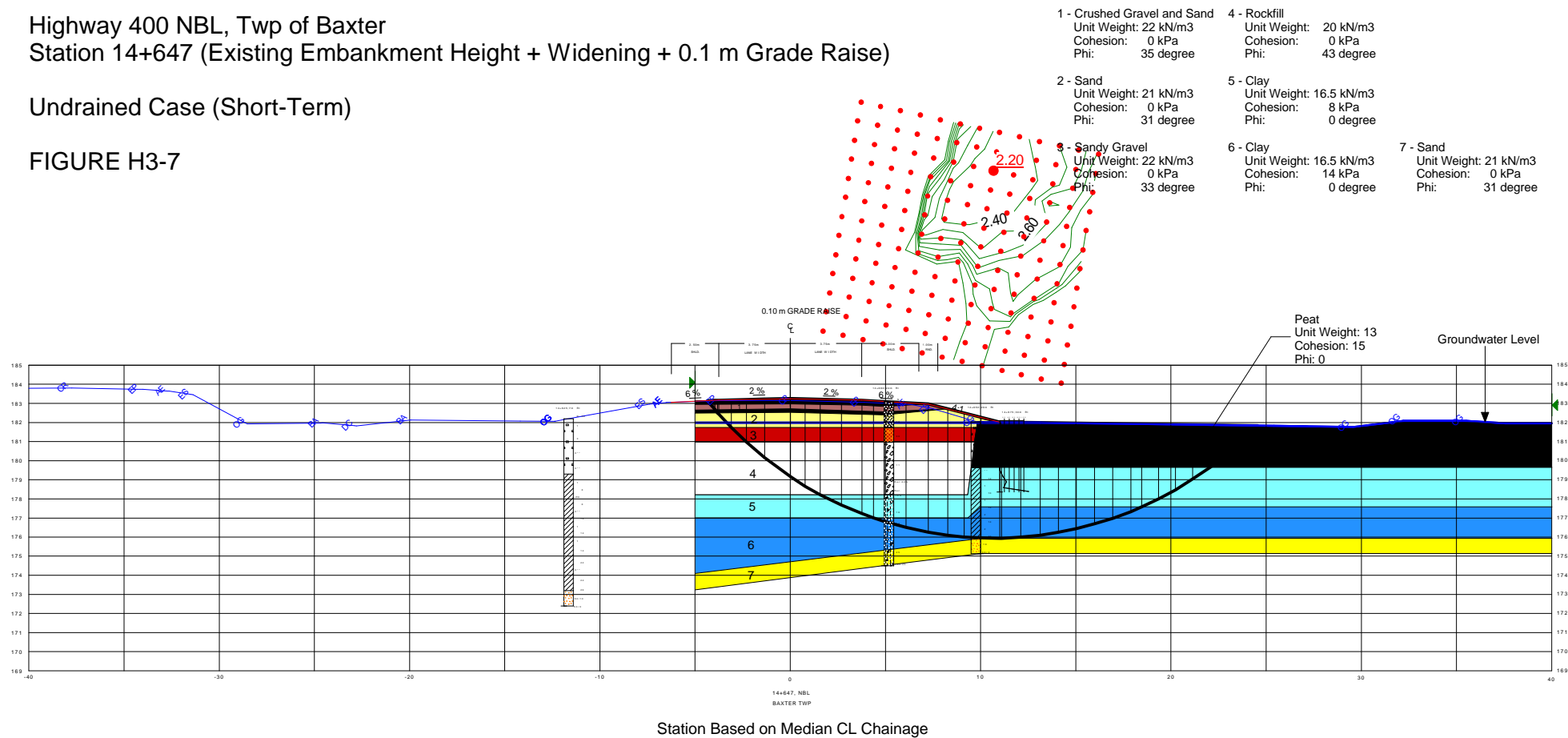
6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 20 degree
- 7 - Sand
Unit Weight: 21 kN/m3
Cohesion: 0 kPa
Phi: 31 degree



Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height + Widening + 0.1 m Grade Raise)

Undrained Case (Short-Term)

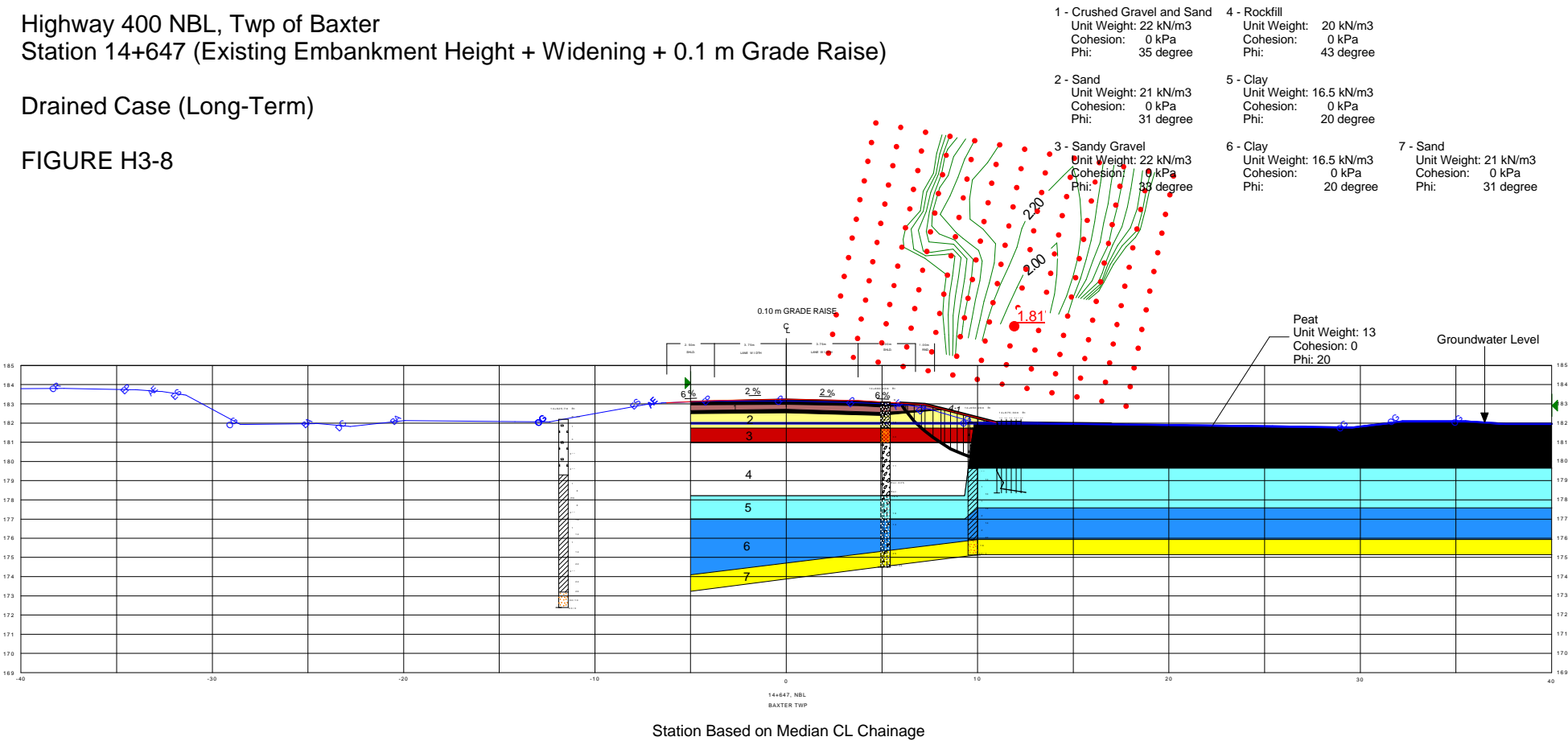
FIGURE H3-7



Highway 400 NBL, Twp of Baxter
Station 14+647 (Existing Embankment Height + Widening + 0.1 m Grade Raise)

Drained Case (Long-Term)

FIGURE H3-8



5.4 SITE 4 (STA. 14+875 TO 14+985)

The relatively short (110 m long) section of the highway between Stations 14+875 and 14+985, which is designated as Site 4 for the purposes of this project, has rock outcrops at both limits, with a water logged swampy area in between. Drainage is provided by CSP under the embankment at about Station 14+948, as shown on the Borehole Location Plan, Drawing No. 4 (included in the Foundation Investigation section of this report).

In this section, the height of the embankment (NBL of the highway) is typically 2 m near the south end at about Station 14+880, increasing to about 3 m towards the middle of the site at about Station 14+930, decreasing rather sharply further north to about 1.2 m at Station 14+940, with a gradual decrease to grade from thereon towards a rock cut at about Station 14+960.

The boreholes contacted up to a maximum of about 9 m overburden to refusal on the augers and the sampler. In the swampy area in the middle portion of the site, typically the sequence of the soil strata consists of organic soils (i.e. peat and organic clayey silt) at surface with a maximum thickness of 3.4 m, underlain by clay to silty clay which is in turn underlain by a basal sand to silty sand overlying a bowl-shaped or more likely a conical-shaped bottom (i.e. the depth of overburden appears to increase in both north-south and east-west directions).

5.4.1 EMBANKMENT STABILITY

As was mentioned before, in this section, the embankment heights reach a maximum of 3.0 m at about Station 14+930 but are typically about 2 m high. The presently proposed profile is given in Figure G4-1 in Appendix G4. We understand little or no adjustments are to be made to the highway where the embankment is about 3 m (i.e. highest) while some adjustments are proposed at and around Station 14+888. A typical cross-section showing these adjustments is presented in Figure G4-2. This shows an up to 0.075 m grade raise by increasing the asphalt thickness along with an up to about 0.3 m widening at the right shoulder. For this reason, we carried out stability analyses at Station 14+888.

As with previous sections, for our analysis we assumed that all the organic soils have been removed from beneath the footprint of the existing embankments, as per normal MTO procedures and as evidenced by the findings of Boreholes 14+917 (24 m Rt) and 14+950 (25 m Rt) which were drilled from the top of the embankment. Analyses were carried out, as before, by the limit equilibrium method, using the computer program Slope/W and Bishop's Simplified method. The results of stability analyses are shown in Appendix H4.

Figures H4-1 and H4-2 show the existing conditions at Station 14+888 and Figures H4-3 and H4-4 show the grade raise and possible modifications on the existing embankment (as shown in Figure G4-2).

The calculations indicate that the factor of safety against a failure when the embankment was first built at this particular station (based on the closest borehole findings to the station) was probably of the order of 3.6 and 2.4 for the short (undrained) and long-term (drained) conditions, respectively, and that when modifications are implemented, the safety factors are still about the same. Based on this, no instability is predicted at this station, due to the proposed grade raise and widening of the right (east shoulder).

5.4.2 SETTLEMENT OF EMBANKMENTS

Our analysis, based on the borehole and laboratory test data, shows that a consolidation settlement of the order of up to 0.4 m is likely to have occurred for embankment heights of the order of 2.0 m, due to the consolidation of the compressible clay to silty clay deposit underlying the site. As at this site, the deposit is relatively thin (i.e. maximum recorded thickness of 3.2 m) the primary portion of these settlements would have been completed while some minor amounts of secondary consolidation settlement can be expected within the next ten years (i.e. several millimeters of settlement within this time-frame). The additional stresses imposed due to the grade raise (using asphaltic concrete) by about 0.075 m are likely to cause a settlement of the order of 10 mm in the next ten-year period. Such settlements are not expected to cause a pavement distress.

If the right shoulder is widened by about 0.3 m in the manner shown in Figure G4-2 (Appendix G4), this will likely cause a settlement of the order of 20 to 25 mm near the right shoulder area. Such settlements may cause some minor cracking of the asphalt pavement along the right shoulder area, as the settlements will present themselves as differential settlements. For this reason, the widening should be kept to a minimum. It should also be pointed out that in our analyses, we assumed that all organic soils were removed from under the entire width of the embankment, including the toe of the embankment. If this is not the case, higher settlements may occur.

Appendix G4

Proposed Profiles and Typical Cross-Section

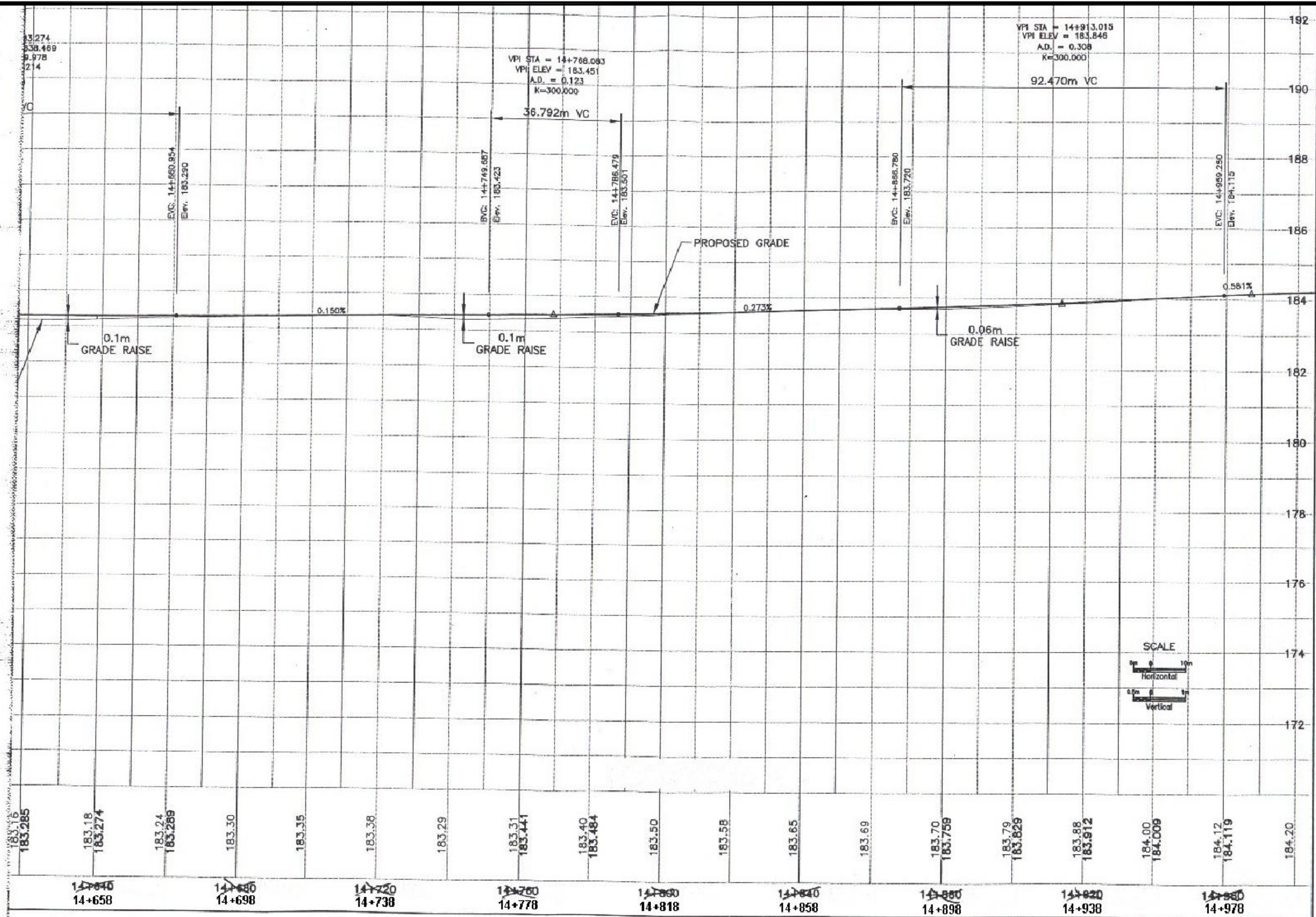
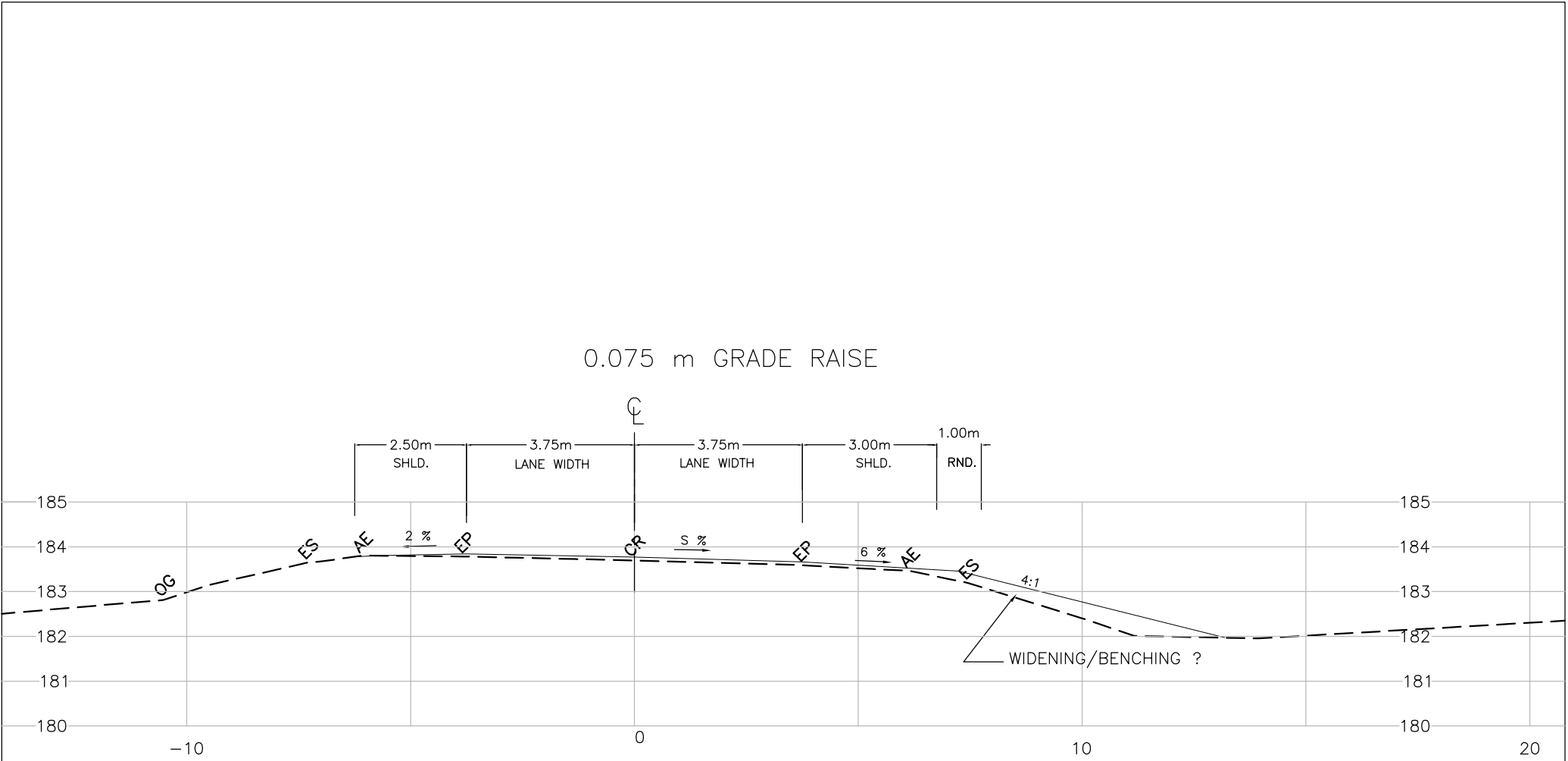


FIGURE G4-1



14+888, NBL
BAXTER TWP

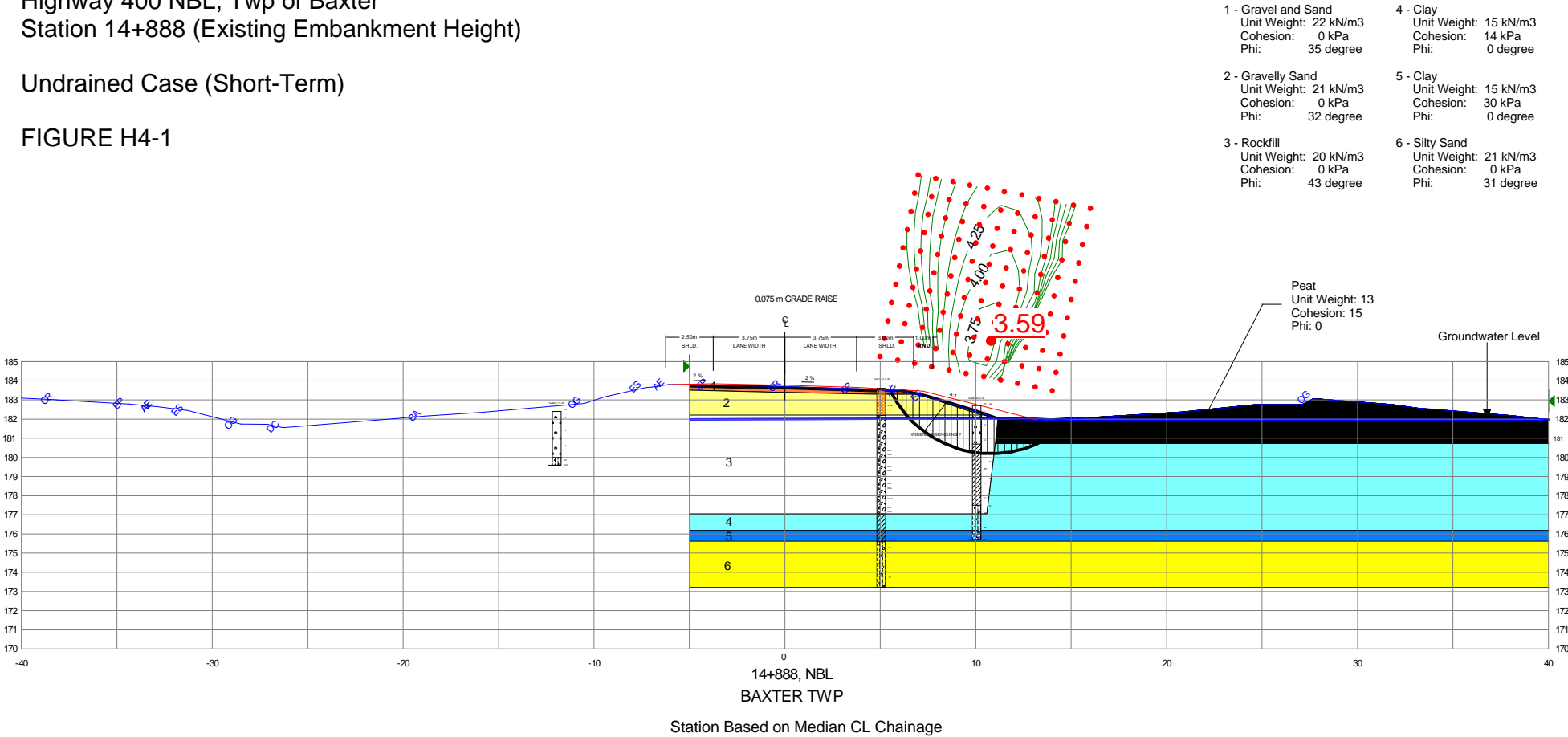
Appendix H4

Slope Stability Analysis Results

Highway 400 NBL, Twp of Baxter
Station 14+888 (Existing Embankment Height)

Undrained Case (Short-Term)

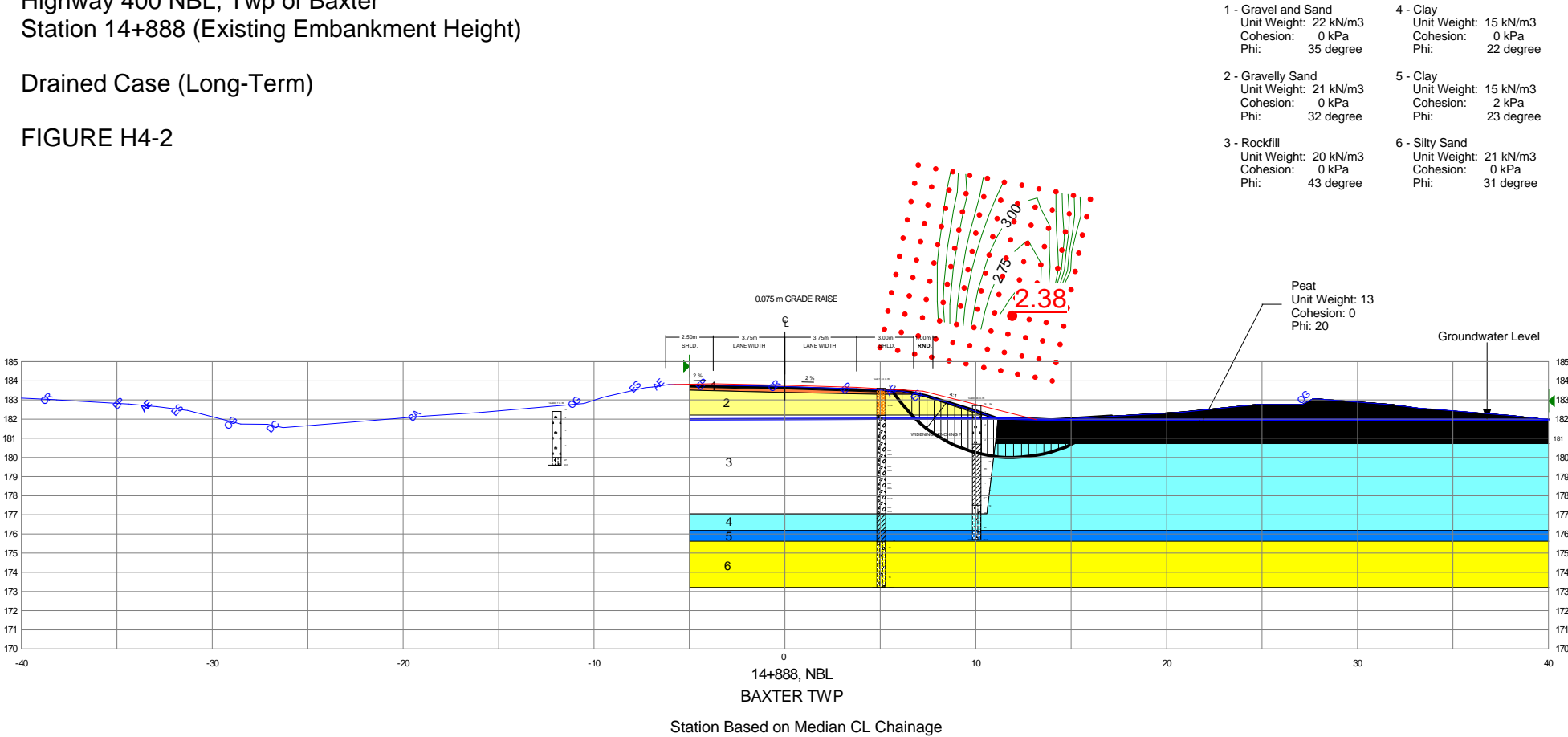
FIGURE H4-1



Highway 400 NBL, Twp of Baxter
Station 14+888 (Existing Embankment Height)

Drained Case (Long-Term)

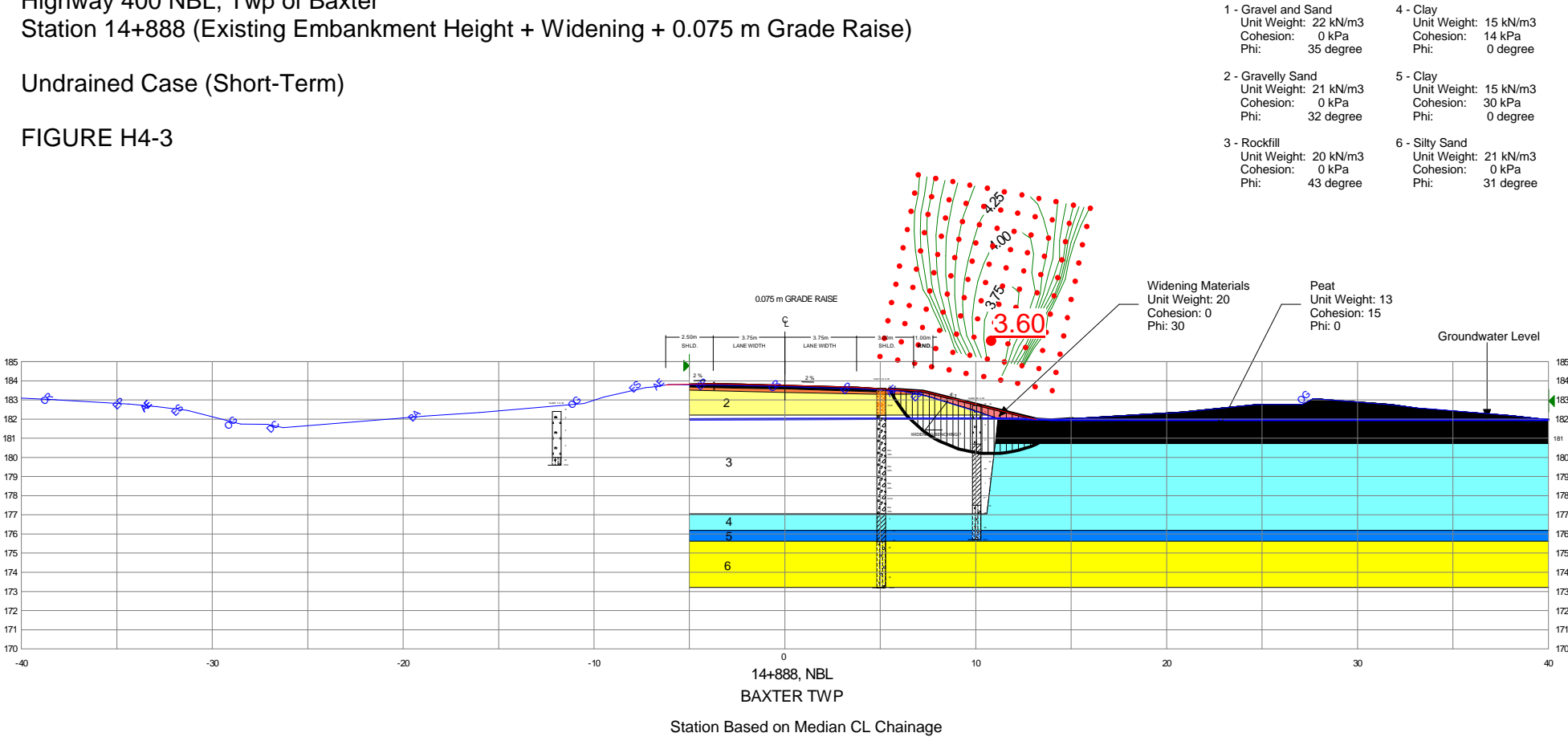
FIGURE H4-2



Highway 400 NBL, Twp of Baxter
Station 14+888 (Existing Embankment Height + Widening + 0.075 m Grade Raise)

Undrained Case (Short-Term)

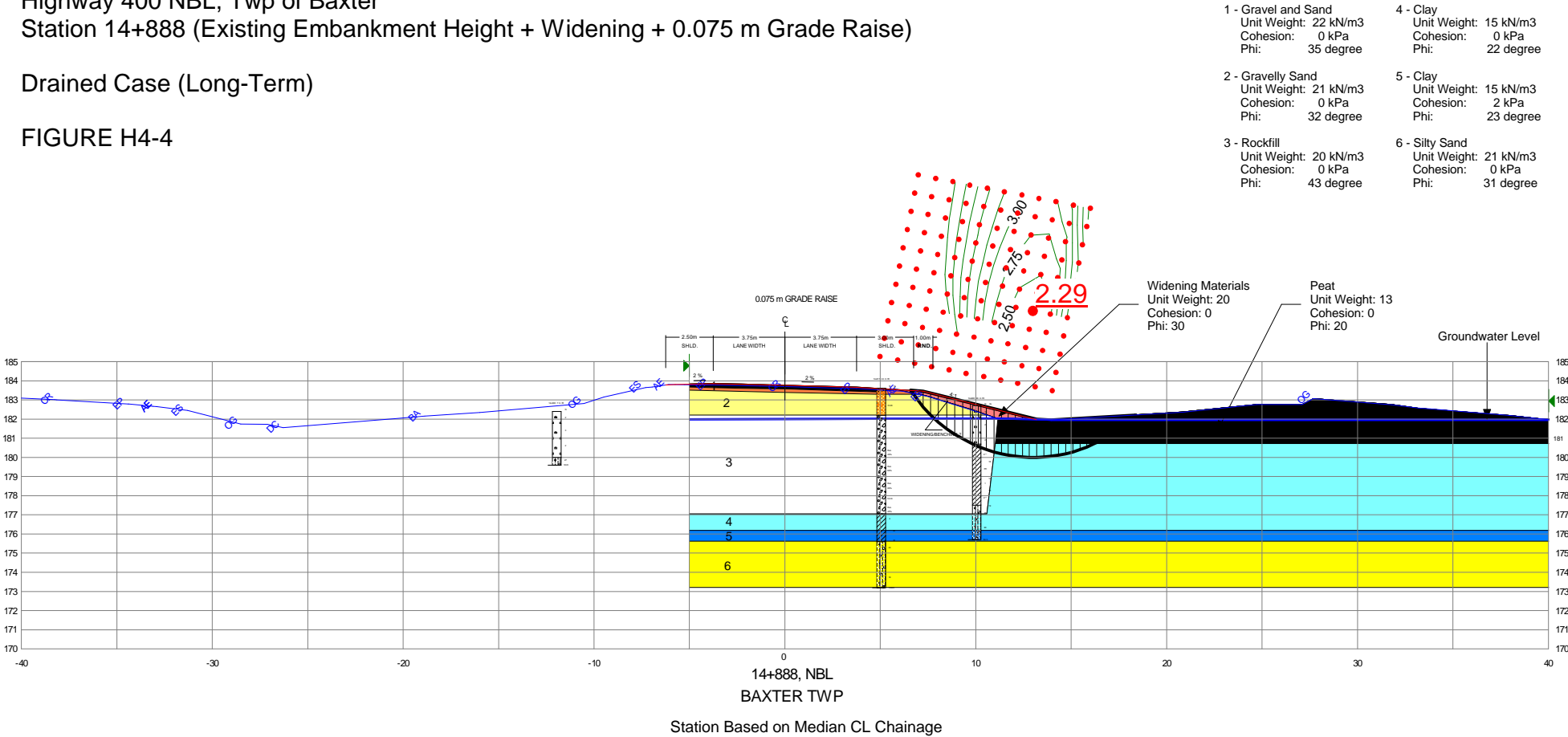
FIGURE H4-3



Highway 400 NBL, Twp of Baxter
Station 14+888 (Existing Embankment Height + Widening + 0.075 m Grade Raise)

Drained Case (Long-Term)

FIGURE H4-4



5.5 SITE 5 (STA. 15+300 TO 15+475)

The 175 m long section of the NBL of Hwy 400 between Stations 15+300 and 15+475 is characterized by rock outcrops near the south and north extremities, with a water logged swampy section in between, similar to previously discussed sites to the south of this section. A 23 m long 1.2 m diameter CSP under the embankment near Station 15+425 provides drainage, as shown on the Borehole Location Plan Drawing No. 5 (see Foundation Investigation section of the report).

In this section, the height of the embankment ranges from about 1 m to about 1.5 m between Stations 15+300 and 15+460. From thereon, the height of the embankment reduces to about 0.8 m at Station 15+470.

In general, the subsurface conditions in the boreholes located in the swampy areas present a similar picture as with the previous sites investigated to the south. The boreholes show in general the presence of between about 1 and 3 m of organic soil, underlain by a deposit of clay to silty clay with a thickness of 2.3 to 12.3 m. The clay to silty clay is weak and compressible and at most borehole locations, it is underlain by basal granular deposits ranging from silt to gravelly sand.

The road surface profile provided to us indicate that in some sections grade raises of up to 0.1 to 0.2 m are possible (see Figure G5-1 in Appendix G5). A cross-section provided at Station 15+430 (presented in Figure G5-2) shows a 0.17 m grade raise on the left side of the existing embankment gradually decreasing to zero towards the right shoulder. Some minor modifications are proposed on the upper portion of the left slope. No grade raise but an approximately 1 m widening is proposed on the right shoulder.

5.5.1 EMBANKMENT STABILITY

Stability analyses were performed at Station 15+430 which was provided to us. For this purpose, as before, limit state analysis by the computer program Slope/W using Bishop's simplified method was employed. The analyses were based on assumed and/or measured soil parameters along with a subsurface profile provided by the nearest boreholes. In the analysis, also as before, all the organic soils were assumed to have been stripped from beneath the footprint of the existing embankment and that the embankment consists of rock fill (as evidenced by Borehole 15+370; 24 m Rt)

The results of the analyses are given in Appendix H5. Figures H5-1, H5-2, H5-3 and H5-4 show the results of the stability analysis which provides probable safety factors when the embankment was first built, while Figures H5-5 through H5-8 show the results of widening and grade raise at both left and right sides of the embankment. These results show a minimum factor of safety of the order of 2.0 which is acceptable.

5.5.2 SETTLEMENT OF EMBANKMENTS

It is understood that in this section, the grades will be raised by up to about 0.20 m. A typical section provided to us (i.e. Section at Station 15+430, as shown in Figure G5-2) shows a 0.17 m grade raise. Some minor modifications are proposed at the left shoulder along with an approximately 1.0 m widening at the right shoulder.

Our settlement analysis, based on the findings of the boreholes and the consolidation tests carried out in the laboratory, indicate that depending on the thickness of the clay to silty clay deposit, a typical embankment, which was 2.0 m high when it was first constructed, would have settled by between 0.3 and 0.5 m since its construction some 18 years ago. Where the clay to silty clay deposit is relatively thick (e.g. 15 m thick), the primary settlement would still be on-going and an additional settlement of up to 40 mm can occur within the next ten years. In addition, a secondary settlement of the order of 5 mm is likely to occur during the same period. In other areas where the clay to silty clay deposit is less than about 10 to 12 m thick, the primary settlement would be essentially completed but some minor secondary settlements would be on-going. Some settlements may also be occurring if modifications were made to the embankment some time after its construction.

A grade raise of about 0.2 m (which is proposed) is likely to cause a further settlement of about 40 mm during a ten-year period after construction (i.e. after the grade raise). For the section given, Figure G5-2 where the grade raise is 0.17 m but decreases to zero from the left shoulder towards the right shoulder, the settlement should not exceed 30 mm near the left shoulder, decreasing towards the right shoulder. A further settlement of about one-third of the figures given is likely to occur during the following (second) ten-year period.

The widening proposed along the right shoulder (see Figure G5-2) is likely to cause a settlement of about 40 mm at the outer edge of the right shoulder (after construction configuration) after a ten-year period with another 15 mm within the following ten-year period thereafter. Such settlements due to their differential nature may cause some cracking of the asphalt along the paved shoulder surface.

It should be pointed out that similar to previous sites discussed in Sections 5.1 through 5.4 in this report, and as was mentioned in Section 5.5.1, in our analysis we have assumed that all the organic soils were removed from under the footprint of the existing embankment during its construction and that the embankment consists of rock fill.

5.5.3 CONSTRUCTION

The widening of the embankment along the east (right) shoulder at or around Station 15+430, should be carried out in accordance with OPSS and established MTO practices, including benching, where applicable. MTO practice also dictates that the organic soils under the widened section of the embankment toe be removed and replaced with suitable soils. In order to prevent instability of the existing embankment, this process should be carried out in short sections (i.e. about 3 m wide sections perpendicular to the length of the embankment). Excavation and backfilling should be carried out concurrently and under water. Excavations may be carried out by backhoe or dredging, although the use of backhoe may prove to be feasible and more economical. Below the groundwater table, granular soils need to be used. In this instance, since the existing embankment appears to be comprised of rock fill, the use of rockfill will likely be more suitable. Rock fill placed below the groundwater table may be end-dumped. The rockfill should be placed according to OPSS 206.07.08 and MTO practice. By means of good construction practices, damage to the existing culvert should be avoided.

Appendix G5

Proposed Profiles and Typical Cross-Section

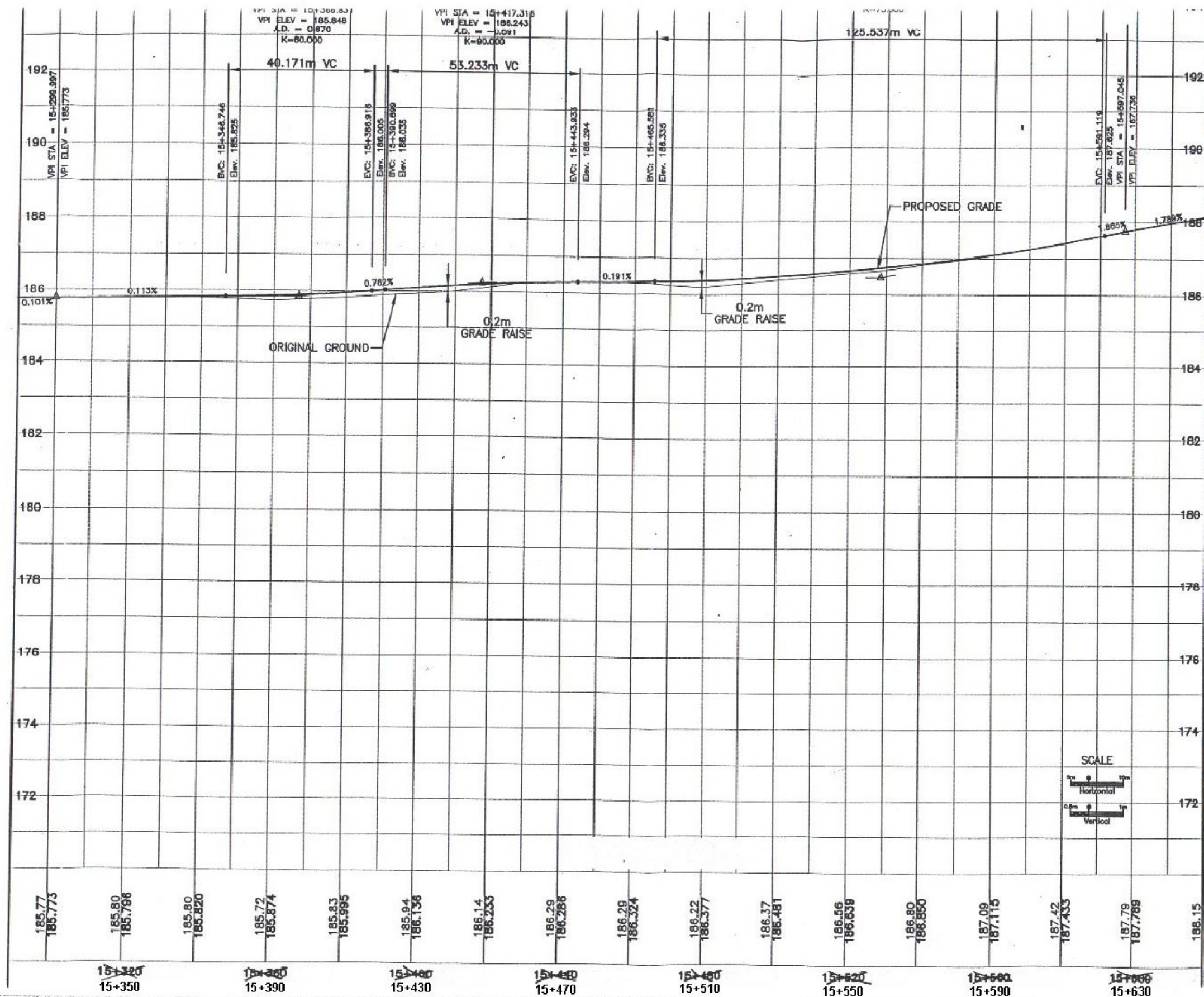
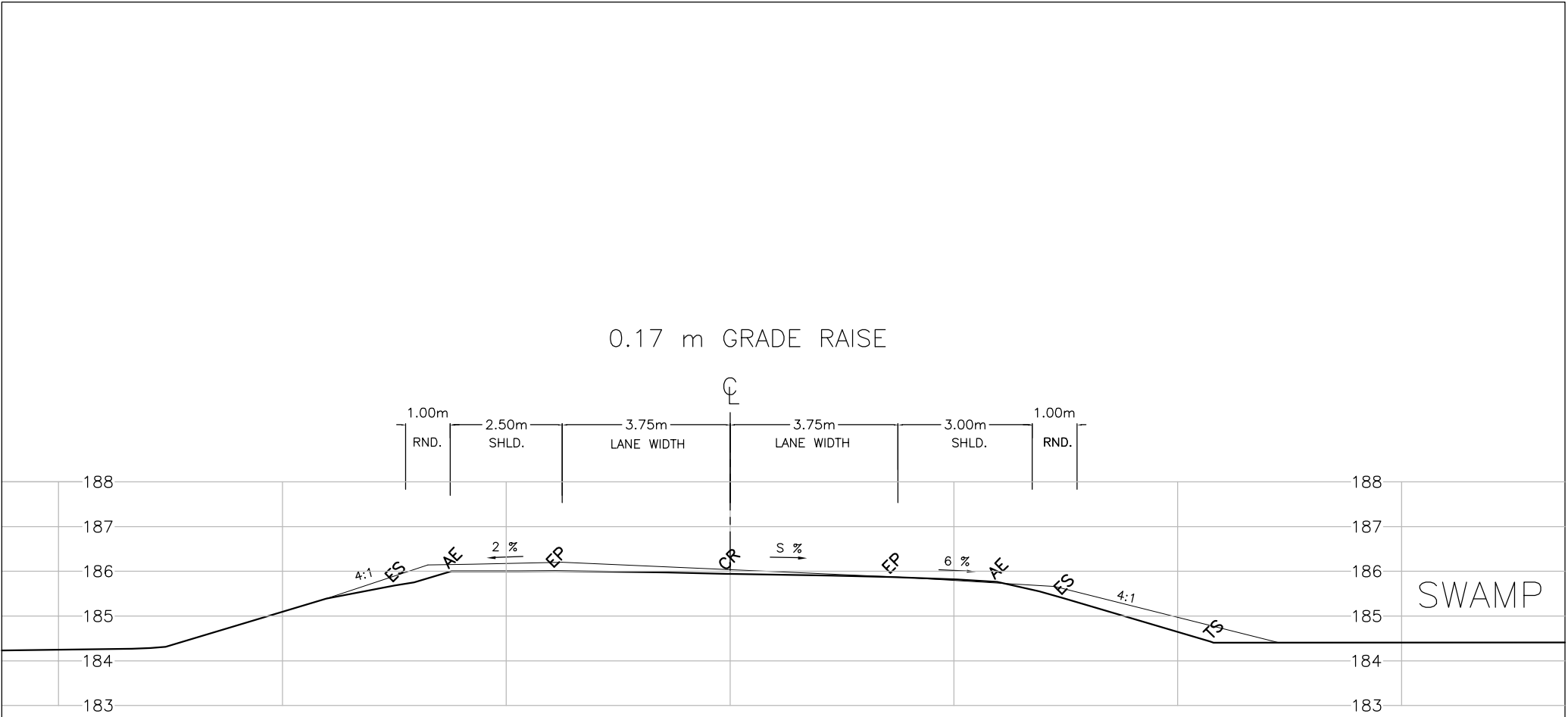


FIGURE G5-1



15+430, NBL
BAXTER TWP

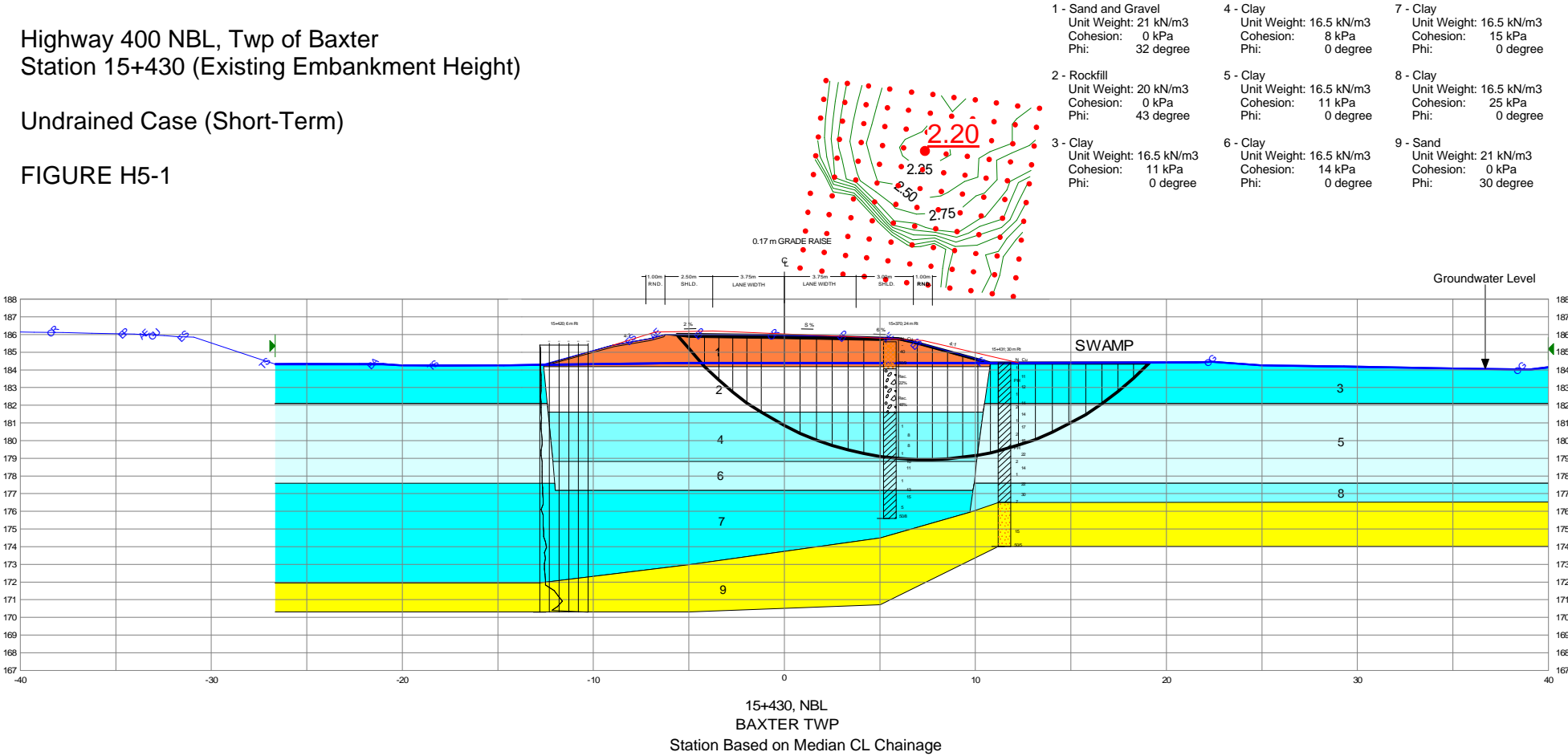
Appendix H5

Slope Stability Analysis Results

Highway 400 NBL, Twp of Baxter
Station 15+430 (Existing Embankment Height)

Undrained Case (Short-Term)

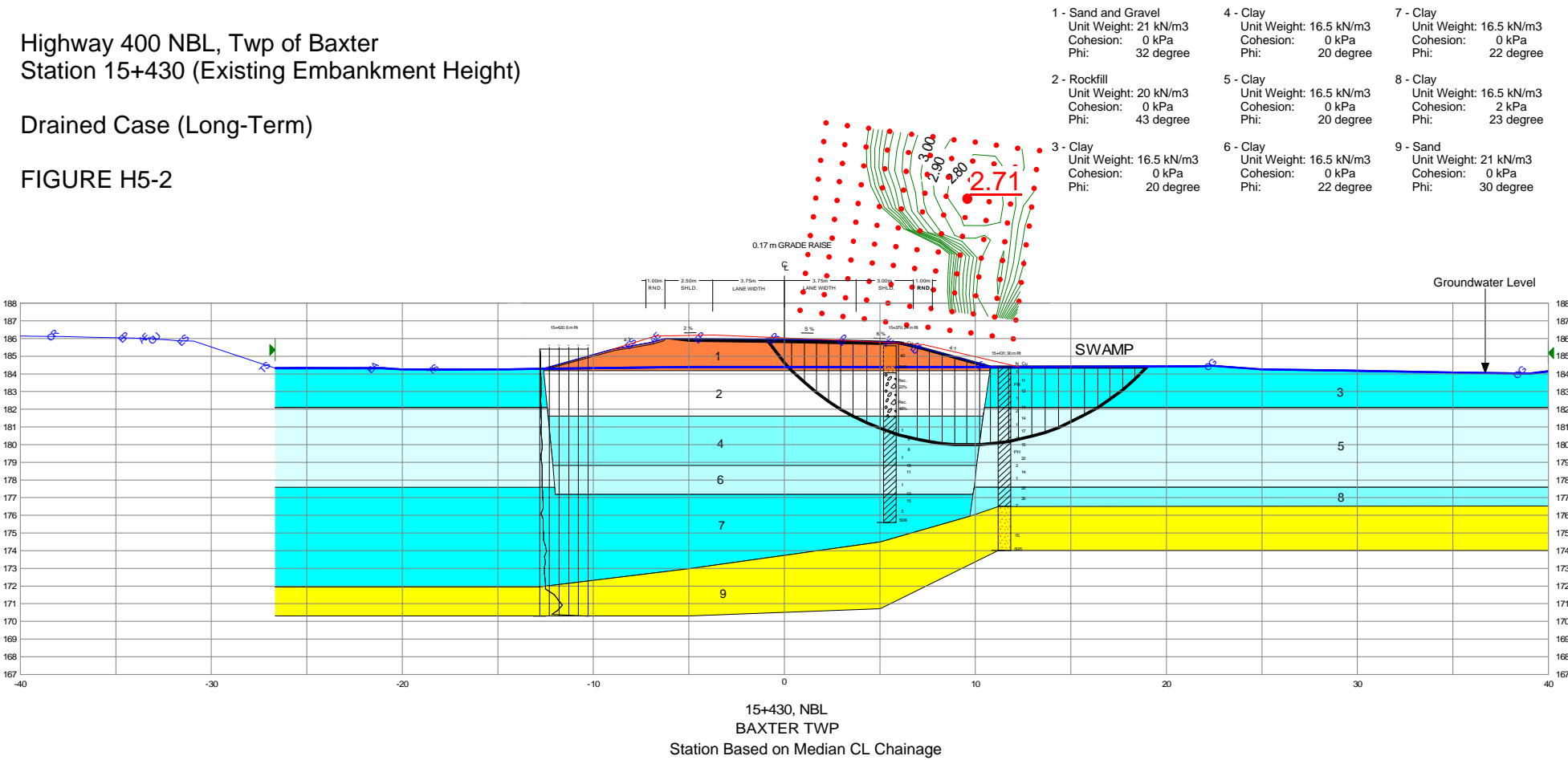
FIGURE H5-1



Highway 400 NBL, Twp of Baxter
Station 15+430 (Existing Embankment Height)

Drained Case (Long-Term)

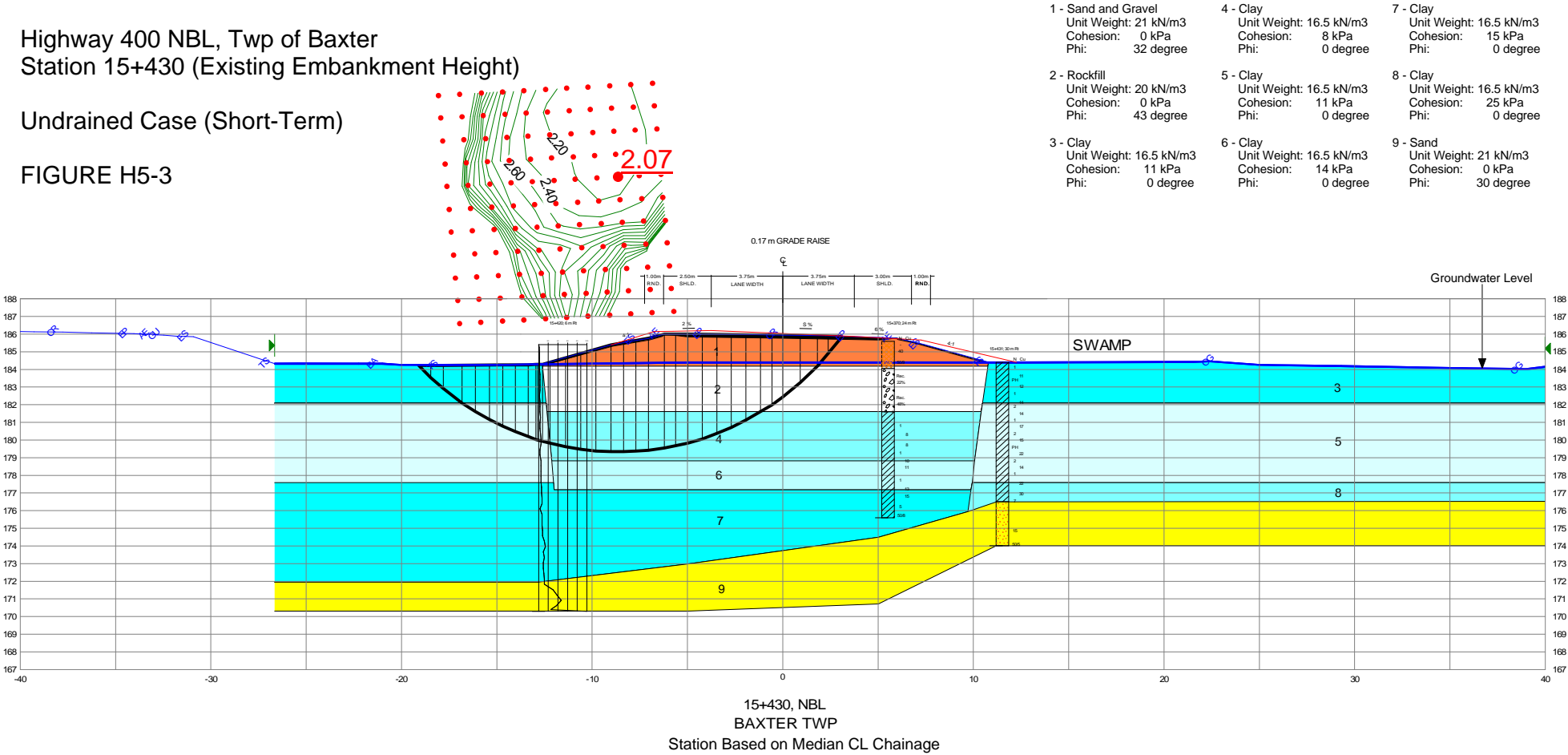
FIGURE H5-2



Highway 400 NBL, Twp of Baxter
Station 15+430 (Existing Embankment Height)

Undrained Case (Short-Term)

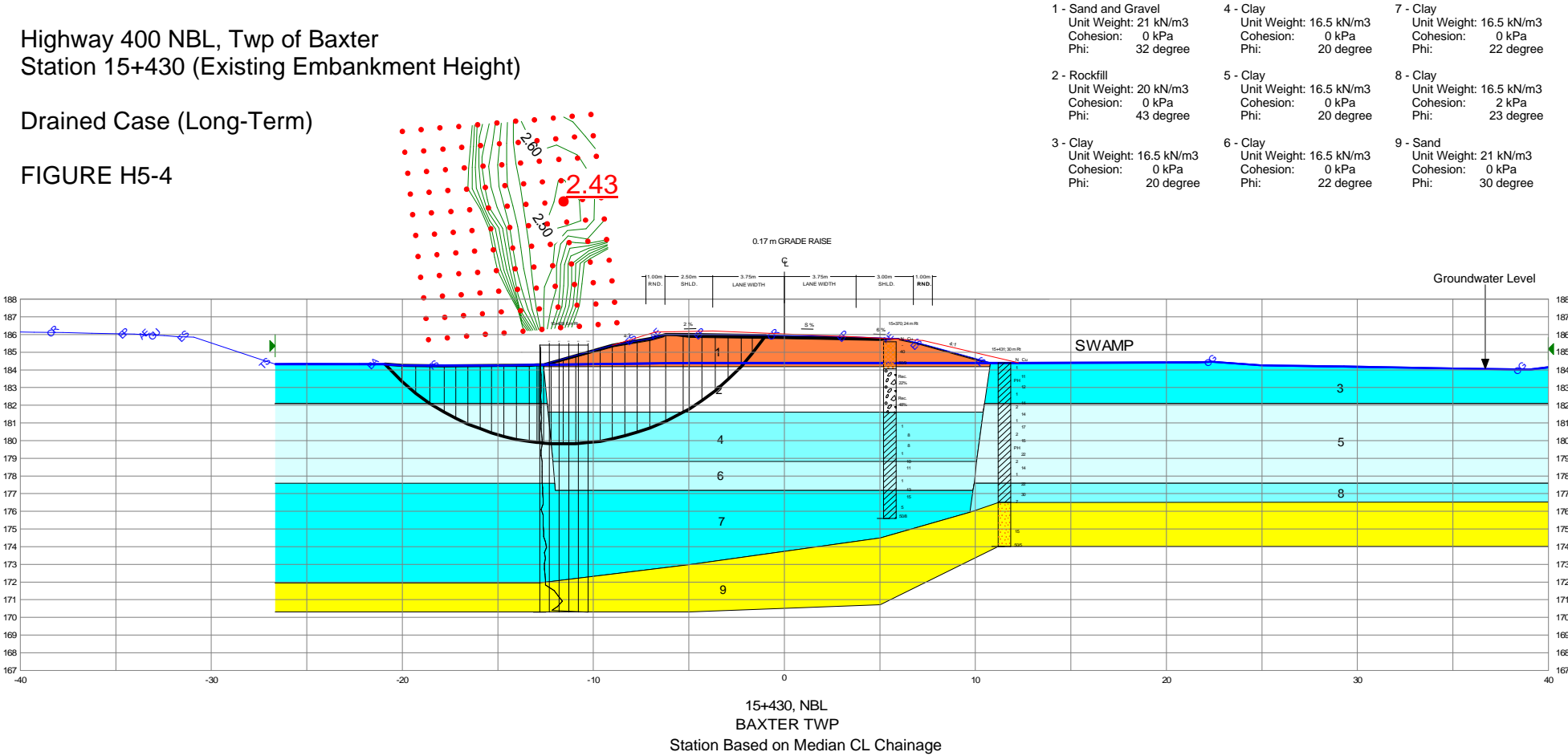
FIGURE H5-3



Highway 400 NBL, Twp of Baxter
Station 15+430 (Existing Embankment Height)

Drained Case (Long-Term)

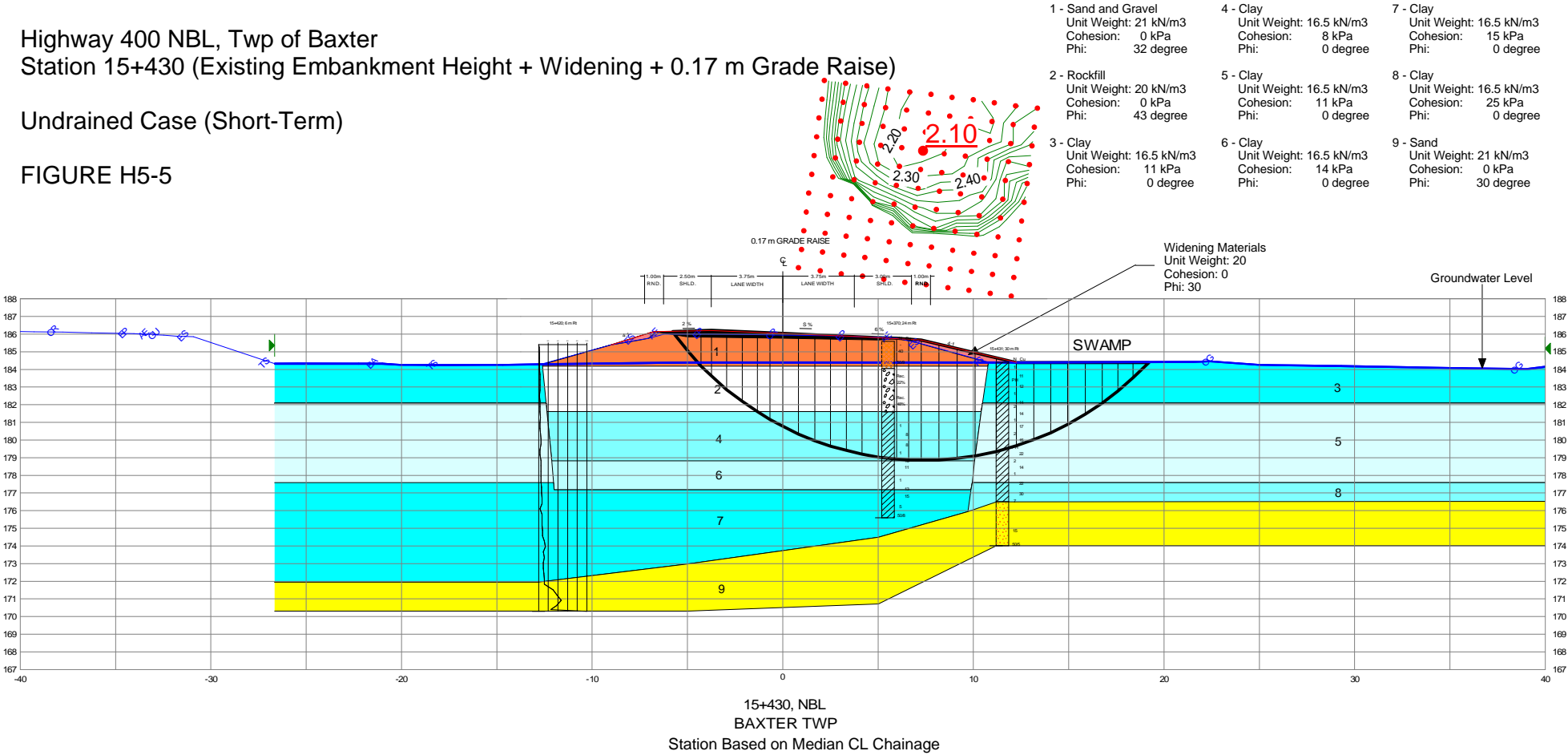
FIGURE H5-4



Highway 400 NBL, Twp of Baxter
Station 15+430 (Existing Embankment Height + Widening + 0.17 m Grade Raise)

Undrained Case (Short-Term)

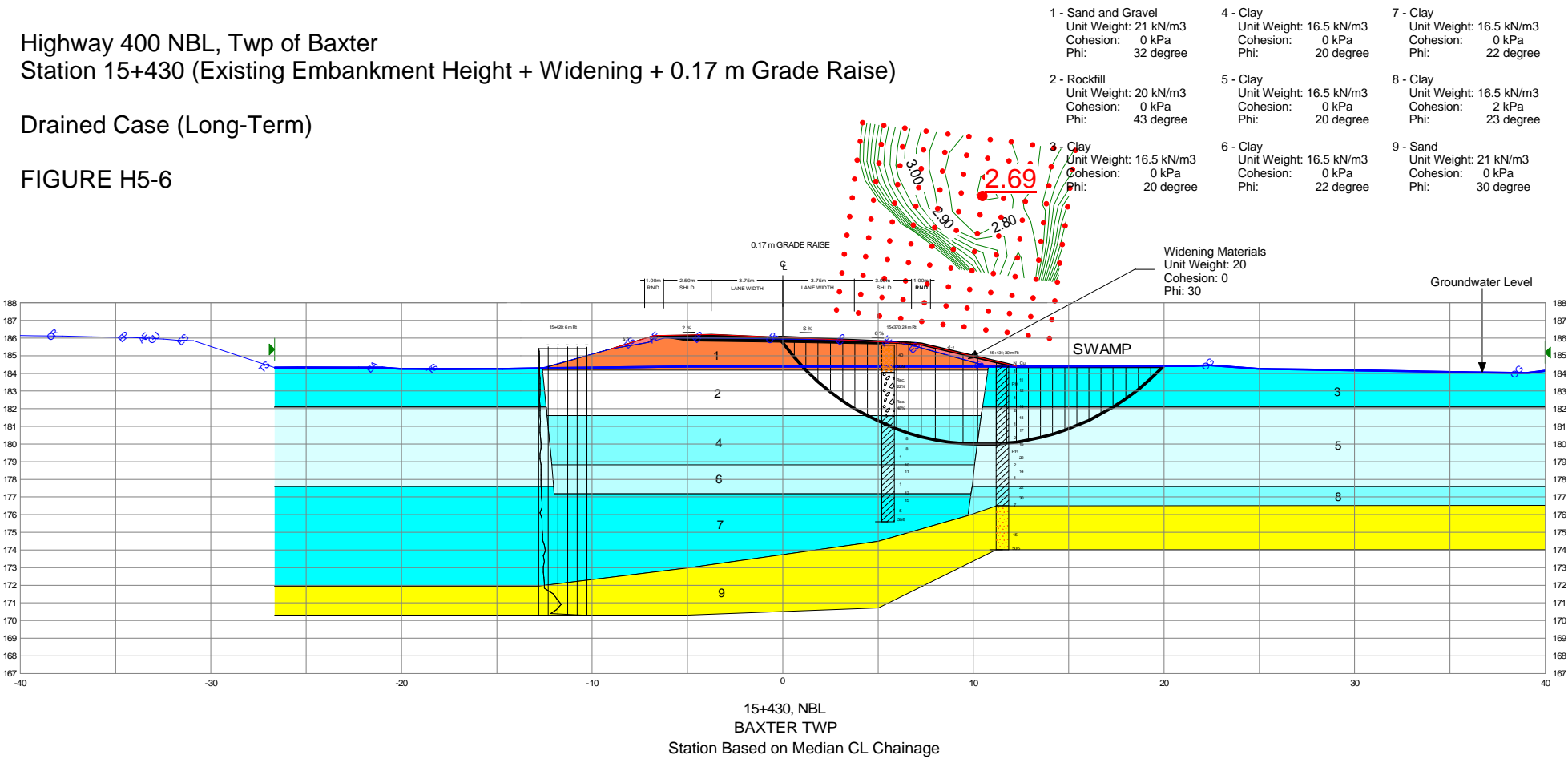
FIGURE H5-5



Highway 400 NBL, Twp of Baxter
Station 15+430 (Existing Embankment Height + Widening + 0.17 m Grade Raise)

Drained Case (Long-Term)

FIGURE H5-6



Highway 400 NBL, Twp of Baxter
Station 15+430 (Existing Embankment Height + Widening + 0.17 m Grade Raise)

Undrained Case (Short-Term)

FIGURE H5-7

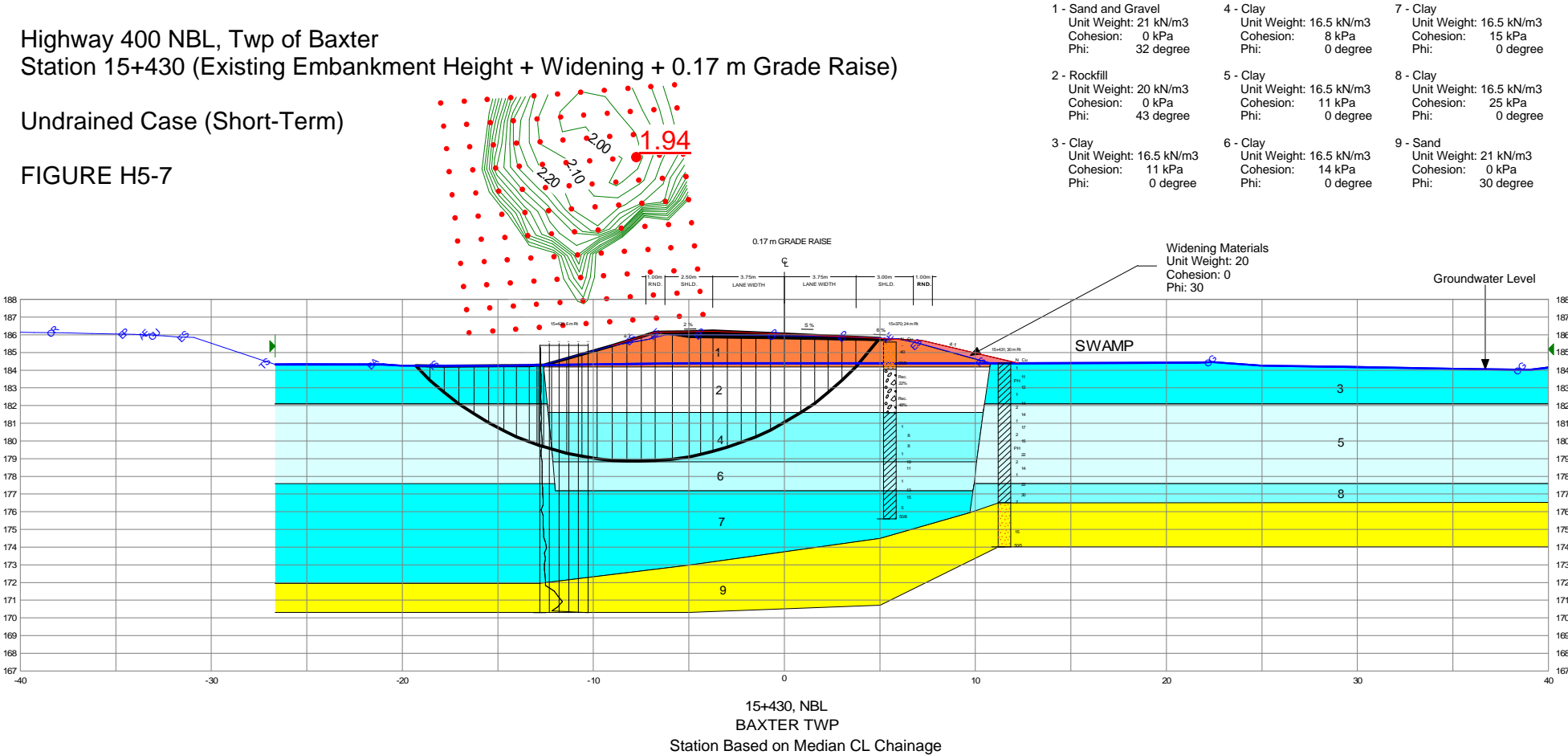
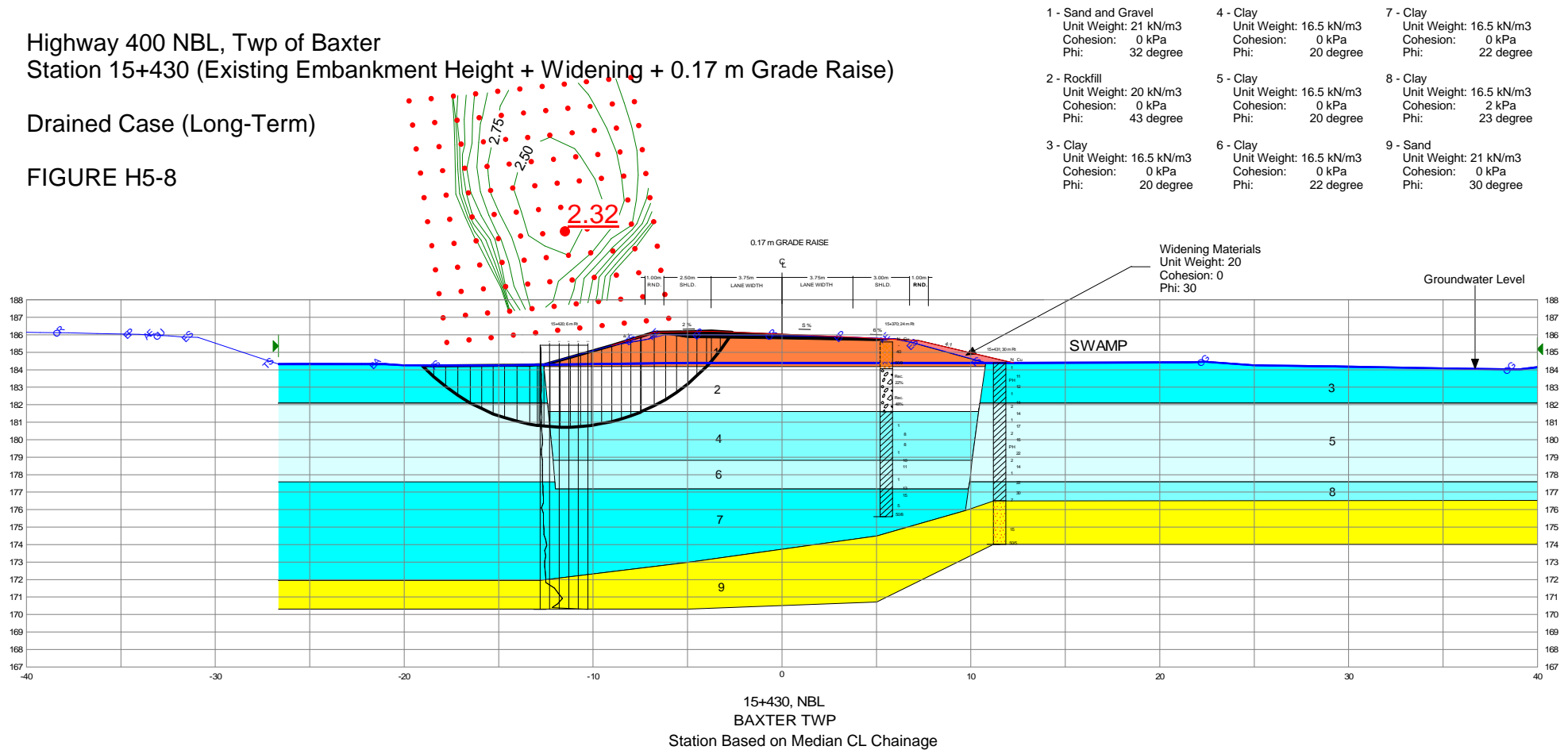


FIGURE H5-8



5.6 SITE 6 (STA. 11+750 TO 12+010-GIBSON TOWNSHIP)

The section designated as Site 6 is the most northerly section dealt within this report. It is located between Stations 11+750 and 12+010 in Gibson Township. The southern tip of this section is characterized by rock outcrops on both sides of the NBL. To the north of the outcrops is a low-lying swampy area. In this area, embankment heights are typically 6 to 7 m. This section of the site is drained by means of a 1219 x 34.39 CSP and the flow is from east to west. Further north, rock outcrops are again visible on the right (east) side within a stretch of nearly 100 m, where the o.g. is relatively higher and embankment height along the east (right) shoulder are typically at grade or less than 2 m in height. On the left hand side (i.e. left or west shoulder), the embankment heights remain high as the o.g. generally slopes down from east to west.

On the right side, another low lying swampy area is prevalent beyond (north of) the rock outcrops. This second swampy area on the east side is drained via a 1400 mm x 36.87 m concrete pipe culvert under the embankment (see Drawing No. 6 of the Foundation Investigation portion of this report). Further north, the o.g. rises towards rock outcrops on both sides of the embankment. In this north half of the site, the embankment reaches a maximum height of about 6.4 m on the LHS and 5.5 m on the RHS.

In summary, the site is characterized by two distinct swampy areas in between rock outcrops on the RHS. These swampy areas are less distinct on the LHS (i.e. nearly one continuous swamp).

The results of the boreholes and the DCPT show that within the southerly swampy area the maximum depths to refusal are about 5 m on the east side of the NBL of the embankment and about 12 m of the west side. Further north, within the second swampy area the maximum depths to refusal (i.e. inferred overburden depths) are 2.4 m and 5.3 m on the east and west sides of the embankment, respectively. Previously drilled MTO boreholes which are located further west (i.e. west of the median centerline) indicate greater overburden depths (of up to about 9.5 m) west of the centerline. In the swampy areas, the subsurface conditions encountered in the boreholes generally consist of surficial organic soils (maximum recorded depth of 3.8 m) underlain by a cohesive soil deposit of clay to clayey silt (typical recorded thickness of between 2.5 to about 6 m) which in some of the boreholes is in turn underlain by a basal granular deposit.

Both surficial organic soils and the underlying clay to clayey silt deposit are considered to be weak and highly compressible materials.

The vertical design profile provided to us (see Figure G6-1) indicate a vertical grade raise which starts at about Station 11+750 increasing to about 0.2 m between Stations 11+780 and 11+800 and gradually decreasing to zero at about Station 11+860. Another grade raise

of up to about 0.08 m is planned between Stations 11+920 and 12+000. Both grade raises will likely be implemented by asphalt surfacing.

5.6.1 EMBANKMENT STABILITY

Similar to previous sections, we performed a stability analyses even though it is unlikely that grade raises of the orders quoted will cause any instability. Another purpose of the stability analyses was to determine whether the weak clayey soils underlying the embankment are likely to have undergone large lateral (i.e. plastic) movements due to stresses caused by the embankments when they were first constructed.

All the analyses were carried out by means of limit state equilibrium (Bishop's simplified method) utilizing the computer program Slope/W.

A cross-section at Station 11+800 was provided to us shown in Figure G6-2. This section is located in the area where the highest grade raise is proposed. Stability analysis was therefore performed at this section, using the subsurface conditions provided by the nearest boreholes. Measured and assumed soil parameters were used for the analysis as shown in Appendix H6. We have assumed that all the organic soils were removed from underneath the footprint of the road embankment (see Record of Borehole 11+812; 23.5 m Rt). Based on the same evidence, as well as the previous sections of the highway NBL findings, we assumed that the embankment was constructed of rock fill. The results of Borehole 11+812 (23.5 m Rt) lead us to believe that either the clay was removed or the rockfill sank into the weak clay deposit by several meters to El. 181.5 m (or a combination of these phenomena) and this aspect was taken into consideration in preparing the subsurface profile for our analyses. As the subsurface conditions present a complex picture (due to the fact that the thickness of the overburden and especially of the weak clay to clayey silt deposit change rapidly not only in the north-south direction along the length of the embankment, but also in the east-west direction perpendicular to the roadway), it is very difficult to present an accurate subsurface profile. The results of our slope stability analyses, based on an assumed subsurface profile from the borehole and DCPT data, are presented in Appendix H6.

Figures H6-1 through H6-4 show a minimum calculated safety factor of 1.59 for the right (east) side of the embankment while the calculated safety factor for the left (west) side is 1.17 (see Figures H6-5 and H6-6). This lower value is due to the fact that the thickness of the weak clay increases from east to west. This low value was obtained for the short-term (undrained) conditions. Obviously, this lower than normally accepted value does not pose any threat to the existing embankment since the embankment has withstood the test of time without any apparent or known failure and also since the calculated long-term stability factors are within a satisfactory range. In addition, a possible increase in the soil strength due to embankment loading towards the west shoulder area of the embankment was only

considered to partial depths and for this reason, the actual safety factor may be somewhat higher. On the other hand, we assumed (as evidenced by the findings of Borehole 11+812; 23.5 m Rt) that all the organic soils were removed and the underlying clay was removed to El. 181.5 m. The significance of the low safety factor which was calculated is nevertheless as follows:

- a) unless the original embankment was stage-constructed and/or the weak clay was removed to greater depths, some plastic yield of the soil is likely to have occurred, in addition to normal consolidation (vertical) settlements, and
- b) any further grade raises may trigger a deep-seated foundation slope failure.

Figures H6-7 through H6-10 show the analyses results on the right (east) side of the embankment when the anticipated 0.21 m grade raise is considered. In this case, the minimum calculated factor of safety is 1.56 (Figure H6-8) which is acceptable.

Figures H6-11 and H6-12 present the results of the analyses for the left side of the embankment and in this case, the anticipated grade raise of 0.21 m reduces the minimum calculated safety factor from 1.17 to 1.10. As was discussed before, this safety factor is marginally low and may pose a slight risk of failure which may not be acceptable to MTO. Our technical personnel visually inspected the pavement and the side slopes for any apparent signs of impending instability. The visual examination did not reveal any obvious signs of instability (see photographs I6-1 and I6-2 in Appendix I6), except for signs of excessive settlement (see photograph I6-10 in Appendix I6). If the risk of a minor risk of instability is unacceptable to MTO, then a berm maybe considered in the area of proposed grade raise (i.e. between Stations 11+760 and 11+840) or any grade raise in this area may be kept to a minimum.

Figures H6-13 through H6-20 show the results of the stability analyses for the portion of the embankment towards the north end of Site 6. On the RHS of the embankment, the borehole results indicate relatively shallower overburden depths and in addition the height of the embankment on this side (i.e. east side of the embankment) is relatively shallower since the o.g. is relatively higher. The calculated minimum factor of safety on this RHS is in excess of 2 (see Figures H6-13 through H6-16, Appendix H6).

On the LHS, only one borehole and one DCPT results are available. Using these information, together with the results of MTO Boreholes BH1-5, BH1-6 and BH1-7, an analysis was carried out assuming all the organic soils were removed from beneath the rock fill embankment during the original construction. These show a safety factor of 1.13 for long-term (drained) condition and 1.23 for short-term (undrained) condition (Figures H6-17 and H6-18). Since the embankment has been in place for a long period of time, in this case, only the calculated long-term safety factor is of concern. While this low safety factor

was obtained assumed soil parameters and assumed subsurface conditions, nevertheless caution is required with any grade raises. The calculated safety factor with the proposed 0.08 m grade raise drops to 1.12 for the long-term and to 1.21 for the short-term conditions (Figures H6-19 and H6-20).

The visual inspection carried out by our technical staff along this section shows some longitudinal cracking (mainly separation and minor amounts of settlement) in the asphalt pavement along the middle of the left shoulder from the culvert location southerly by about 25 m (see photographs I6-3 through I6-8, Appendix I6). Some separation cracking is also evident at the very edge of the embankment shoulder, as can be seen in the same photographs. These may be signs of possible impending instability, especially since our stability calculations show safety factors only marginally higher than 1.0. For this reason, any further increase in grade in this area should, in our opinion, only be implemented after the construction of a toe berm unless some risk of a foundation failure would be acceptable to MTO. Stability analysis results show that a 6.0 m wide toe berm constructed to El. 186.5 m increases the calculated long-term factor of safety against a failure from 1.12 to 1.34, as shown in Figure H6-21 in Appendix H6. It is suggested that the toe berm be constructed in the area of apparent cracking (i.e. starting from about Station 11+930 northerly to as close to the existing culvert at about Station 11+960, as practically and hydrologically feasible). The berm should be constructed prior to the implementation of the proposed grade raise.

Normally, MTO procedures require that all the organic materials be removed from the bottom of any berms constructed. (This is because if future widening occurs, the procedure will ensure that no organic soils will be left underneath the embankment.) The removal of organics should be carried out in very short sections perpendicular to the longitudinal direction along the length of the highway to prevent a possible slope and/or foundation failure during the excavation (i.e. no wider than about 3 m widths). Excavations may likely be carried out by a backhoe. Excavation and backfilling of each short section must be carried out concurrently and underwater. Granular soils must be used below the groundwater table. Since the existing embankments consist of rock fill, use of rock fill is preferred. Rock fill placed below the groundwater table may be end dumped. The rock fill should be placed according to OPSS 206.07.08 and MTO practice. Damage to existing culvert should be prevented. As well, any hydrological impact of the construction of a berm very close to the existing culvert may need to be considered.

5.6.2 SETTLEMENT OF EMBANKMENTS

A laboratory consolidation test was carried out on a sample recovered from Borehole 11+820 (34 m Rt), while two laboratory consolidation test results are available from the MTO work which was carried out in 1993 for the construction of SBL (see Appendix B6-1 and Appendix B6-2 of the Foundation Investigation section of this report).

A settlement analysis using the results of the consolidation test carried out for this investigation indicates that where the clay to clayey silt deposit is thickest in the southerly swamp section (i.e. between Stations 11+750 and 11+820) an originally 7.0 m high embankment may have settled by about 0.7 m. If MTO consolidation test results are used, settlements in excess of 1.0 m are predicted. In our opinion, settlements of up to 0.6 to 0.7 m are likely have occurred in an area where the embankments are high and the clay to clayey silt deposit is thick (i.e. between Stations 11+760 and 11+810), unless all or some of the clay deposit was removed. Photograph I6-10 shows undulating pavement condition at about Station 11+800 which leads us to believe that settlements are likely to have occurred. These settlements would likely have been completed within about eight years of the construction of the embankments but some minor secondary consolidation would still be occurring. In addition, stability calculations indicate a low factor of safety and unless staging construction was implemented when the embankments were first constructed, some lateral movements of the soft to very soft clay to clayey silt soils would likely have occurred.

Based on the available data, a grade raise of about 0.20 m (as proposed) will likely cause a settlement of the order of 20 mm which is expected to take place within the next six to eight years, along with secondary consolidations of about 10 mm from the previous loadings, bringing the total to about 30 mm. Such settlements are not expected to cause pavement distress, since differential settlements are expected to be minimal (e.g. while the left shoulder area can be expected to settle more than the right shoulder area because the thickness of the overburden appears to increase from right to left, but a sharp and sudden increase is not anticipated). In our opinion, the differential settlements are likely to be within the capacity of a flexible pavement to withstand such settlements without showing signs of significant distress.

It is however recommended that since due to low shear strength of the soil the stability of the embankment is only marginally on the safe side, the grade raises be kept to a minimum and if possible to less than 0.20 m, unless other measures were taken as was discussed before.

Further north, beyond Station 11+820 the thickness of the overburden (in particular, the clay to clayey silt deposit) at the borehole locations are less than the southern section discussed in the previous paragraphs. Therefore, the calculated settlements for this section are also less. The estimated settlements which are likely to have occurred to date under the weight

of the embankment range from about 0.2 to 0.4 m depending on the thickness of the clay to clayey silt deposit (i.e. 0.4 m settlement is likely to have occurred on the left side of the embankment between about Stations 11+880 and 11+930, while settlements on the right side would be less than this amount).

Settlements due to the proposed grade raise of about 0.08 m are likely to cause settlements of less than 10 mm. Again, the settlements would be negligible on the right side but may approach a maximum of 10 mm on the left side. Such settlements should not present a concern for the integrity of the pavement and should be completed within a period of about five years after construction. However, as discussed before, stability concerns may need to be addressed.

5.7 CONSTRUCTION CONSIDERATIONS

At the time of our investigation, the groundwater levels at all six sites were at or very close to the existing ground surface (o.g.) levels. This, together with the presence of peat and other organic soils, may present problems with access for construction equipment in the low lying areas.

As was discussed in the previous sections of this report, where there is any widening or berm construction required, all the organic soils will need to be removed and replaced with granular soils (most likely rock fill for this project) as per established MTO practice. Excavations and backfilling will need to be carried out in short sections to prevent instability of the existing embankments. The process of excavation and backfilling of each sufficiently narrow section should be carried out concurrently and under water. Rock fill placed below the groundwater table may be end dumped. Rock fill should be placed according to OPSS 206.07.08 and MTO practice. The materials used for the construction of the embankment should consist of approved, acceptable rock fill in accordance with OPSS 206 and its amendment(s).

The fact of the existing slope should be properly prepared for any widening and/or berm construction. Where required, benching should be implemented as per MTO procedures and in accordance with OPSD 208.01.

Proper erosion control measures should be implemented on the face of the newly constructed slopes, both during the construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

6. CLOSURE

The Limitations of Report, as quoted in Appendix J, are an integral part of this report.

SHAHEEN & PEAKER LIMITED

Z.S. Ozden, P.Eng.



ZO:tr/idrive

K. R. Peaker, Ph.D., P.Eng.



Appendix G6

Proposed Profiles and Typical Cross-Sections

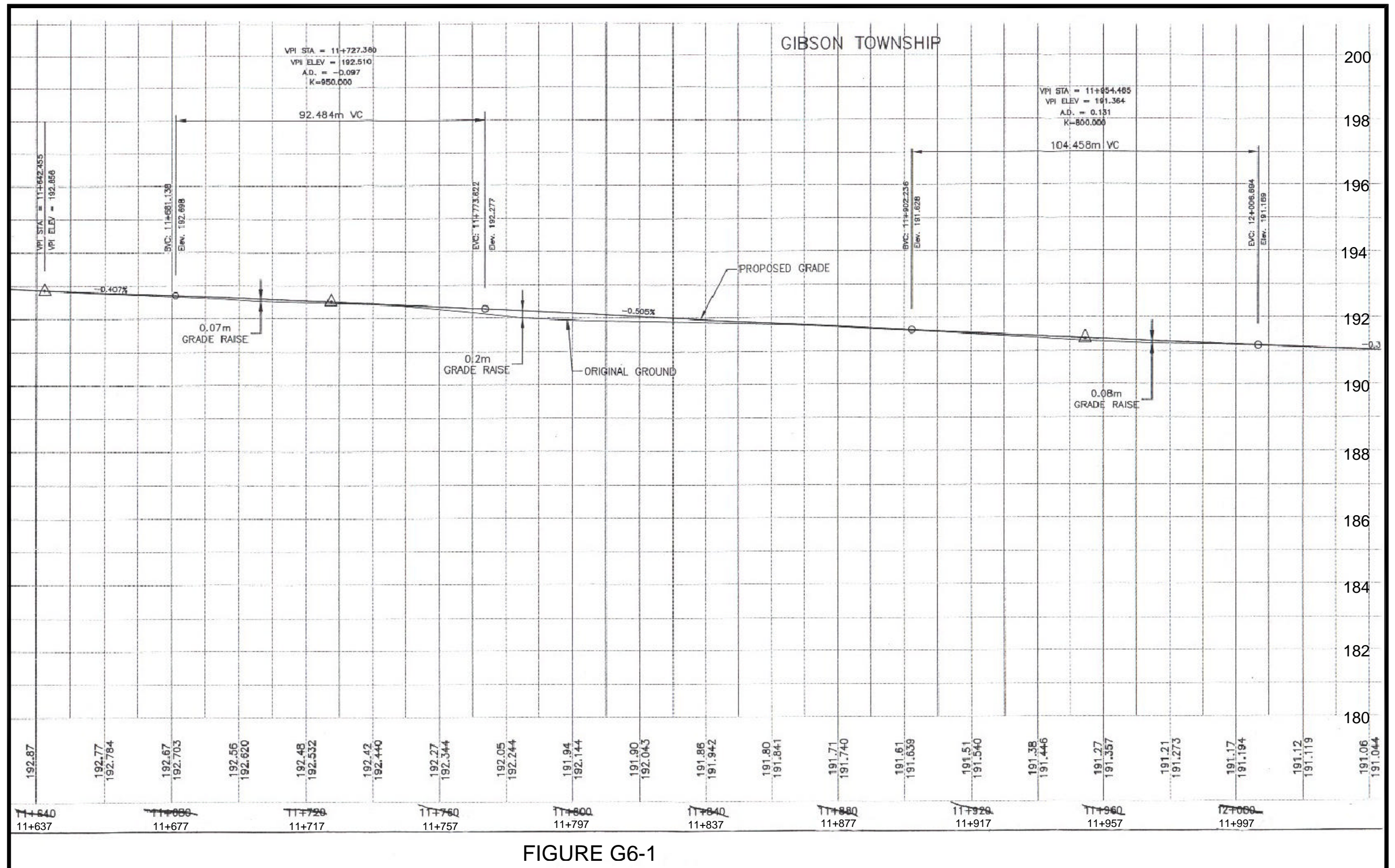
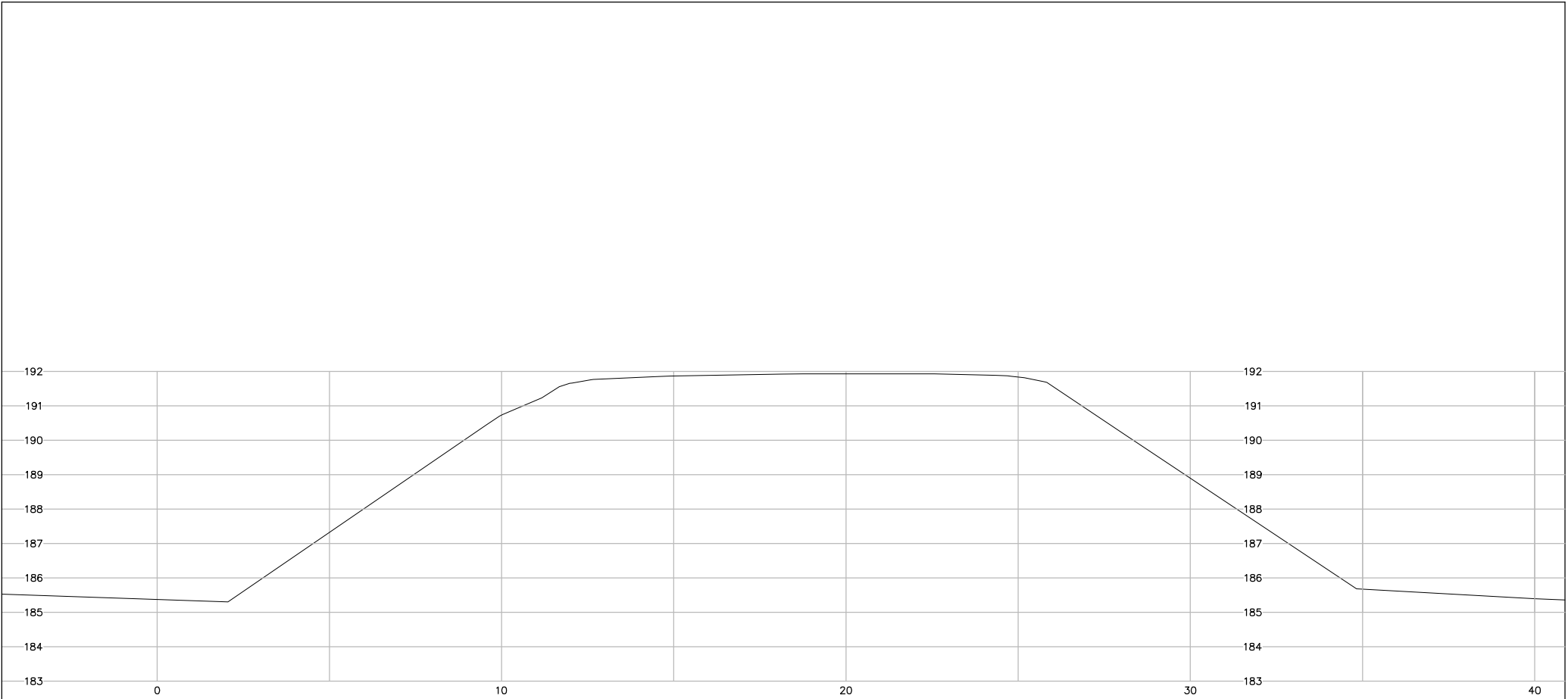
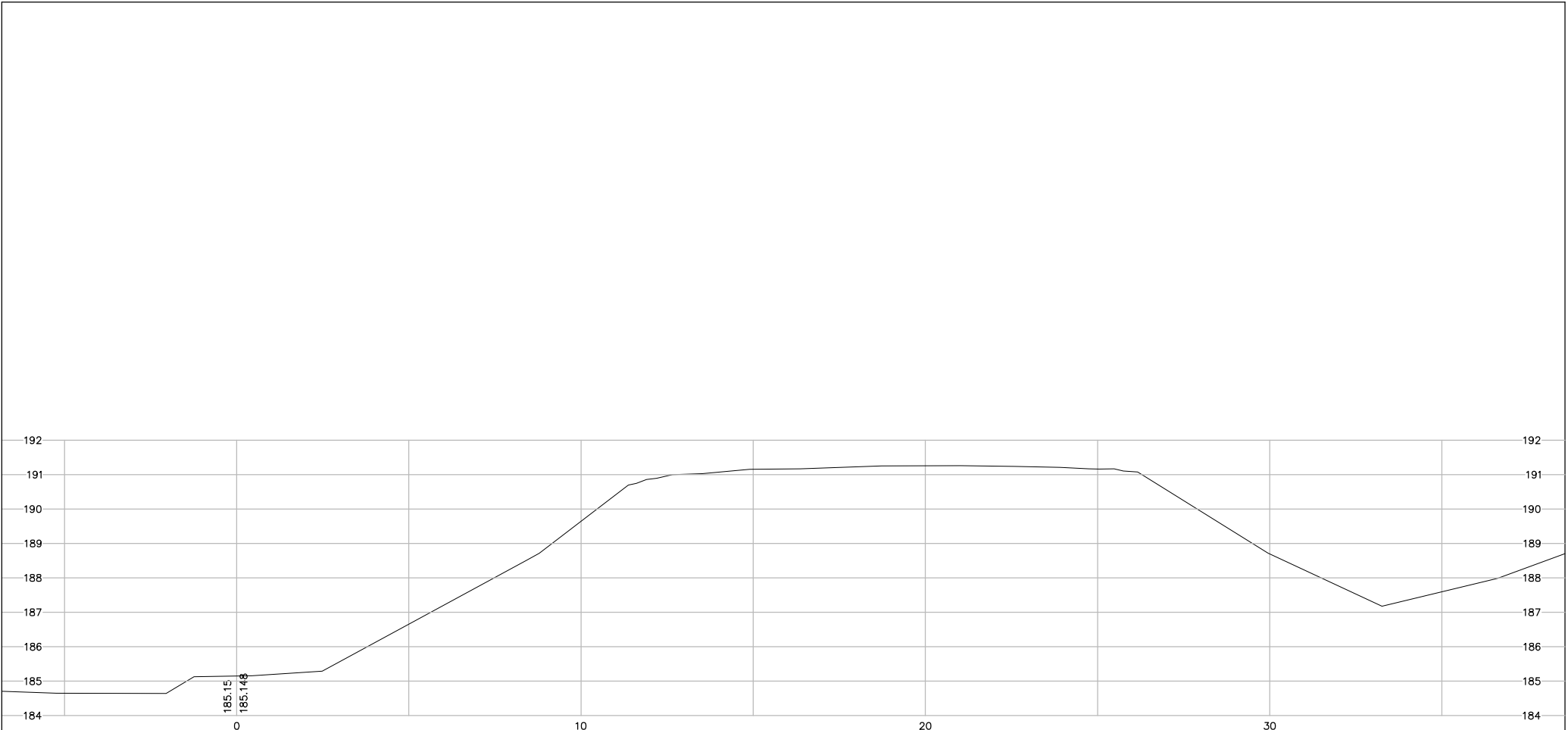


FIGURE G6-1



11+800, NBL
GIBSON TWP



11+960, NBL
GIBSON TWP

Appendix H6

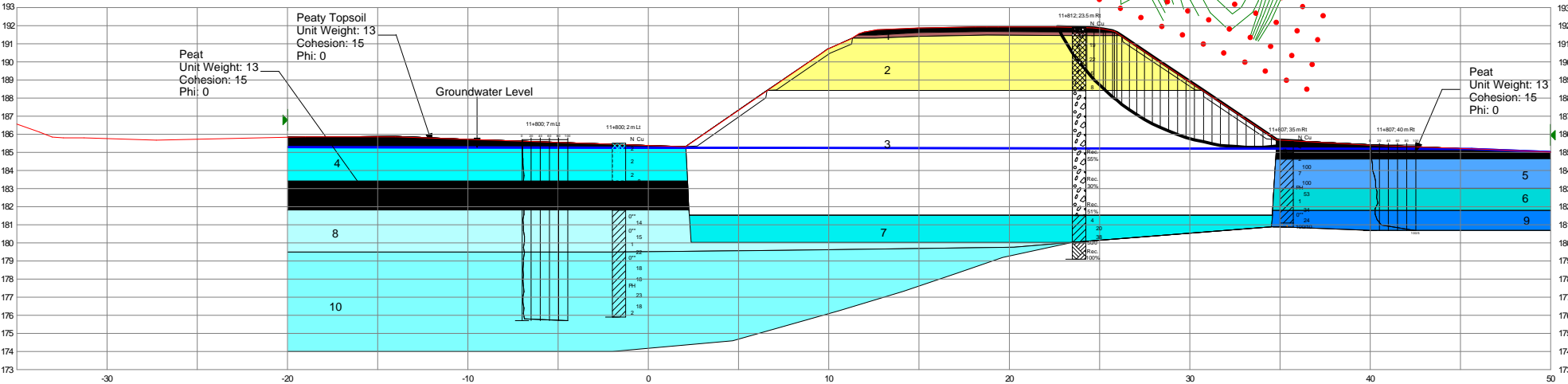
Slope Stability Analysis Results

Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H6-1

- | | | |
|---|---|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 8 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 20 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 80 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 14 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 28 kPa
Phi: 0 degree |
| | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 18 kPa
Phi: 0 degree | |



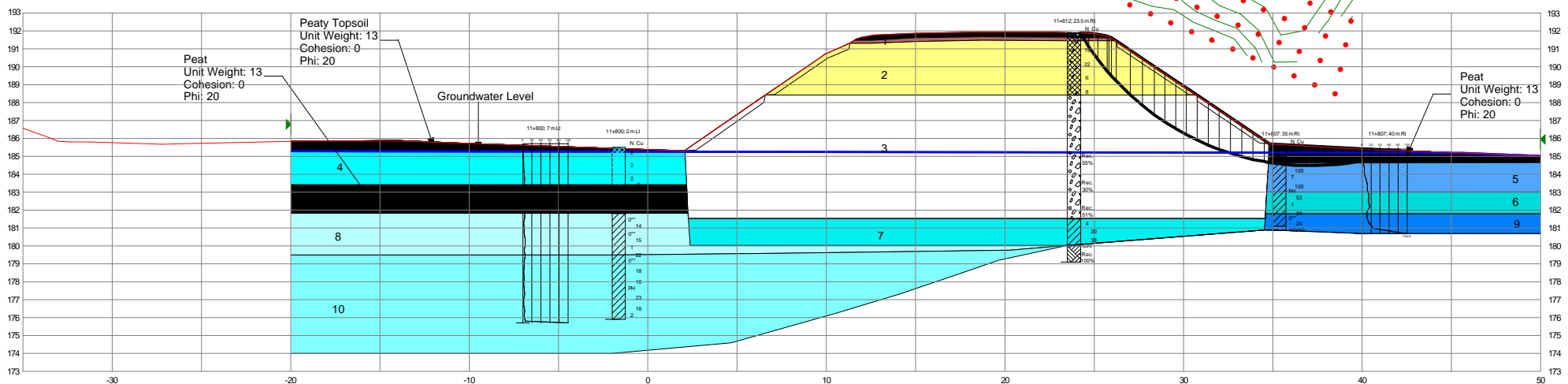
11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H6-2

- | | | |
|---|--|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 0 kPa
Phi: 20 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 24 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |



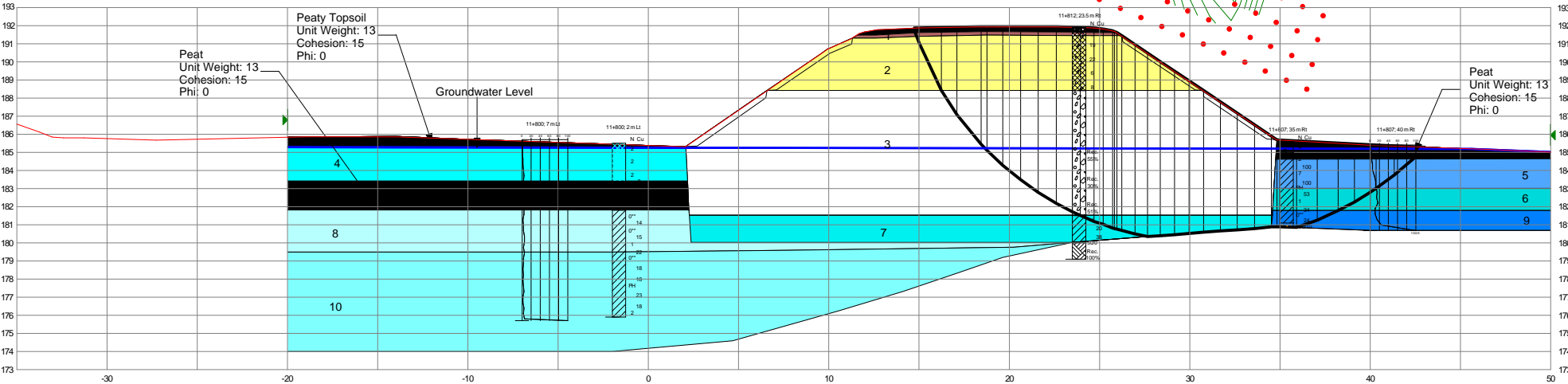
11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H6-3

- | | | |
|---|---|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 8 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 20 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 80 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 14 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 28 kPa
Phi: 0 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 18 kPa
Phi: 0 degree |



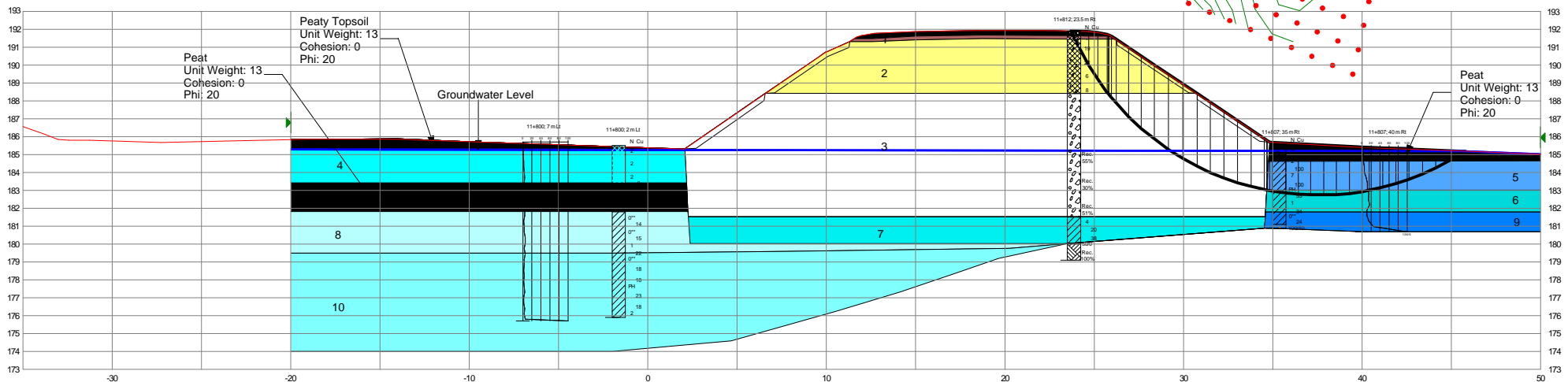
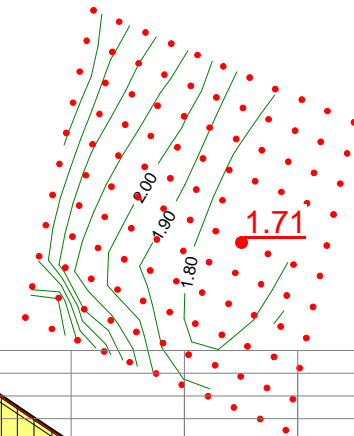
11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H6-4

- | | | |
|---|--|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 0 kPa
Phi: 20 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 24 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |



11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

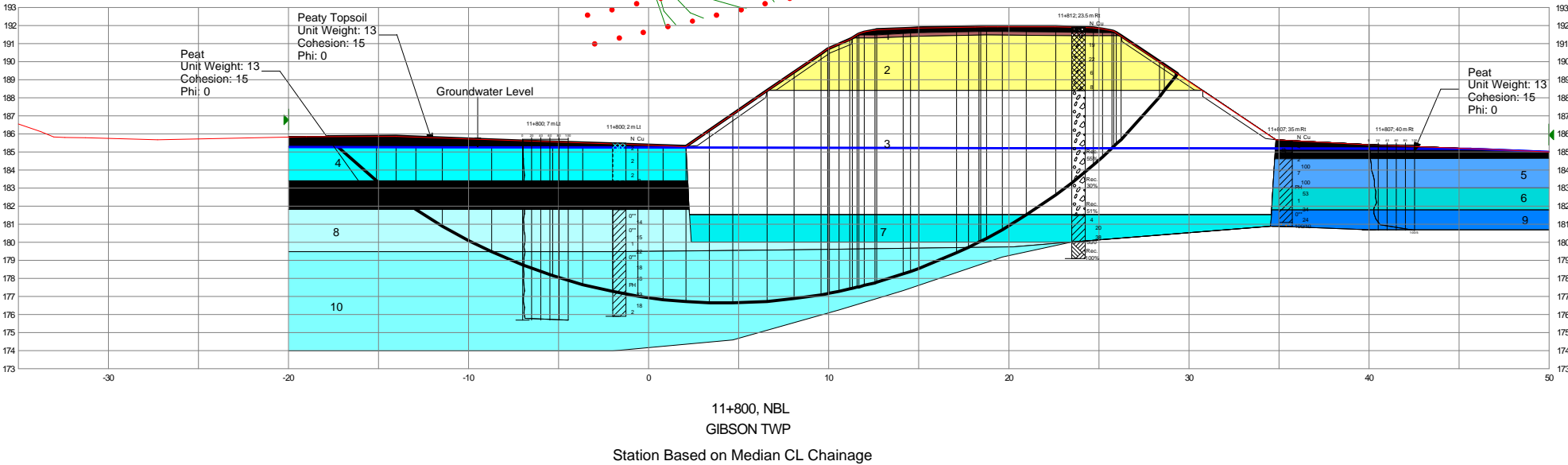
Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H6-5



- | | | |
|---|---|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 8 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 20 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 80 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 14 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 28 kPa
Phi: 0 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 18 kPa
Phi: 0 degree |

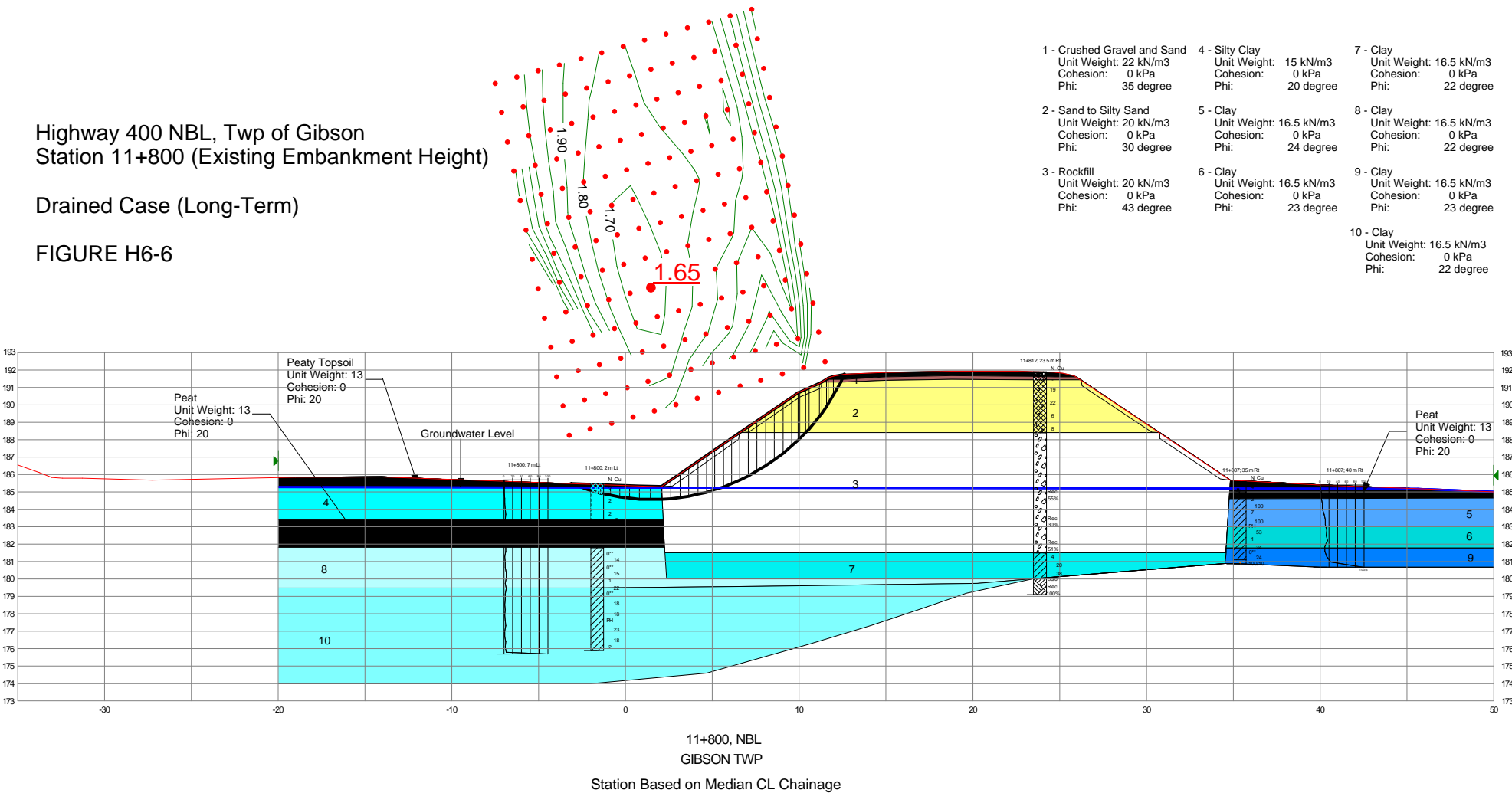


Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H6-6

- | | | |
|---|--|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m3
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m3
Cohesion: 0 kPa
Phi: 20 degree | 7 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 24 degree | 8 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 23 degree | 9 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 23 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m3
Cohesion: 0 kPa
Phi: 22 degree |

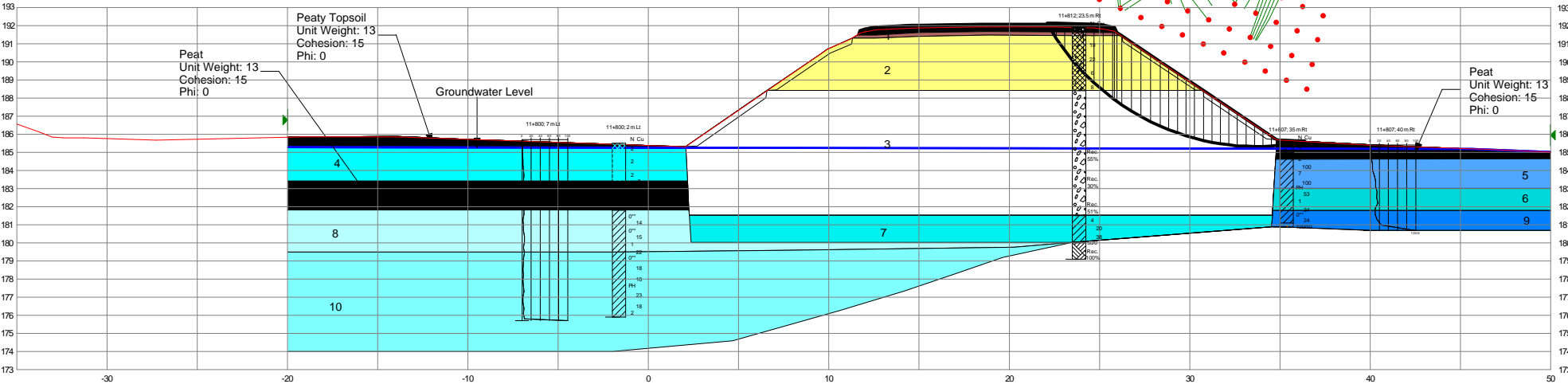


Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)
[+ 0.21 m Grade Raise]

Undrained Case (Short-Term)

FIGURE H6-7

- | | | |
|---|---|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 8 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 20 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 80 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 14 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 28 kPa
Phi: 0 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 18 kPa
Phi: 0 degree |



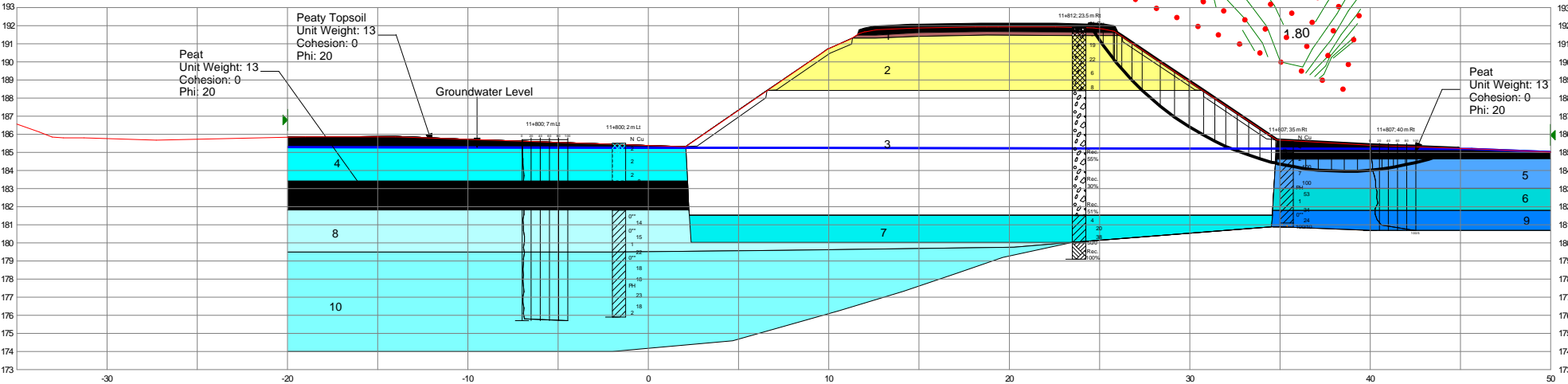
11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)
[+ 0.21 m Grade Raise]

Drained Case (Long-Term)

FIGURE H6-8

- | | | |
|---|--|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 0 kPa
Phi: 20 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 24 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |



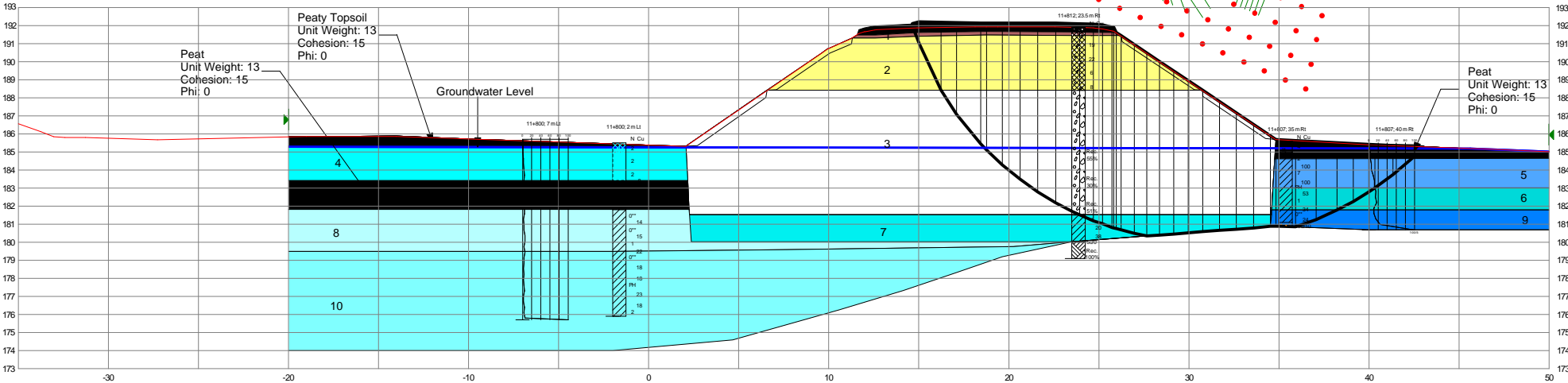
11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)
[+ 0.21 m Grade Raise]

Undrained Case (Short-Term)

FIGURE H6-9

- | | | |
|---|---|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 8 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 20 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 80 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 14 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 28 kPa
Phi: 0 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 18 kPa
Phi: 0 degree |



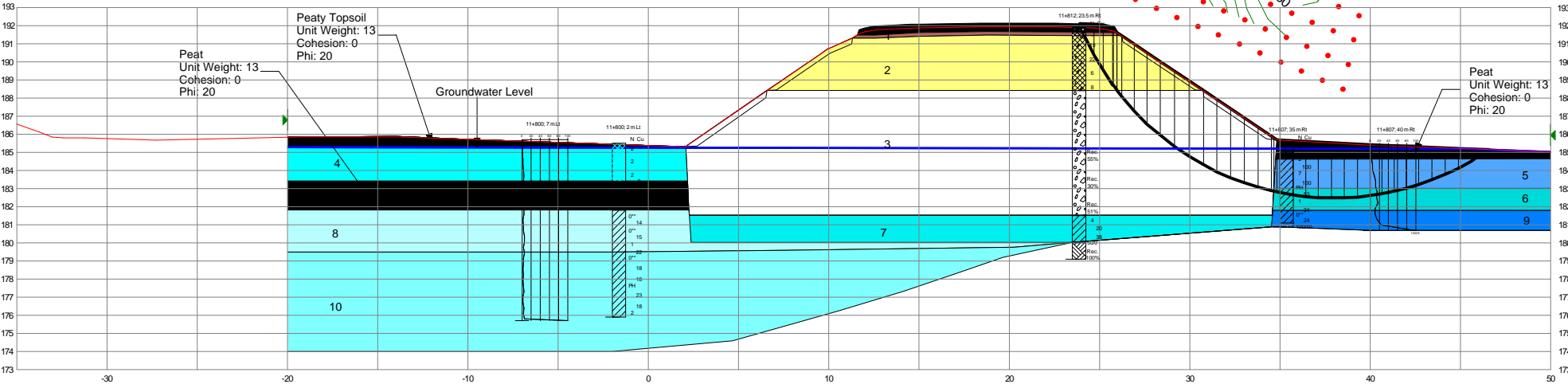
11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)
[+ 0.21 m Grade Raise]

Drained Case (Long-Term)

FIGURE H6-10

- | | | |
|---|--|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 0 kPa
Phi: 20 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 24 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 23 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |

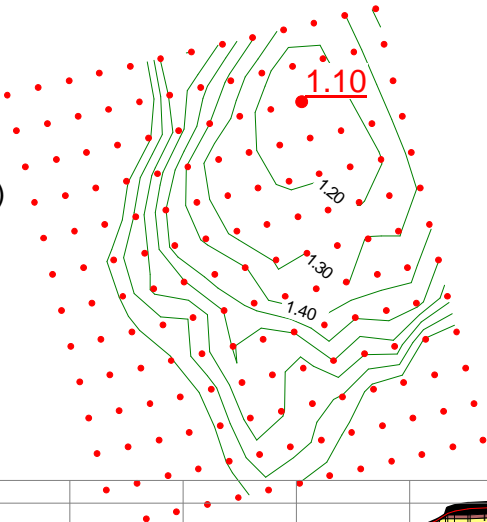


11+800, NBL
GIBSON TWP
Station Based on Median CL Chainage

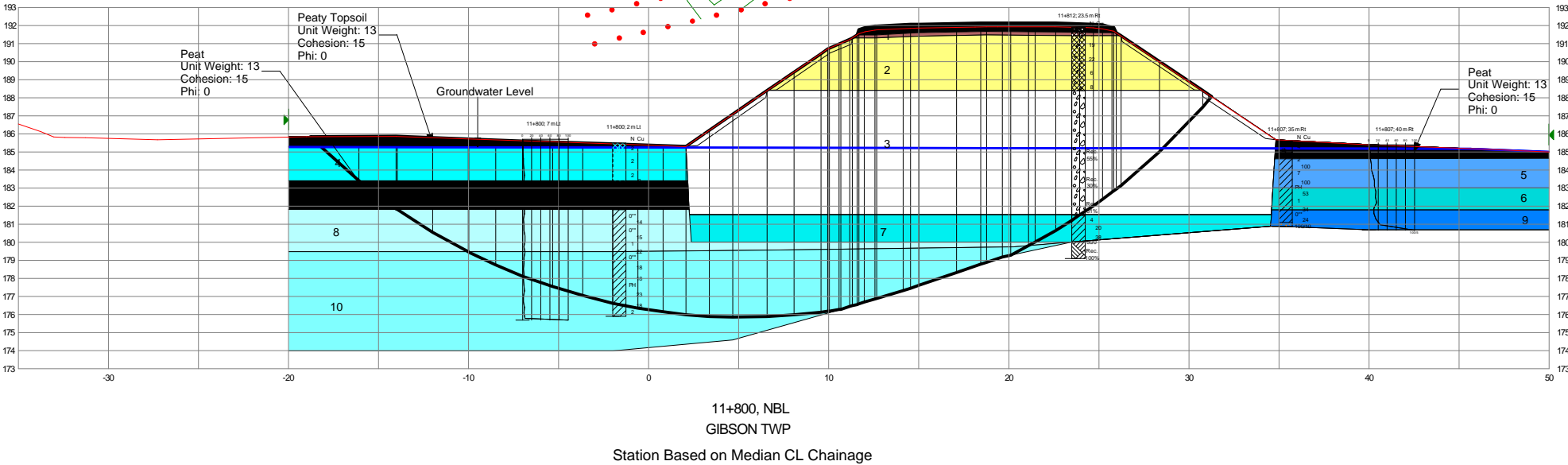
Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)
[+ 0.21 m Grade Raise]

Undrained Case (Short-Term)

FIGURE H6-11



- | | | |
|---|---|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Silty Clay
Unit Weight: 15 kN/m ³
Cohesion: 8 kPa
Phi: 0 degree | 7 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 20 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 80 kPa
Phi: 0 degree | 8 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 14 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree | 9 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 28 kPa
Phi: 0 degree |
| | | 10 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 18 kPa
Phi: 0 degree |



Highway 400 NBL, Twp of Gibson
Station 11+800 (Existing Embankment Height)
[+ 0.21 m Grade Raise]

Drained Case (Long-Term)

FIGURE H6-12

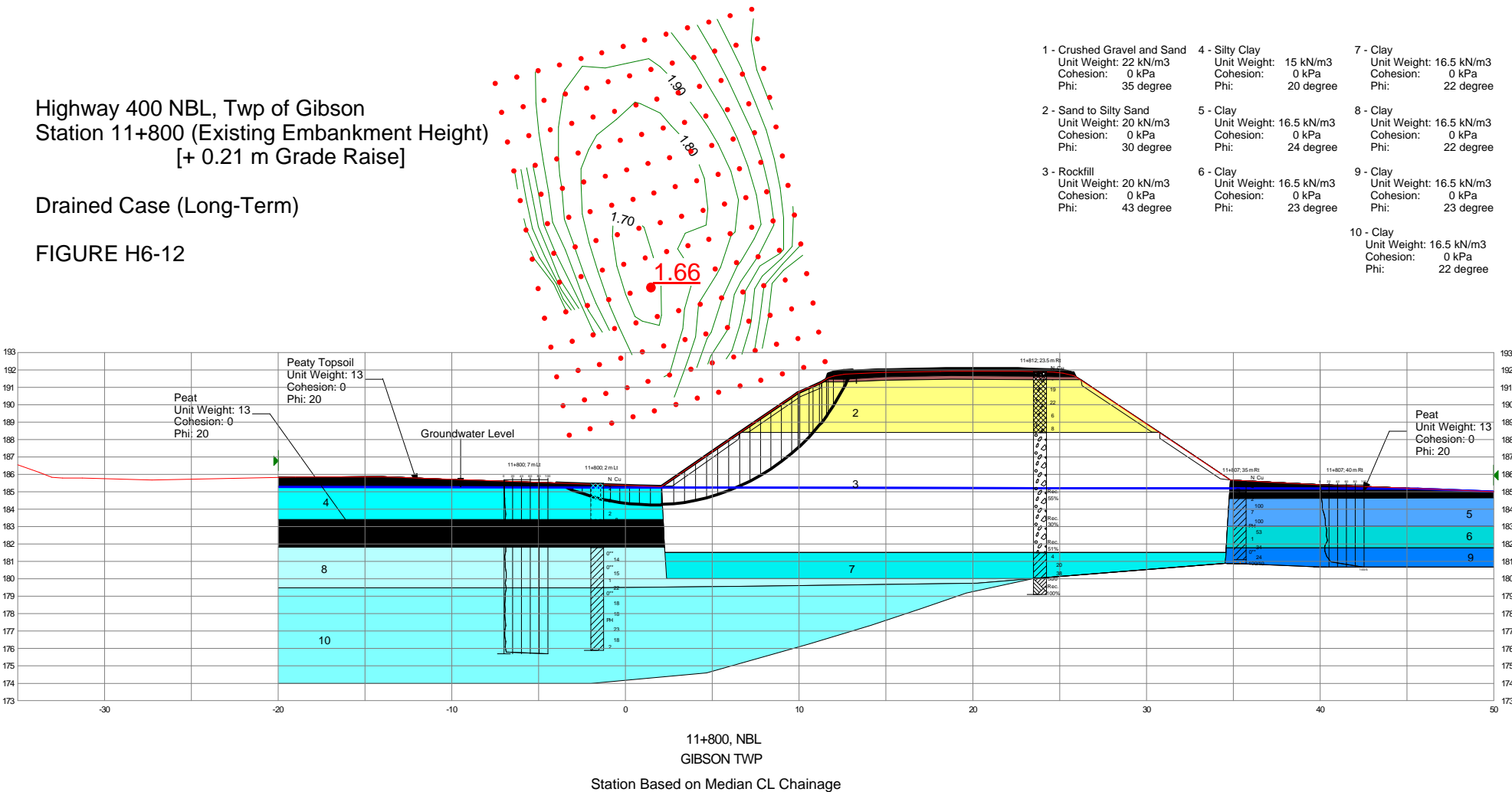


FIGURE H6-13

- | | |
|---|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 22 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Sandy Silt to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 27 degree |

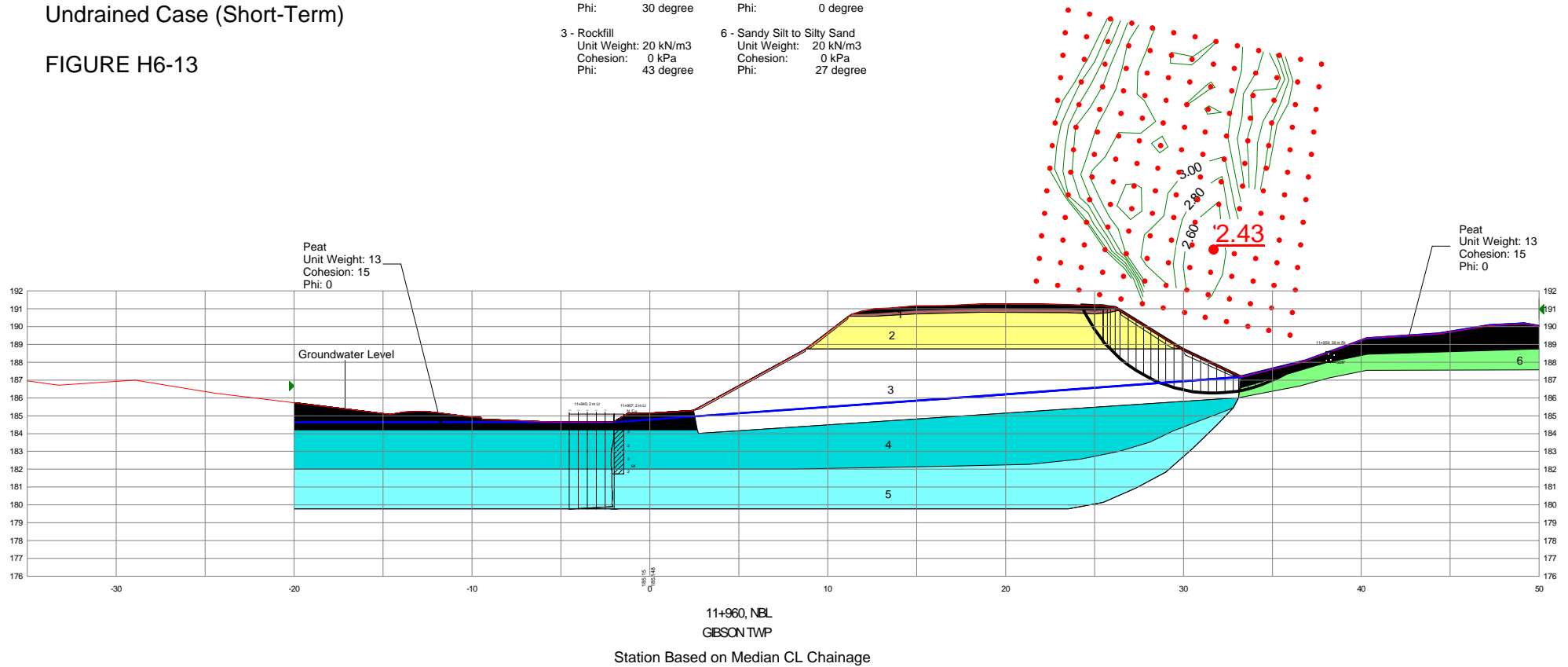
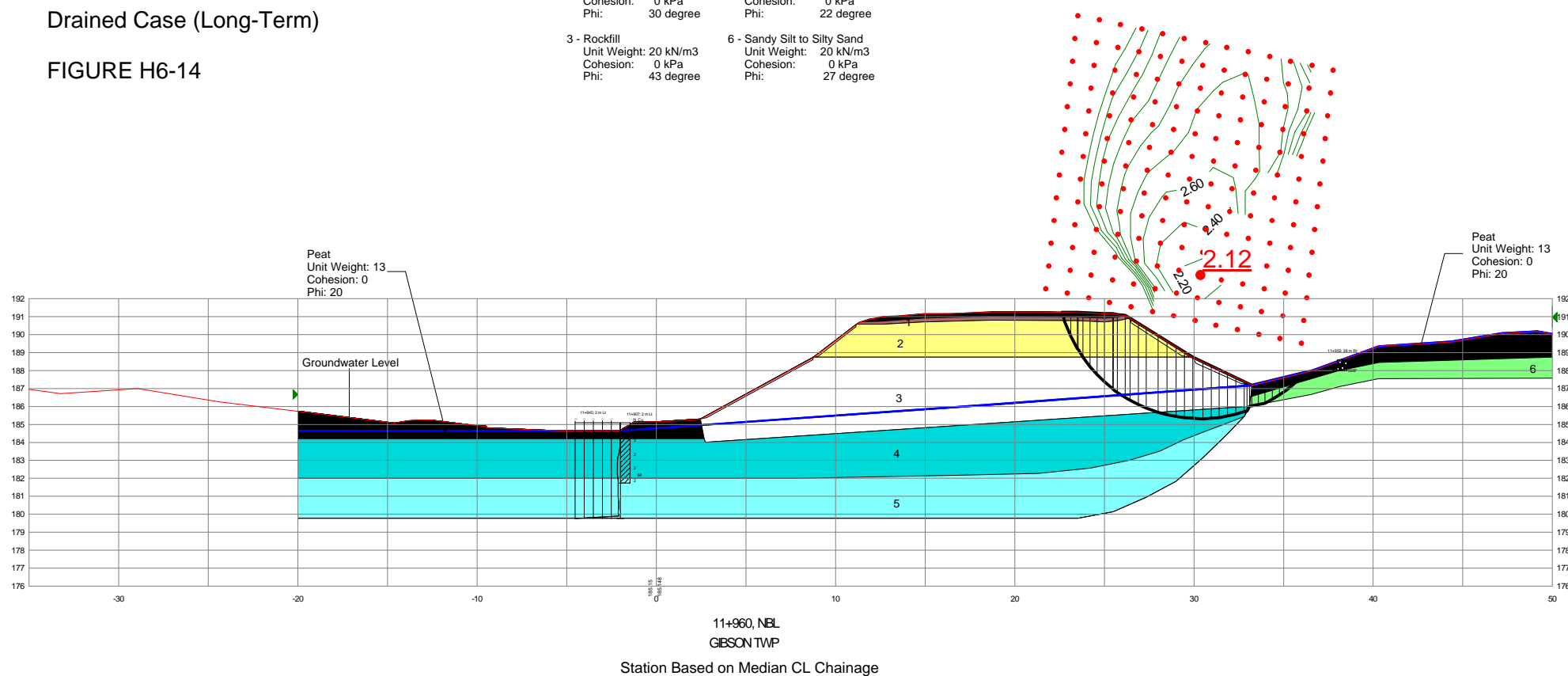


FIGURE H6-14

- | | |
|---|---|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Sandy Silty to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 27 degree |

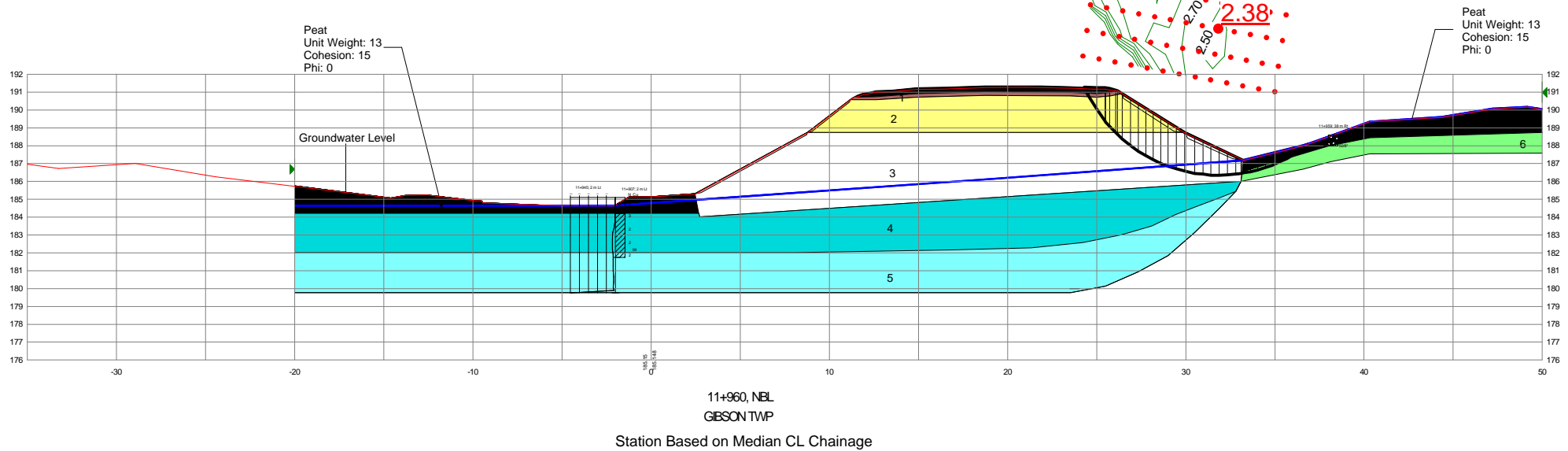


Highway 400 NBL, Twp of Gibson
Station 11+960 (Existing Embankment Height + 0.08 m Grade Raise)

Undrained Case (Short-Term)

FIGURE H6-15

- | | |
|---|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 22 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Sandy Silt to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 27 degree |

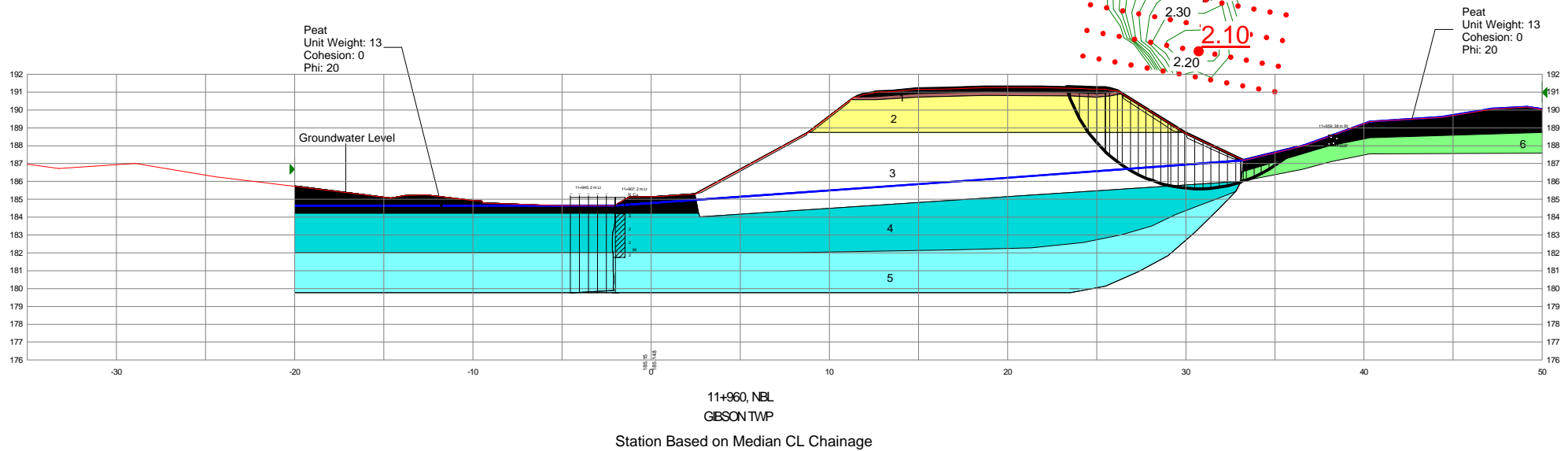


Highway 400 NBL, Twp of Gibson
Station 11+960 (Existing Embankment Height + 0.08 m Grade Raise)

Drained Case (Long-Term)

FIGURE H6-16

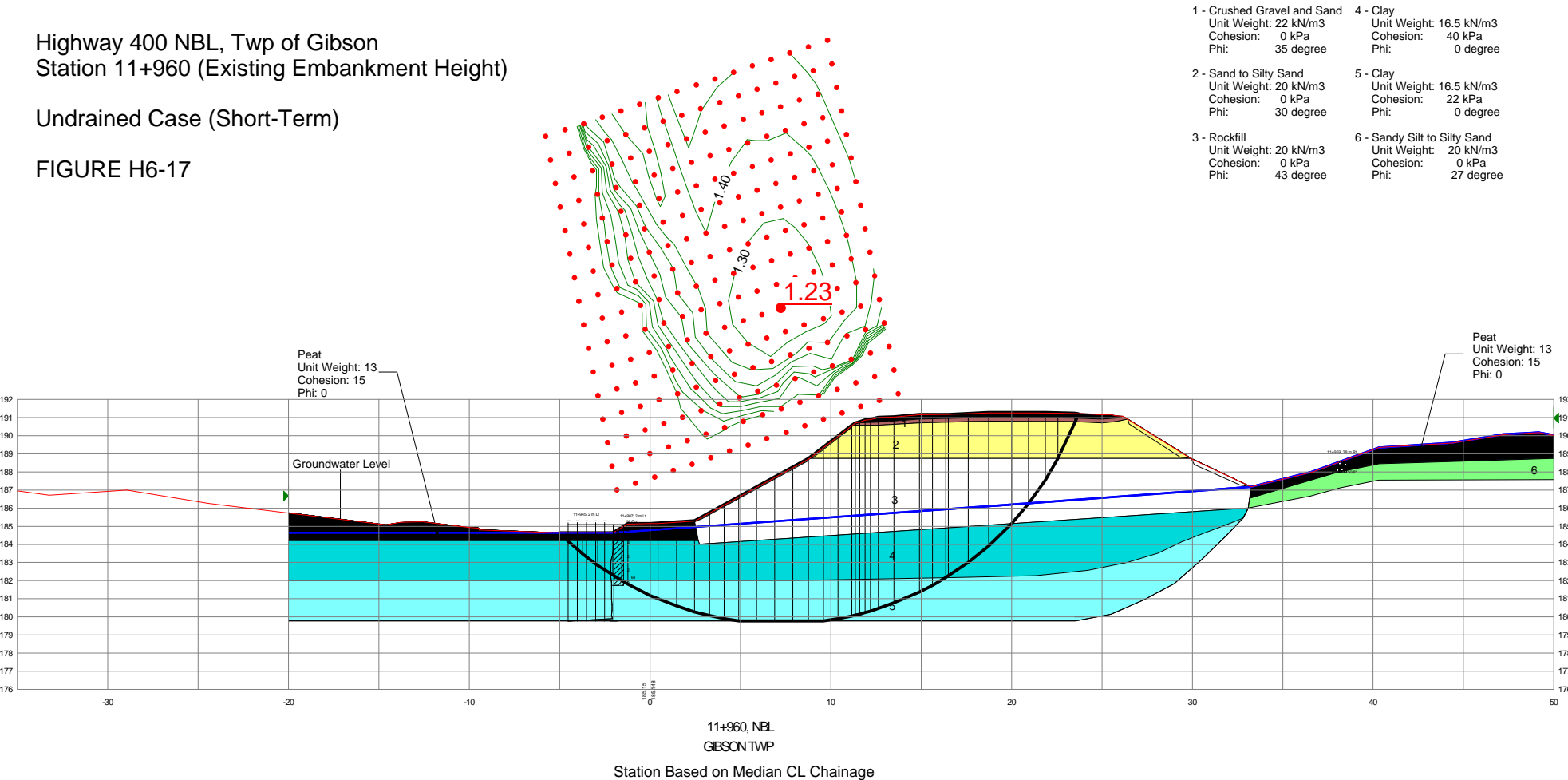
- | | |
|---|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 0 kPa
Phi: 22 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Sandy Silt to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 27 degree |



Highway 400 NBL, Twp of Gibson
Station 11+960 (Existing Embankment Height)

Undrained Case (Short-Term)

FIGURE H6-17



Highway 400 NBL, Twp of Gibson
Station 11+960 (Existing Embankment Height)

Drained Case (Long-Term)

FIGURE H6-18

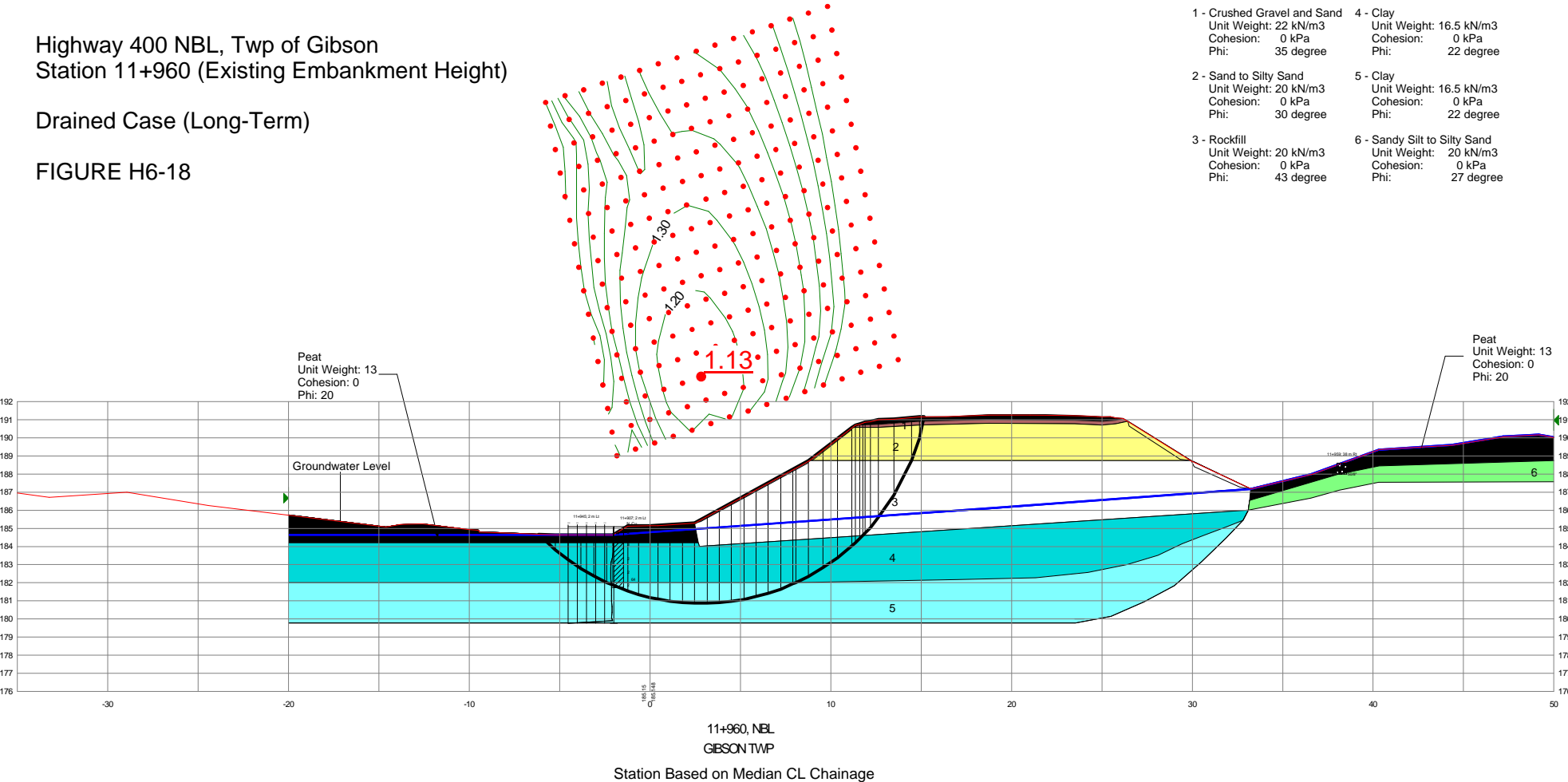
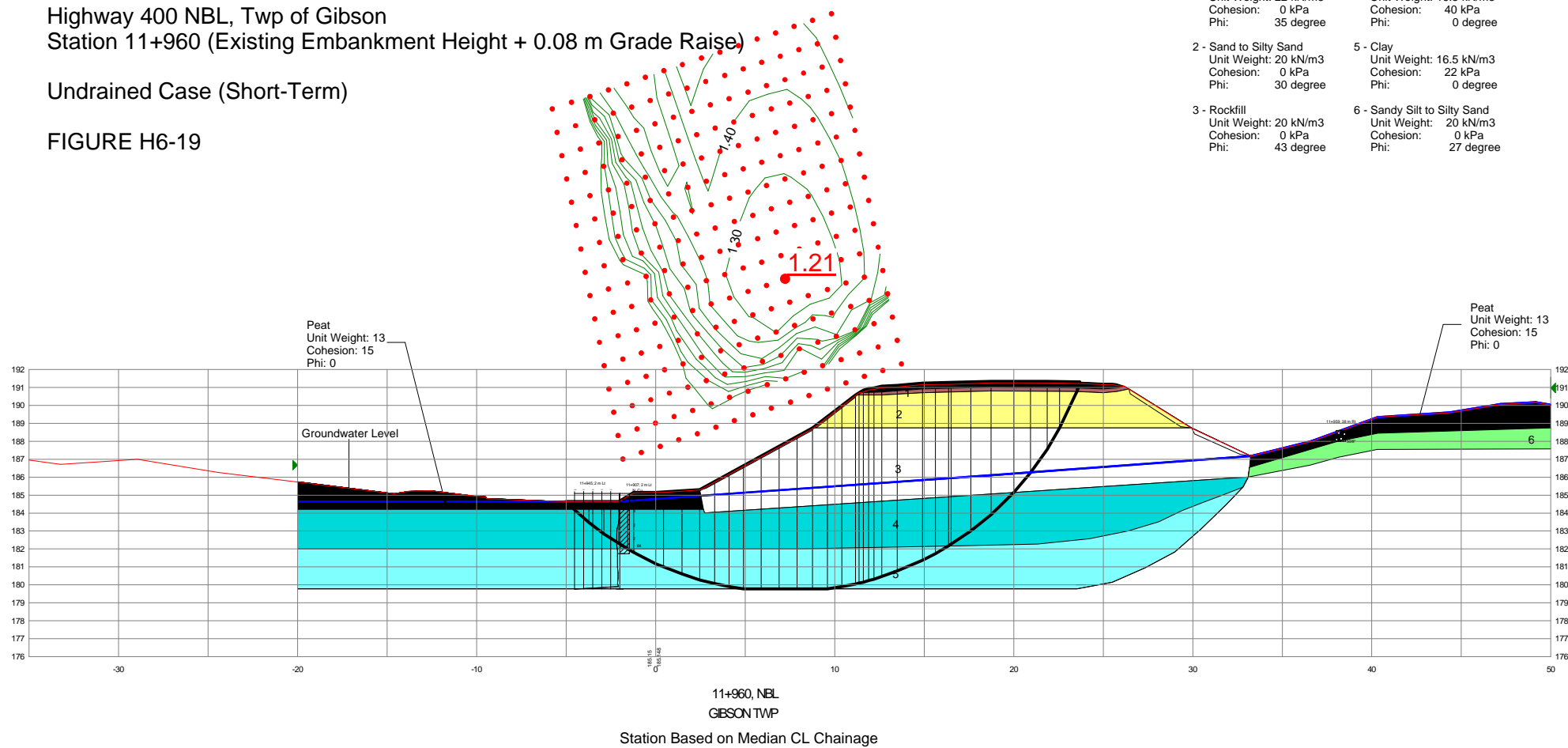


FIGURE H6-19

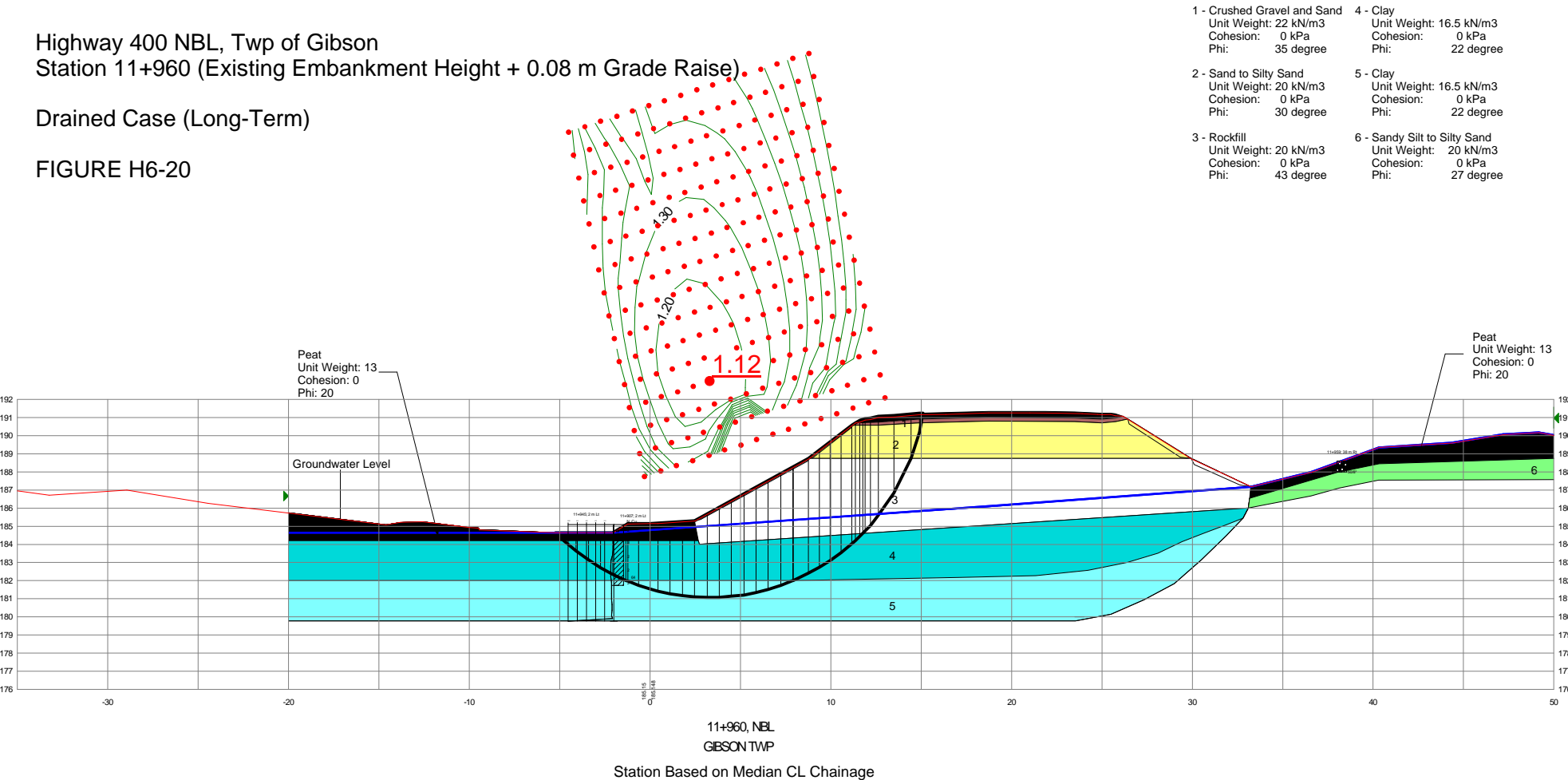
- | | |
|---|--|
| 1 - Crushed Gravel and Sand
Unit Weight: 22 kN/m ³
Cohesion: 0 kPa
Phi: 35 degree | 4 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 40 kPa
Phi: 0 degree |
| 2 - Sand to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 30 degree | 5 - Clay
Unit Weight: 16.5 kN/m ³
Cohesion: 22 kPa
Phi: 0 degree |
| 3 - Rockfill
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 43 degree | 6 - Sandy Silt to Silty Sand
Unit Weight: 20 kN/m ³
Cohesion: 0 kPa
Phi: 27 degree |



Highway 400 NBL, Twp of Gibson
Station 11+960 (Existing Embankment Height + 0.08 m Grade Raise)

Drained Case (Long-Term)

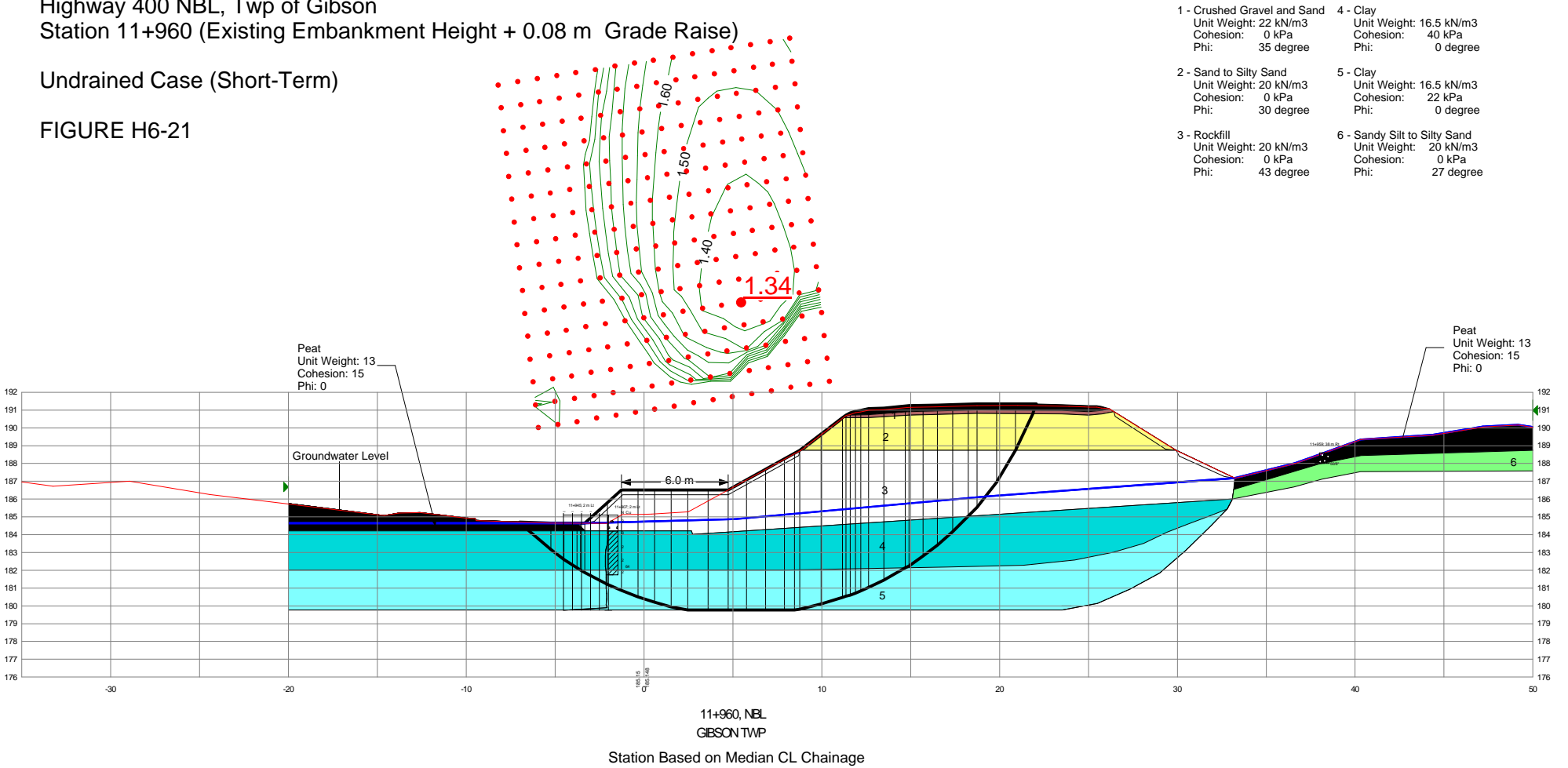
FIGURE H6-20



Highway 400 NBL, Twp of Gibson
Station 11+960 (Existing Embankment Height + 0.08 m Grade Raise)

Undrained Case (Short-Term)

FIGURE H6-21



Appendix I6

Photographs



Photograph I-1 View to the North at about Station 11+800



Photograph I-2 Pavement condition at about Station 11+800 (left shoulder)



Photograph I-3 View to the South at about Station 11+930



Photograph I-4 View to the North at about Station 11+940



Photograph I-5 View to the North at about Station 11+940



Photograph I-6 Cracks on Left Shoulder



Photograph I-7 View to the North at about Station 11+960



Photograph I-8 Cracks on Left Shoulder



Photograph I-9 View to the South at about Station 11+960



Photograph I-10 View to the South at about Station 11+930

Appendix J

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time. Any user of this report specifically denies any right to claims against the Consultant, Sub-Consultants, their officers, agents and employees in excess of the fee paid for professional services.