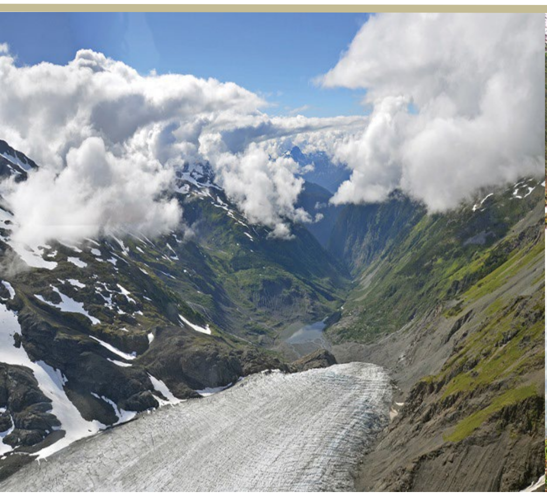
Highway 1
P7 – King Road Realignment
100% Detailed Design Geotechnical Report



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McElhanney Ltd.

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Revision History

Date	Revision Version	Comments
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ACRONYMS & ABBREVIATIONS

Acronyms/Abbreviations	Definition
BC MoTI	British Columbia Ministry of Transportation and Infrastructure
CFEM	Canadian Foundation Engineering Manual
CSA S6-19	CSA S6:19, Canadian Highway Bridge Design Code
DCP	Double Corrosion Protection
DCPT	Dynamic Cone Penetration Test
DST	Downhole Seismic Tests
ETR	Energy Transfer Ratio
FoS	Factor of Safety
GSC	Geological Survey of Canada
McElhanney	McElhanney Ltd.
NBCC	National Building Code of Canada
NCHRP	National Cooperative Highway Research Program
PTI	Post Tensioning Institute
SGEP	Supplementary Ground Exploration Program
SPT	Standard Penetration Test
S_u	Undrained Shear Strength
Tetra Tech	Tetra Tech Canada Inc.
TRB	Transportation Research Board
V_s	Shear Wave Velocity
ϕ'	Drained Peak Friction Angle
C’	Effective Cohesion

LIMITATIONS OF REPORT

This report and its contents are intended for the sole use of McElhanney Ltd. and British Columbia Ministry of Transportation and Infrastructure and their agents. Tetra Tech Canada Inc. (Tetra Tech) does not accept any responsibility for the accuracy of any of the data, the analysis, or the recommendations contained or referenced in the report when the report is used or relied upon by any Party other than McElhanney Ltd. and British Columbia Ministry of Transportation and Infrastructure, or for any Project other than the proposed development at the subject site. Any such unauthorized use of this report is at the sole risk of the user. Use of this document is subject to the Limitations on the Use of this Document attached in the Appendix A.

1.0 INTRODUCTION

Tetra Tech Canada Inc. (Tetra Tech) is pleased to present McElhanney Ltd. (McElhanney) and the British Columbia (BC) Ministry of Transportation and Infrastructure (MoTI) with the following geotechnical design recommendations as part of the Detailed Design submission for the King Road Realignment (here forth referred to as Project 7) for the Fraser Valley Highway 1 Corridor Improvement Program in Abbotsford, BC.

The project scope is established to progress early work activities involving the realignment of King Road, which has been designated for a future widening of Highway 1. The P7 scope requires excavation of a soil cut approximately 185 m in length with a maximum cut height of approximately 8 m along the southern side of King Road.

The contents of this report are limited to the geotechnical assessment that is carried out to define the geotechnical requirements and recommendations for the proposed structure. The Limitations on the Use of this Document, attached in Appendix A, forms an integral part of this report.

2.0 DESIGN CRITERIA

2.1 Design Basis

The following design codes and documents will be used to develop the basis of design:

- BC MoTI Bridge Standards and Procedures Manual, Volume 1, Supplement to CHBDC S6:19 (Referred to herein as the MoTI Supplement).
- Canadian Highway Bridge Design Code (CHBDC), CSA S6-19.
- AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020 – for those items not covered in the two references above.
- T04-17 - Geotechnical Design Criteria.
- FHWA-NHI-14-007 “Soil Nail Walls Reference Manual” February 2015.
- Canadian Foundation Engineering Manual (CFEM), 4th Edition, 2006.

2.2 Importance Category

The classification of the structures has been considered as “Major Route”, as recommended by MoTI.

2.3 Consequence and Site Understanding Classification

The consequence classification used in the geotechnical design is “Typical”, as recommended by MoTI. As per Table 6.1 of CSA S6-19, the consequence factor for Typical Consequence is 1.0.

Based on the scope and results of the completed site exploration, the Degree of Understanding (DoU) is considered to be “Typical” according to Tables 6.2a and 6.2c of the MoTI Supplement.

2.4 Static Design

The Geotechnical Resistance Factors for static design will be determined in accordance with Table 6.2 of CSA S6-19. The minimum Factor of Safety (FoS) for static global stability of embankments is defined in Table 6.2b of the MoTI Supplement.

2.5 Seismic Design

2.5.1 Seismic Hazard

BC MoTI has confirmed that the 6th Generation seismic hazard model, developed to accompany the National Building Code of Canada 2020 (NBCC 2020), should be used for geotechnical and structural design.

2.5.2 Geotechnical Resistance Factors

Geotechnical Resistance Factors for seismic design will be determined in accordance with Clause 6.14.4.1 of the MoTI Supplement. For both force-based and performance-based design, including the design of capacity-protected elements, the seismic geotechnical resistance factor should be taken as the static value + 0.2 (but not greater than 1.0).

2.5.3 Seismic Performance Criteria

The minimum seismic performance levels defined in the MoTI Supplement for a Major-Route structures outside of bridge approach embankment are summarized below.

- 475-Year EQ: Major-Route geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years. If damaged, normal service shall be restorable within one month.
- 2475-Year EQ: Lifeline, Major-Route, and other geotechnical systems shall meet the life safety requirement of no collapse following ground motions with a return period of at least 2475 years. Large permanent foundation deformations may be acceptable provided specified post-seismic travelled lane functionality can be achieved.

3.0 REVIEW OF AVAILABLE GEOTECHNICAL INFORMATION

The geotechnical conditions at the project location have been evaluated using the background information and data available from the exploration that was carried out in 2021 by Golder Associates Ltd (Golder). The information used for the preliminary geotechnical soil characterization is presented in the following reports:

- Golder Associates Ltd. (2021), Geotechnical Desktop Review, Trans Canada Highway Fraser Valley Upgrades – Segment 3, Abbotsford, BC. Reference No. 19136159-015-TM-Rev0-3000.
- Golder Associates Ltd. (2021), Trans Canada Fraser Valley Segment 3 – Factual Report – Geotechnical Investigation at 50% Preliminary Design Stage. Reference No. 19136159-031-R-Rev0.
- Golder Associates Ltd. (2021), Trans Canada Fraser Valley Segment 3 – Preliminary Geotechnical Design Report. Reference No. 19136159-036-R-RevA.

- Golder Associates Ltd. (2022), Earthquake Scenario Spectra and Acceleration Time Histories for 1/475, 1/975 and 1/2475 Annual Exceedance Probabilities for Trans Canada – Fraser Valley Project, Abbotsford, British Columbia, Canada. Reference No. 21498748-001-TM-Rev0.
- Tetra Tech Canada Inc. (2022), 00815-264ST-WR-GE-P2-P3-P4-430-Highway 1 – 264th Street to Whatcom Road Segments 2/3 Supplemental Geotechnical Exploration Program.
- Tetra Tech Canada Inc. (2024), P7 Highway 1 – King Road Realignment Supplementary Geotechnical Exploration Factual Geotechnical Report (Ref. No.: 704-ENG.VGEO04080-07_02 Rev. 0).

A general site plan indicating the locations of the proposed alignment, and boreholes that were considered for this study is provided on Figure 1.

4.0 GEOLOGICAL AND GEOTECHNICAL CONDITIONS

4.1 Surficial Geology

According to Geological Survey of Canada (GSC) Surficial Geology Map 1485A, the project site is located in an area of transition between Sumas Drift and Postglacial sediments. The area to the west of Riverside Road is underlain by glaciofluvial and minor flow till deposits consisting of gravel, sand and sandy till units.

The following sections outline our preliminary interpretation of the subsurface soil and groundwater conditions based on the review of the available and collected geotechnical data at the project site to date.

4.2 2024 Tetra Tech Supplemental Geotechnical Exploration Program

A soil nail reinforced slope is being considered for a section of the proposed cut on the south side of King Road. Site-specific geotechnical information about the characteristics and relative density/consistency of the subsurface soils were not available at or near the specific area of the proposed soil-nail-reinforced slope. Considering the cost associated with the installation of the soil nails, a Supplementary Ground Exploration Program (SGEP) was proposed and completed to evaluate the potential elimination and/or optimization of the soil nails during the value engineering process.

The intent of the supplementary geotechnical investigation program was to reduce uncertainties and risk regarding subsurface soil and groundwater conditions and to re-evaluate geotechnical engineering design parameters considered for the detailed design of the proposed reinforced and unreinforced soil cuts along King Road.

Details of the field and laboratory activities relevant to the 2024 supplementary geotechnical site exploration as part of the P7 scope of work are presented in 2024 Tetra Tech report titled “P7 Highway 1 – King Road Realignment Supplementary Geotechnical Exploration Factual Geotechnical Report”.

4.3 Subsurface Ground Conditions

The findings from the various ground explorations indicate that the area immediately west of the Riverside overpass is predominantly characterized by stiff to very stiff glacial soil deposit referred to as “weathered glacial soils”, followed by very stiff to hard and dense to very dense glacial soils described as “Till-Like”. Bedrock was encountered at elevations below El. -30 m west of Riverside Overpass.

The current ground surface along the existing King Road alignment (south of the existing Highway varies in elevation between El. 31 m and El. 38 m in the project area.

The generalized soil stratigraphy, in terms of depth below existing ground surface, is described below.

Based on the information collected from test holes AH/MW24-01 and AH24-02, the typical soil stratigraphy upslope south of King Road consists of the following units:

- **TOPSOIL:** consists primarily of organic materials, fine to coarse sand and silt, with some fine gravel, generally moist and very loose. The thickness is generally about 0.2 m to 0.3 m.
- **Glacial Soils (Soils Unit II):** this unit primarily comprises non-plastic silt with poorly graded fine to medium sand, and trace fine to coarse gravel. This soil unit extends from a depth of approximately 0.3 m to between 6.0 m and 7.0 m. In the upper 3 m, the soil is generally firm to stiff and becomes very stiff to hard as the depth increases. This increase in consistency with depth is evidenced by Dynamic Cone Penetration Test (DCPT) blow counts, which range from 24 to 80.
- **Glacial Soils/Till-Like (Soils Unit III):** this unit primarily comprises well graded fine to coarse sand with approximately 20% to 38% low plastic fines, and trace fine gravel. Occasional lenses (~50 mm) of very stiff clay are present within this unit. This unit is generally very dense extending from about 6.0 m to 7.0 m depth to the maximum depth of exploration. DCPT refusal was encountered within this silty sand unit at about 6.0 m to 7.5 m depth below ground surface.

The typical soil stratigraphy at the toe of the slope along King Road, based on the subsurface information collected from testholes AH/MW24-03 and AH24-04, consists of:

- **SAND to Gravelly SAND (FILL – Soil Unit I):** this unit primarily comprises well graded, fine to coarse sand and fine to coarse gravel, with trace fines. This soil unit is generally brown and moist and is approximately 2.0 m thick. This unit is compact to dense with DCPT blow counts ranging between 15 and 47. Lower DCPT blow counts were measured near ground surface and increasing rapidly with depth.
- **Glacial Soils/Till-Like (Soils Unit III):** this unit primarily comprises well graded fine to coarse silty sand with fines content of approximately 23% and some fine to coarse gravel and occasional cobbles. This unit extends from approximately 2.0 m depth to the maximum exploration depth of about 9.2 m. This unit is dense to very dense with DCPT blow counts ranging from 46 to 98. DCPT refusal was encountered at about 4.5 m below ground surface within this unit.
- **Sedimentary Bedrock (Soil Unit IV)**: Bedrock consists of weak to strong sandstone and mudstone. Bedrock is present at about 48 m depth (El. -30 m) west of Riverside Road.

In addition to the generalized soil layers described above, the inferred soil profile and soil type variation along the current King Road alignment is shown on Figure 2.

4.4 Groundwater Conditions

At the boreholes completed south of King Road, upslope of the proposed cut on MoTI property, two distinct groundwater regimes were observed. Groundwater was observed at depth of approximately 3.0 m in both testholes AH/MW24-01 and AH24-02. A deeper groundwater regime at a depth of about 7.5 m was also observed AH/MW24-01. The soils immediately below the shallower groundwater level were observed to have relatively low moisture content extending to a depth of about 7.5 m where the deeper groundwater regime was observed. It should be noted that groundwater measurements were completed on March 27, 2024 following well installation and development.

At the location of AH/MW24-03, near the toe of the existing slope along King Road, the groundwater level was detected at an approximate depth of 7.9 m below the ground surface. No evidence of groundwater was observed at AH24 04 location which terminated at a depth of 6.1 m. The reported groundwater depths are from measurements completed on the last day of drilling (March 27, 2024) corresponding to 1 day and 2 days after the installation of monitoring wells at AH24-01 and AH24-03, respectively. The groundwater level may vary with seasonal changes, precipitation, and local infiltration of surface water. Given the uncertainty involved with the continuity of the perched groundwater and the potential long-term effects, engineering analyses were completed considering the presence groundwater regime within the overburden above the dense to very dense stratum.

5.0 GEOTECHNICAL PARAMETERS

Geotechnical design parameters, including total unit weight, drained (effective) peak friction angle (ϕ') and Apparent/Effective cohesion (c') , undrained shear strength (S_u), and shear wave velocity (V_s) were defined using the factual data from the 2021 exploration program (Golder) and the 2022 and 2024 exploration programs (Tetra Tech).

The range of geotechnical parameters of the main soil units used to carry out the engineering analyses are presented in Table 5-1. Additional details about the estimated and selected strength parameters are provided in Sections 5.3 and 5.4.

Table 5-1: Geotechnical Engineering Parameters

Soil Type	Unit Weight (kN/m³)	Strength Parameters *			V_s (m/s)
		Drained Condition ϕ' (°)	Apparent Cohesion, c' (kPa)	Undrained Condition S_u (kPa)	
Fill (I)	19	36	–	–	180 – 200
Glacial Silt / sandy Silt (II) (weathered)	20	26	10 – 12	130 – 150	260 – 270
Glacial Soils / Till-like (III)	20	38	5 – 10	-	320 – 380
Bedrock / Sandstone (IV)	23	–	–	–	> 600

* Strength parameters selection is further discussed in Section 5.3 and 5.4.

The apparent/effective cohesion in the fine-grained soils will be significantly reduced if the soil becomes saturated.

5.1 Penetration Resistance

Penetration resistance values were obtained within the sonic testholes using the Standard Penetration Test (SPT) The penetration resistance values obtained from the field were used to correlate static engineering parameters and to assess the soil resistance to liquefaction.

Based on the information provided in the 2021 Golder report, measurement of Energy Transfer Ratio (ETR) performed on the SPT hammer, and the energy ratios were measured to vary between 75% and 81%. No site-specific energy measurements were performed on the SPT hammers that were used in the 2022 site exploration program. Based on the information provided by the drilling contractor and recent energy measurements completed for these SPT hammers at other projects, an average ETR of 85% was considered appropriate for the 2022 hammers. For the design average energy ratios of 78% for the 2021 hammer and 85% for the 2022 hammer were considered to correct the penetration values to standard values (N_{60}) corresponding to an energy ratio of 60% (Seed et al., 1984).

The energy correction factor (C_E) is computed as follows:

$$C_E = ER_m / 60\text{, where } ER_m \text{ is the measured energy ratio.}$$

The ETR value was used, along with other standard correction factors, to correct the field SPT-N values to standardized values (N_{60}) corresponding to an energy ratio of 60% (Seed et al. 1984).

Plots of the calculated $(N_1)_{60}$ values are presented on Figure 3.

5.2 Undrained Shear Strength

The undrained shear strength (S_u) of the near surface soils was estimated based on the result of an Unconsolidated Undrained (UU) Triaxial test completed on a relatively undisturbed sample obtained at borehole SH22-P2-06. Based on the UU Triaxial test result, the undrained shear strength of the cohesive soils in the upper 10 m is between 130 kPa to 150 kPa.

5.3 Strength Properties (Friction Angle and Cohesion)

Peak friction angle values were calculated based on the SPT data, which showed considerable variability due to the heterogeneous nature of the soil deposits. The design friction angles were conservatively selected considering SPT-based correlations along with published data and previous experience with similar soils.

Due to the relatively high content of fine-grained material (such as silt and clay) within the weathered glacial soils, a drained friction angle of 26 degrees was considered for the design. To inform selection of overburden strength properties used in the design, back analyses were carried out at representative sections spanning the proposed alignment. The slopes considered are generally 12 m to 14 m tall with an average slope angle of ranging between 20° and 30°.

In the domain of slope stability, slopes with static Factor of Safety (FoS) values greater than 1.2 to 1.3 are typically considered stable, exhibiting negligible to no movement. When observable slope movement is detected, a back analysis will often be conducted to consider FoS values of 1.1 or 1.2. For deep-seated failures (> 3m) the minimum global FoS value typically falls below 1.25 have resulted in ground movements such as landslides or slope cracking based on case histories and Tetra Tech's past project experiences. In the absence of observable movements or indication of cracking in the existing slopes, a FoS of 1.3 for back analysis was considered. For the back analysis the soil was modelled with a friction angle of 26° and cohesion was increased until the critical FoS was about 1.3.

Based on the back-analysis results presented on Figure 6, an apparent cohesion of 10 kPa to 12 kPa was calculated for the weathered glacial soils. The apparent cohesion in the fine-grained soils will be reduced if the soil becomes saturated, and/or the local vegetation or tree roots are removed.

Additionally, back analyses were also conducted assuming a higher friction angle (36 degrees) with no cohesion, and fully undrained behavior considering an undrained shear strength of 100 kPa. In both scenarios, the calculated factor of safety for the existing slope exceeded 1.40.

5.4 Re-evaluation of Design Strength Parameters by SGEP

The supplementary ground exploration completed during the 90% detailed design stage has re-evaluated the interpreted subsurface ground and groundwater condition and the geotechnical engineering parameters. From the 2024 data, the near surface soils (Unit II) along the proposed cut consist mainly of firm to hard fine-grained soils. The weathered glacial soils contain a high percentage of fines (> 50%). It is anticipated that the behavior of the weathered glacial soil will be governed by its cohesive characteristics with plasticity index (PI) ranging between 10 to 20.

The un-weathered glacial soils/till-like material (Unit III) comprise of very dense fine to coarse sand with fines content ranging between about 20% and 40%. Measured DCPT blow counts and fines content for the various soil units are present on Figure 5 (a & b). Furthermore, the depth variation of the identified soil units, as determined from the results of the 2024 Supplementary Ground Exploration Program (SGEP), has been incorporated into the stability analyses.

5.5 Shear Wave Velocity

The variation of the shear wave velocity (V_s) with depth was obtained from in-situ measurements performed during Downhole Seismic Tests (DST). The 2022 exploration program included DST for shear wave velocity measurements within the glacial and bedrock units.

The measured values are shown on Figure 4 and were used to determine the site designation.

6.0 SEISMIC DESIGN CONSIDERATIONS

The seismic performance of the proposed retaining wall was evaluated using a 475-year and 2,475-year return period EQ event. Seismic design parameters based on the 2020 NBCC seismic hazard model were obtained from Earthquake Canada website considering the 6th generation seismic hazard model developed for NBCC 2020 (Natural Resources Canada). The parameters include the Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), and 5% damped Spectral Acceleration (Sa(T), where T is the period in seconds) values for specified periods of motion.

Using the V_s data presented on Figure 4, the time-average shear wave velocity over the upper 30 m (V_{s30}) is estimated to be about 312 m/s and was used to define the code-based design response spectra.

Table 6-1 presents the seismic design parameters for earthquakes with Annual Exceedance Probability (AEP) of 1/475 and 1/2475 corresponding to a site with V_{s30} of 312 m/s.

Table 6-1: 2020 NBCC Seismic Design Parameters for a Site with V_{s30} of 312 m/s (5% Damping)

Annual Exceedance Probability	PGA	PGV	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)
1/475	0.218	0.222	0.511	0.409	0.227	0.115	0.029	0.011
1/2475	0.417	0.486	0.963	0.810	0.480	0.285	0.087	0.037

The natural soil deposits encountered at the project site were typically stiff to hard or dense to very dense. The near surface soils at the project site are considered to have a low potential for cyclic mobility, while the till-like soil is not susceptible to liquefaction.

7.0 ENGINEERING ASSESSMENT AND RECOMMENDATIONS

As indicated on the McElhanney drawings attached in Appendix B, to allow for the realignment of King Road a soil cut is proposed between about Sta. 4001+50 to Sta. 4003+30 for a total length of about 180 m long. The first portion of the proposed cut denotes a 1.5H:1V cut between about Sta. 4001+50 to Sta. 4001+90 (approximately 40 m long). Following the 1.5H:1V cut the soil cut gradually becomes shallower to a final cut angle of 2.0H:1V at about Sta. 4002+00 until the end of the cut (Sta. 4003+30).

For the engineering analysis, the material within the soil cut were considered to have a drained friction angle of 26 degrees and cohesion of 10 kPa. A horizontal seismic coefficient (k_h) equal to one-half of the site-adjusted peak horizontal ground acceleration of 0.417g, corresponding to the 2475-year earthquake, was used for the seismic analysis.

The following has been considered for the detailed design:

- Geotechnical Resistance Factor (GRF) for soil nail pullout resistance = 0.4 (non-seismic)
- Geotechnical Resistance Factor (GRF) for soil nail pullout resistance = 0.6 (seismic)

7.1 Global Stability

Global stability analyses were conducted using 2-dimensional limit equilibrium slope stability analysis software, GeoStudio 2023 Slope/W by Geo-Slope International Ltd. (version 2023.1.2) to assess the stability of the proposed soil cuts, for both static and seismic conditions. The analyses were performed for the typical sections along the soil cut alignment provided by McElhanney.

The stability evaluation was performed considering:

- Geotechnical Resistance Factor (GRF) for global stability = 0.65 (non-seismic), corresponding to a FoS greater than or equal to 1.54 under static conditions.
- A horizontal seismic coefficient (k_h) equal to one-half of the site-adjusted peak horizontal ground acceleration for pseudo-static analysis.
- Geotechnical Resistance Factor (GRF) for global stability = 0.75 (seismic), corresponding to a global FoS for the overall stability greater than or equal to 1.3 under pseudo-static conditions considering k_h = 50% PGA. A performance target of lateral deformations ≤ 300 mm was considered for FoS less than 1.3 under the aforementioned pseudo-static conditions. The lateral deformations were estimated with simplified Newmark based displacement methods outlined by Bray, Macedo and Travasarou 2018 and Bray and Macedo 2019 considering the median displacement prediction.

Global stability analyses were performed for critical and representative sections of the soil cut. The results of the stability analyses for stations L4001+60, L4001+80 and L4002+30 corresponding to the various sections of the proposed soil cut are presented on Figures 7 through 9. Included as part of the analyses, are the stability checks performed for the existing/current condition that informed the soil nail reinforced slope design. The results of the stability analyses indicate that the soil nail wall design meets the typical design and performance requirements for both the static and seismic conditions.

Additional sensitivity analyses were performed considering the loss of cohesion due to saturation in the top 1 m of the weathered glacial soil. As illustrated on Figure 10, the sensitivity case decreases the FoS for surficial failures to values around 1.0 illustrating the need to protect the slope from saturation both during and after construction.

Considering this case (loss of apparent cohesion) as an extreme condition, given the low permeability of the soils, it does demonstrate the potential for material sluffing without appropriate controls. Additional discussion on required slope protection is provided in Section 8.0.

7.1.1 Unreinforced Soil Cut

Limit equilibrium analyses were completed for Section L4002+30 (Figure 9) to evaluate the maximum unreinforced face angle to fulfill the design criteria. To fulfill the design criteria of the soil cut without reinforcement the slopes of 2.0H:1V or shallower will be required.

Further information on the constructability including surface protection during construction and revegetation following construction is discussed further in Section 8.0.

7.1.2 Soil Nail Reinforced Slope

Additional reinforcement in the form of soil nails will be required for slopes steeper than 2.0H:1V. As shown on the drawings provided by McElhanney included in Appendix B, soil cuts between Sta. 4001+50 and Sta. 4002+00 are proposed with slopes ranging from 1.5H:1V to 2.0H:1V. Stability analyses were completed to evaluate the reinforcement requirements for the 1.5H:1V slope section of the soil cut as the most critical section. The soil cut reviewed considered a total cut height of about 8 m, with the total slope height from crest to toe of about 13 m. Analyses were carried out to check the slip surface intersecting the soil nails, or behind the bonded zone.

BC MoTI Supplement to CSA S6-19, Section 2.3.6.10 Soil and Rock anchors states “unless otherwise Consented by the Ministry, soil and rock anchors shall be a Post Tensioning Institute (PTI) – Class 1, Double Corrosion Protected (DCP) system”. Protection method for PTI - Class 1 nails/anchors is “encapsulation”, which is equivalent to the Class A protection as defined in FHWA-NHI-14-007. Based on the analyses performed, geotechnical recommendations and comments related to soil nail requirements are presented below:

- Soil nails are to be DCP, installed at 1.75 m horizontal and 1.75 m vertical spacing. The first row of nails is to be installed at 1.5 m above the top of the soil cut. The vertical spacing provided is measured along the slope face.
- DCP soil nails should have a minimum bond diameter of 152 mm (6 inches) and a minimum length of 10 m based on an ultimate bond strength of 60 kPa.
- Soil nails should be installed with an inclination angle of 30 degrees from horizontal.
- Use of DCP nails may necessitate the use of a temporary casing for drilling and installation of the soil nail.
- The soil nails should have a minimum ultimate load and ultimate strength of 351 kN and 685 MPa, respectively, and a minimum yield load and strength of 264 kN and 517 MPa, respectively, equivalent to a Dywidag #8 hot-rolled thread bar.
- The maximum Design Load (DL) is estimated to be 38 kN, considering the 30 kN pre-tensioning load for the soil nails and earth pressure load factor of 1.25 (CSA S6-19, Table 3.3). The pre-tensioning is intended to limit deformation, engage the resistance of the soil nails against the steel mesh, and tension the mesh facing.
- Testing of soil nails shall be conducted to verify the bond strength and nominal load transfer rate between soil and soil nails. A minimum of four verification tests are to be conducted on sacrificial nails installed and tested prior to production nail installation. Additional testing requirements are discussed in Section 7.2.
- Additional full-length soil nails shall be installed to allow for future extraction for long-term inspection and testing. Requirements for the additional soil-nails are discussed in Section 7.2.

In addition to the soil nails providing support for the slope a high tensile strength wire mesh spanning between the installed soil nails will be required to provide support and mitigate erosion of the surficial materials of the slope. The steel mesh will be pre-tensioned and will transfer forces to the nails efficiently, preventing deformation within the system. The mesh system shall incorporate an erosion blanket, and a structural high-strength steel mesh. The erosion blanket is included to reduce the impact of surficial runoff and prevent material loss while allowing vegetation growth. The high-strength steel mesh is the primary means of support for the slope to stabilize surficial instabilities on the slope. The erosion blanket and structural mesh will be secured to the slope using the soil nails and additional driven pins as required by the manufacturer/supplier.

The structural high-strength steel mesh should be at least 3.0 mm in diameter with a minimum tensile strength of 1770 MPa with a single twist mesh diamond. The erosion blanket should be a three-dimensional three-layer polypropylene mat specifically designed and tested for erosion control. Additional information on the soil nails, structural steel mesh, and erosion blanket will be incorporated into the special provisions developed for this project.

Further information on the constructability including surface protection during construction and revegetation following construction is discussed further in Section 8.0.

7.2 Installation and Testing

Testing of soil nails shall be conducted to verify the bond strength and nominal load transfer rate between soil and soil nails. The following tests should be completed during the construction process.

- A minimum of four verification tests are to be conducted on sacrificial nails installed and tested prior to production nail installation to review the bond capacity of the grout soil bond. Verification tests are to be performed per soil strata; two within the weathered and two within the unweathered glacial soils.
- Verification test nails must have a nominal bonded length of 2 m.
- Verification test nails must have an unbonded length. For testing within the weathered glacial soils, the unbonded length in ground must be at least 3.5 meters. For verification tests within the unweathered glacial soil, the unbonded length in ground must be at least 8 meters. Additional unbonded length for connecting the bar with the load test assembly may be required and is to be determined by the installation contractor.
- Saturation of the bond-soil interface resulting in reduced bond capacity shall be avoided. Given the high fines content of the subsurface soils, use of water or mud for drilling during the installation of soil nails is not recommended. The borehole should be protected for water runoff or heavy rain during construction.
- Carry out performance tests on the first three soil nails, and thereafter a further three soil nails or 2% of the total installed soil nails whichever is greater.
- Carry out proof tests on 5% soil nails in a nail row or a minimum of 2 nails not subjected to performance tests.
- Proof and performance tests shall be completed to a test load of 65 kN, estimated based on the recommendations in FHWA-NHI-14-007. The test load is estimated based on the smaller of 3 m or maximum bonded length in FHWA-NHI-14-007.
- Soil nail installation and verification load and proof testing should be performed in accordance with the recommendations of Soil Nail Walls Reference Manual (FHWA-NHI-14-007). Performance testing is to be completed in accordance to Post Tensioning Institute (PTI, 2014 or latest) for pre-stressed rock and soil anchors.

- Upon completion of proof and performance testing, soil nails shall be grouted to the final configuration and pre-tensioned to a 30 kN lock-off load. The pre-tensioning is intended to limit deformation, engage the resistance of the soil nails against the steel mesh, and tension the mesh facing.

- To ensure structural integrity and long-term performance, provisions have been made for the installation of three (3) additional soil nails specifically for future extraction for inspection and testing during the design life of the reinforced slope. This will involve periodic inspections, load testing, and the extraction of select soil nails to assess their condition and effectiveness over time, to review he ongoing stability and performance of the reinforcement system. The testing schedule during the design life of the structure is to be decided by the Ministry.

8.0 CONSTRUCTABILITY AND POTENTIAL RISKS

Some of the recommendations and potential risks involved with both unreinforced cut and soil nail reinforced soil cut construction are highlighted below:

- Temporary stability of the excavation is the responsibility of the Contractor.

- Excavation shall consider a phased approach by exposing the slope face in sections to evaluate ground condition and stability. If ground conditions are not consistent with ground conditions considered for the design, this information shall be immediately provided to the geotechnical designer. The Contractor shall provide an excavation and contingency plan if unfavorable/ unforeseen conditions are found.

- To avoid the surficial materials becoming saturated and thereby losing their apparent cohesion the slopes should be protected during excavation and revegetated as soon as practicable. It is envisioned that the slopes will be revegetated through hydroseeding. Erosion protection matts including drainage in addition to hydroseeding may be required for unreinforced slope cuts depending on the local ground and groundwater conditions.

- Prior to the slope being revegetated, polyethylene sheeting should be draped over the soil cut during construction to protect the slope from saturation during rain events. The polyethylene sheeting will also aid in reviewing if there is groundwater seepage during the construction activities.

- The proposed soil nails are intended to be installed within the glacial soils units. There is potential for some grout loss during the installation due to presence of voids. There is also potential for presence of large diameter particles within the glacial soil unit that pose risk during soil nail installation.

9.0 CLOSURE

We trust this report meets your present requirements. If you have any questions or comments, please contact the undersigned.

Respectfully submitted,
Tetra Tech Canada Inc.

PROFESSIONAL
PROVINCE OF
S. RASHIDI
46535
BRITISH COLUMBIA
ENGINEER
2024-05-16
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PERMIT TO PRACTICE
TETRA TECH CANADA INC.

PERMIT NUMBER: 1001972

SR/TS/EP/sy/cee

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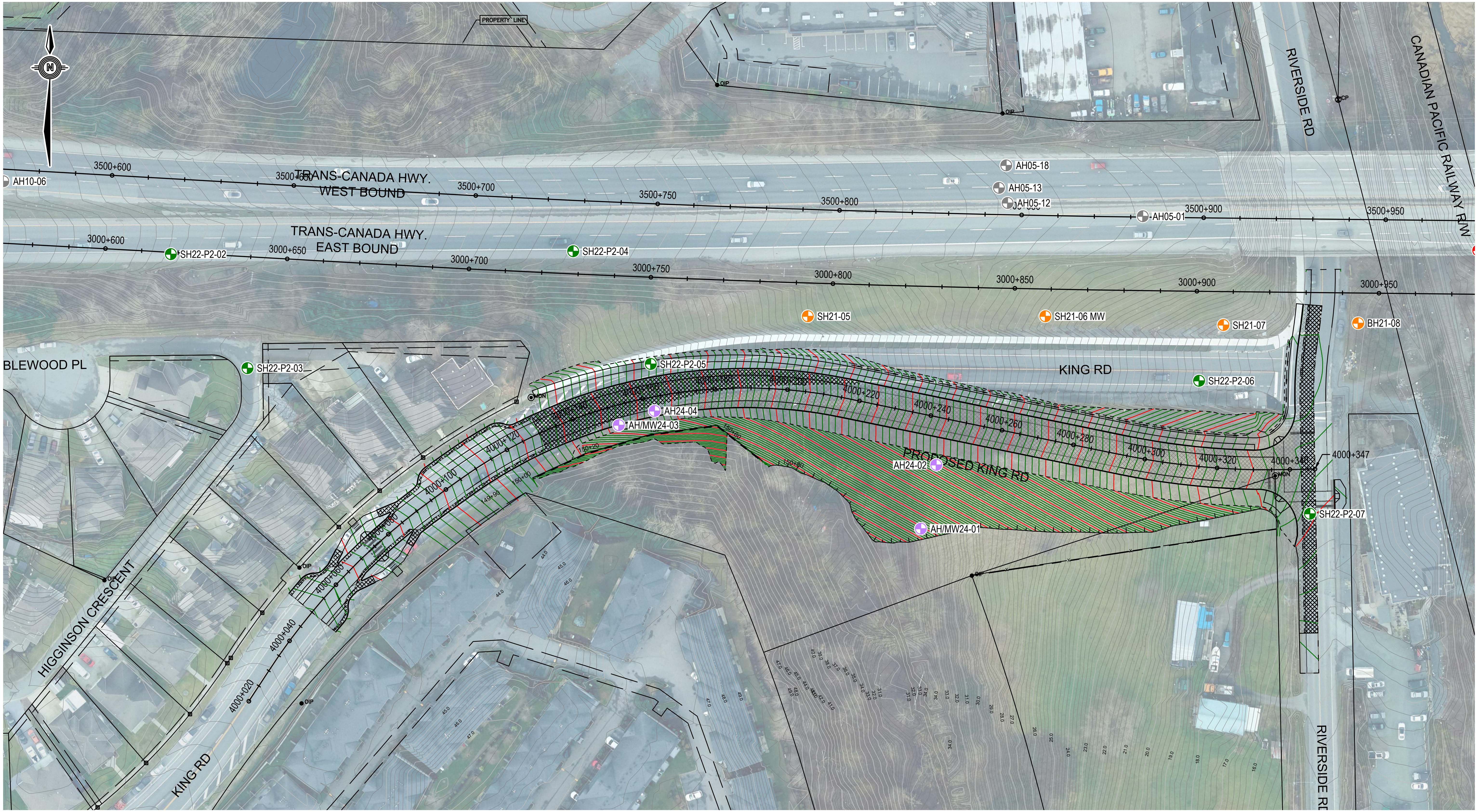
FIGURES



LEGEND

- Testhole by Tetra Tech (2024)
- Testhole by Tetra Tech (2022)
- CPT/SCPT by Tetra Tech (2022)
- Cancelled Testhole by Tetra Tech (2022)
- Cancelled CPT/SCPT by Tetra Tech (2022)
- Testhole by Golder (2021)
- CPT/SCPT by Golder (2021)
- Testpit by Golder (2021)
- Historic Testhole
- Historic Testpit

NOTES

1. Layout based on dwg. no SKT-002-P7-00815 dated June 28, 2023.

0 50m
Scale: 1:1,000 @ 11"x17"

ISSUED FOR USE

CLIENT
BRITISH COLUMBIA
Ministry of Transportation and Infrastructure

TETRA TECH

P7 - KING ROAD REALIGNMENT

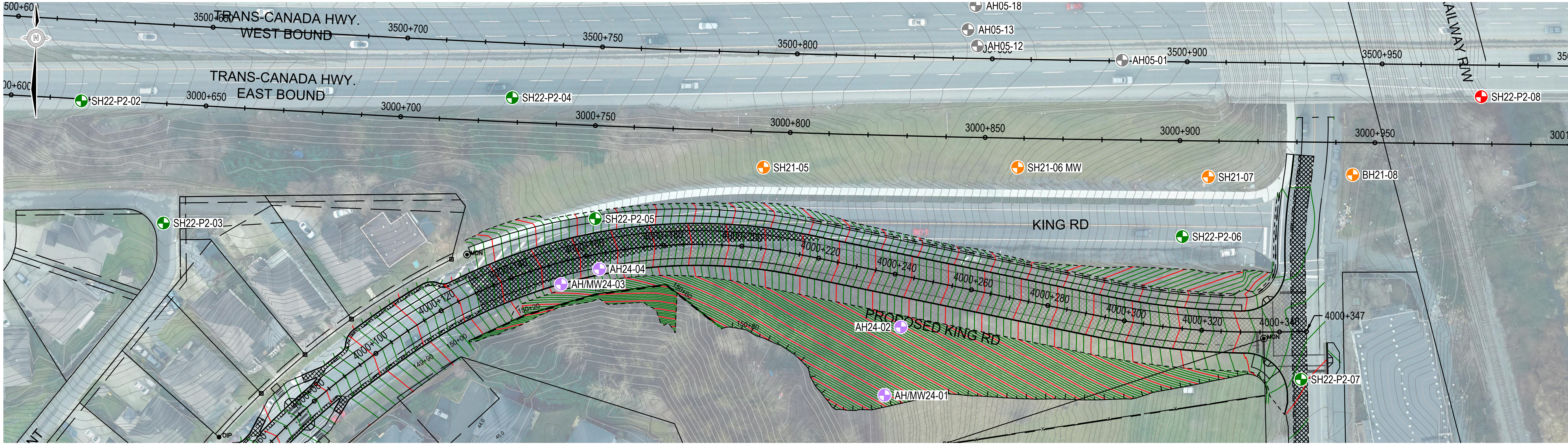
SITE PLAN

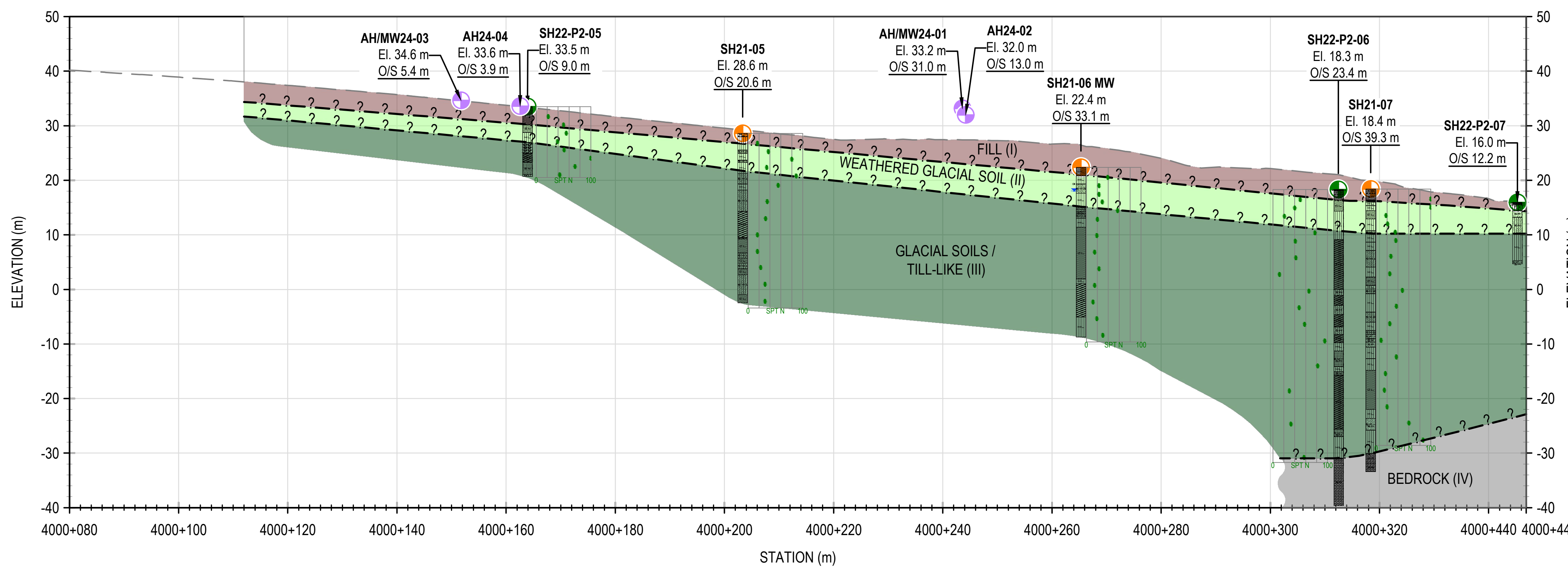
PROJECT NO.	DWN	CKD	REV
ENG.VGEO04080-07	RH	SR	0

OFFICE	DATE
VANC	April 18, 2024

Figure 1

O:\Vancouver\Drafting\Engineering\VGEO\ENG.VGEO04080-07\ENG.VGEO04080-07 2024 P7 King Road Realignment R0d.dwg [DP7KING01] April 18, 2024 - 9:38:33 am (BY: HALL, ROBERT J)





LEGEND
- Existing ground
- Testhole by Tetra Tech (2024)
- Testhole by Tetra Tech (2022)
- CPT/SCPT by Tetra Tech (2022)
- Testhole by Golder (2021)
- CPTs/SCPT by Golder (2021)
- Testpit by Golder (2021)
- Historic Testhole
- Historic Testpit
- Water level

STICKLOG LEGEND
- Fill
- Topsoil
- Peat
- Clay / gravel
- Silt / gravel
- Silt (low plasticity)
- Silt (high plasticity)
- Clay (low plasticity)
- Clay (high plasticity)
- Silty Sands
- Gravel
- Sand (well graded)
- Sand (poorly graded)
- Sand and gravel
- Organic silt
- Bedrock

SOIL LEGEND
- Fill (I)
- Weathered glacial soil (II)
- Glacial soil/till-like (III)
- Bedrock (IV)

NOTES
1. Layout based on dwg. no SKT-002-P7-00815 dated June 28, 2023.

ISSUED FOR USE

0 — 50m
Horizontal and Vertical Scale: 1:1000 @ 11"x17"

CLIENT
British Columbia — Ministry of Transportation and Infrastructure

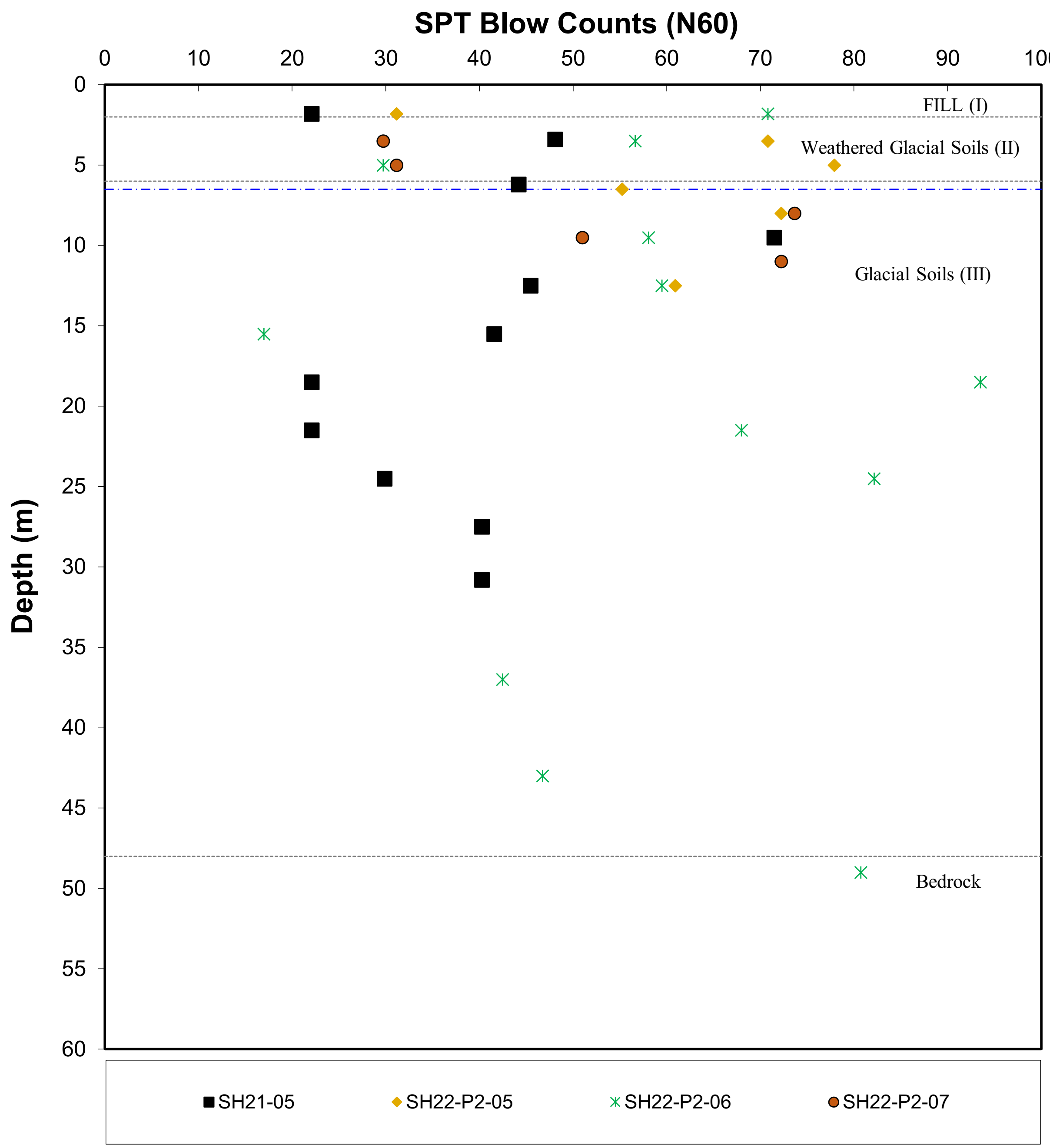
TETRA TECH

P7 - KING ROAD REALIGNMENT

L-4000 KING ROAD PLAN AND PROFILE

PROJECT NO.	DWN	CKD	REV
ENG.VGEO04080-01	RH	SR	0
OFFICE	DATE		
VANC	April 18, 2024		

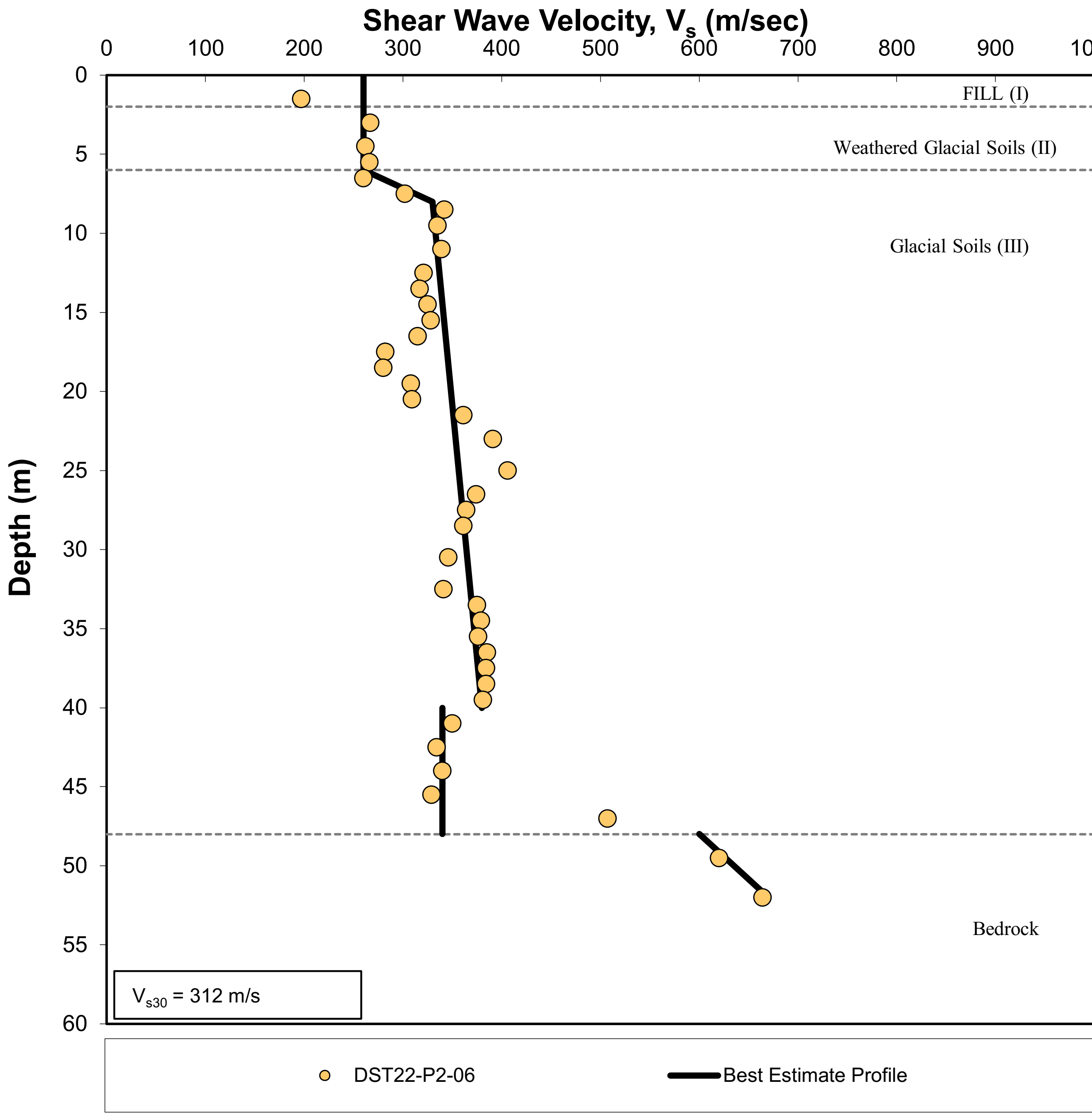
Figure 2



TETRA TECH

SPT BLOW COUNT, N60 PROFILE

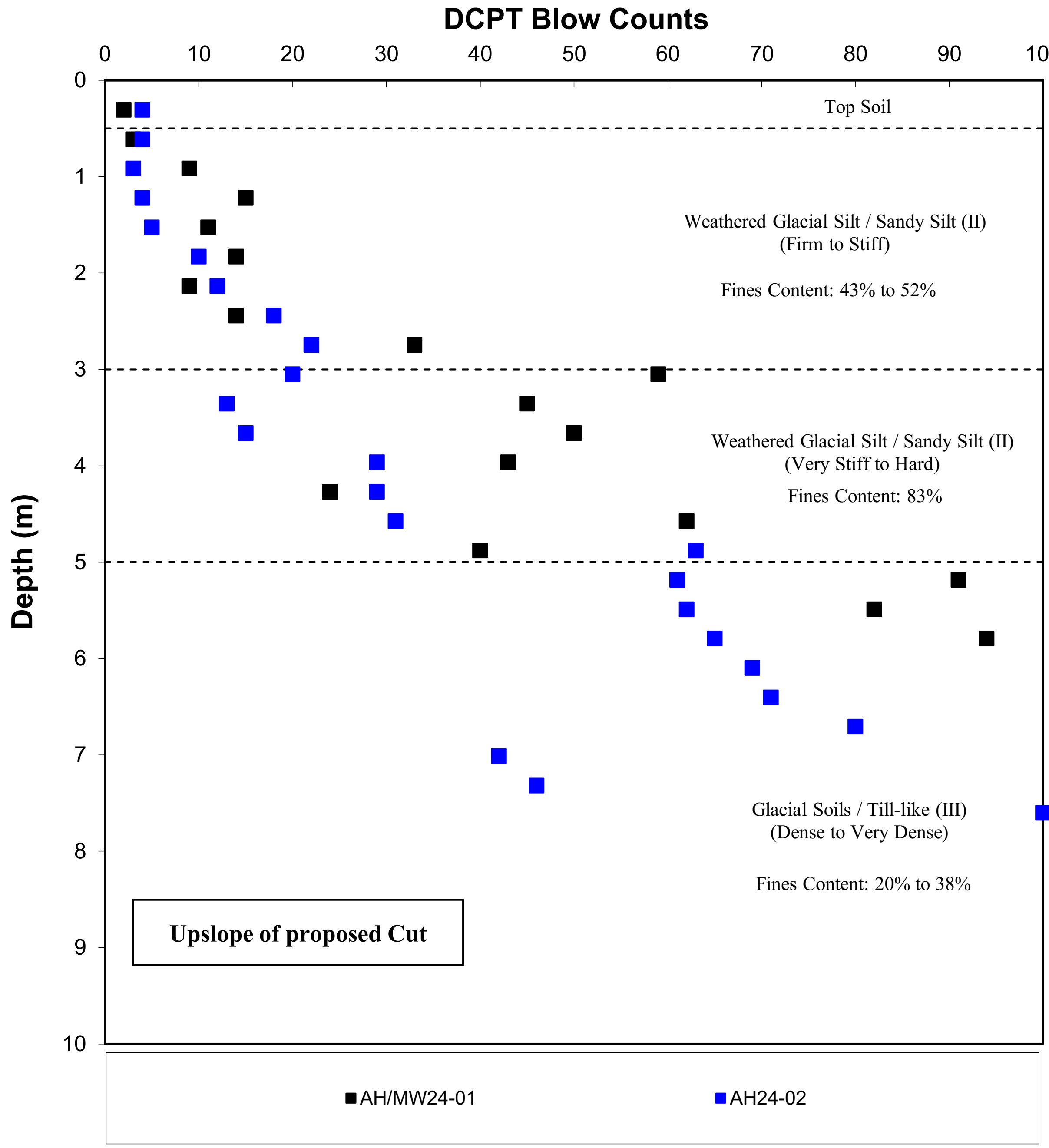
Figure 3



TETRA TECH

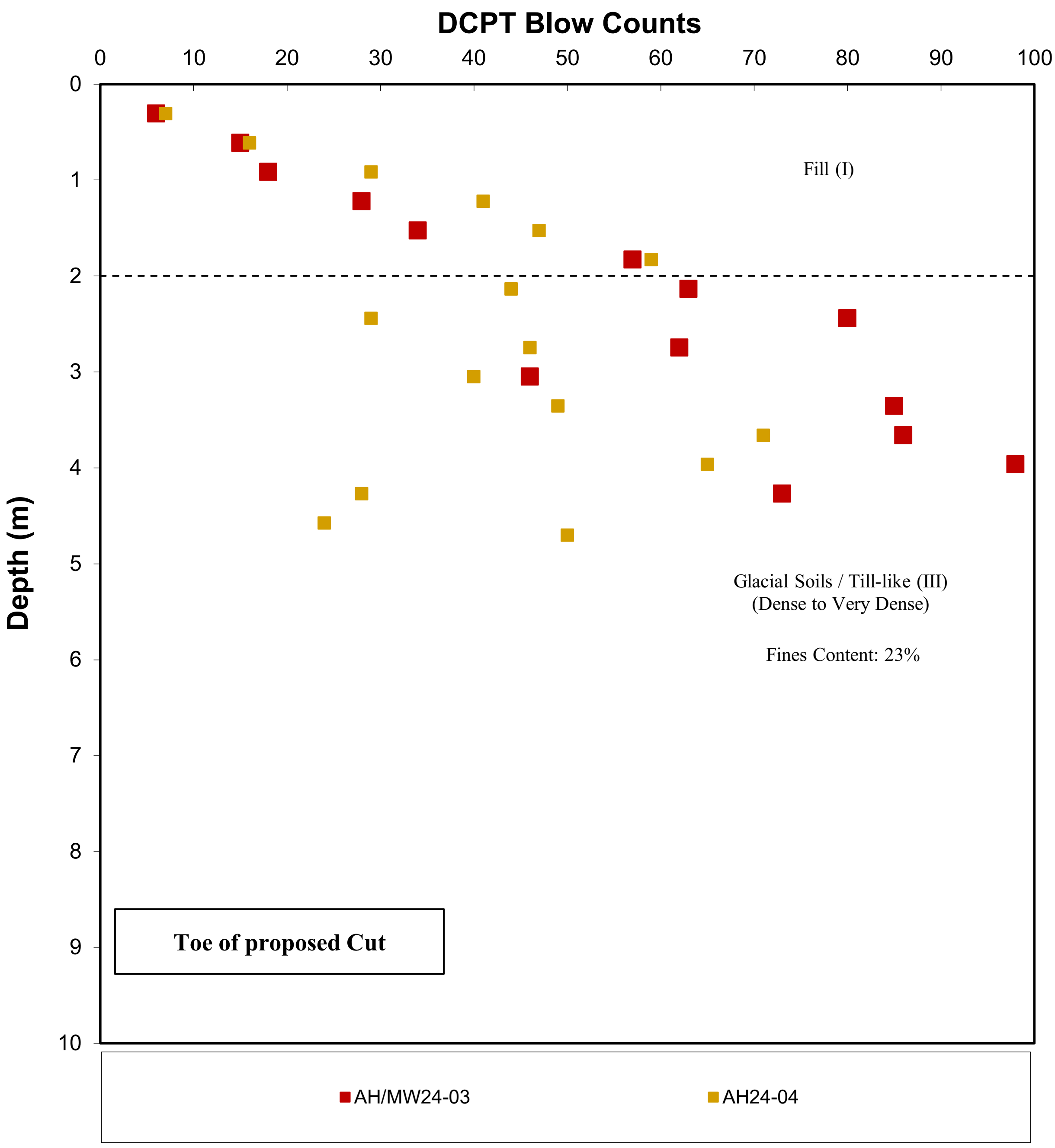
SHEAR WAVE VELOCITY PROFILE

Figure 4



DCPT BLOW COUNT PROFILE
Upslope of Proposed Cut

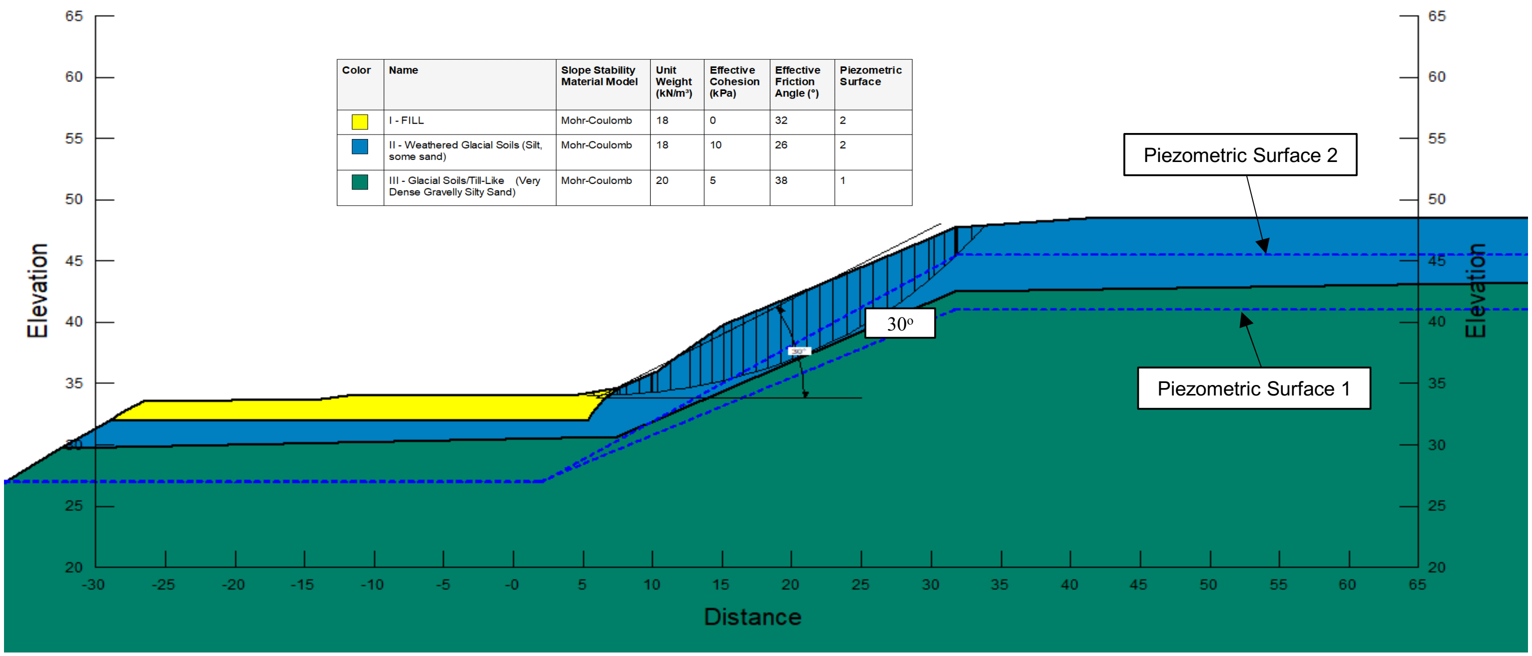
Figure 5a



TETRA TECH

DCPT BLOW COUNT PROFILE
Toe of Proposed Cut

Figure 5b



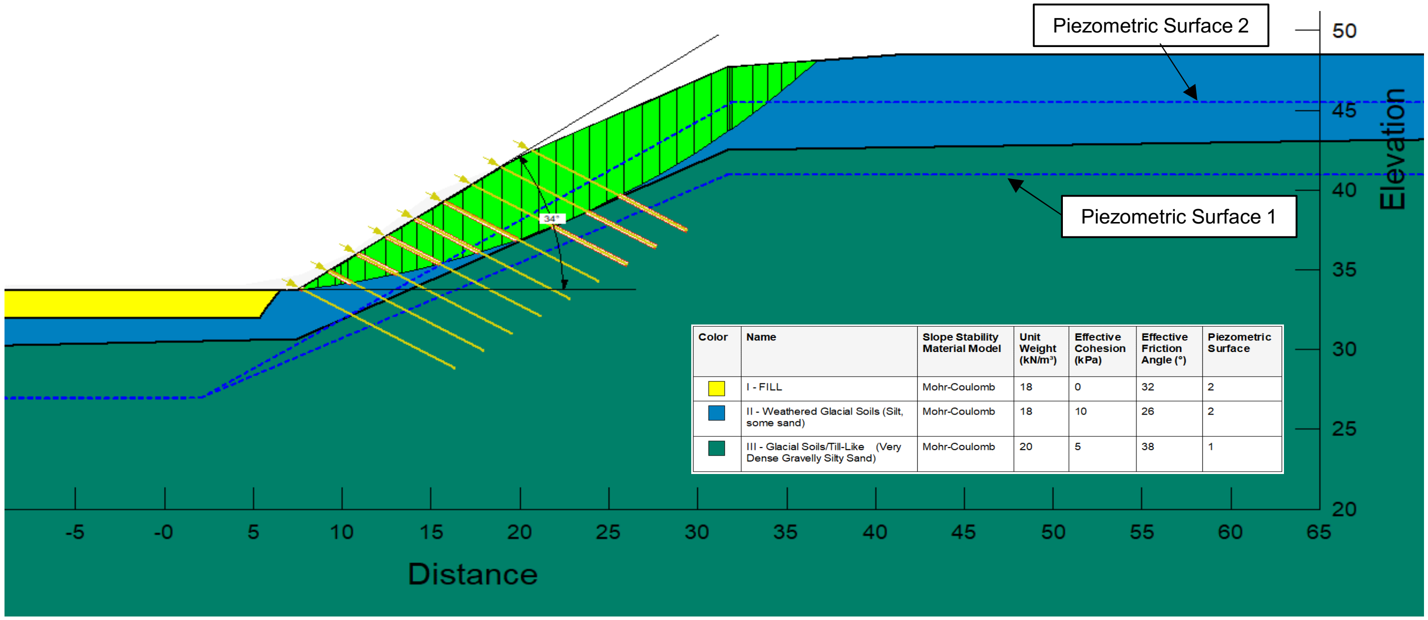
No.	Case	K_h	FoS
1	Static	-	1.35
2	Pseudo-static: 475-Year	0.11	1.07
3	Pseudo-static: 2475-Year	0.21	0.88

Sta. L4001+60

TETRA TECH

GLOBAL STABILITY ANALYSES
Existing Condition - Station L4001+60

Figure 6



No.	Case	K_h	FoS	Comment
1	Existing slope	-	1.35	-
2	Existing slope Pseudo-static: 2475-Year	0.21	0.88	-
3	Proposed - Static	-	1.26	Without soil nail reinforcement
4	Static	-	1.59	-
5	Pseudo-static: 475-Year	0.11	1.38	-
6	Pseudo-static: 2475-Year	0.21	1.11	-
7	Pseudo-static: Yield Acc.	0.26	1.00	Lateral ground deformation < 85mm

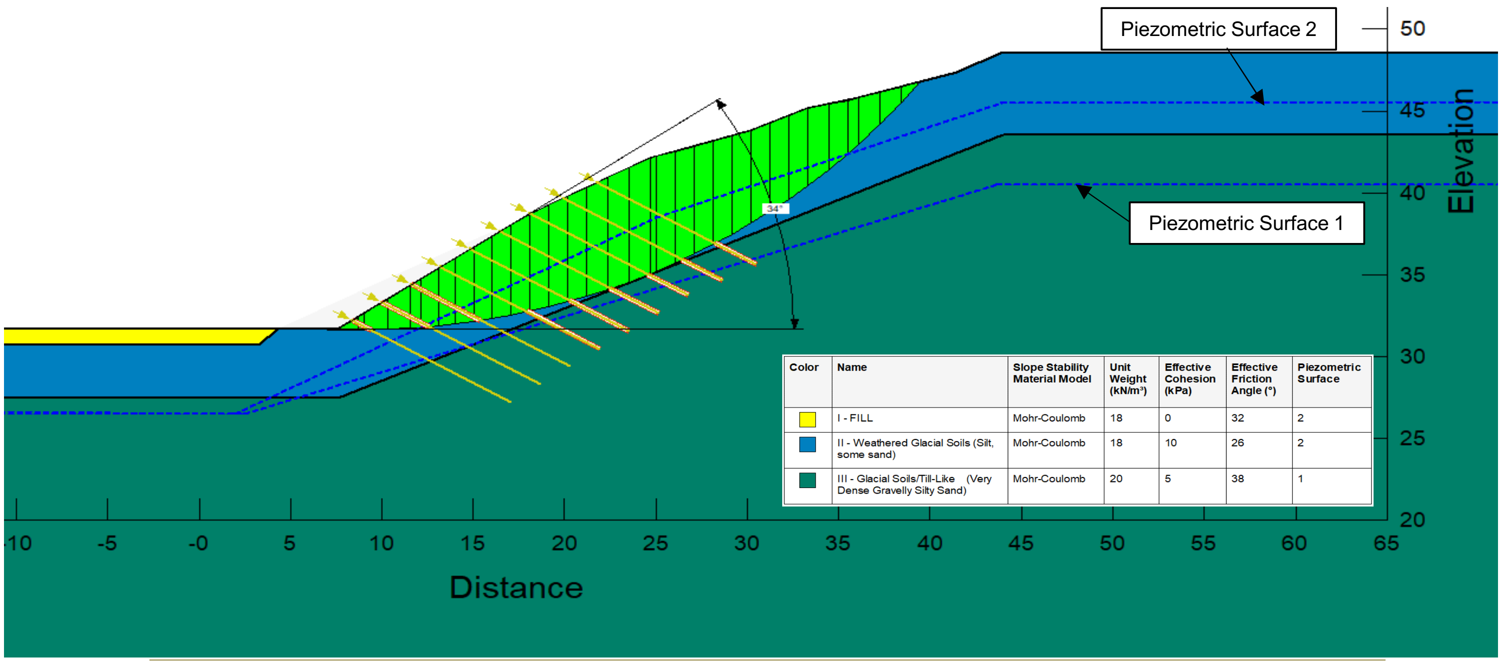
Station L4001+60 (8.2 m 1.5:1V Reinforced Slope)

Notes:
- Soil nails are to be DCP, installed at 1.75 m horizontal and 1.75 m vertical spacing.
- DCP soil nails should have a minimum bond diameter of 152 mm (6 inches) and a minimum length of 10 m based on an ultimate bond strength of 60 kPa.
- Soil nails should have a minimum ultimate load and ultimate strength of 351 kN and 685 MPa, respectively.
- Soil nail should have a minimum yield load and strength of 264 kN and 517 MPa, respectively, equivalent to DCP 25 mm (#8) bar.
- Proof and performance tests shall be completed to a load test of 65 kN (estimated based on the recommendations in FHWA-NHI-14-007).
- Additional full-length soil nails shall be installed to allow for future extraction for long-term inspection and testing.

TETRA TECH

GLOBAL STABILITY ANALYSES
Station L4001+60 (1.5:1V Reinforced Slope)

Figure 7



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	I - FILL	Mohr-Coulomb	18	0	32	2
	II - Weathered Glacial Soils (Silt, some sand)	Mohr-Coulomb	18	10	26	2
	III - Glacial Soils/Till-Like (Very Dense Gravelly Silty Sand)	Mohr-Coulomb	20	5	38	1

No.	Case	K_h	FoS	Comment
1	Existing slope	-	1.40	-
2	Existing slope Pseudo-static: 2475-Year	0.21	0.90	-
3	Proposed - Static	-	1.28	Without soil nail reinforcement
4	Static	-	1.54	-
5	Pseudo-static: 475-Year	0.11	1.24	-
6	Pseudo-static: 2475-Year	0.21	1.00	-
7	Pseudo-static: Yield Acc.	0.21	1.00	Lateral ground deformation < 90 mm

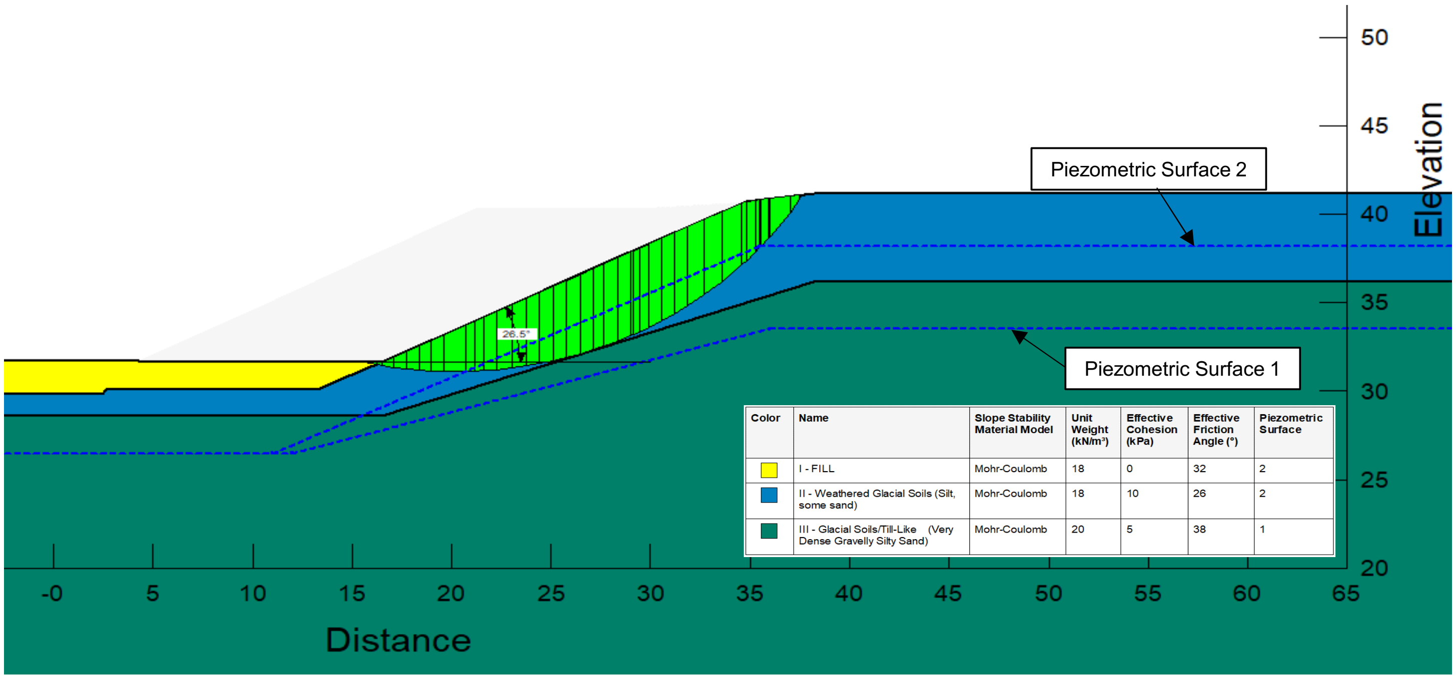
Station L4001+80 (7.0 m 1.5:1V Reinforced Slope)

Notes:

- Soil nails are to be DCP, installed at 1.75 m horizontal and 1.75 m vertical spacing.
- DCP soil nails should have a minimum bond diameter of 152 mm (6 inches) and a minimum length of 10 m based on an ultimate bond strength of 60 kPa.
- Soil nails should have a minimum ultimate load and ultimate strength of 351 kN and 685 MPa, respectively.
- Soil nail should have a minimum yield load and strength of 264 kN and 517 MPa, respectively, equivalent to DCP 25 mm (#8) bar.
- Proof and performance tests shall be completed to a load test of 65 kN (estimated based on the recommendations in FHWA-NHI-14-007).
- Additional full-length soil nails shall be installed to allow for future extraction for long-term inspection and testing.

GLOBAL STABILITY ANALYSES
Station L4001+80 (1.5:1V Reinforced Slope)

Figure 8



No.	Case	K_h	FoS	Comment
1	Existing slope Static	-	> 1.54	-
2	Existing slope Pseudo-static: 2475-Year	0.21	1.06	-
3	Static	-	1.56	-
4	Pseudo-static: 475-Year	0.11	1.26	-
5	Pseudo-static: 2475-Year	0.21	1.05	-
6	Pseudo-static: Yield Acc.	0.25	1.00	Lateral ground deformation < 85 mm

<u>Station L4002+30 (9.1 m 2:1V Cut Slope)</u>

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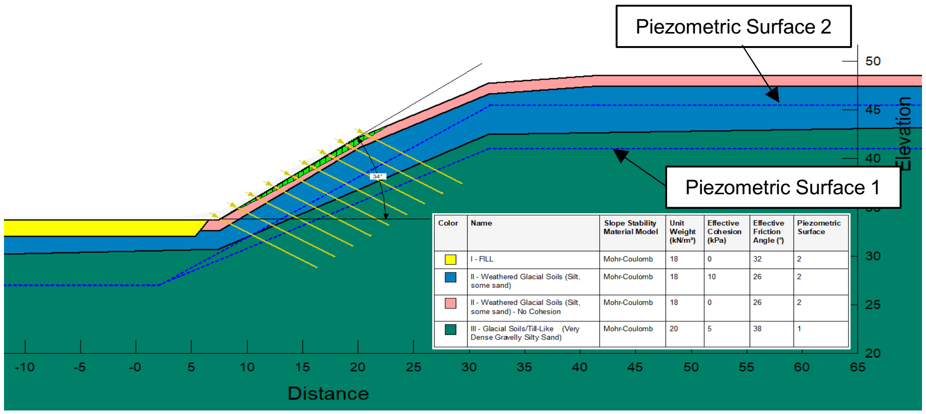
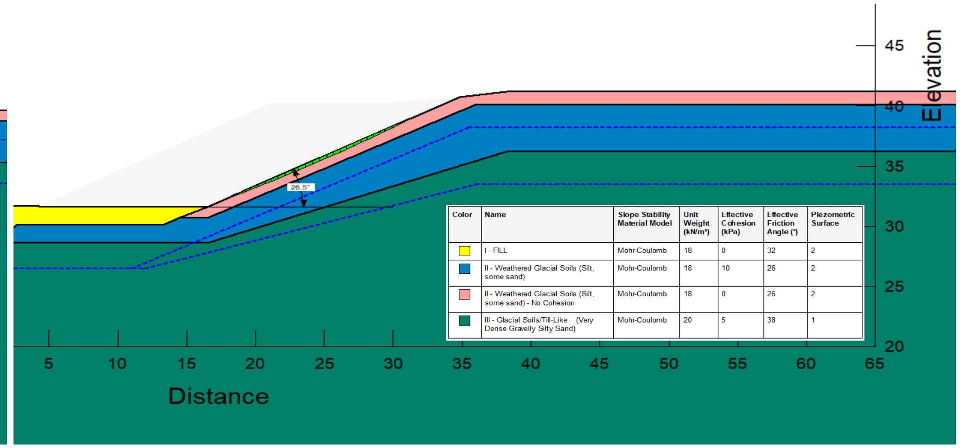
GLOBAL STABILITY ANALYSES
Station L4002+30 (2:1V Cut Slope)

Figure 9



Station L4001+60



Station L4002+30

No.	Case	Case	FoS	Comment
1	L4001+60	Static	1.10	Sensitivity analyses - loss of cohesion in top 1 m due to saturation
2	L4001+80	Static	1.15	
3	L4002+30	Static	0.98	

Note: Failures are limited to surficial layers < 1 m deep

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GLOBAL STABILITY ANALYSES
Sensitivity Analyses

Figure 10

APPENDIX A

TETRA TECH'S LIMITATIONS ON THE USE OF THIS DOCUMENT

LIMITATIONS ON USE OF THIS DOCUMENT

GEOTECHNICAL

1.1 USE OF DOCUMENT AND OWNERSHIP

This document pertains to a specific site, a specific development, and a specific scope of work. The document may include plans, drawings, profiles and other supporting documents that collectively constitute the document (the “Professional Document”).

The Professional Document is intended for the sole use of TETRA TECH’s Client (the “Client”) as specifically identified in the TETRA TECH Services Agreement or other Contractual Agreement entered into with the Client (either of which is termed the “Contract” herein). TETRA TECH does not accept any responsibility for the accuracy of any of the data, analyses, recommendations or other contents of the Professional Document when it is used or relied upon by any party other than the Client, unless authorized in writing by TETRA TECH.

Any unauthorized use of the Professional Document is at the sole risk of the user. TETRA TECH accepts no responsibility whatsoever for any loss or damage where such loss or damage is alleged to be or, is in fact, caused by the unauthorized use of the Professional Document.

Where TETRA TECH has expressly authorized the use of the Professional Document by a third party (an “Authorized Party”), consideration for such authorization is the Authorized Party’s acceptance of these Limitations on Use of this Document as well as any limitations on liability contained in the Contract with the Client (all of which is collectively termed the “Limitations on Liability”). The Authorized Party should carefully review both these Limitations on Use of this Document and the Contract prior to making any use of the Professional Document. Any use made of the Professional Document by an Authorized Party constitutes the Authorized Party’s express acceptance of, and agreement to, the Limitations on Liability.

The Professional Document and any other form or type of data or documents generated by TETRA TECH during the performance of the work are TETRA TECH’s professional work product and shall remain the copyright property of TETRA TECH.

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1.2 ALTERNATIVE DOCUMENT FORMAT

Where TETRA TECH submits electronic file and/or hard copy versions of the Professional Document or any drawings or other project-related documents and deliverables (collectively termed TETRA TECH’s “Instruments of Professional Service”), only the signed and/or sealed versions shall be considered final. The original signed and/or sealed electronic file and/or hard copy version archived by TETRA TECH shall be deemed to be the original. TETRA TECH will archive a protected digital copy of the original signed and/or sealed version for a period of 10 years.

Both electronic file and/or hard copy versions of TETRA TECH’s Instruments of Professional Service shall not, under any circumstances, be altered by any party except TETRA TECH. TETRA TECH’s Instruments of Professional Service will be used only and exactly as submitted by TETRA TECH.

Electronic files submitted by TETRA TECH have been prepared and submitted using specific software and hardware systems. TETRA TECH makes no representation about the compatibility of these files with the Client’s current or future software and hardware systems.

1.3 STANDARD OF CARE

Services performed by TETRA TECH for the Professional Document have been conducted in accordance with the Contract, in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions in the jurisdiction in which the services are provided. Professional judgment has been applied in developing the conclusions and/or recommendations provided in this Professional Document. No warranty or guarantee, express or implied, is made concerning the test results, comments, recommendations, or any other portion of the Professional Document.

If any error or omission is detected by the Client or an Authorized Party, the error or omission must be immediately brought to the attention of TETRA TECH.

1.4 DISCLOSURE OF INFORMATION BY CLIENT

The Client acknowledges that it has fully cooperated with TETRA TECH with respect to the provision of all available information on the past, present, and proposed conditions on the site, including historical information respecting the use of the site. The Client further acknowledges that in order for TETRA TECH to properly provide the services contracted for in the Contract, TETRA TECH has relied upon the Client with respect to both the full disclosure and accuracy of any such information.

1.5 INFORMATION PROVIDED TO TETRA TECH BY OTHERS

During the performance of the work and the preparation of this Professional Document, TETRA TECH may have relied on information provided by third parties other than the Client.

While TETRA TECH endeavours to verify the accuracy of such information, TETRA TECH accepts no responsibility for the accuracy or the reliability of such information even where inaccurate or unreliable information impacts any recommendations, design or other deliverables and causes the Client or an Authorized Party loss or damage.

1.6 GENERAL LIMITATIONS OF DOCUMENT

This Professional Document is based solely on the conditions presented and the data available to TETRA TECH at the time the data were collected in the field or gathered from available databases.

The Client, and any Authorized Party, acknowledges that the Professional Document is based on limited data and that the conclusions, opinions, and recommendations contained in the Professional Document are the result of the application of professional judgment to such limited data.

The Professional Document is not applicable to any other sites, nor should it be relied upon for types of development other than those to which it refers. Any variation from the site conditions present, or variation in assumed conditions which might form the basis of design or recommendations as outlined in this document, at or on the development proposed as of the date of the Professional Document requires a supplementary exploration, investigation, and assessment.

TETRA TECH is neither qualified to, nor is it making, any recommendations with respect to the purchase, sale, investment or development of the property, the decisions on which are the sole responsibility of the Client.

1.7 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, TETRA TECH has not been retained to explore, address or consider and has not explored, addressed or considered any environmental or regulatory issues associated with development on the subject site.

1.8 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems, methods and standards employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. TETRA TECH does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

1.9 LOGS OF TESTHOLES

The testhole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

1.10 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historical environment. TETRA TECH does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional exploration and review may be necessary.

1.11 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

1.12 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

1.13 INFLUENCE OF CONSTRUCTION ACTIVITY

Construction activity can impact structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques, and construction sequence are known.

1.14 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, and the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

1.15 DRAINAGE SYSTEMS

Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function. Where temporary or permanent drainage systems are installed within or around a structure, these systems must protect the structure from loss of ground due to mechanisms such as internal erosion and must be designed so as to assure continued satisfactory performance of the drains. Specific design details regarding the geotechnical aspects of such systems (e.g. bedding material, surrounding soil, soil cover, geotextile type) should be reviewed by the geotechnical engineer to confirm the performance of the system is consistent with the conditions used in the geotechnical design.

1.16 DESIGN PARAMETERS

Bearing capacities for Limit States or Allowable Stress Design, strength/stiffness properties and similar geotechnical design parameters quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition used in this report. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions considered in this report in fact exist at the site.

1.17 SAMPLES

TETRA TECH will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the Client's expense upon written request, otherwise samples will be discarded.

1.18 APPLICABLE CODES, STANDARDS, GUIDELINES & BEST PRACTICE

This document has been prepared based on the applicable codes, standards, guidelines or best practice as identified in the report. Some mandated codes, standards and guidelines (such as ASTM, AASHTO Bridge Design/Construction Codes, Canadian Highway Bridge Design Code, National/Provincial Building Codes) are routinely updated and corrections made. TETRA TECH cannot predict nor be held liable for any such future changes, amendments, errors or omissions in these documents that may have a bearing on the assessment, design or analyses included in this report.

APPENDIX B

DRAWINGS PROVIDED BY MCELHANNEY

HIGHWAY 1 - CORRIDOR IMPROVEMENTS
KING ROAD & RIVERSIDE ROAD
Cross-Section KEY

ISSUED FOR
INTERNAL USE ONLY
DATE: April 24, 2024

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BRITISH COLUMBIA

MINISTRY OF TRANSPORTATION
AND INFRASTRUCTURE
SOUTH COAST REGION
HIGHWAY ENGINEERING AND GEOMATICS

SCALE: 0 1:500 25

ORIGINAL DWG SIZE: ANSI D (22" x 34")

PROJECT NUMBER: 2121-00815-07

DATE: 2023-11-01

DRAWING NUMBER: SKT-005-P7-00815 __

REV: B

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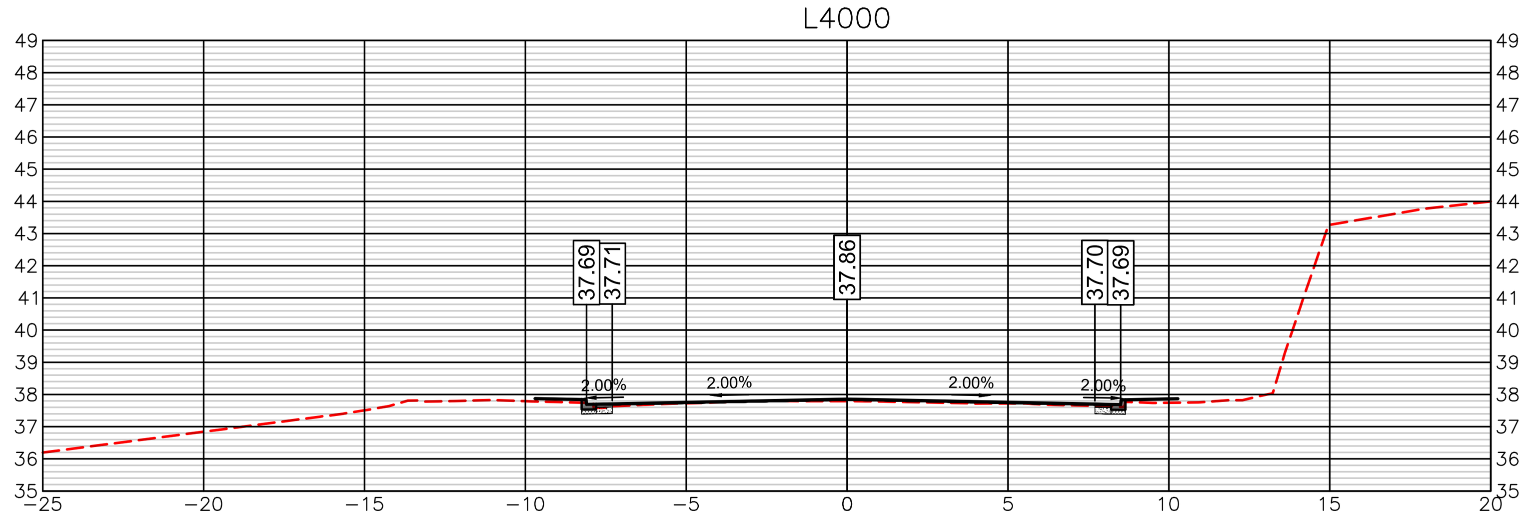

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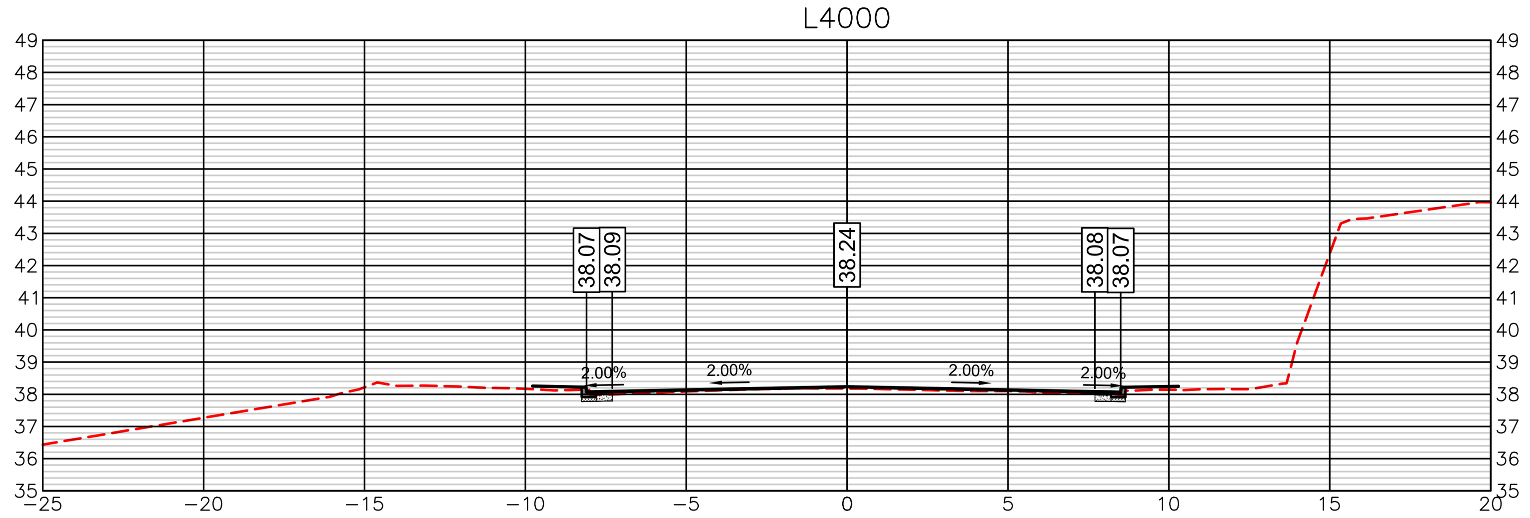

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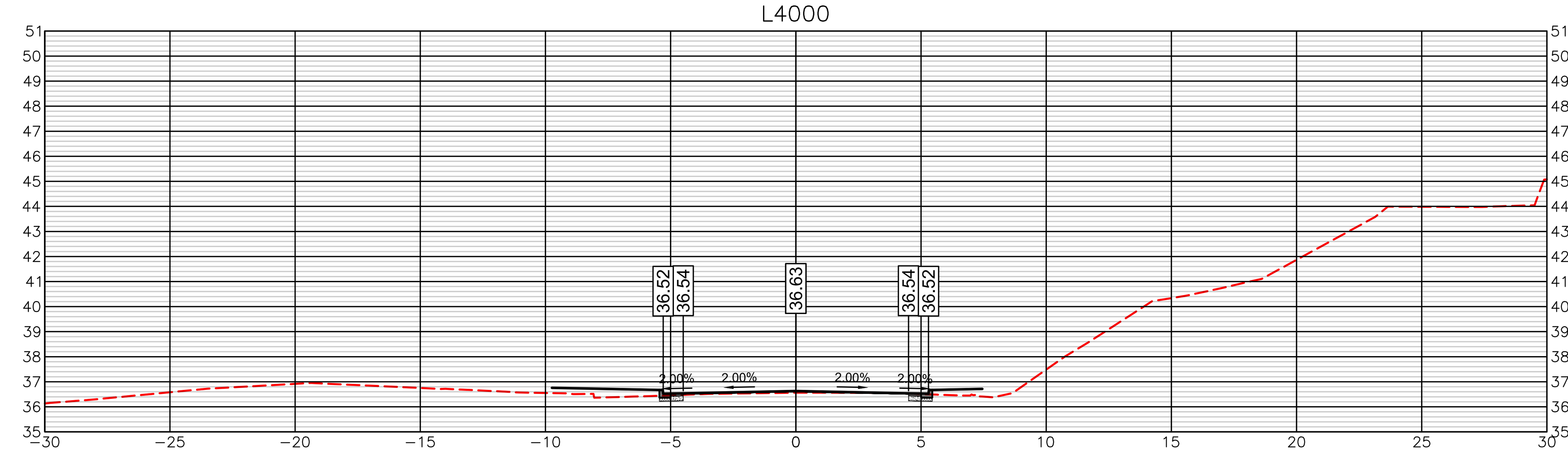

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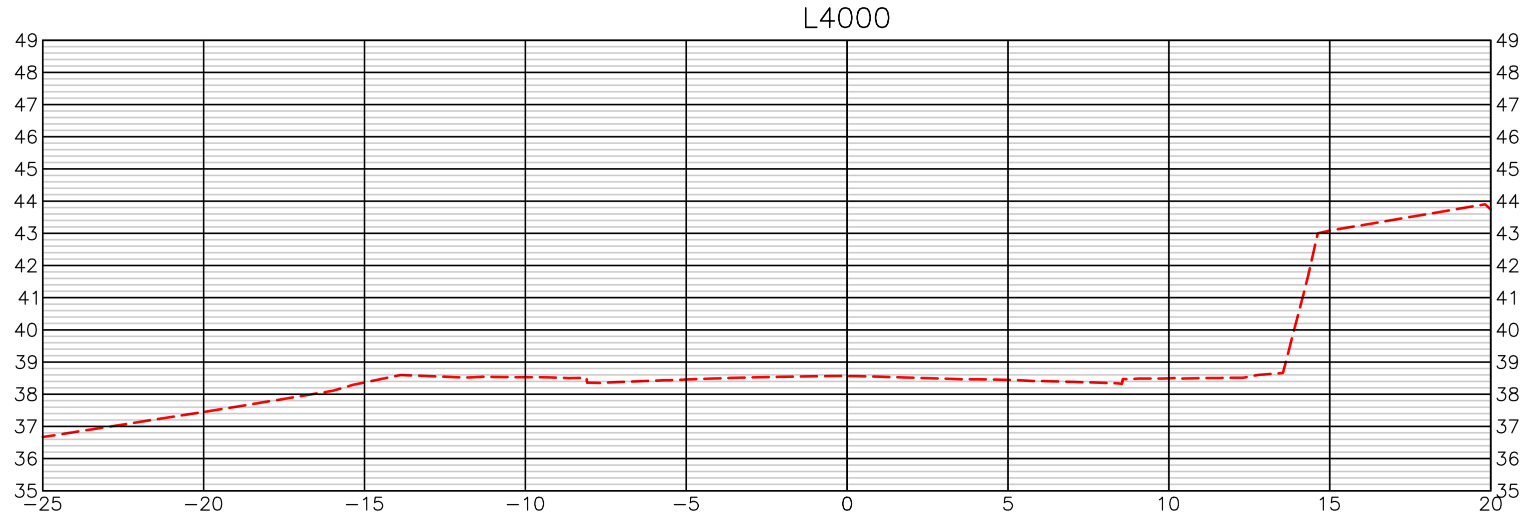

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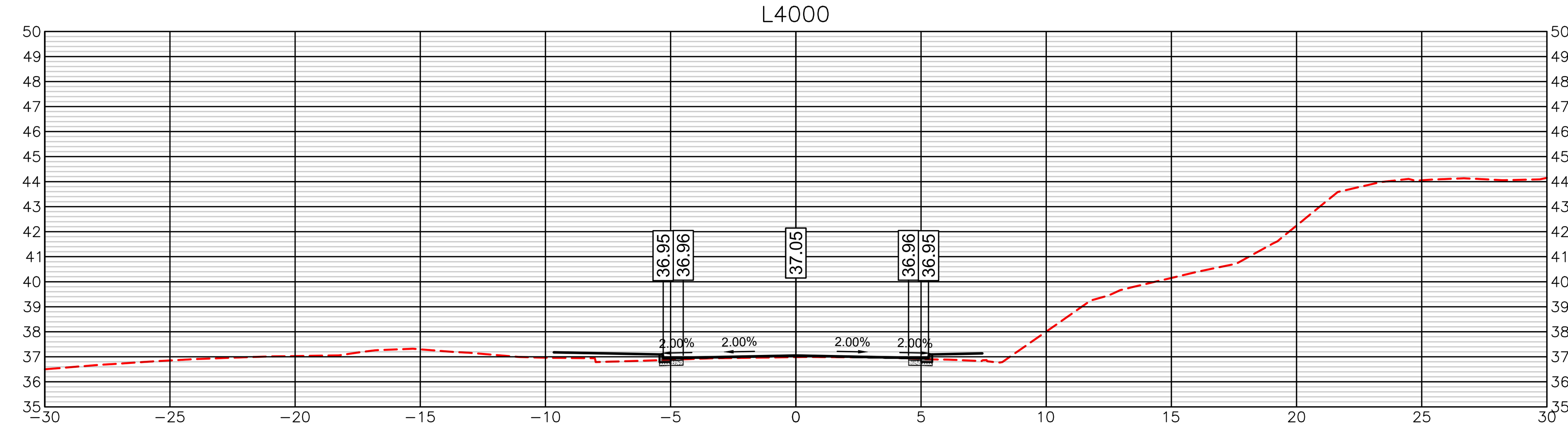

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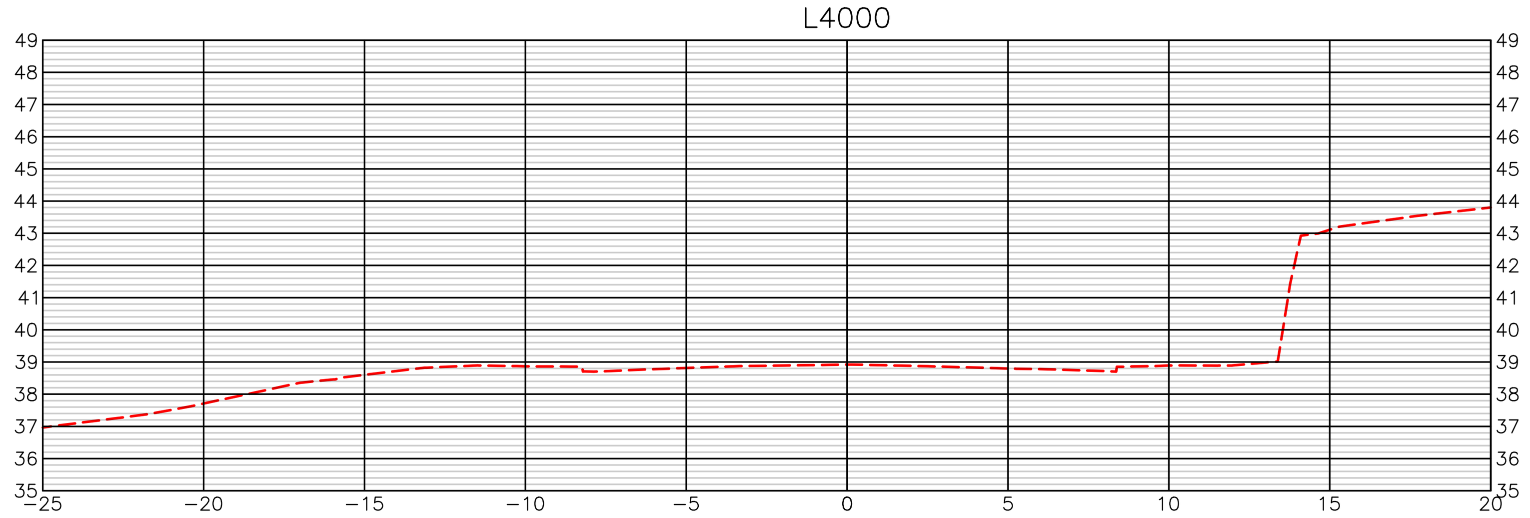

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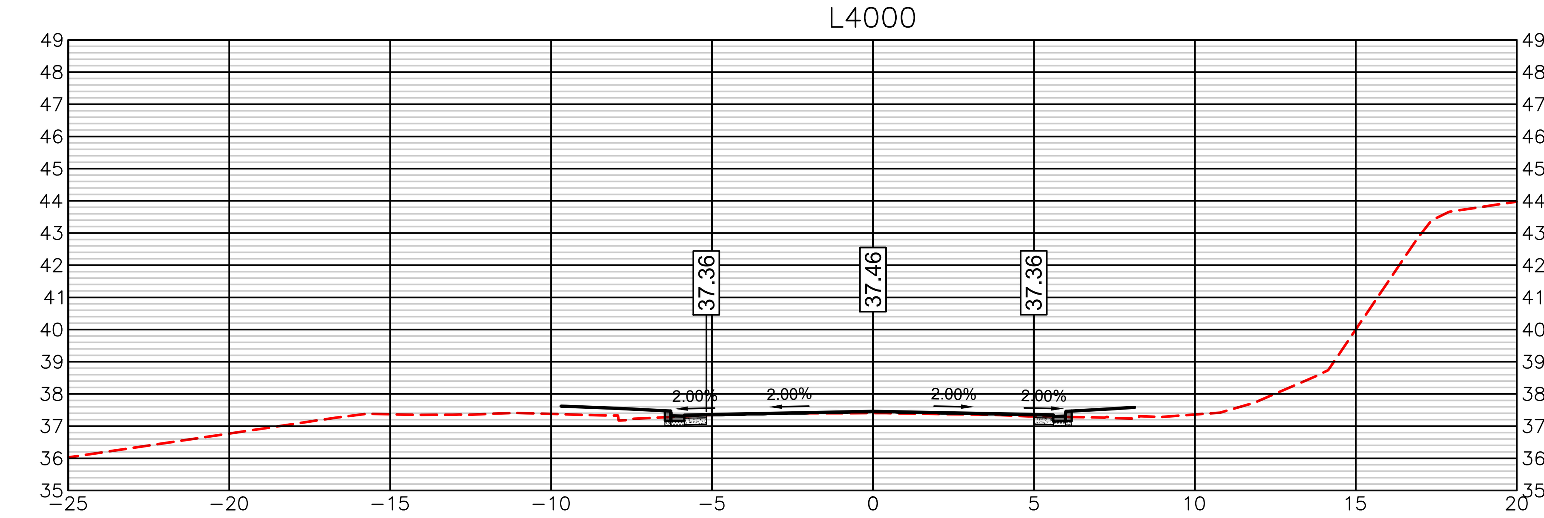

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4001+45
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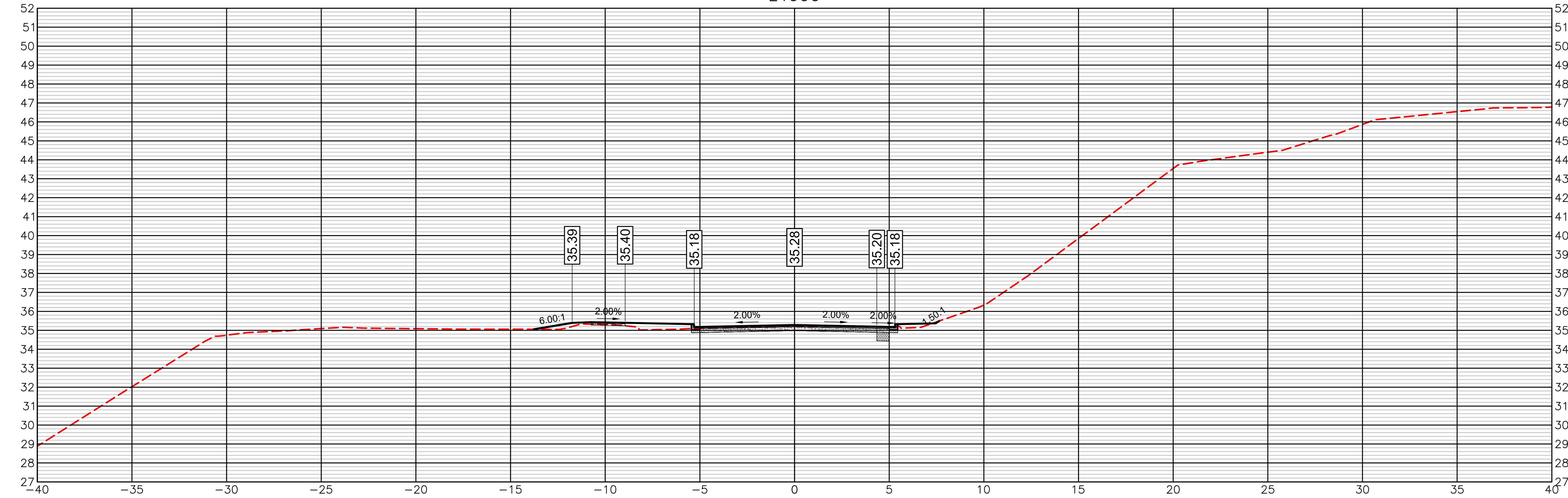

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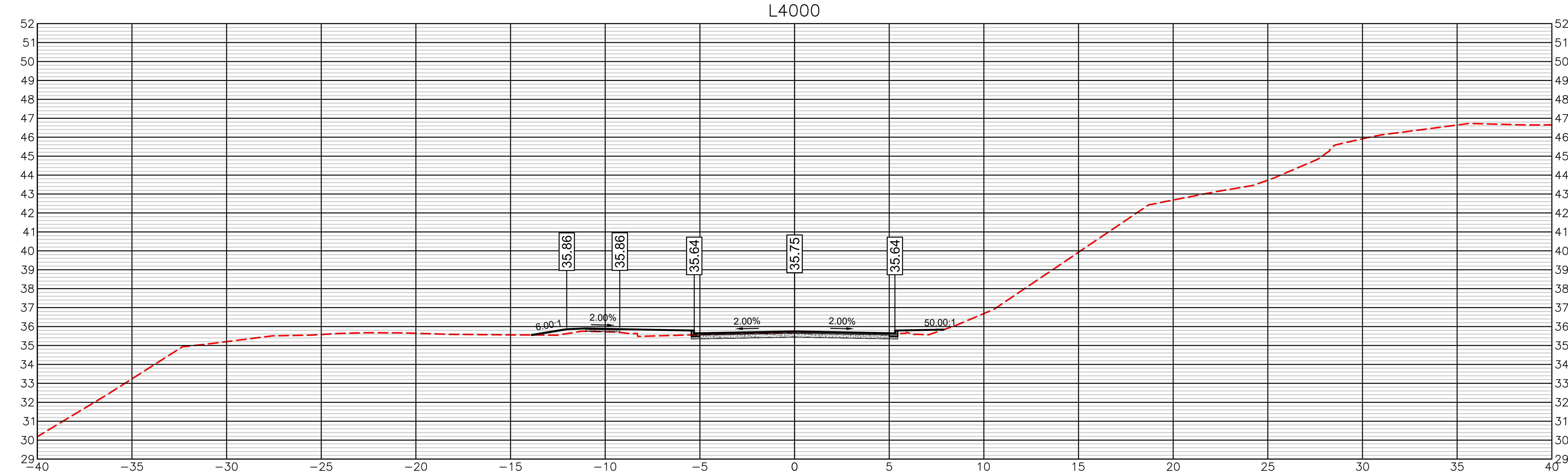

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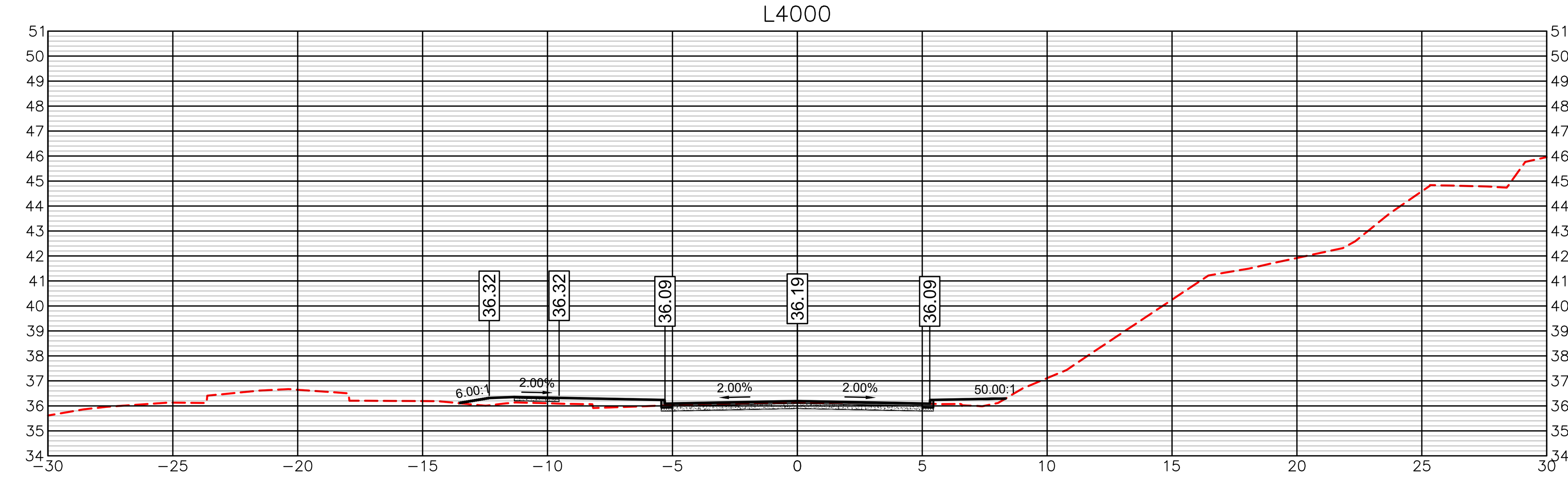

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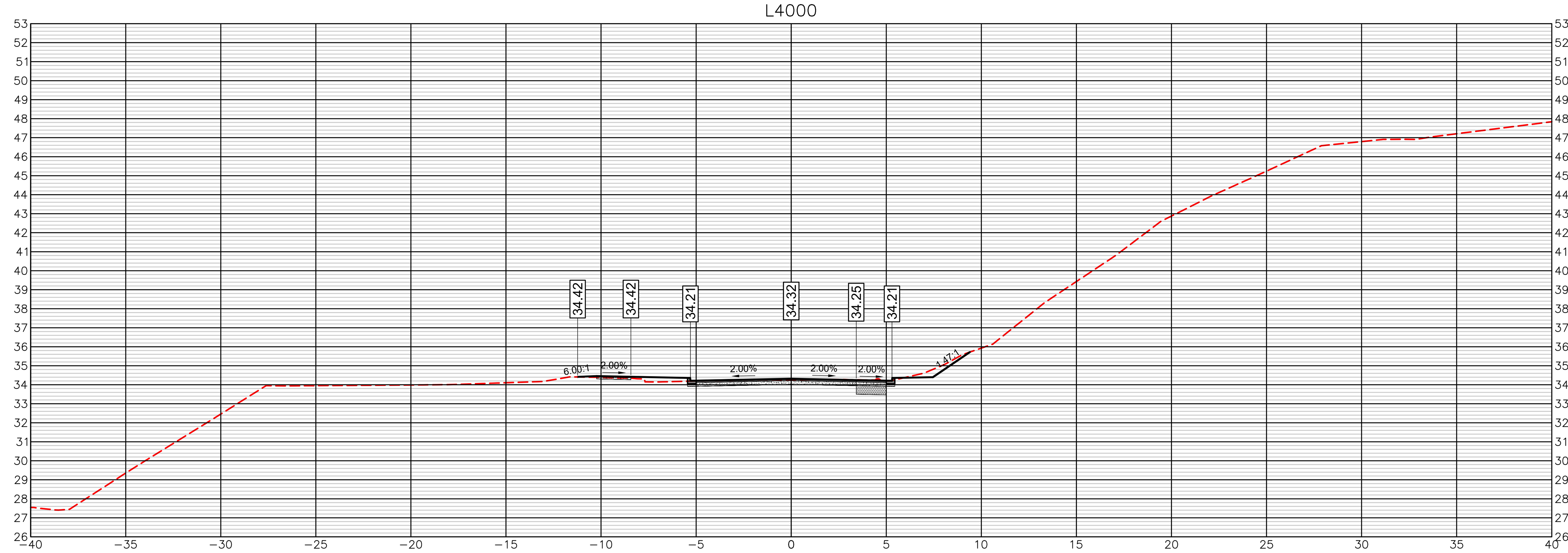

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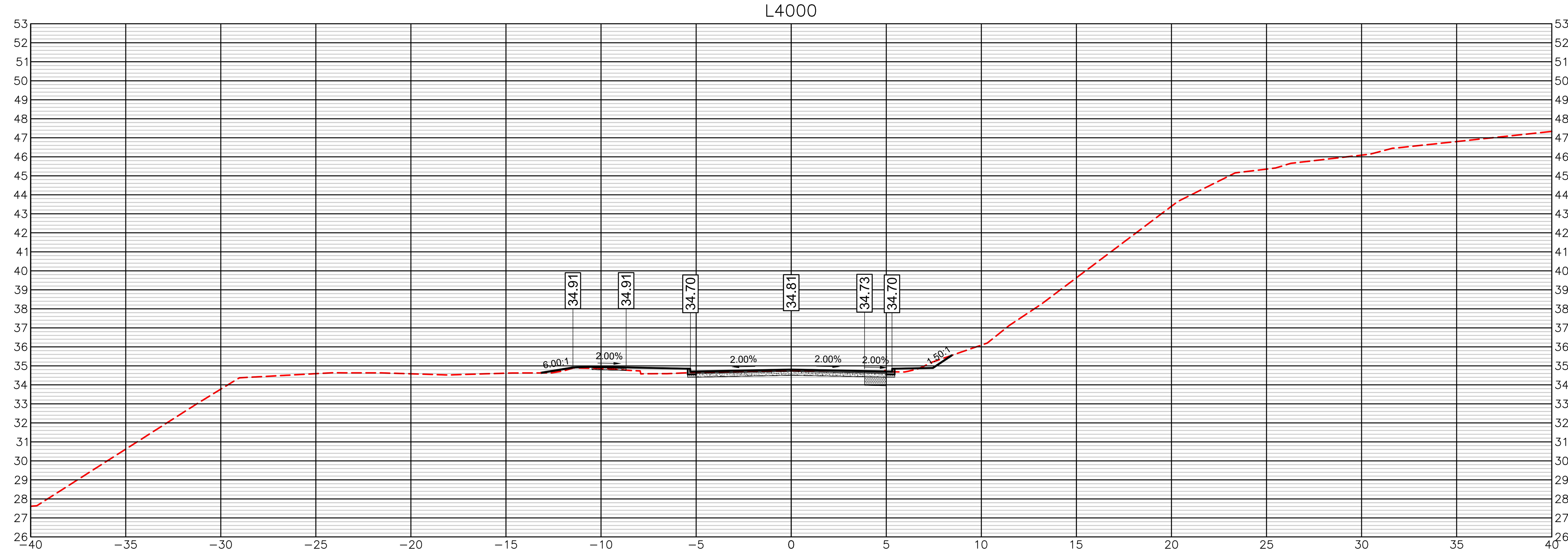

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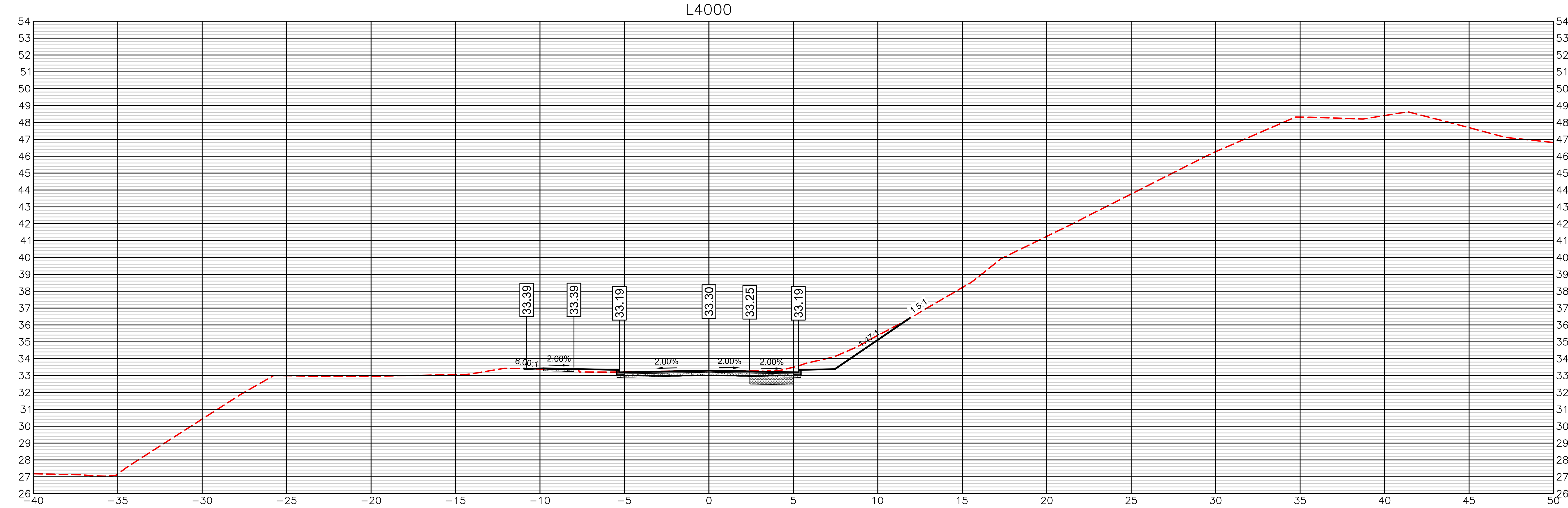

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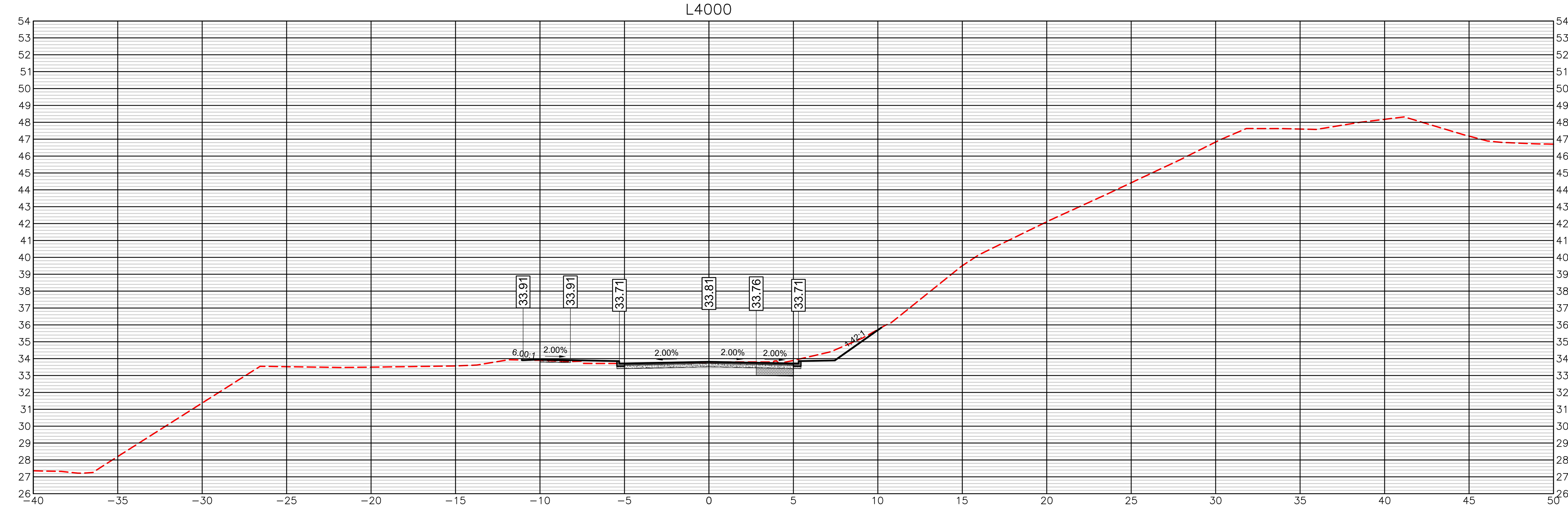

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4001+75
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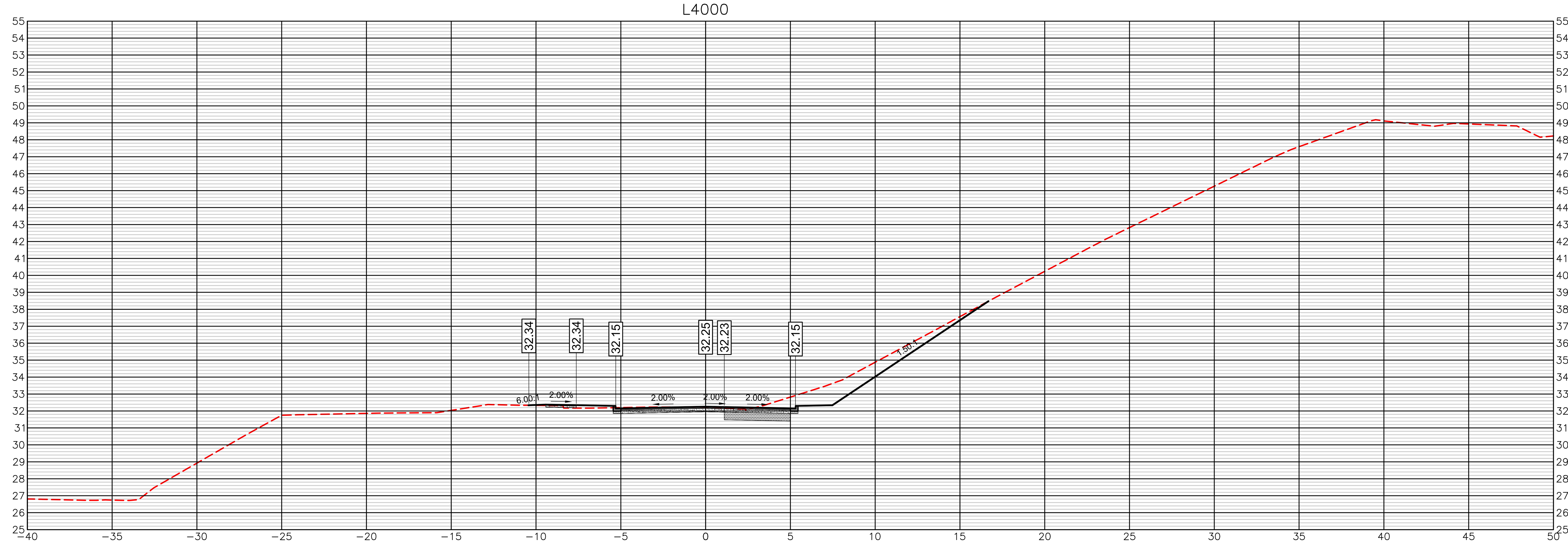

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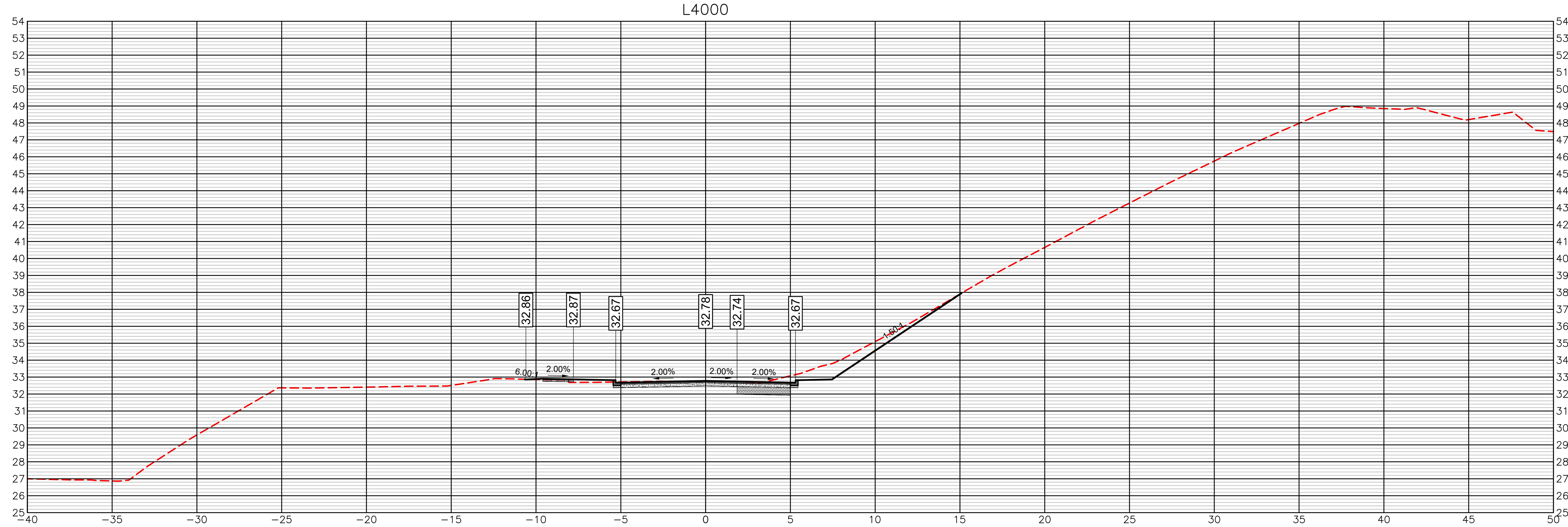

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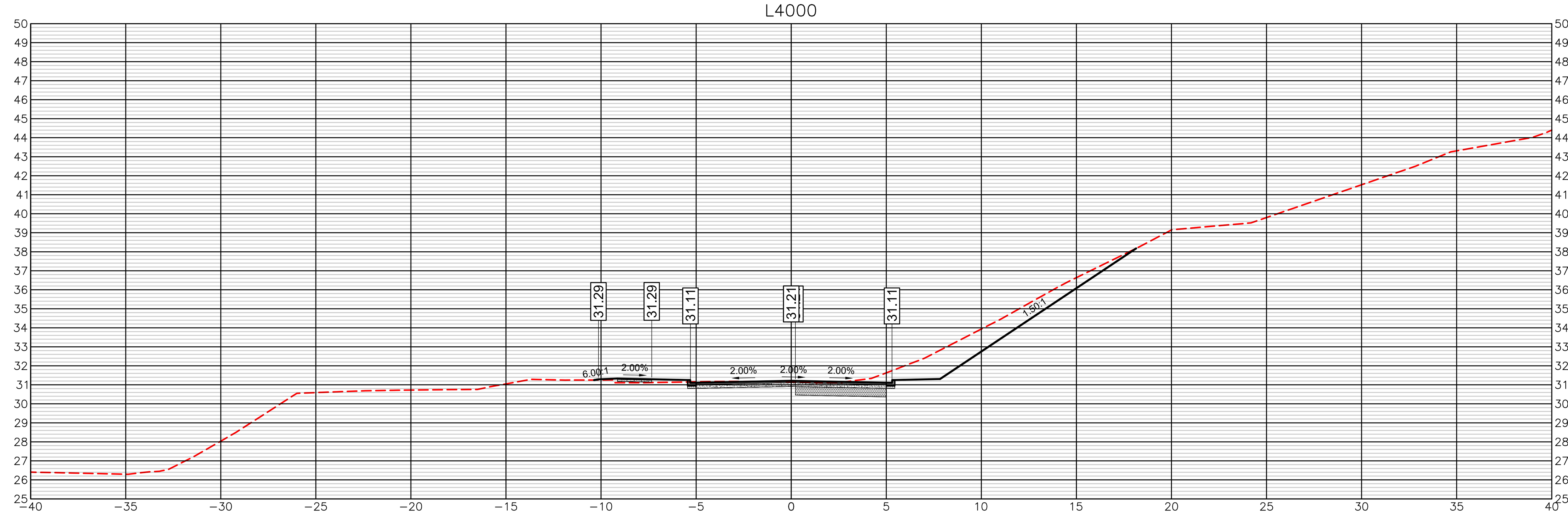

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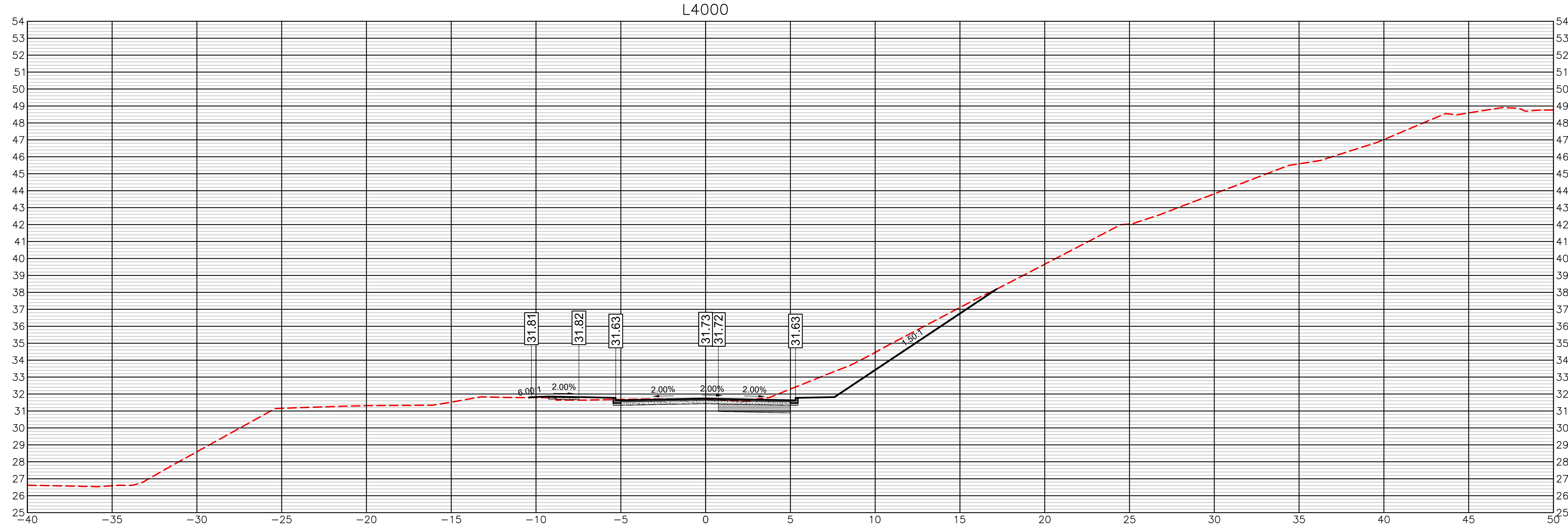

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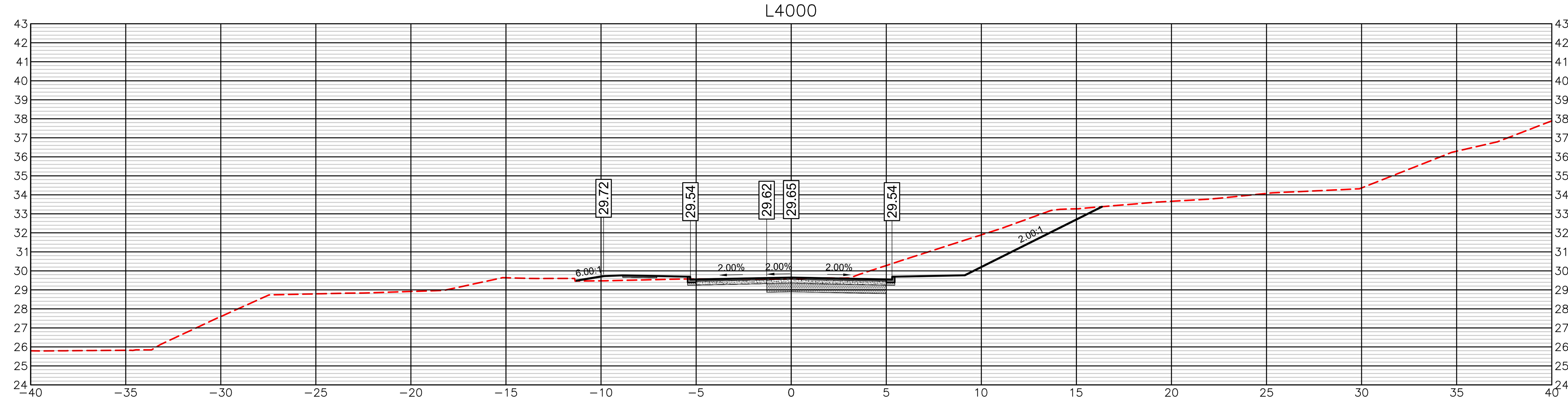
4002+00
L4000



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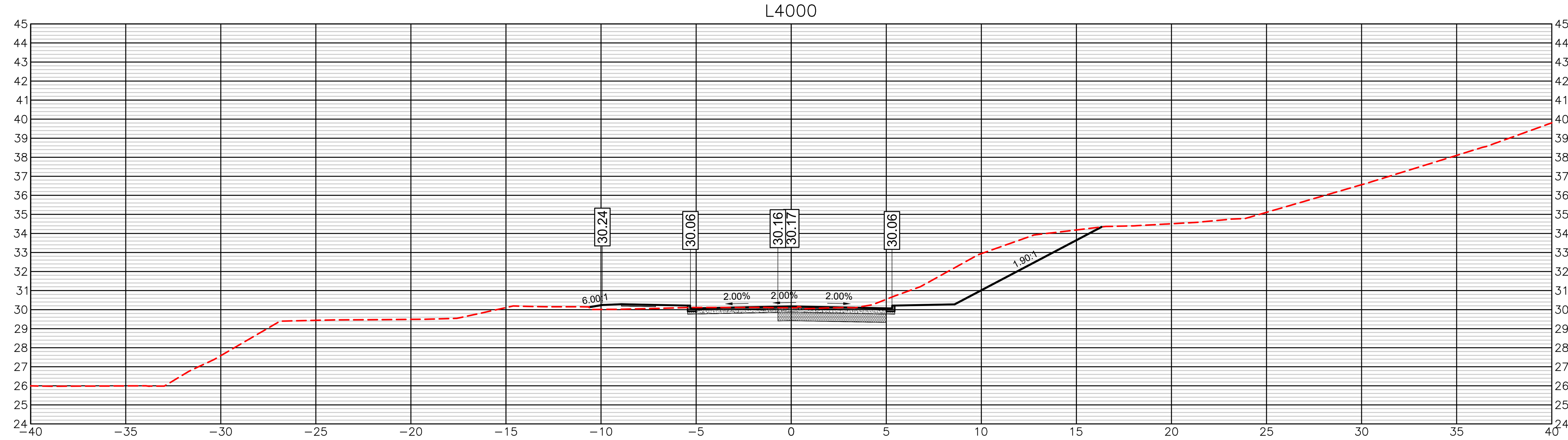

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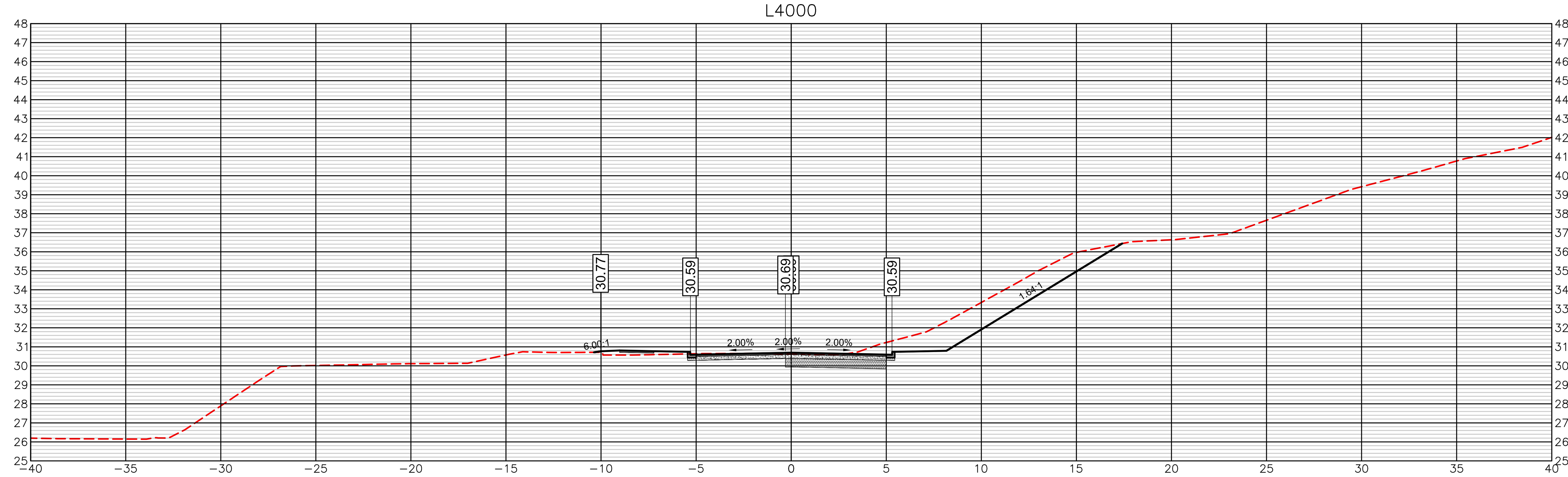

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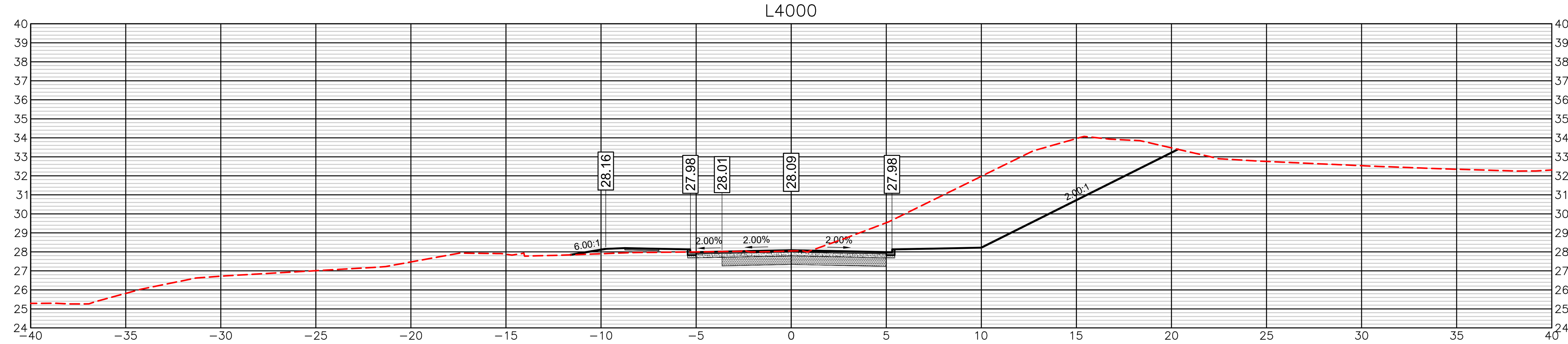

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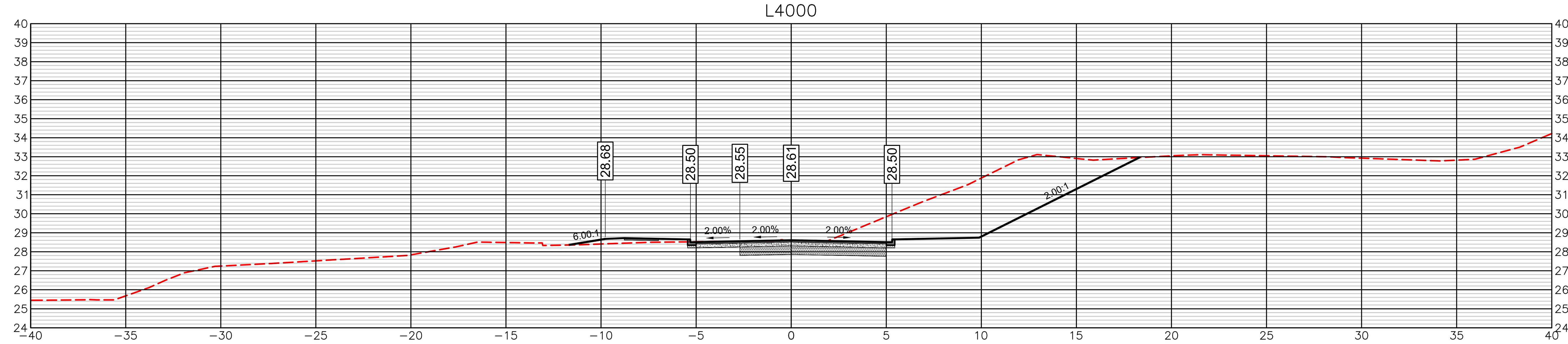

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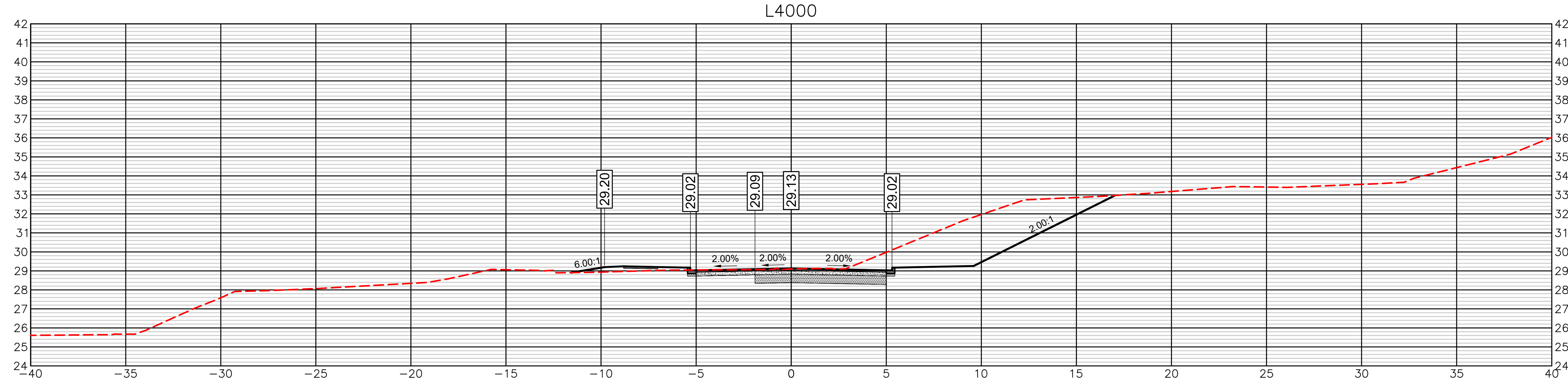

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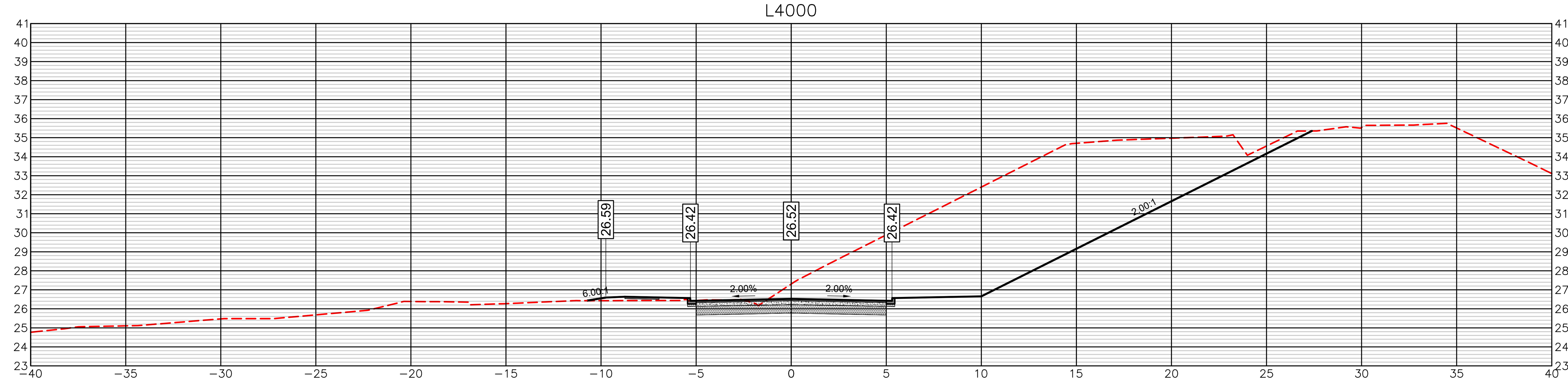

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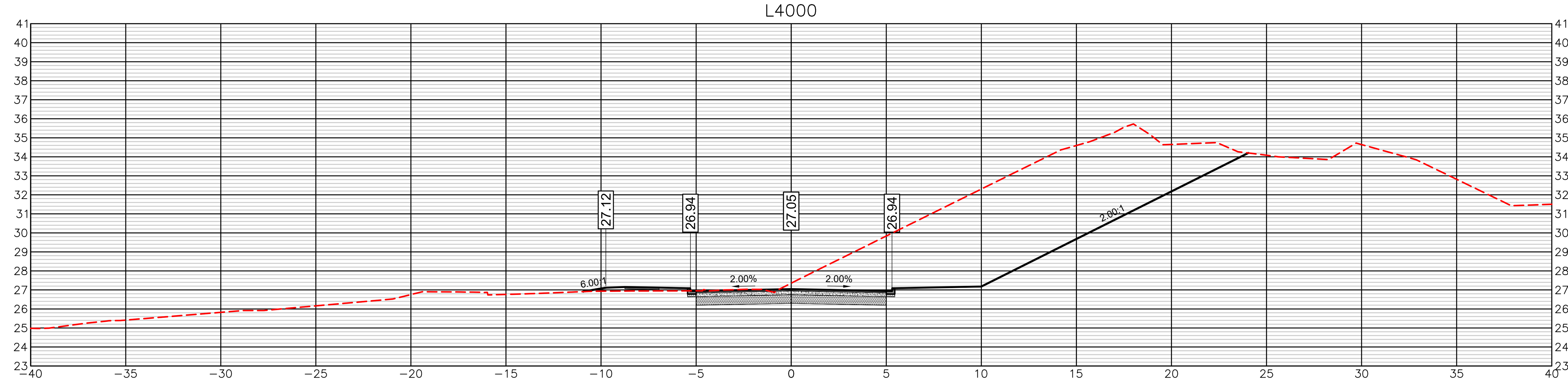

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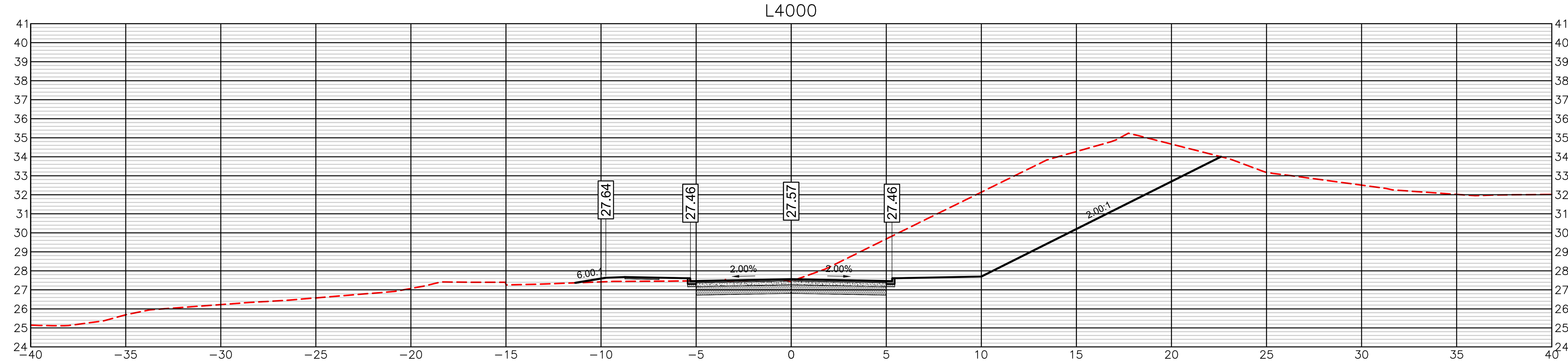

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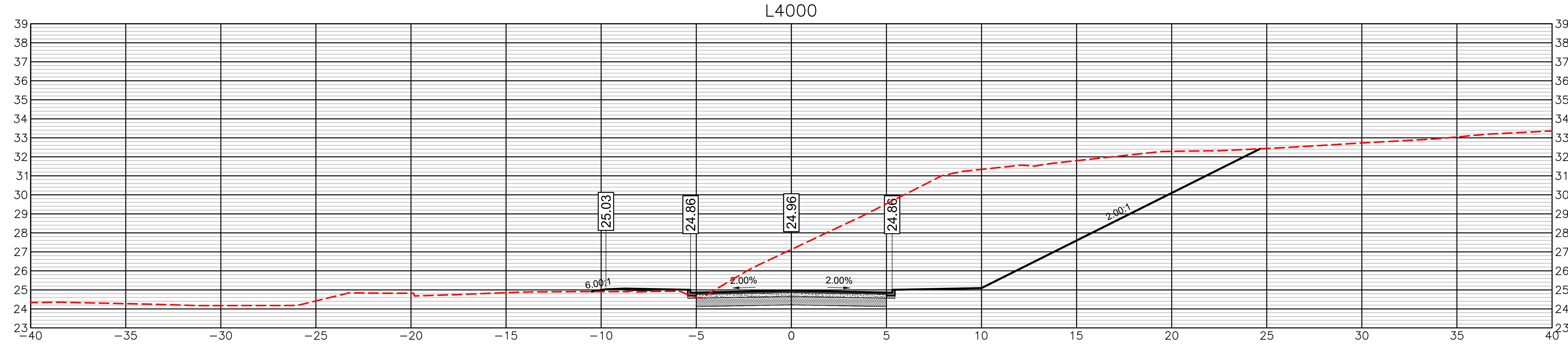

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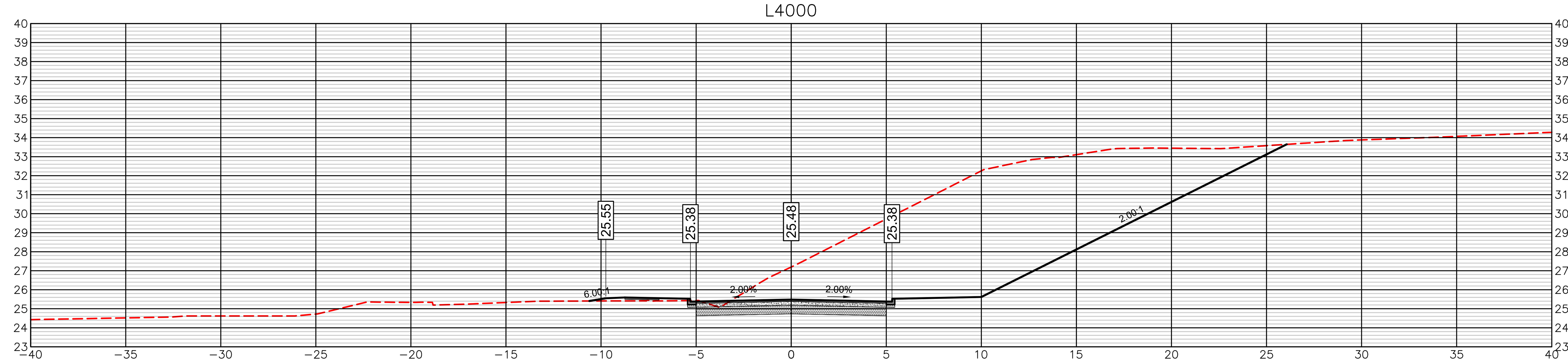

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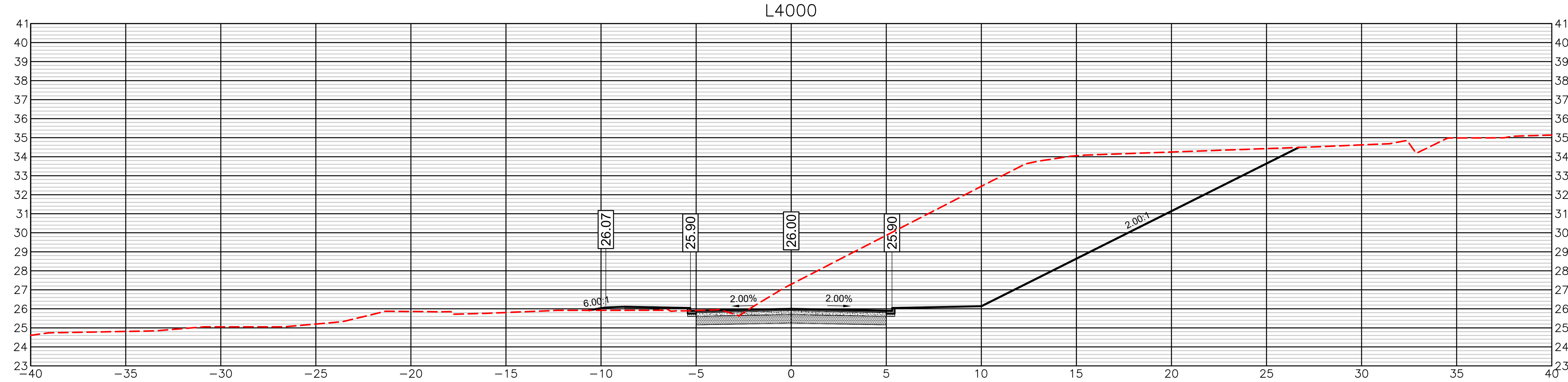

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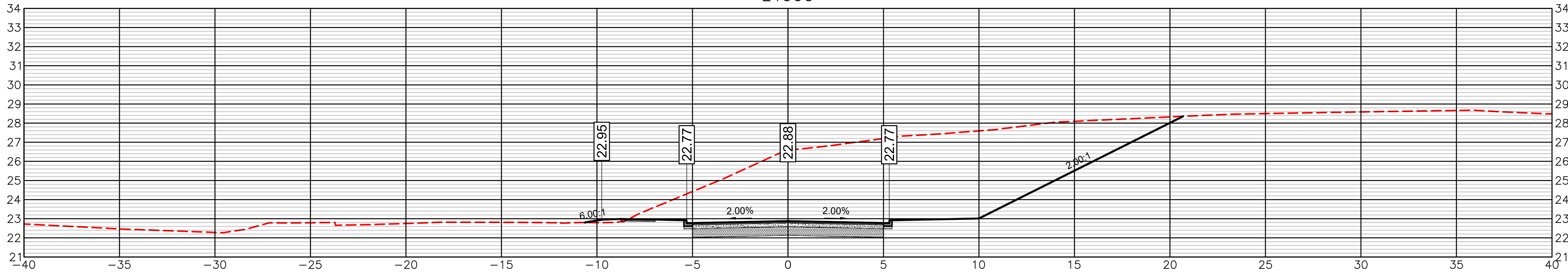

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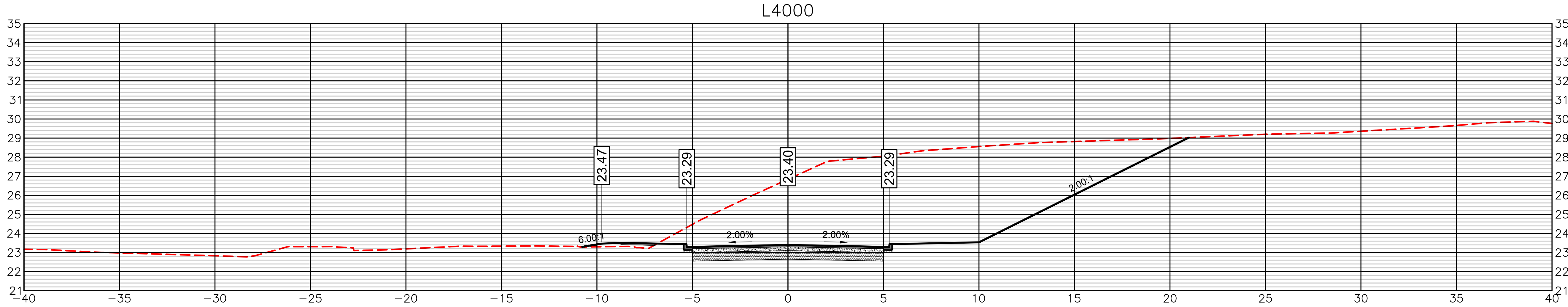

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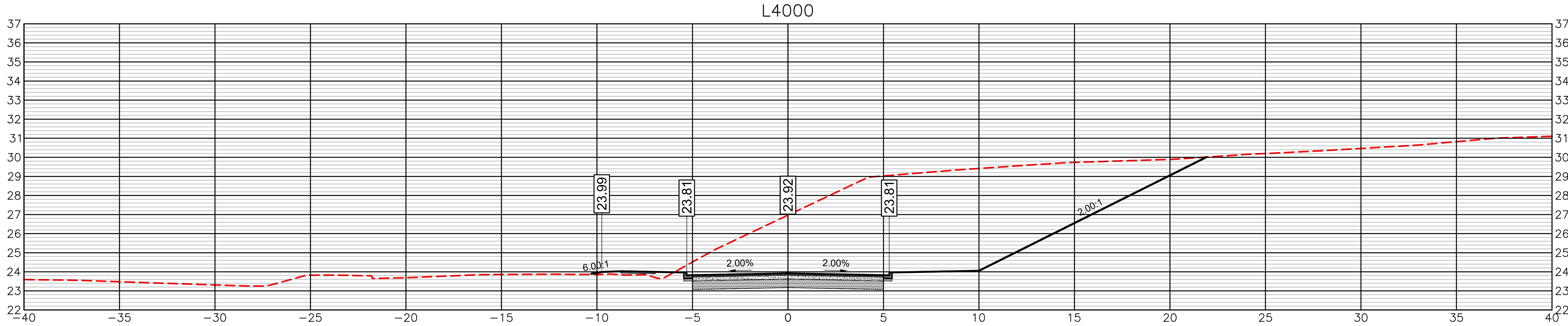

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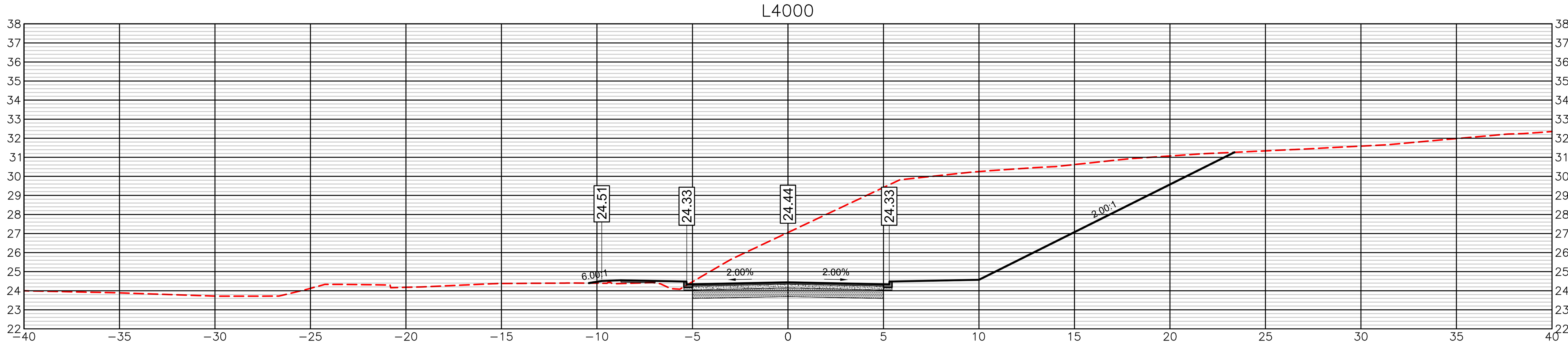

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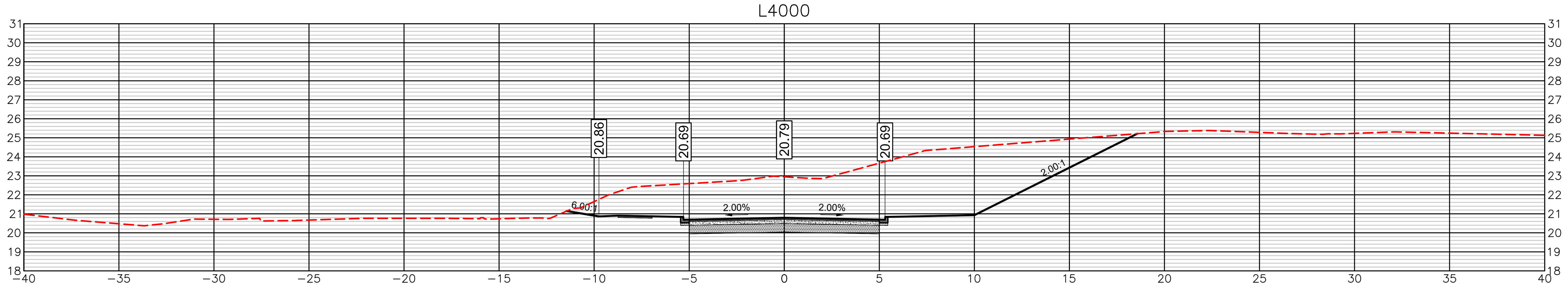

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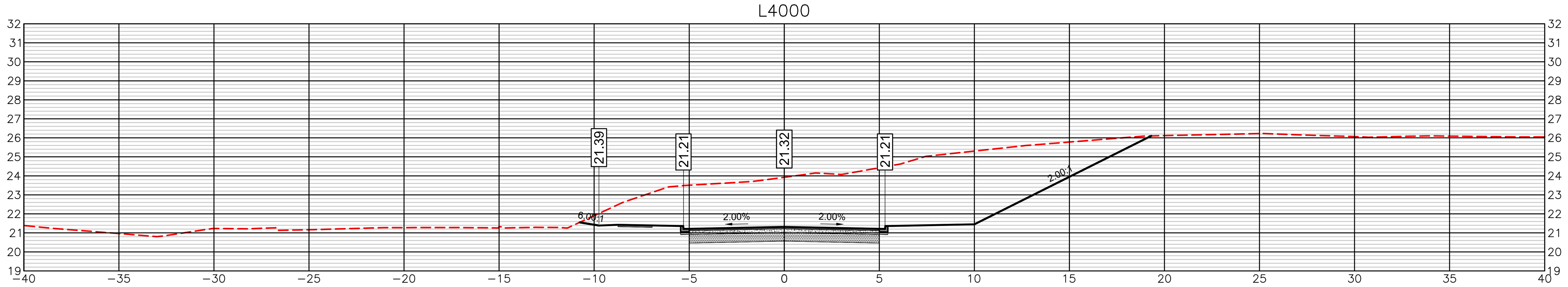

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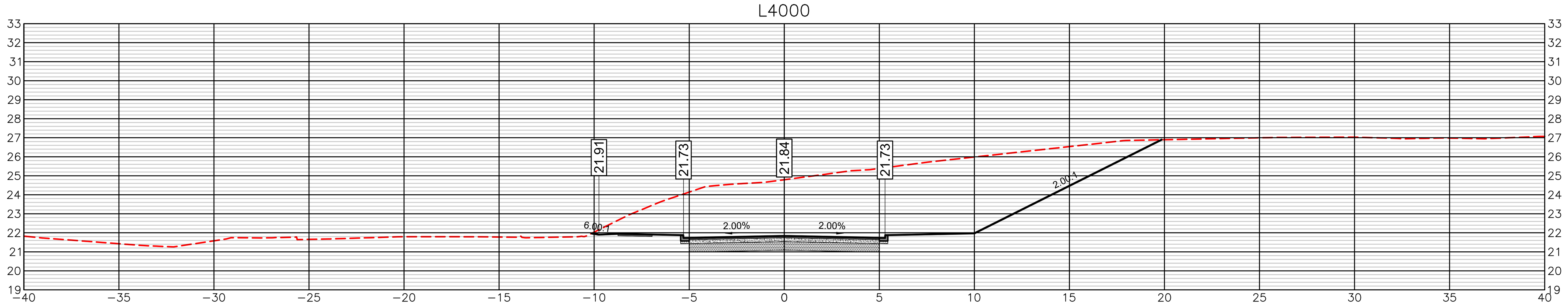

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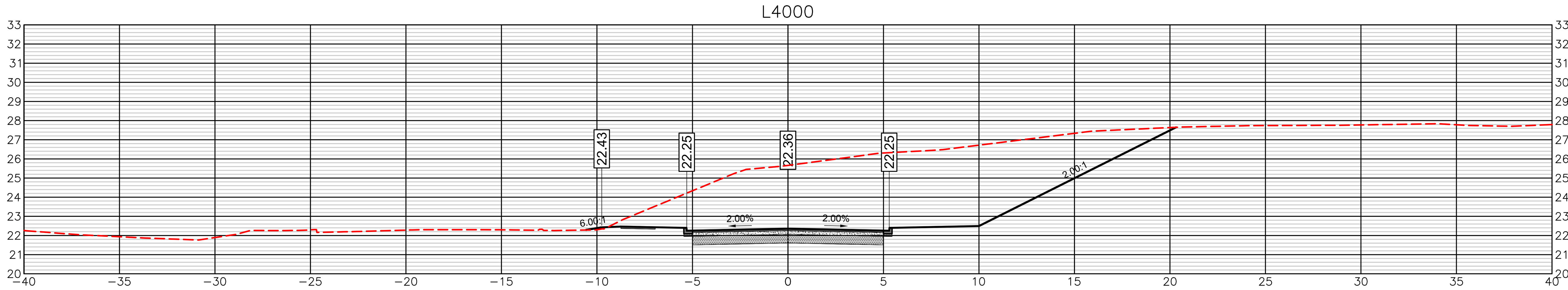

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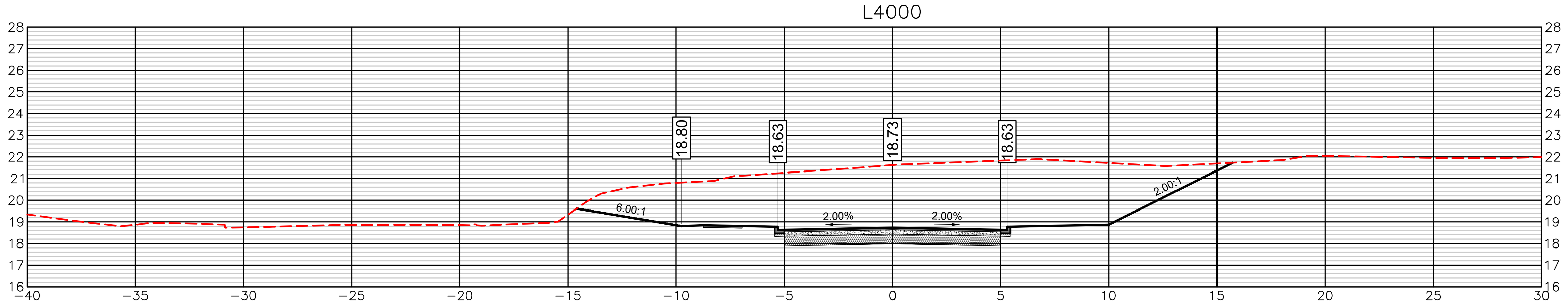

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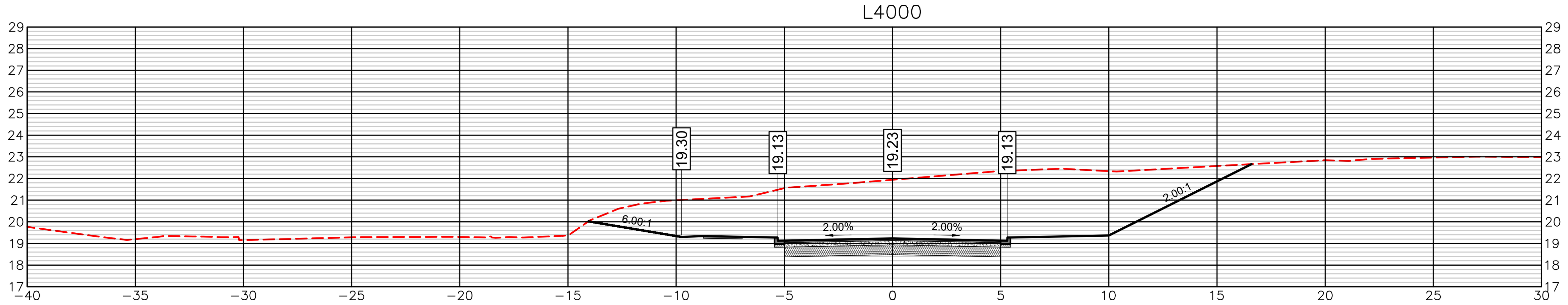

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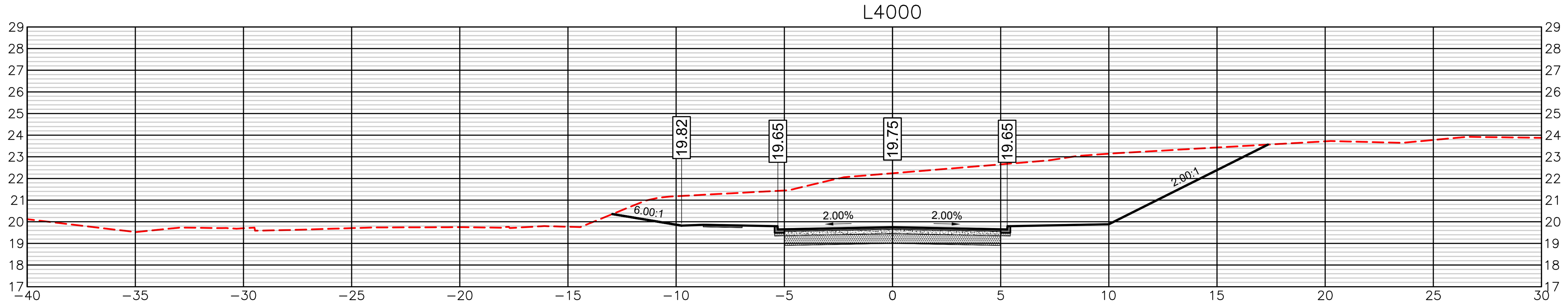

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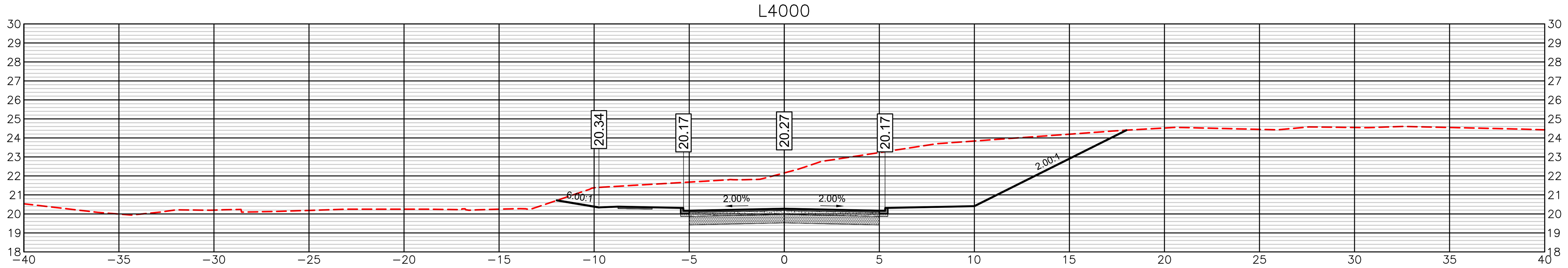

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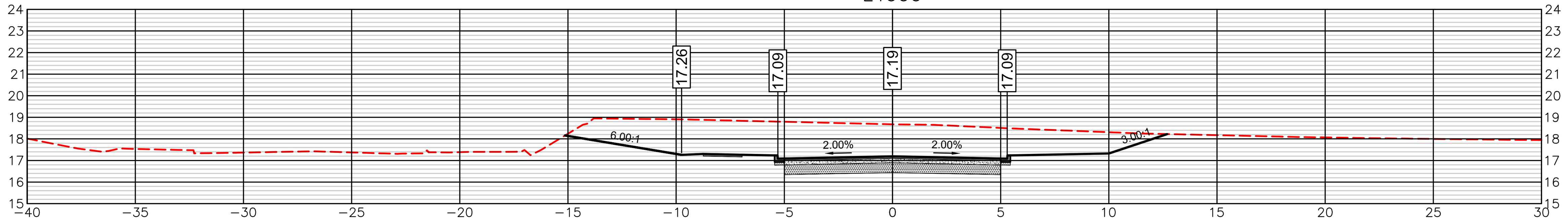

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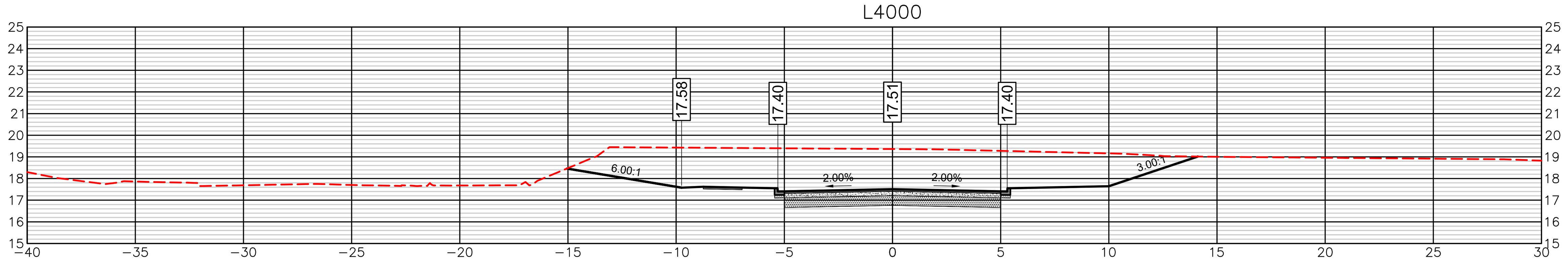

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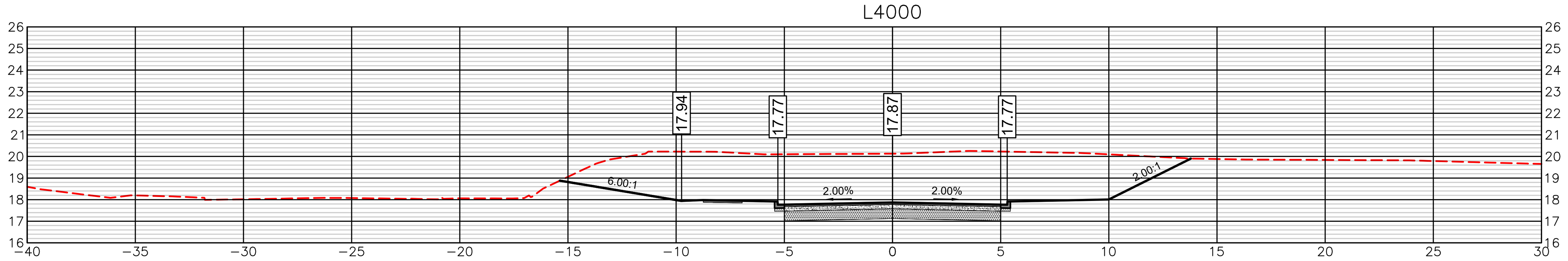

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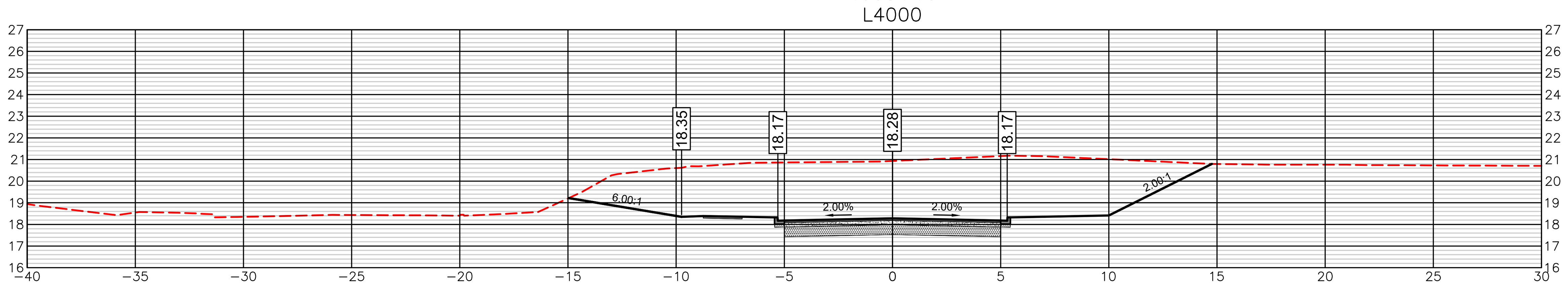

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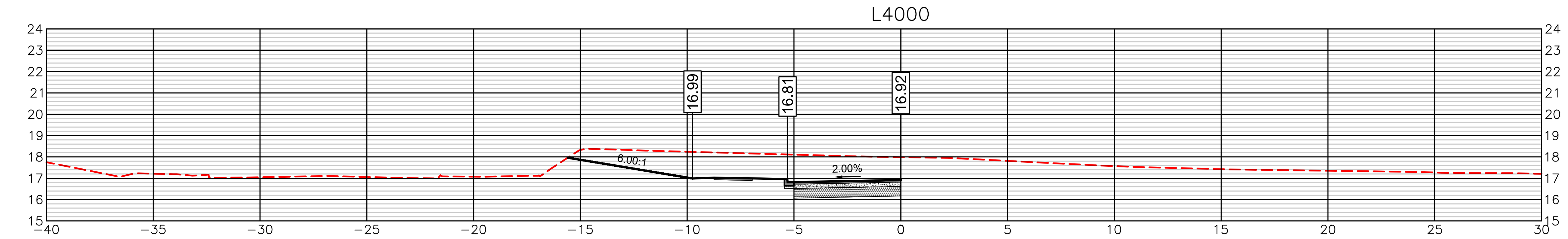

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