GEOCRES #31L-75

Golder Associates Ltd.

662 Falconbridge Road
Sudbury, Ontario, Canada P3A 4S4
Telephone (705) 524-6861
Fax (705) 524-1984

Golder Associates

FINAL REPORT ON

GEOTECHNICAL/FOUNDATION INVESTIGATION AND DESIGN FOR PROPOSED RETAINING WALL ALONG HIGHWAY 63 W.P. 167-90-00 MTO DISTRICT 54, SUDBURY

Submitted to:
Proctor & Redfern Limited
222 McIntyre Street West, Suite 410
North Bay, Ontario
P1B 2Y8

SEE TWO ADDENDUMS AT THE BACK OF THE REPORT (STAPLED)

DISTRIBUTION:

6	Copies	- Proctor & Redfern Limited North Bay, Ontario
1	Copy	- Golder Associates Ltd., Sudbury, Ontario
1	Copy	- Golder Associates Ltd., Mississauga, Ontario

December 1998

981-9109

TABLE OF CONTENTS

SECTION **PAGE**

In Order
Following
Page No. 16

List of Tables

List of Figures

List of Appendices

1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Proctor & Redfern Limited (Proctor & Redfern) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a geotechnical investigation at the site of the proposed horizontal realignment of Highway 63 in Poitras Township. The realignment requires widening of the highway and cutting as much as about 5 m into the toe of the steep slope along the west (left) side. The options for accommodating the cut include a retaining wall at the toe of the slope, a permanent cut slope within the overburden, a cleaned bedrock surface, and/or a rock cut, depending on the subsoil conditions. It is understood that the length of the cut, which may require treatment, is 360 m (from Station 15+000 to Station 15+360).

The purpose of this investigation was to delineate the subsurface conditions in the vicinity of the cut limits utilizing air track drilling, hand augered probeholes, hand sampled boreholes, hand dug test pits and mapping of bedrock outcrops by a geologist. Based on our interpretation of the data obtained, recommendations on the geotechnical aspects of design of the proposed works are provided. Digital drawings of the proposed realignment and cross-sections at 15 m intervals along the highway were provided by Proctor & Redfern.

The terms of reference for the scope of work are outlined in our proposal number P81-9110, dated June 22, 1998. The work was carried out in accordance with the "Guideline for Professional Engineers Providing Geotechnical Engineering Services" (1993) and the provisions in the Request For Proposal (RFP) Terms of Reference.

2.0 SITE DESCRIPTION

The site is located along Highway 63 in Poitras Township approximately 7 km west of the Ottawa River and 9 km east of the Big Jocko River. The site is located within the MTO District 54, Sudbury, some 60 km northeast of the City of North Bay.

The Village of Eldee is located to the south of the proposed cut, while the road to the north of the cut follows the Ottawa River until it reaches the Quebec/Ontario border. The terrain is relatively steep along the cut, with the ground surface decreasing from Elevation 249 m to 236 m towards the north.

Along the east (right) side of the road, moderate to severe erosion has been noted in the shoulder granulars. These granulars overlay exposed rock fill which extends towards the base of the slope (>30m) where a wooded area is present. To the west (left) side of the road, the embankment slope comprises essentially two grades. The steeper, lower grade has indication of slope failure in the exposed native sand and the upper, less steep grade is vegetated with trees, shrubs and a variety of growth on the forest floor.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out in three phases and consisted of air track probeholes, hand augered probeholes, hand sampled boreholes, hand dug test pits and outcropping bedrock mapping. These phases are discussed in more detail below.

The air track probing was carried out on September 8 and 9, 1998, using an air track drill with a detached compressor. The actual drilling involved driving the air track from a transport trailer parked directly adjacent to the gabion basket wall up the slope as far as reasonably possible. Limitations included the steepness of the slope and overhead Bell and Hydro lines (see photos 1 and 2 – Appendix A). A total of 49 air track probeholes (AT-1 to AT-49) were advanced at 15 m stations (with up to 3 holes at each station) along the project length to depths of between 2.4 m to 5.8 m. As a minimum, one hole was advanced behind the existing gabion basket wall and, where possible, holes were advanced up the slope. The upper holes were advanced at various angles in an attempt to better define the bedrock surface within the proposed excavation limits. This angled drilling proved to be difficult at locations with thick amounts of overburden because the granular material would "fall" back onto the drill rods and cause difficulties during retrieval of the rods. Air track probehole data is summarized in Table 1 and the location of the air track probeholes is shown on Figure 1. The results are also shown on the 15 m profile sections on Figures 2 to Figures 25.

The hand augered probeholes were completed on October 15, 1998, and comprised advancing 50 mm diameter augers using a small hand held power auger. These holes were advanced at left offsets from the centre line of the proposed retaining wall in the upper wooded area, in an attempt to determine types and depths of overburden. The depths of these holes ranged from 0.2 m to 0.9 m and typically experienced refusal on boulders within the overburden. Hand augered probehole data is summarized in Table 2. Two small test pits were excavated with hand shovels to investigate refusal depths but they proved to be impractical to advance because of extensive boulders. Test pit data is summarized in Table 3. The location of the hand augered probeholes and test pits is shown on Figure 1. The results are also shown on the 15 m profile sections on Figures 2 to Figures 25.

The boreholes put down using manual equipment were also completed on October 15, 1998, and comprised advancing a 50 mm diameter split spoon sampler through the overburden to obtain Standard Penetration Test (SPT) 'N' values. These holes were advanced at Stations 15+135 (BH-1) and 15+195 (BH-2), directly adjacent the gabion basket wall. The boreholes were advanced to approximate depths of 1.5 m and 1.8 m, respectively, to obtain subsoil information for assessing the bearing capacity of the native soils. The detailed subsurface soil conditions encountered in the boreholes are given on the attached Record of Borehole Sheets, following the text of this report. The location of the boreholes and a section along the wall/edge of the cut is shown on Figure 1.

Detailed geotechnical mapping of exposed bedrock outcrops within the site area was carried out. The mapping included an assessment of the various rock types, weathering effects, the presence of striations, the dip and strike of fractures, and grain size. The locations of outcrops identified during this phase were also surveyed by Proctor & Redfern in order to accurately plot them.

All phases of the field work were supervised on a full-time basis by one or two members of our technical staff who located the testholes and outcropping bedrock in the field. This staff also directed the drilling, sampling and in situ testing operations, and logged the testholes. The soil samples were identified in the field, placed in labelled containers and transported back to our laboratory in Sudbury for further examination. Index and classification tests were carried out on selected samples.

The as-drilled test hole locations were determined by our field personnel based on the highway chainages as marked in the field. Test hole elevations were interpolated from cross-sections provided by Proctor & Redfern and we understand that the elevations are referenced to Geodetic Datum. The locations of all testholes are shown on attached Figure 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published geologic information, the site is located in the physiographic region known as the Central Gneiss Belt which is part of the Grenville Province (OGS Map 2543). The Central Gneiss Belt is from the Mesoproterozoic (0.9 to 1.6 billion years) and primarily comprises of migmatitic rocks and gneisses of undetermined protolith. This geologic area commonly has layered biotite gneisses and migmatites that locally could include quartzofeldspathic gneisses, orthogneisses and paragneisses.

Major Geological Structure

The results of the geotechnical mapping carried out at the various bedrock outcrops are presented in the form of a stereoplot on Figure 27 (refer to Appendix B for a more complete description of stereograghic techniques). Based on the stereoplot there are there are 3 major discontinuity sets evident and these are summarized below:

Joint Set	Dip	Dip Direction	Remarks
J1	80° to 90°	130° or 310°	Rough, uneven joints
J2	80° to 90°	50° or 250°	Rough, uneven joints
J3	30° to 50°	230° to 350°	Foliation

Note: dip directions are referenced to true north.

Joint Set 1 dips steeply to the northwest or southeast (i.e. strikes in a northeast to southwest direction) while Joint Set 2 is a conjugate set striking northwest to southeast. Discontinuity Set 3, which is the foliation, has a much shallower average dip angle (30° to 50°). The foliation appears to dip at a shallower angle to the northwest at the south end of the proposed rock cut becoming somewhat steeper to the north and also dipping more to the southwest.

4.2 Site Stratigraphy

This section of Highway 63 is currently located on the side of a steep slope. The elevation of the road decreases from south to north with the road cut into the side of the slope along the left. As a result, the height of the slope to the west of the road decreases as the elevation of the road increases to the south. The rock type at the site is a heavily weathered, strong to very strong, medium to fine grained, foliated micaceous gneiss. The outcrops in the southern portion of the site are mafic in nature resulting in a grey to black, salt and pepper appearance. Primary mineral components of this rock are quartz, biotite mica, feldspar, pyroxene, and amphibole. The rock outcrops in the northern portion of the site are more felsic in nature resulting in a lighter, whitish-pink to grey appearance. The primary mineral components of this rock include quartz, muscovite mica, and feldspar.

Significant amounts of unconsolidated overburden and vegetation make it difficult to determine the location of outcrops and to distinguish large boulders from actual bedrock. Large boulders are present in the unconsolidated material on the slope, however, it is unclear whether the origin of these boulders is glacial or erosional. The outcrops that are obviously bedrock, are generally massive but have been subjected to heavy weathering and may not be indicative of the insitu structure of the rock mass that would be exposed when a road cut is made into the slope. Strong sub-horizontal foliation is evident in most outcrops as well as some random jointing. The joint spacing varies from moderately close (0.3 m to 1 m) to wide (1 m to 3 m). The joints typically appear to be tight without alteration or infilling. The joint texture was relatively smooth while the planarity of the joints had significant undulations. Lithological and Geotechnical Rock Description Terminology is included in Appendix C.

A detailed description of the subsurface conditions encountered in the testholes is provided in the following sections. A List of Abbreviations and Symbols is included in Appendix C.

4.2.1 Sand and Gravel

Boreholes 1 and 2 were put down at Stations 15+135 and 15+195, respectively, adjacent to the existing gabion basket wall. The subsoils encountered in the boreholes generally comprise compact to very dense, brown sand and gravel. SPT values of 14 to 120 blows per 0.3 m were obtained; however, the higher SPT values may reflect the presence of soil particles that are larger in diameter than the 50 mm split spoon opening.

This information coincides with the large amounts of exposed sand and gravel along this section of cut. Bulk samples were obtained from the slope and the results of grain size distribution tests are shown on Figure 26. The overburden is a well-graded sand and gravel with cobbles and boulders and little silt content.

4.2.2 Bedrock

Bedrock is exposed at numerous locations along the cut section and within the wooded area to west of the proposed cut. The location of bedrock outcrops is shown on Figure 1 in plan view and approximate locations are also shown in profile along the proposed retaining wall centre line on the same drawing. The profile of the bedrock along the proposed retaining wall centre line was developed by transposing locations from the plan view and information from the air track probehole data.

Based on the mapping, the bedrock consists of a heavily weathered, strong to very strong, medium to fine grained, foliated micaceous gneiss. Also, the bedrock surface appears to slope/step downwards from west to east, following the natural surface grade.

4.3 Groundwater Conditions

No water was encountered in the testhole excavations. It is likely that groundwater, when present, would flow at or near the bedrock surface and flow towards the lower, Ottawa River. Isolated pockets of perched water may be present during wetter periods, but these levels will be subject to fluctuations.

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of design of the Highway 63 realignment based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The works described in this report are associated with the proposed highway realignment and the design of a potential retaining structure. It is understood that the centre line of Highway 63 will be shifted 3.0 m to the left and a 4 m shoulder provided, which will result in a cut into the existing side hill cut. The existing left side hill cut is at or near limiting equilibrium and the existing gabion wall is in place only to catch soil sloughing from the slope. The fill embankment on the right hand side shows signs of movement as indicated by settlement of the shoulder and tilting of the guardrail. It is also understood that the highways vertical alignment will be maintained through this section.

The survey for Highway 63 realignment and the location of the proposed retaining structure were completed by Proctor & Redfern survey staff and digital files were provided to our office.

The realignment of Highway 63, by shifting the road centre line 3.0 m left, requires excavation limits that measure approximately 11.8 m left of the existing road centre line. The options for cut face treatment at this excavation limit will depend on whether soils or bedrock will be encountered and include the excavation of overburden to a safe slope, cleaning the surface of shallow bedrock, blasting a bedrock face and constructing a retaining structure.

The field work data from the air track probeholes, hand augered probeholes, shallow hand dug test pits, mapping bedrock outcrops and accurately surveying bedrock outcrops were all utilized to determine which treatment will be applied along the 360 m road realignment. The boreholes put down with manual equipment were advanced to assess the bearing capacity for the native soils, where a retaining structure would be founded.

5.2 Excavation

The present investigation has found the presence of bedrock in reasonable proximity to the proposed roadway for the majority of the route presently being considered in our study (see red hatched areas on Figure 1). Excavation of the overburden along the roadway section can be carried out on a conventional slope (2 horizontal to 1 vertical) until the slope approaches the existing grade or a bedrock outcrop is reached. In some sections the excavation will be within the bedrock itself.

5.2.1 Excavations – Soil

During construction, the excavations operations must be carried out in accordance with the most recent guidelines of the Occupational Health & Safety Act and good construction practice.

Excavations through the overburden soil on this site will not be straightforward. The current configuration of the existing slope will make excavations at the base difficult because material will ravel as the excavation proceeds into the side hill cut. Also, the presence of cobbles and large boulders will cause excavation difficulties. It should be noted that clearing of part of the upper forested area will also be required.

The sides of temporary excavations established during the construction stage should remain stable if cut back at an angle no steeper than 1.5 horizontal to 1 vertical (1.5H:1V). The interception of groundwater during excavations is not anticipated, although some seepage may be experienced depending on the time of year in which construction is completed. Any groundwater seepage through the relatively permeable overburden should be controlled through the use of properly filtered pumps and sumps.

5.2.2 Excavations – Bedrock

Excavations in bedrock should be carried out using controlled blasting methods in order to minimize damage to the remaining rock mass. Detailed blasting method statements and blast designs should be submitted by the contractor and these should include details on the hole diameter, inclination, depth, burden, spacing and pattern type, amount and distribution of explosives material in each hole and type, sequence and number of delays. Consideration should also be given to using pre-splitting techniques in order to preserve the integrity of the rock mass as denoted by a high percentage (>80%) of drillhole traces ("half barrels") on the blasted face after scaling.

In order to achieve an acceptable blasting standard, consideration should be given to using a performance based specification. Due to the existence of steeply dipping joint sets some overbreak may be inevitable and proper allowances should be incorporated into the construction contract. Also, it should be noted that blasting techniques may be required to remove some large boulders.

5.3 Permanent Cut Slopes

Where permanent cut slopes in bedrock are required the final face should be cut back to form an overall slope 0.25 H: 1 V. The final blasted faces should be inspected once exposed to assess the orientation of the major structural features (fractures, joints etc.) and determine if additional rock reinforcement is required to provide adequate long term stability as discussed below.

Based on the structural data collected (refer to the steroeplot shown on Figure 27) a kinematic analysis of possible failure modes has been carried out. Based on the results of the analysis, which looks at the orientation of potential sliding planes and wedge intersection lines with respect to the rock face, major planar or wedge type failures are not anticipated for the final rock cuts. Some variation in the overall foliation angles has been noted based on the geotechnical mapping, however, the foliation generally dips into the hillside.

Other minor instabilities associated with small loose blocks or larger instabilities associated with irregular joints are always a possibility in all areas and as such a provision should be made for scaling of loose rock after blasting and for rock dowels or bolts, if required. The actual conditions of the final rock faces and any requirements for rock reinforcement cannot be finalized until following the field assessment after the completion of the excavations.

The surface of cut slopes in bedrock should have the overburden removed to a distance of 3 m from the slope edge. The remaining overburden can then be graded at 2:1, as discussed below.

Based on the field investigation data and the survey information, we have summarized the required excavation treatments along the realignment in the following table.

FROM STATION	TO STATION	TREATMENT
15+000	15+015	• Grade overburden to 2H:1V or to clean bedrock surface to intercept existing grade. • Bedrock outcrop at 15+015.
15+015	15+030	• Bedrock excavation. • Overburden to the north of 15+030.
15+030	15+060	• Grade overburden to 2H:1V or to clean bedrock surface to intercept existing grade.
15+060	15+285	• Grade overburden to 2H:1V or to clean bedrock surface to intercept exposed bedrock outcrop. • Bedrock excavation may be required near Station 15+115, 15+170 & 15+230 (exposed in slope). • Extended earth excavation limits may be required between Stations 15+165 to 15+195, where bedrock outcrop is at distance from roadway. • Alternatively, excavate a temporary slope (1.5H:1V) for anticipated wall construction between Stations 15+165 to 15+195. If bedrock is intercepted, grade overburden to 2H:1V or to clean bedrock surface.
15+285	15+345	• Excavate a temporary slope (1.5H:1V) for anticipated wall construction. If bedrock is intercepted, grade overburden to 2H:1V or to clean bedrock surface, otherwise implement retaining structure design.
15+345	15+360	• Grade overburden to 2H:1V to intercept existing grade.

preferred solution is excavation.

The natural upper (left hand) slope along most of the area under investigation as surveyed is typically about 1.7:1. The portion of the slope adjacent to the roadway, and behind the gabion wall, has been steepened to between 0.9:1 and 1.3:1 with a slope of 1:1 being most common. A slope of 1:1 in this material is near limiting equilibrium. The flattening of the slope to 2:1 should increase the stability by some 30% to 40%, which is satisfactory from a geotechnical view point. In addition, the present MTO Standard OPSD 202, calls for bench(es) on high cut slopes. The benches should be at 8 m vertical spacing on the slope and of 2 m width.

Drainage should be provided at the contact between the bedrock and overburden in the form of rock filled interceptor drains. The interceptor drains will be constructed in the overburden and provided with a 150 mm diameter sub-drain pipe within a Granular "A" surround. The upper portion of the drains will be backfilled with maximum 150 mm diameter rock fill encompassed in geotextile. These drains should be led to rock filled counterfort drains provided on the slope leading to each of the existing catch basins. A typical drainage detail is shown on Figure 28. In addition, the bench(es) should be graded to direct runoff to the counterfort drains. The capacity of the catch basin drainage facilities should be checked to determine their ability to handle the flow from the slope drainage works. With surface drainage provided for, the 2:1 cut slope could be seeded to limit surface erosion. Alternatively, the cut slope could be faced with fine rock fill (maximum size 300 mm) which would improve its resistance to surface erosion, though the placement of this rock fill may present some difficulties.

5.4 Proposed Retaining Structure

Where it is anticipated that a retaining structure will be required, several options are available and they include but are not limited to the following:

- Precast concrete component wall;
- Cantilever retaining wall;
- Reinforced earth retaining wall;
- Gabion basket wall;
- Geo-web wall with soil nailing; and
- Soldier pile and lagging wall.

The construction of a retaining wall within the toe of the steep overburden slope presents construction problems. According to construction considerations, the wall types are grouped below.

5.4.1 Gravity Retaining Wall

Gravity type retaining walls, either precast component bin walls or cast in place cantilever retaining walls will involve additional excavation of some 6 m width into the steep overburden slope. The temporary excavation slope could be dug in short sections with side slopes inclined at about 1H:1V to intersect the existing slope. A temporary safety wall would have to be constructed behind the working area to allow the workmen to carry out construction of the cantilever retaining wall. The necessity for this safety wall for the bin type construction would depend on whether workmen would be exposed to soil sloughing during installation of the precast component wall.

The safety wall could consist of a soldier pile and lagging wall. Some difficulties would be encountered in driving or augering for the soldier piles in the overburden, which contains cobbles and boulders.

The use of a reinforced earth wall would run into similar difficulties as the gravity walls, as excavation would have to be carried out to install the reinforcing strips, including a safety wall to protect the workmen. The use of a gabion basket wall would involve similar excavation problems. As well, the height of the wall would require extensive stacking of the units.

5.4.2 Factored Geotechnical Resistance

A gravity retaining structure will require the preparation of the founding surface using a compacted granular mat. This founding surface preparation should include excavation into the native sand and gravel, free of organics and deleterious materials. Cobbles and boulders should be removed from the surface and replaced with Granular "A" fill. The sand and gravel surface should be proof rolled and the excavation partially backfilled with 300 mm of compacted, OPSS Granular A. The Granular A should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The Granular A should extend outwards from either edge of the structure/footing at least 300 mm. Where required concrete spread footing can then be formed and poured on the compacted mat.

Spread footings placed on the compacted granular mat may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa. A factored geotechnical resistance at Serviceability Limit States (SLS) of 300 kPa may be used. This value is for vertical concentric loads only. Effects of load inclination and eccentricity need to be taken into account as appropriate. The settlement of the structure will be less than 25 mm.

All footing excavations should be inspected prior to placing Granular A fill to ensure that the base has been adequately prepared (including proof rolling) and that the soil conditions as exposed at the founding level are consistent with the design assumptions. All large cobbles and boulders within the footprint of the founding area should be removed and replaced with compacted Granular A.

5.4.3 Tied Back Walls

A geo-web wall with soil nailing could be a viable option, as it would minimize the excavation required. The soil nails would be installed from benches, working from the top of the slope down. Some difficulty would be encountered in installing the soil nails because of the presence of cobbles and boulders. The installation could be accomplished by drill rigs utilizing casing. Where bedrock and/or boulders are encountered within the design length of the soil nail, diamond drilling techniques could be employed to advance the nail. A geo-web facing would be placed on the slope face, which could be inclined up to 60 degrees from the horizontal.

Another tie back wall that could be utilized is a soldier pile and lagging type. The soldier pile could be installed by pile driving methods, although, it is considered that the pile alignment would be affected by the boulders within the overburden. Alternatively, the soldier piles could be installed by first augering an oversized hole, which is subsequently cased within the granular overburden. The augering would also encounter difficulties and alignment problems with the boulders, but should be sufficiently aligned to allow the proper positioning of a soldier pile in a weak concrete backfill. Churn drilling techniques could also be employed to install the soldier piles. The precast panels could then be subsequently inserted behind the flange of the soldier pile as the weak concrete is excavated in that area.

5.4.4 Horizontal Resistance

Resistance to lateral forces/sliding resistance between the concrete footings and the compacted Granular A mat should be calculated in accordance with Section 6-8.4.3 of the Ontario Highway Bridge Design Code (OHBDC) assuming an unfactored angle of friction of 35 degrees.

5.4.5 Frost Protection

The frost penetration depth for the project area is 2 m. The overburden is nominally frost susceptible, therefore, spread footings placed on compacted Granular A require soil frost protection cover of 2 m. For bin type walls or gabion basket walls where some movement is allowable, the frost protection cover could be reduced to 1 m.

5.5 Lateral Earth Pressures

The lateral pressures acting on the retaining structure will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the retaining structure in accordance with the OHBDC:

- Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B but with less than 5 per cent passing the 200 sieve should be used as backfill behind the structure. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill where concrete walls are used.
- The granular fill may be placed either in a zone with width equal to at least 2.0 m behind the back of the stem (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).
- It is assumed that the retaining structure allows lateral yielding of the stem (unrestrained structure) and active earth pressures may be used in the geotechnical design of the structure.
- A compaction surcharge equal to 16 kPa should be included in the lateral earth pressures for the structural design of the retaining structure.
- For Case I, the pressures are based on the insitu soils in the slope and the following parameters (unfactored) may be assumed:

Soil unit weight (assuming clean earth fill)	21 kN/m^3
Coefficients of 'active' lateral earth pressure:	0.47

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular A	**Granular B**
Soil Unit Weight	22 kN/m^3	21 kN/m^3
Coefficient of 'active' lateral earth pressure	0.40	0.47

It should be noted that the above design parameters assume backfill and ground surface behind the wall rising at 2 horizontal to 1 vertical.

5.6 Stability of Lower (Right Hand) Slope

There is some evidence of movement at the shoulder of the road (tilting of the guard rail, depression in the asphalt shoulder, lateral movement of the granular fill). It is considered that the shifting of the roadway by 3 m to the left and the lowering of the grade by about 1 m at the edge of the shoulder / top of slope will correct this shoulder movement.

The original construction of the highway resulted in some dumping of fill on the lower slope. As recently surveyed by Proctor & Redfern, the right hand slope between about Station 15+090 and 15+255 is between 1.4:1 and 1.5:1 in steepness. These slope angles are steeper than the natural slope to the left of the roadway but less than the cut slope adjacent to the gabion wall. To the north and south of the area, the lower slope is generally 1.75:1 to 2:1. Consideration should be given at this time, if sufficient property is available, to flattening the slope to the MTO Standard of 2 horizontal to 1 vertical, plus 2 m benches at 8 m vertical spacing. Excess material from the left hand cut could be used, after the bush cover had been removed.

Alternatively, an earth reinforced type wall could be provided along the shoulder of much of the total length of the project under consideration.

GOLDER ASSOCIATES LTD.

Dan Cacciotti

Dan M. Cacciotti, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
A. S. POSCHMANN
PROVINCE OF ONTARIO

Anne S. Poschmann, P.Eng.
Principal

REGISTERED PROFESSIONAL ENGINEER
F. J. HEFFERNAN
PROVINCE OF ONTARIO

F. J. Heffernan

Fintan J. Heffernan, P.Eng.
Designated MTO Contact

DMC/FJN/ASP/tl

TABLE 1

AIR TRACK PROBEHOLE DATA

Station	Hole No.	Depth (m)	Angle of Mast	Distance	Reference
15 + 345	AT-1	2.4	90.0	4.4	EOP
	AT-2	2.7	56.3	7.7	EOP
15 + 330	AT-3	2.7	90.0	3.6	EOP
	AT-4	2.7	65.4	6.4	EOP
	AT-5	2.7	36.0	8.0	EOP
15 + 315	AT-6	2.7	90.0	0.7	Back Gab
	AT-7	2.7	69.4	3.2	Back Gab
	AT-8	5.5	37.7	4.8	Back Gab
15 + 300	AT-9	2.7	90.0	1.1	Back Gab
	AT-10	5.8	65.4	3.5	Back Gab
15 + 285	AT-11	2.7	90.0	1.1	Back Gab
	AT-12	5.8	63.4	5.1	Back Gab
15 + 270	AT-13	2.7	90.0	0.3	Back Gab
	AT-14	5.8	69.4	4.4	Back Gab
	AT-15	5.8	46.2	5.7	Back Gab
15 + 255	AT-16	2.7	90.0	0.4	Back Gab
	AT-17	5.8	46.2	5.1	Back Gab
15 + 240	AT-18	2.7	90.0	0.2	Back Gab
	AT-19	5.2	50.2	4.0	Back Gab
15 + 225	AT-20	2.7	90.0	0.1	Back Gab
	AT-21	5.2	61.6	3.1	Back Gab
15 + 210	AT-22	2.7	90.0	0.0	Back Gab
	AT-23	5.8	63.4	5.0	Back Gab
15 + 195	AT-24	2.7	90.0	0.3	Back Gab
	AT-25	5.5	67.4	6.0	Back Gab
15 + 180	AT-26	5.2	59.7	3.3	Back Gab
15 + 165	AT-27	2.7	90.0	1.2	Front Gab
	AT-28	5.8	58.0	4.4	Front Gab
15 + 150	AT-29	2.7	90.0	1.0	Front Gab
	AT-30	5.8	65.4	4.4	Front Gab
15 + 135	AT-31	2.7	90.0	1.2	Front Gab
	AT-32	5.8	71.6	4.8	Front Gab

TABLE 1 (CONT'D)

AIR TRACK PROBEHOLE DATA

Station	Hole No.	Depth (m)	Angle of Mast	Distance	Reference
15 + 120	AT-33	2.7	90.0	1.5	Front Gab
	AT-34	5.8	78.2	4.7	Front Gab
15 + 105	AT-35	2.7	90.0	0.9	Front Gab
	AT-36	5.8	71.6	4.8	Front Gab
15 + 090	AT-37	2.7	90.0	1.6	Front Gab
	AT-38	5.8	73.7	4.8	Front Gab
15 + 075	AT-39	2.7	90.0	1.5	Front Gab
	AT-40	5.8	71.6	4.9	Front Gab
15 + 060	AT-41	2.7	90.0	1.4	Front Gab
	AT-42	5.8	71.6	4.2	Front Gab
15 + 045	AT-43	2.7	90.0	1.4	Front Gab
	AT-44	5.8	78.2	4.9	Front Gab
15 + 030	AT-45	2.7	90.0	1.3	Front Gab
	AT-46	2.7	71.6	4.5	Front Gab
15 + 015	AT-47	2.7	90.0	1.2	Front Gab
	AT-48	2.7	71.6	4.0	Front Gab
15 + 000	AT-49	2.7	71.6	5.4	EOP

Notes:

1. Air track probeholes were advanced on September 8 and 9, 1998.
2. Air track probehole locations are shown on Figure 1.
3. EOP: Edge of Pavement
   Front Gab: Front of Gabion Basket Wall
   Back Gab: Back of Gabion Basket Wall

TABLE 2

HAND AUGERED PROBEHOLE DATA

Station	Probe Hole #	Lt. Offset from Centreline of Proposed Wall (m)	Depth (m)	Comments
15+045	PH-1	2.0	0.35	Type of overburden: boulders in overburden Type of refusal: boulders
	PH-2	10.0	0.90	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
15+060	PH-3	0.0	0.65	Type of overburden: boulders in overburden Type of refusal: boulders
	PH-4	8.5	0.72	Type of overburden: boulders in overburden Type of refusal: boulders
	PH-5	17.0	0.44 0.37	Type of overburden: boulders Type of refusal: boulders
15+075	PH-6	0.0	0.70	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-7	6.5	0.76 0.76	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-8	17.0	0.90	Type of overburden: boulders Type of refusal: boulders
15+090	PH-9	0.0	0.85	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-10	6.5	0.85	Type of overburden: boulders Type of refusal: boulders
15+105	PH-11	0.0	--	Type of overburden: soil overburden Type of refusal: bedrock
	PH-12	8.0	0.33	Type of overburden: soil overburden Type of refusal: bedrock or boulders
15+120	PH-13	0.0	--	Type of overburden: boulders Type of refusal: boulders
	PH-14	7.5	0.35, 0.55, 0.75	Type of overburden: boulders Type of refusal: bedrock or boulders
15+135	PH-15	0.0	0.90	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders

TABLE 2 (CONT'D)

HAND AUGERED PROBEHOLE DATA

Station	Probe Hole #	Lt. Offset from Centreline of Proposed Wall (m)	Depth (m)	Comments
	PH-16	5.2	0.70	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-17	7.5	0.40	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-18	13.5	0.20 5 holes	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
15+150	PH-19	0.0	0.50	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-20	7.5	0.75 0.85	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-21	13.5	0.35	Type of overburden: boulders Type of refusal: boulders
15+165	PH-22	0.0	0.25	Type of overburden: boulders Type of refusal: bedrock or boulders
15+170	PH-23	0.0	0.55	Type of overburden: boulders Type of refusal: bedrock or boulders
	PH-24	9.0	0.75, 0.80	Type of overburden: boulders Type of refusal: bedrock or boulders
	PH-25	14.0	0.70	Type of overburden: boulders Type of refusal: bedrock or boulders
15+180	PH-26	0.0	0.45	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-27	6.5	0.25	Type of overburden: boulders Type of refusal: bedrock or boulders
	PH-28	11.0	0.20, 0.55, 0.75	Type of overburden: boulders Type of refusal: bedrock or boulders
	PH-29	13.5	--	Type of overburden: boulders in overburden Type of refusal: boulders
15+195	PH-30	0.0	0.80	Type of overburden: boulders in overburden Type of refusal: bedrock

TABLE 2 (CONT'D)

HAND AUGERED PROBEHOLE DATA

Station	Probe Hole #	Lt. Offset from Centreline of Proposed Wall (m)	Depth (m)	Comments
	PH-31	5.0	0.95	Type of overburden: boulders in overburden Type of refusal: bedrock or boulders
	PH-32	12.0	0.80	Type of overburden: boulders in overburden Type of refusal: bedrock
	PH-33	15.0	0.24	Type of overburden: boulders Type of refusal: boulders and cobbles
15+210	PH-34	0.0	0.80	Type of overburden: boulders Type of refusal: bedrock
	PH-35	8.0	0.80	Type of overburden: boulders Type of refusal: bedrock
	PH-36	12.0	0.40, 0.20	Type of overburden: boulders Type of refusal: bedrock
15+225	PH-37	0.0	0.90	Type of overburden: boulders in overburden Type of refusal: bedrock
	PH-38	10.0	0.20, 0.20, 0.80	Type of overburden: boulders Type of refusal: boulders and cobbles
	PH-39	17.0	0.40, 0.60	Type of overburden: boulders in overburden Type of refusal: boulders
15+240	PH-40	0.0	0.55	Type of overburden: boulders Type of refusal: bedrock or boulders
	PH-41	6.0	0.75, 0.75	Type of overburden: boulders Type of refusal: bedrock or boulders
	PH-42	11.0	0.75, 0.50, 0.30	Type of overburden: boulders Type of refusal: boulders
15+315	PH-43	6.0	0.40	Type of overburden: boulders in overburden Type of refusal: boulders
	PH-44	9.0	0.75	Type of overburden: boulders in overburden Type of refusal: boulders

TABLE 2 (CONT'D)

HAND AUGERED PROBEHOLE DATA

Station	Probe Hole #	Lt. Offset from Centreline of Proposed Wall (m)	Depth (m)	Comments
	PH-45	13.0	0.25, 0.35	Type of overburden: boulders in overburden Type of refusal: boulders
15+330	PH-46	0.0	0.30	Type of overburden: boulders Type of refusal: boulders
	PH-47	3.0	0.75	Type of overburden: boulders Type of refusal: boulders
	PH-48	10.0	0.70, 0.65	Type of overburden: boulders Type of refusal: boulders
	PH-49	16.0	0.75, 0.45	Type of overburden: boulders Type of refusal: boulders
15+345	PH-50	0.0	0.25, 0.30	Type of overburden: boulders in overburden Type of refusal: boulders
	PH-51	6.0	0.70	Type of overburden: boulders Type of refusal: bedrock or boulders
	PH-52	12.0	0.80, 0.80	Type of overburden: boulders Type of refusal: boulders

TABLE 3

TEST PIT DATA

Station	Test Pit #	Lt. Offset from Centreline of Proposed Wall (m)	Depth (m)	Comments
15+045	TP-1	8.8	0.65	Type of overburden: boulders in overburden Type of refusal: boulders
15+060	TP-2	8.8	0.90	Type of overburden: boulders in overburden Type of refusal: boulders

DIST 54 HWY 63
CONT. No.
WP No. 167-90-00

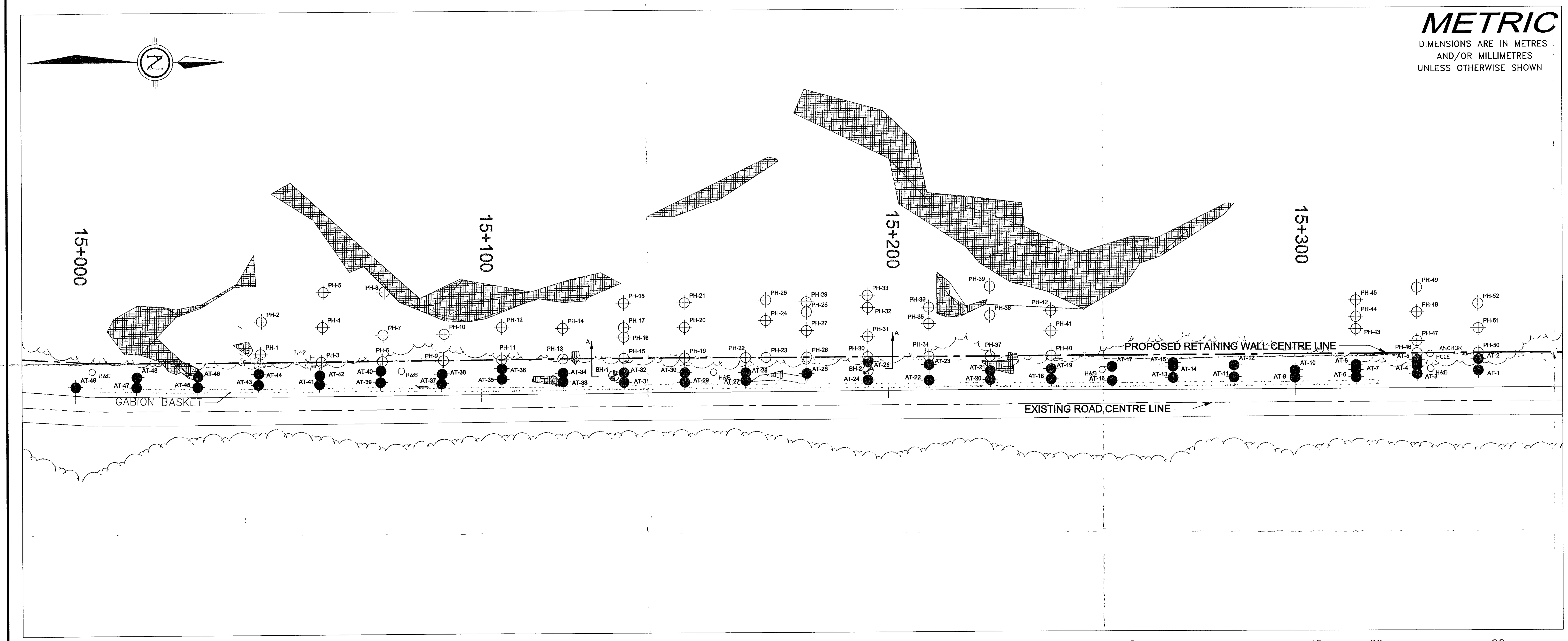
SHEET

HIGHWAY 63 REALIGNMENT
STATION 15+000 TO 15+360
TESTHOLE LOCATIONS & PROFILE

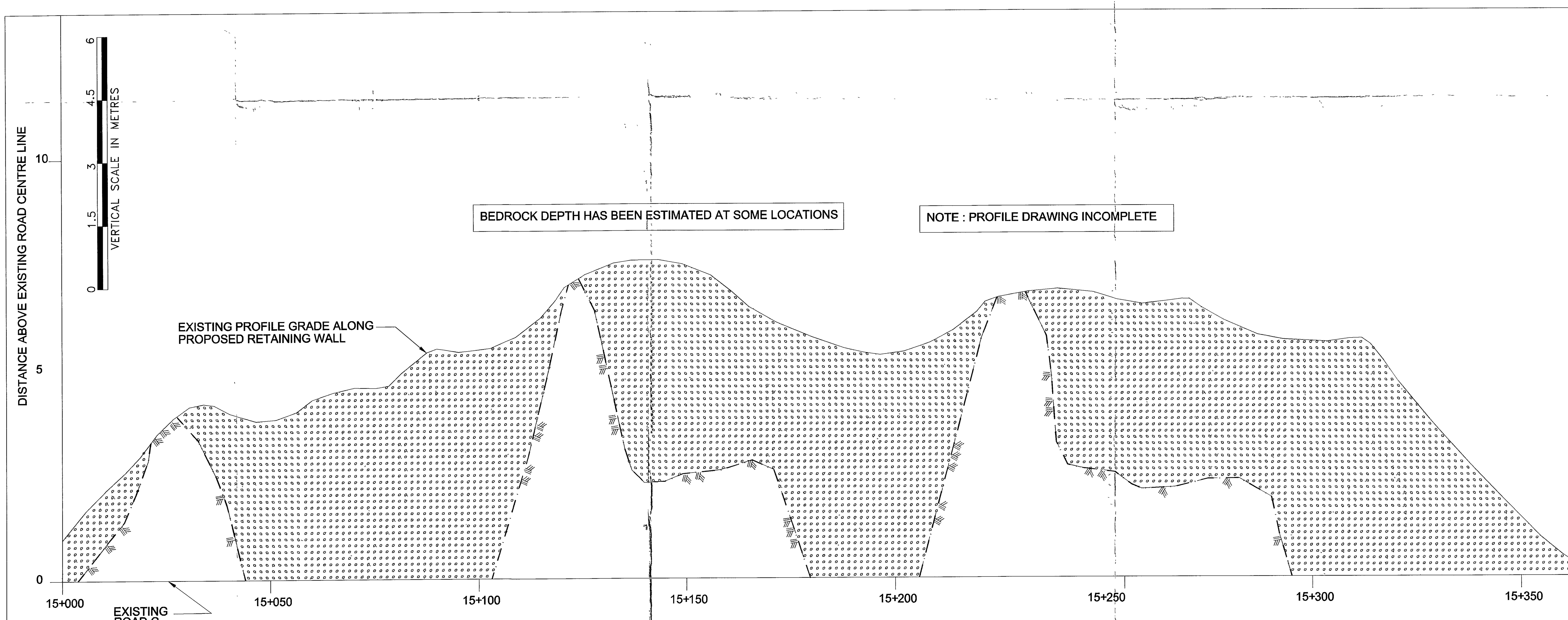
Golder Associates

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



PLAN



PROFILE ALONG PROPOSED RETAINING WALL CENTRE LINE

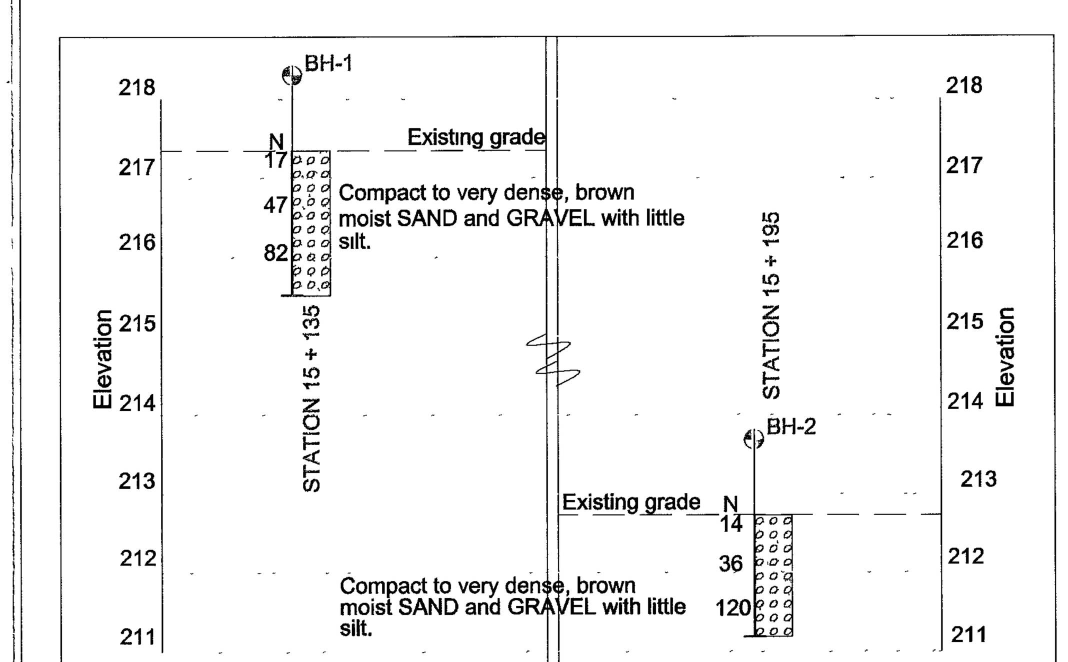
KEY PLAN

LEGEND

- Borehole
- Hand Augered Probehole
- Air Track Probehole
- Test Pit
- N Blows/0.3m (Std. Pen. Test, 475 j/blow)
- Exposed Bedrock
- Overburden
- Estimated Bedrock Surface

GENERAL NOTES

1. Digital drawing and survey information were provided by Proctor and Redfern.



SECTION A-A

NO.	DATE	BY	REVISION
B	98/12/03	DMC	ISSUED FOR FINAL REPORT
A	98/10/23	DMC	ISSUED FOR REVIEW

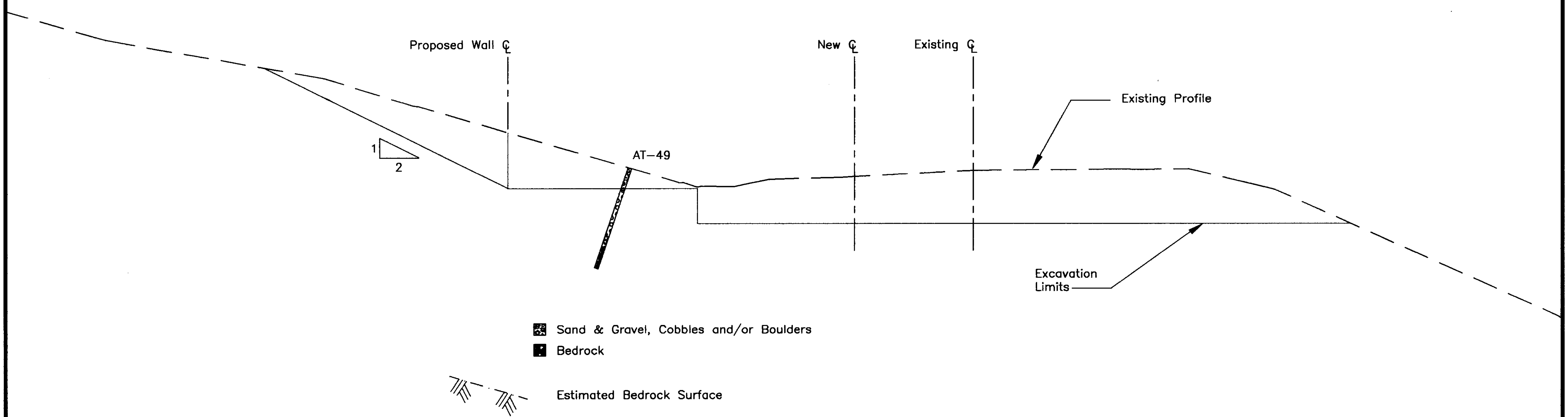
Geocres No.

HIGHWAY 63		PROJECT NO.: 981-9109	DIST. 52
SUBM'D. DMC	CHKD: DMC	DATE: 1998 10 23	SITE
DRAWN: PZ	CHKD. FJH	APPD.	DWG. FIGURE 1

ACAD FILE 9819109FIG1.DWG

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+000

FIGURE 2



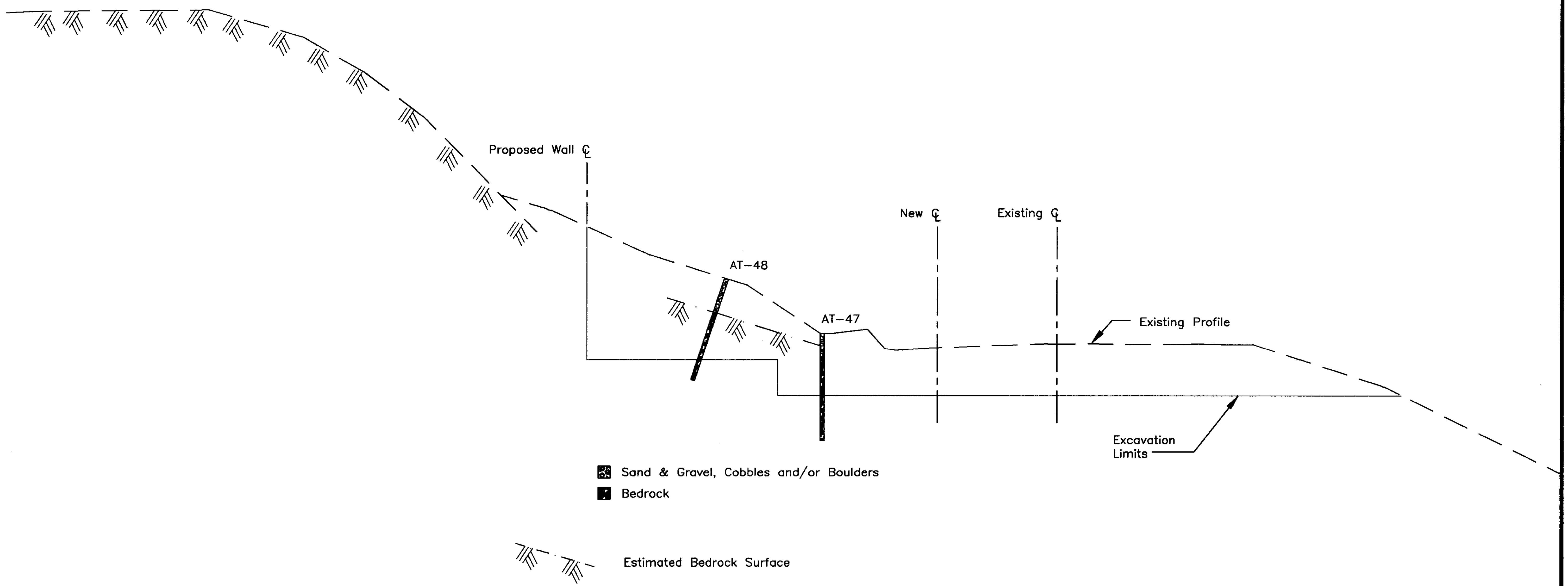
Scale 1:100
File 9819109FIGS.dwg
Date DEC/98
Project 981-9109

Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+015

FIGURE 3



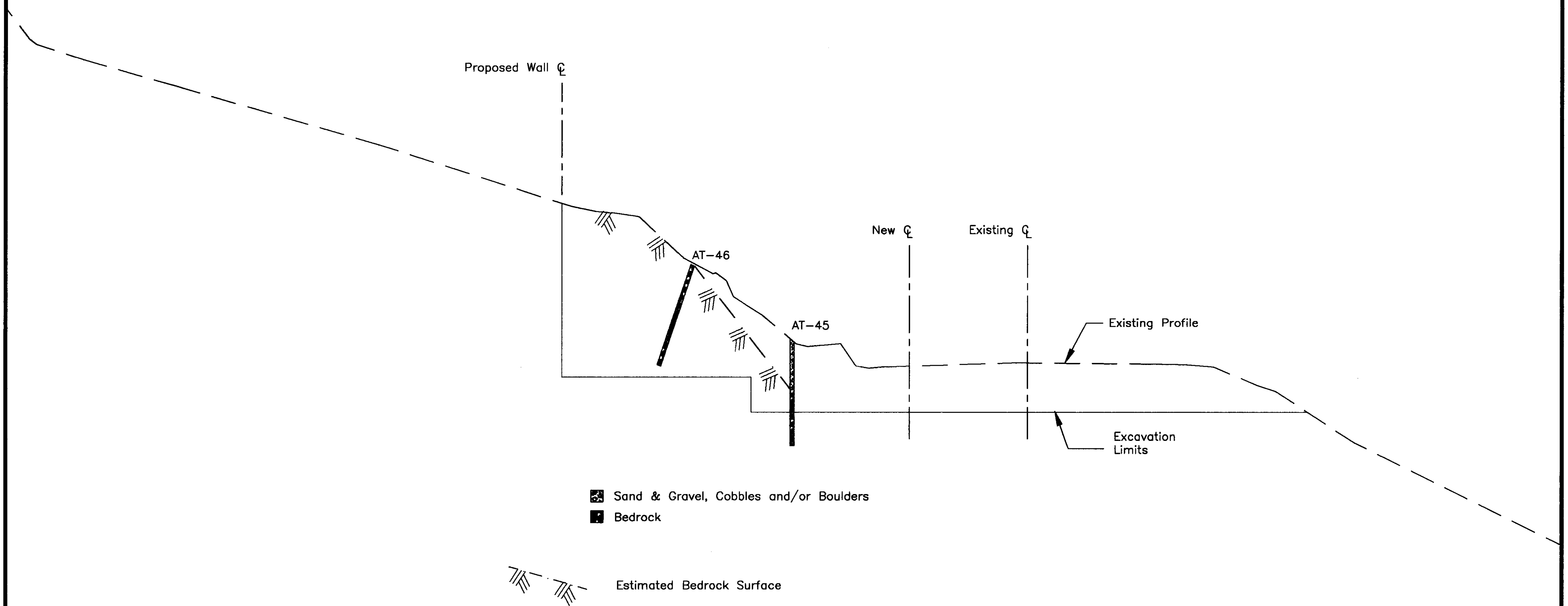
Scale ....1:100.........
Date ....DEC/98....
File 9819109FIGS.dwg
Project 981-9109

Golder Associates

Drawn ...KLS....
Chkd ....DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+030

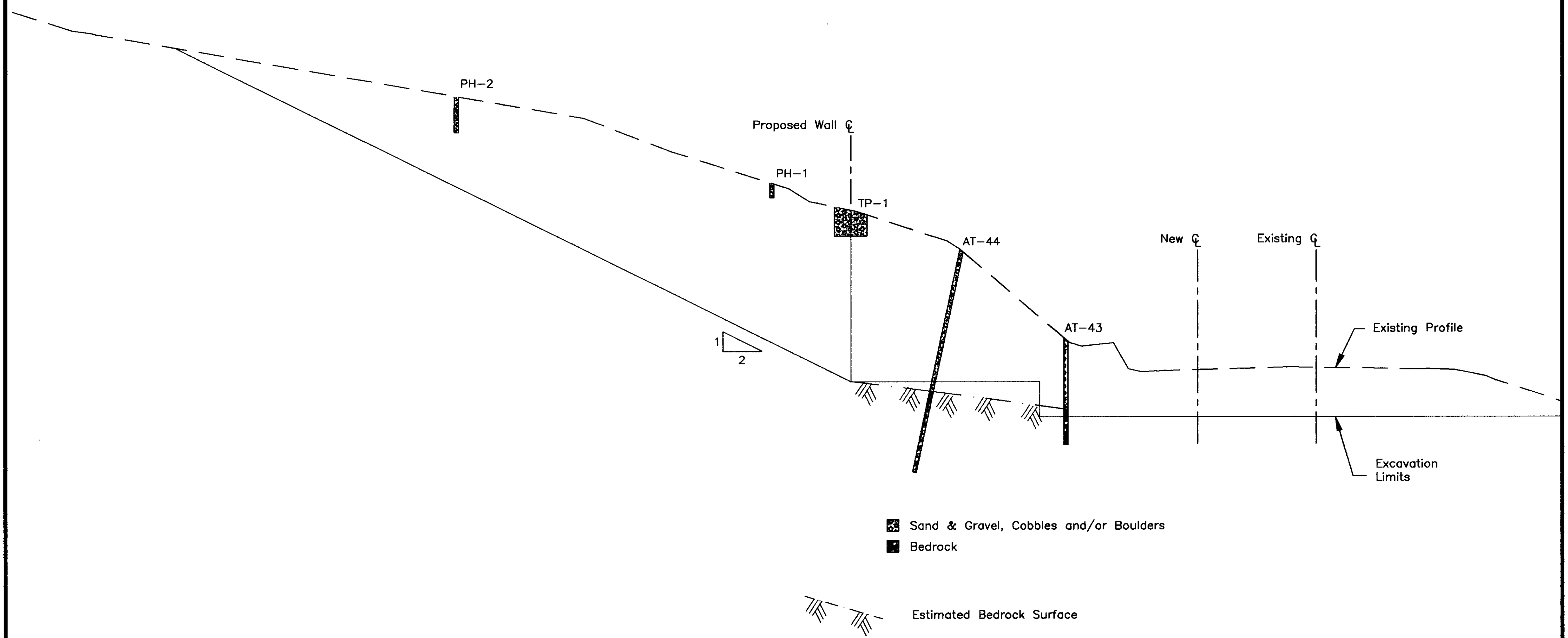
FIGURE 4



Scale 1:100	Date DEC/98	Golder Associates	Drawn KLS
File 9819109FIGS.dwg	Project 981-9109		Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+045

FIGURE 5



Scale ......1:100........ Date ......DEC/98....

File 9819109FIGS.dwg Project 981-9109

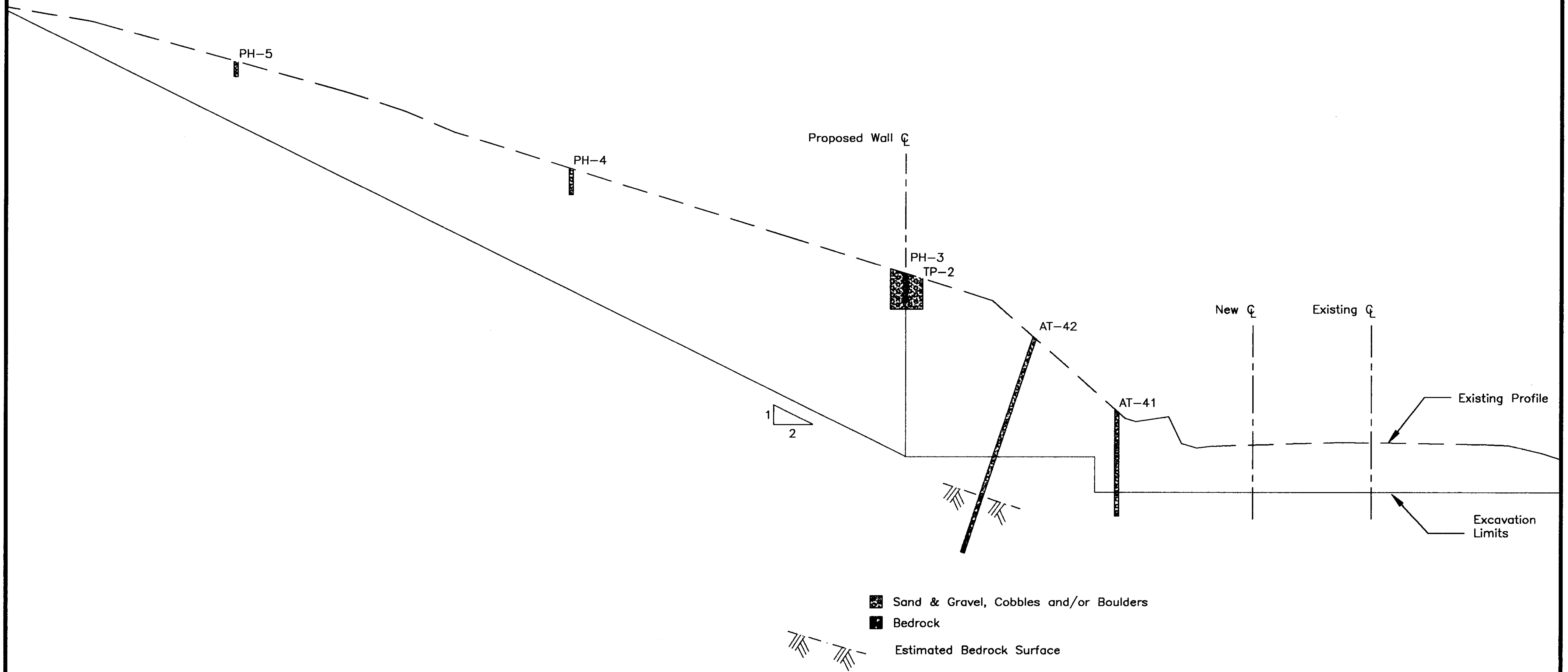
Golder Associates

Drawn ..KLS....

Chkd ......DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+060

FIGURE 6



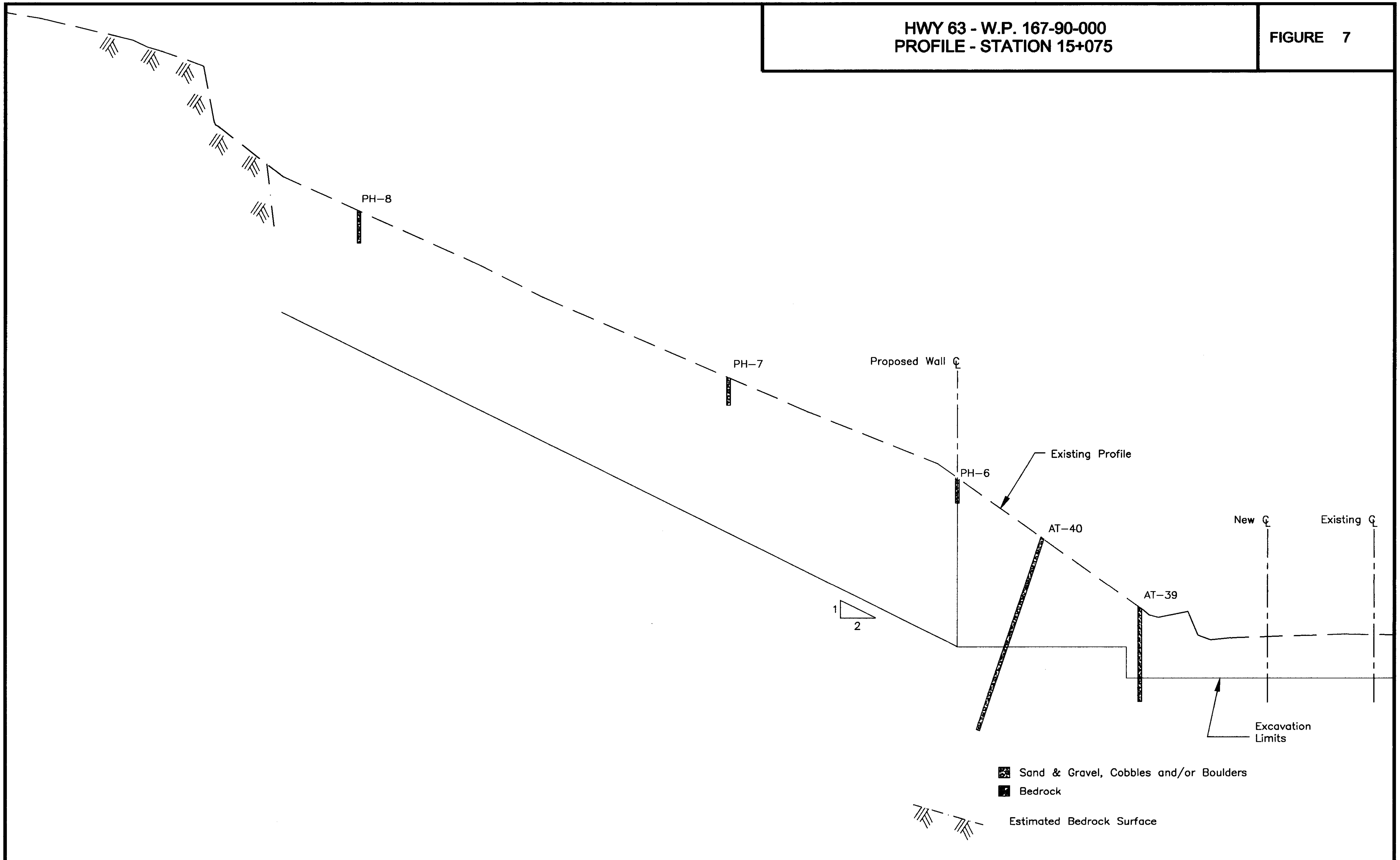
Scale ....1:100.... Date ....DEC/98....
File 9819109FIGS.dwg Project 981-9109

Golder Associates

Drawn ..KLS...
Chkd ....DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+075

FIGURE 7



Scale ....1:100....

Date ....DEC/98....

File 9819109FIGS.dwg

Project 981-9109

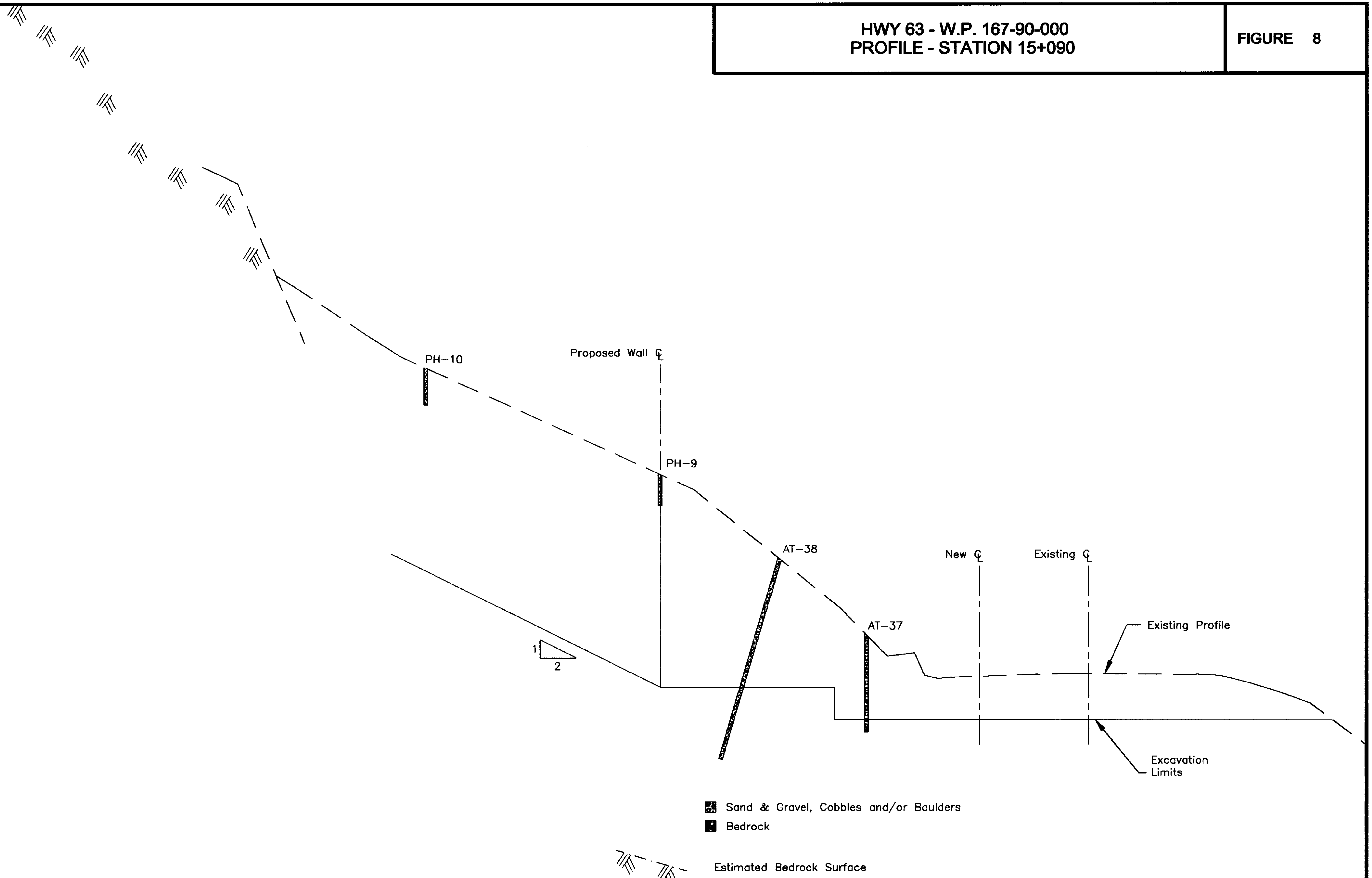
Golder Associates

Drawn ..KLS...

Chkd ....DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+090

FIGURE 8



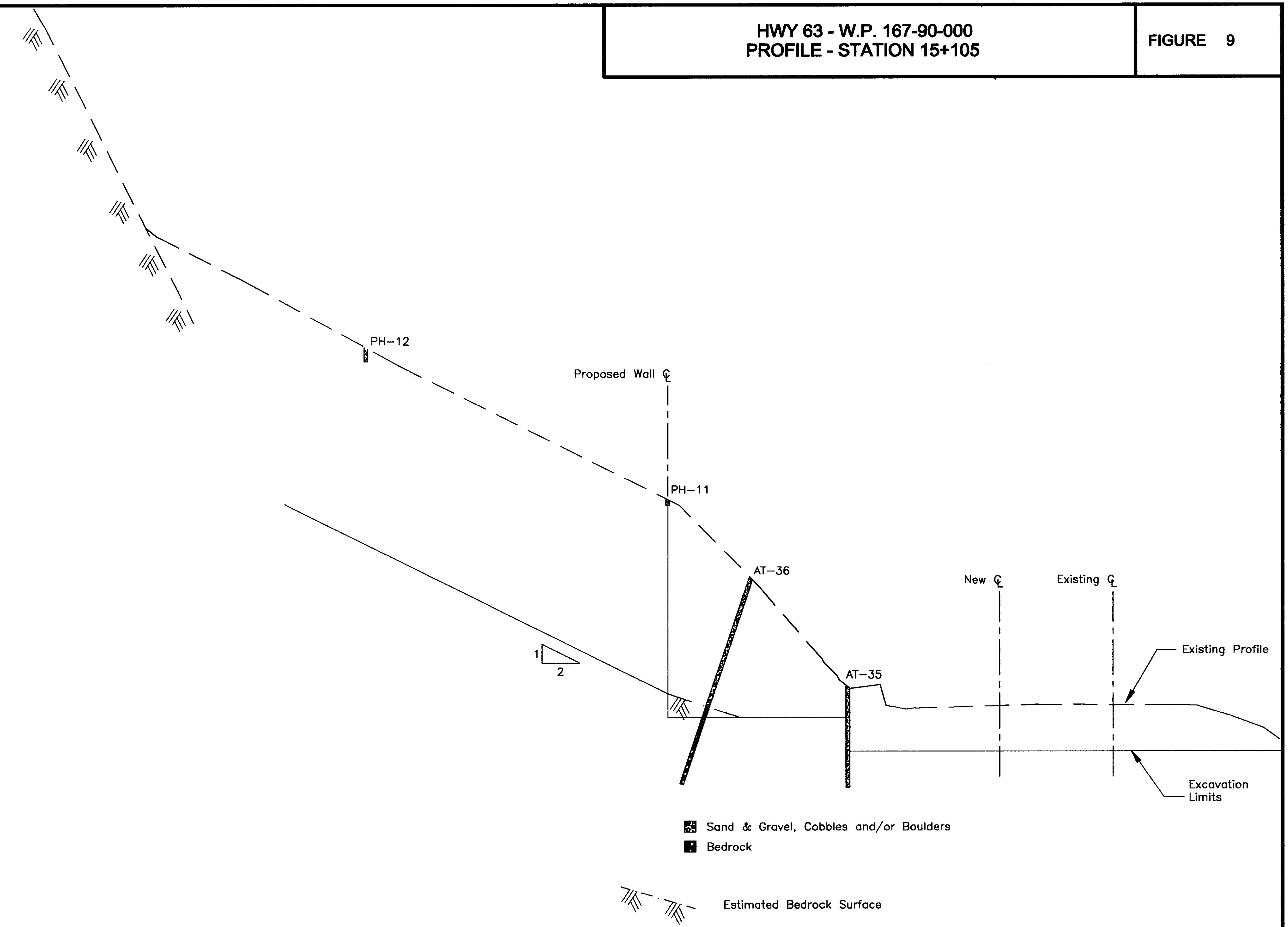
Scale ....1:100......... Date ....DEC/98.... File 9819109FIGS.dwg Project 981-9109

Golder Associates

Drawn ...KLS.... Chkd ....DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+105

FIGURE 9



Scale ....1:100........

Date ....DEC/98....

Golder Associates

Drawn ..KLS....

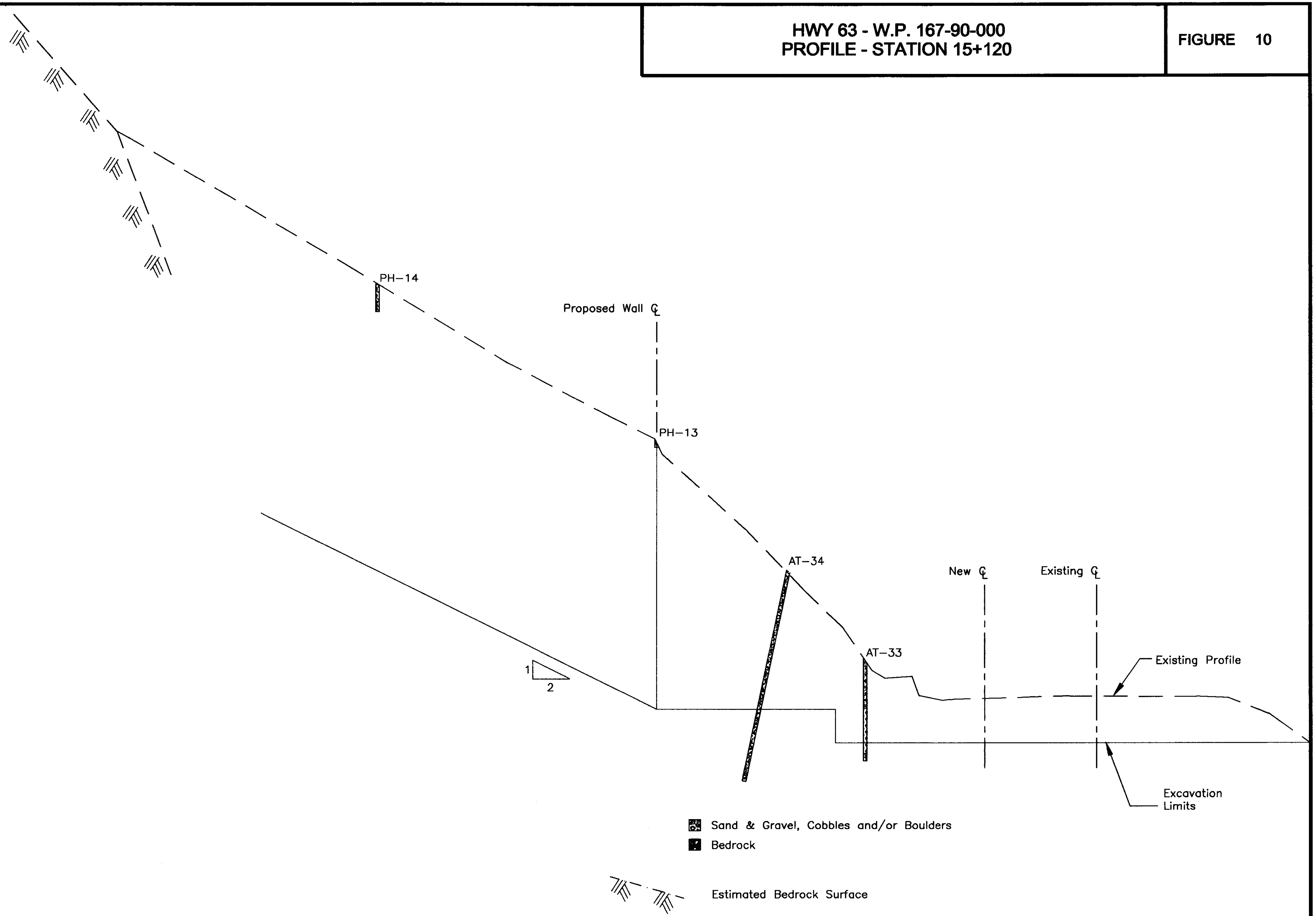
File 9819109FIGS.dwg

Project 981-9109

Chkd ....DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+120

FIGURE 10

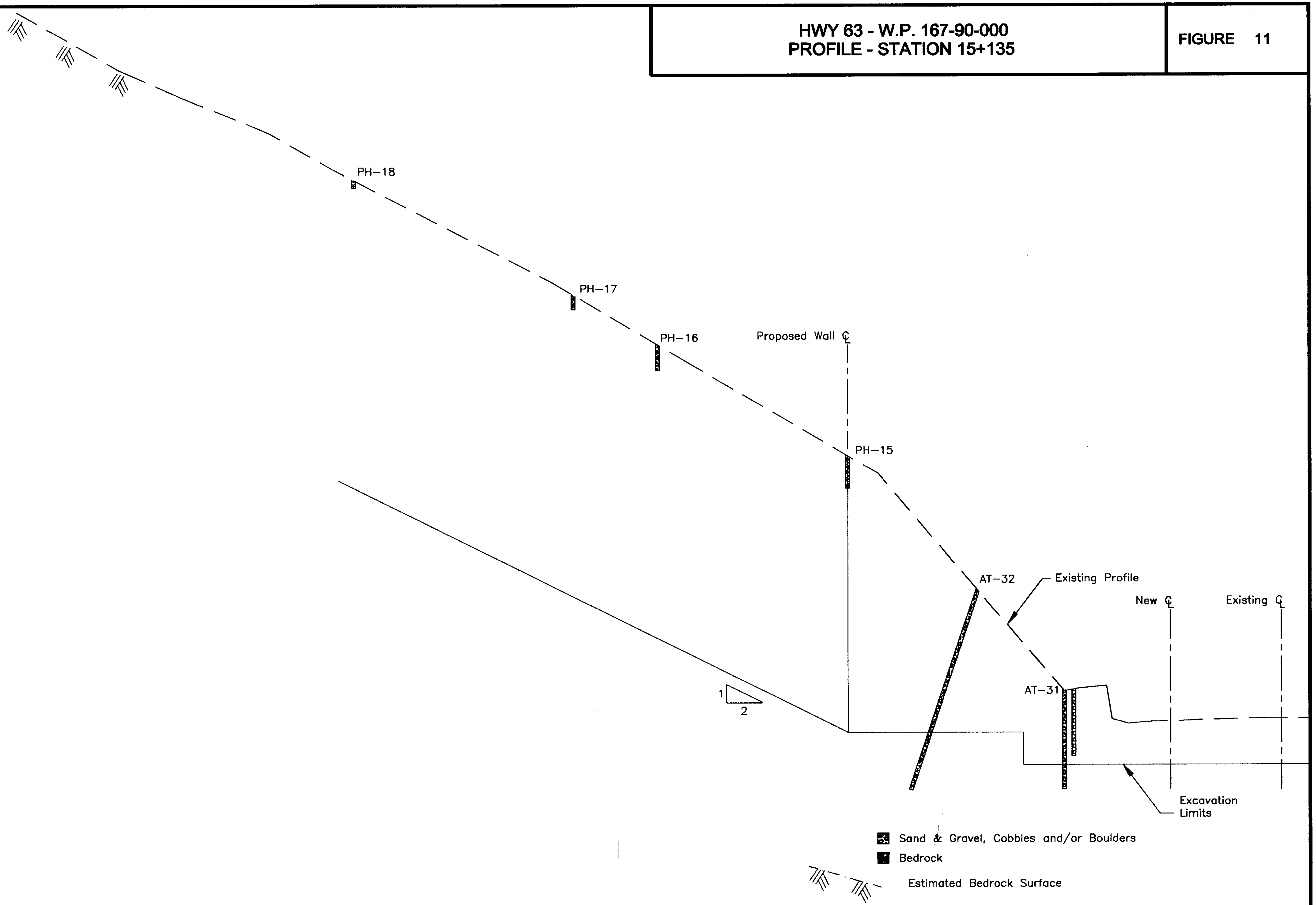


Scale 1:100 | Date DEC/98 | Drawn KLS
File 9819109FIGS.dwg | Project 981-9109 | Chkd DMC

Golder Associates

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+135

FIGURE 11



Scale 1:100 | Date DEC/98
File 9819109FIGS.dwg | Project 981-9109

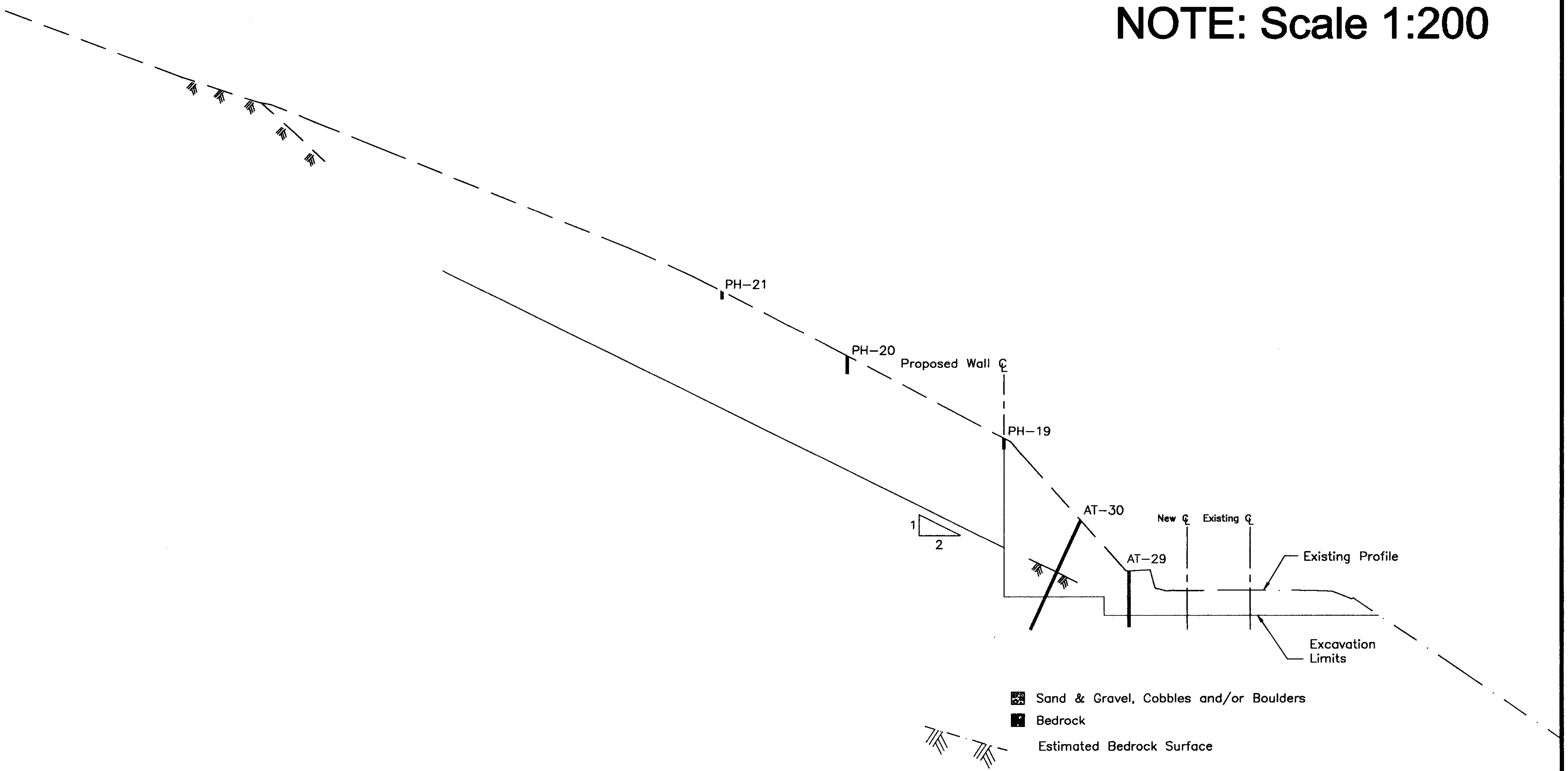
Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+150

FIGURE 12

NOTE: Scale 1:200



Scale 1:200
Date DEC/98
File 9819109FIGS.dwg
Project 981-9109

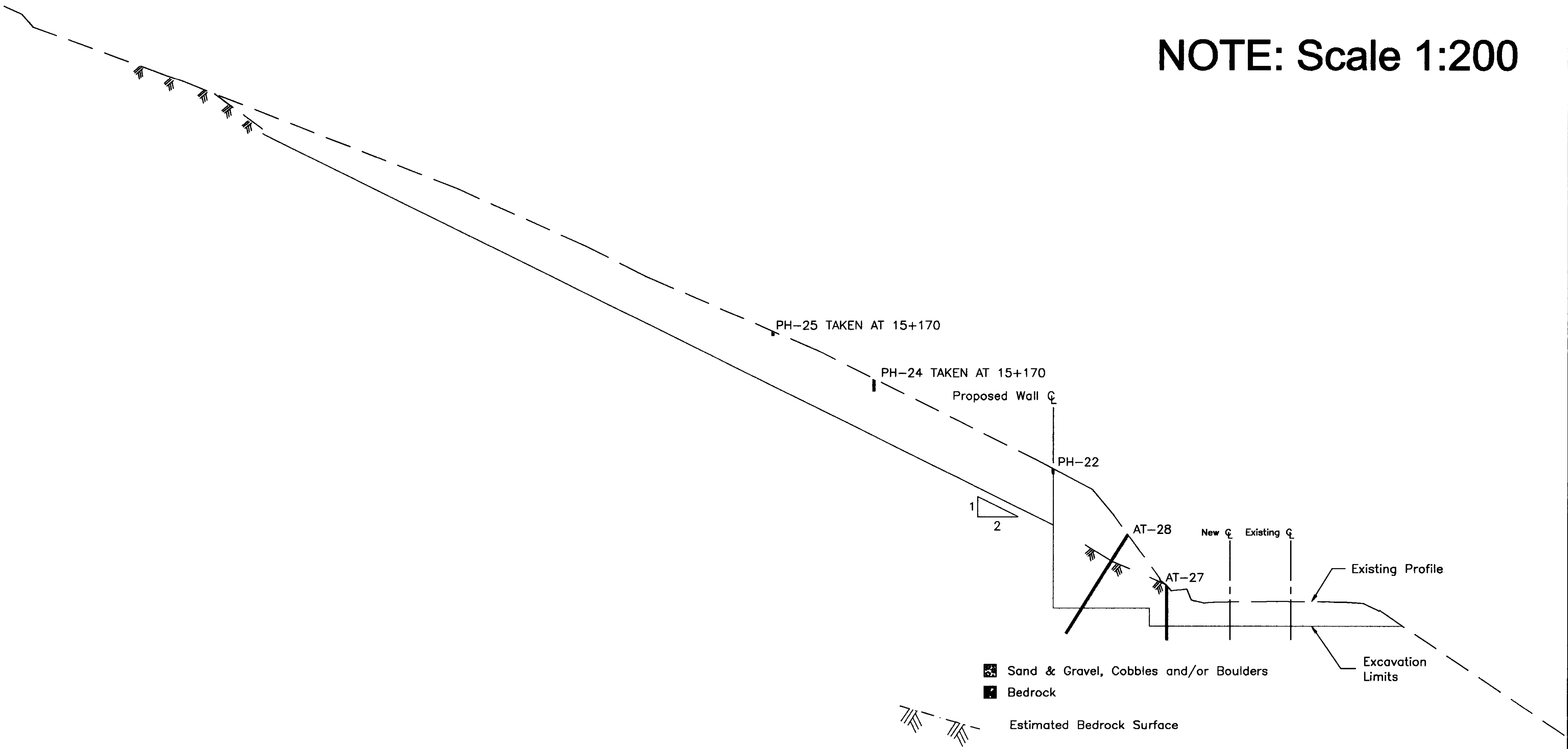
Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+165

FIGURE 13

NOTE: Scale 1:200

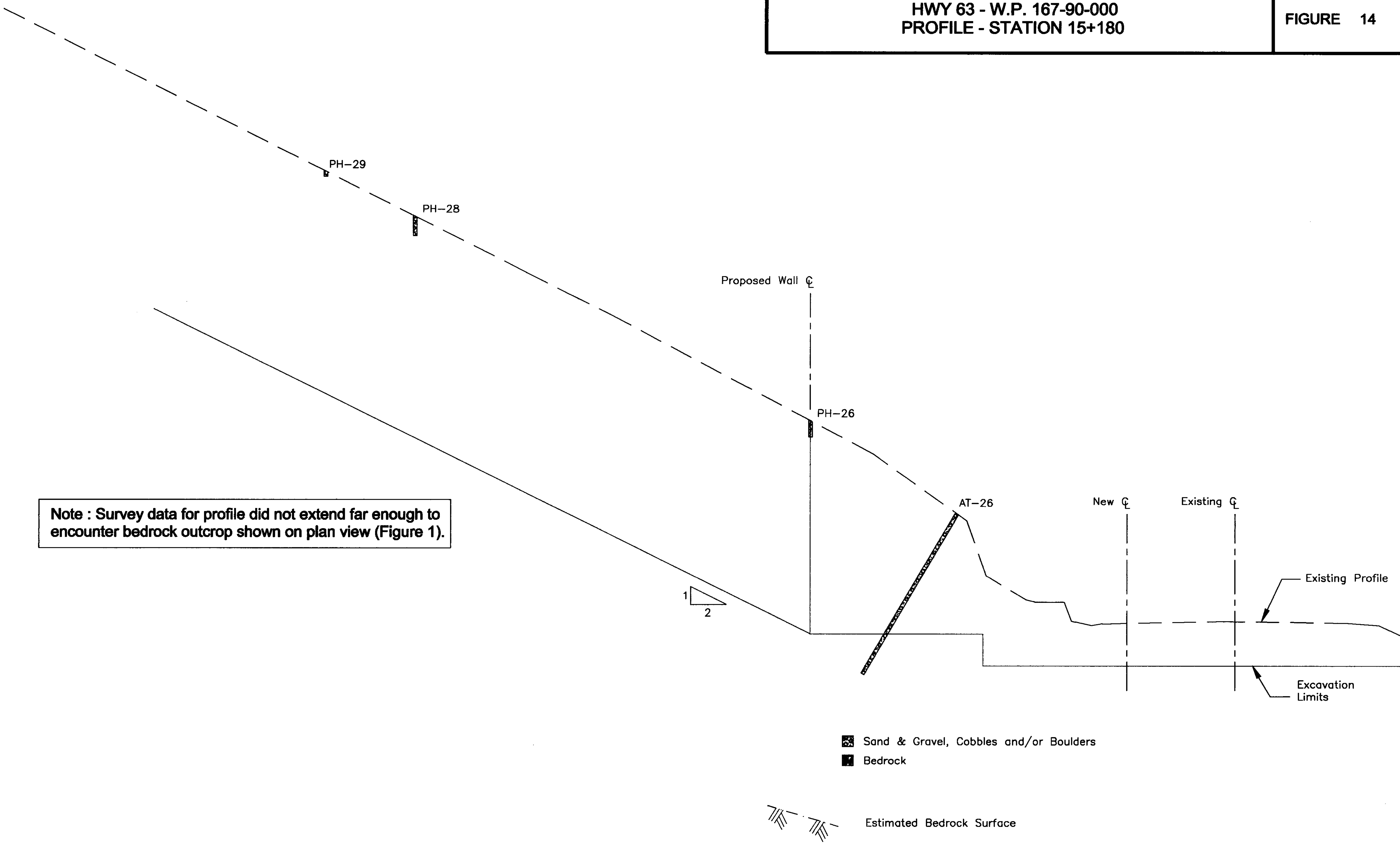


Scale 1:200 | Date DEC/98 | Drawn KLS
File 9819109FIGS.dwg | Project 981-9109 | Chkd DMC

Golder Associates

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+180

FIGURE 14



Note : Survey data for profile did not extend far enough to encounter bedrock outcrop shown on plan view (Figure 1).

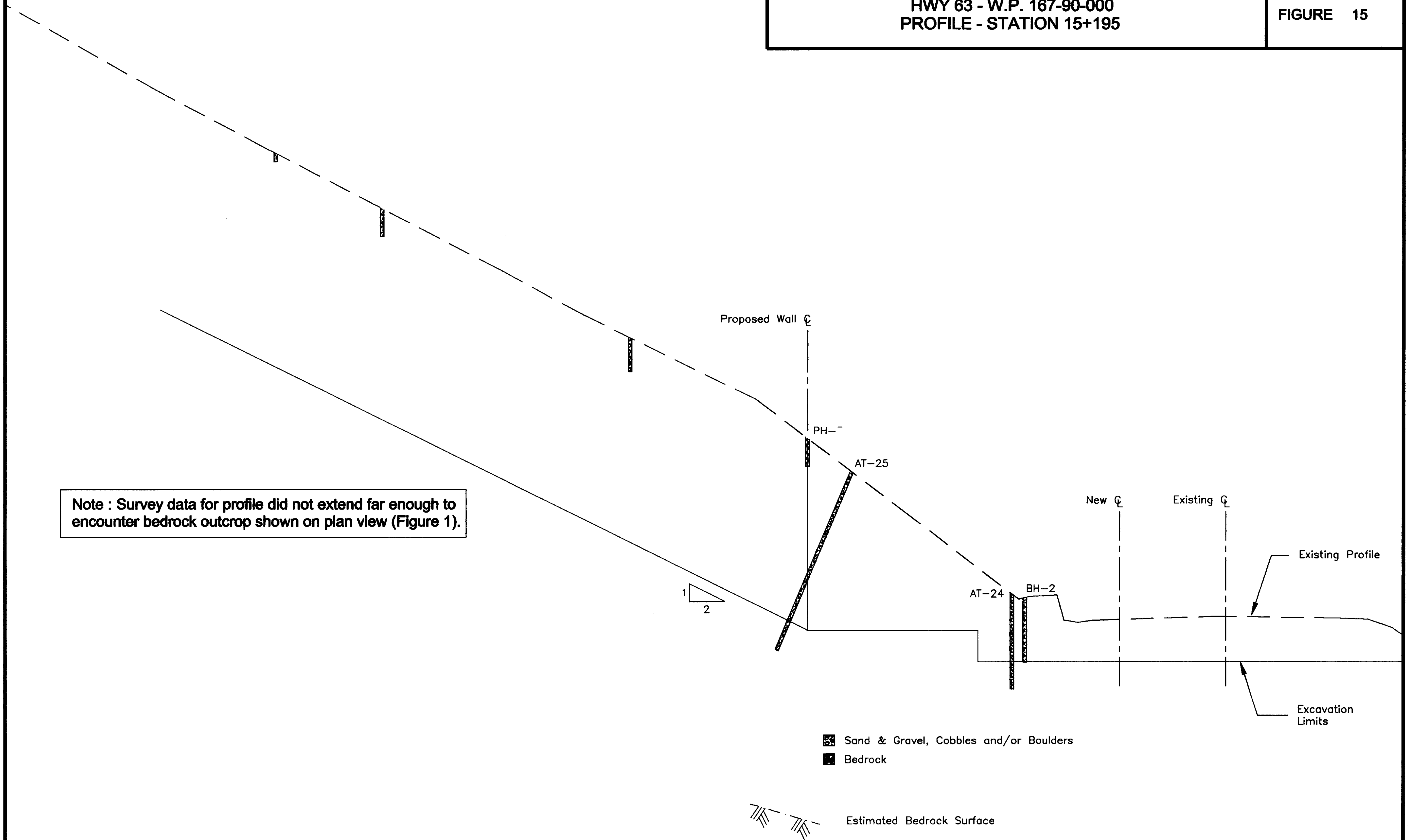
Scale 1:100
File 9819109FIGS.dwg
Date DEC/98
Project 981-9109

Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+195

FIGURE 15



Note : Survey data for profile did not extend far enough to encounter bedrock outcrop shown on plan view (Figure 1).

Scale ....1:100.........
File 9819109FIGS.dwg

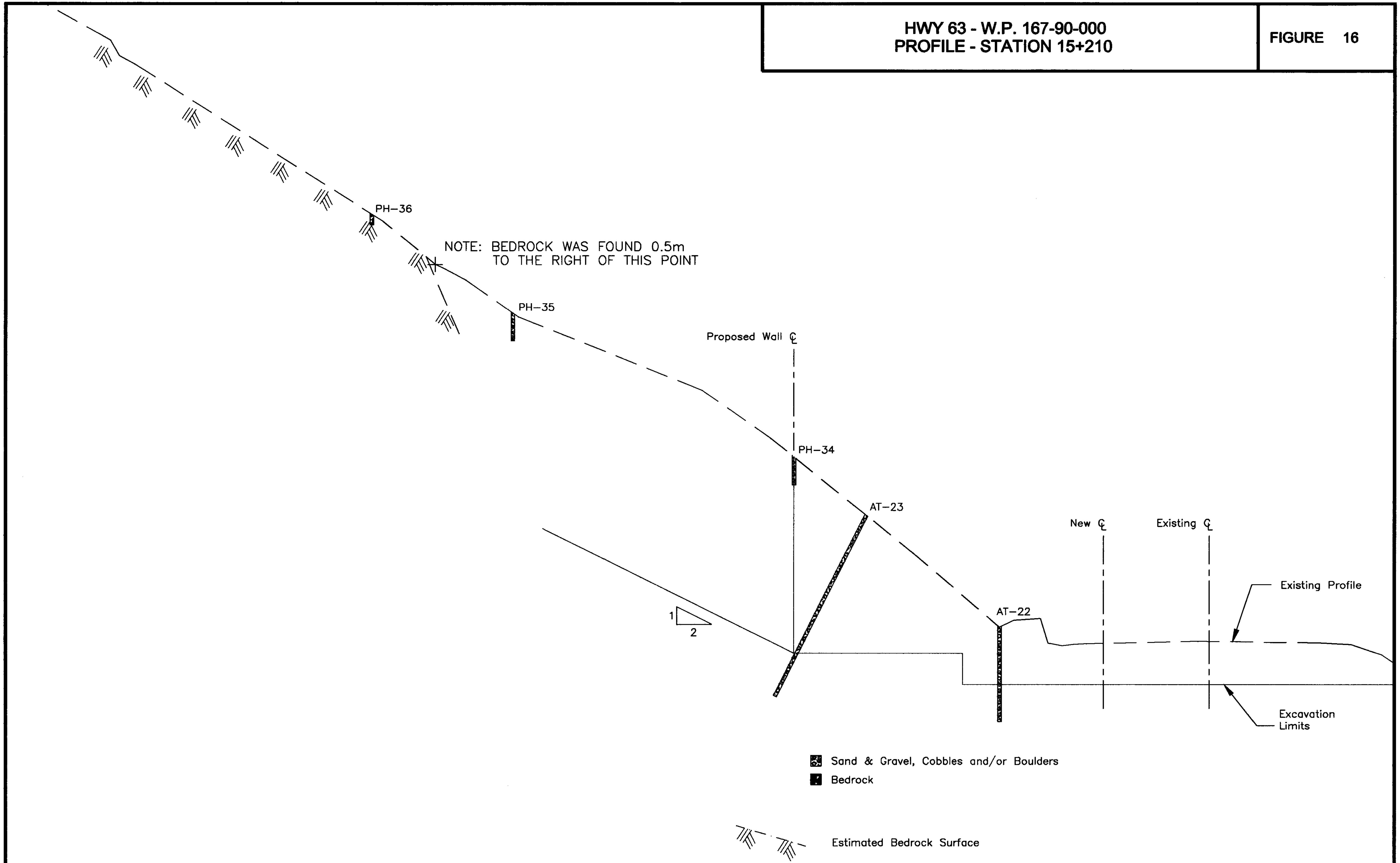
Date ....DEC/98....
Project 981-9109

Golder Associates

Drawn ...KLS....
Chkd ....DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+210

FIGURE 16



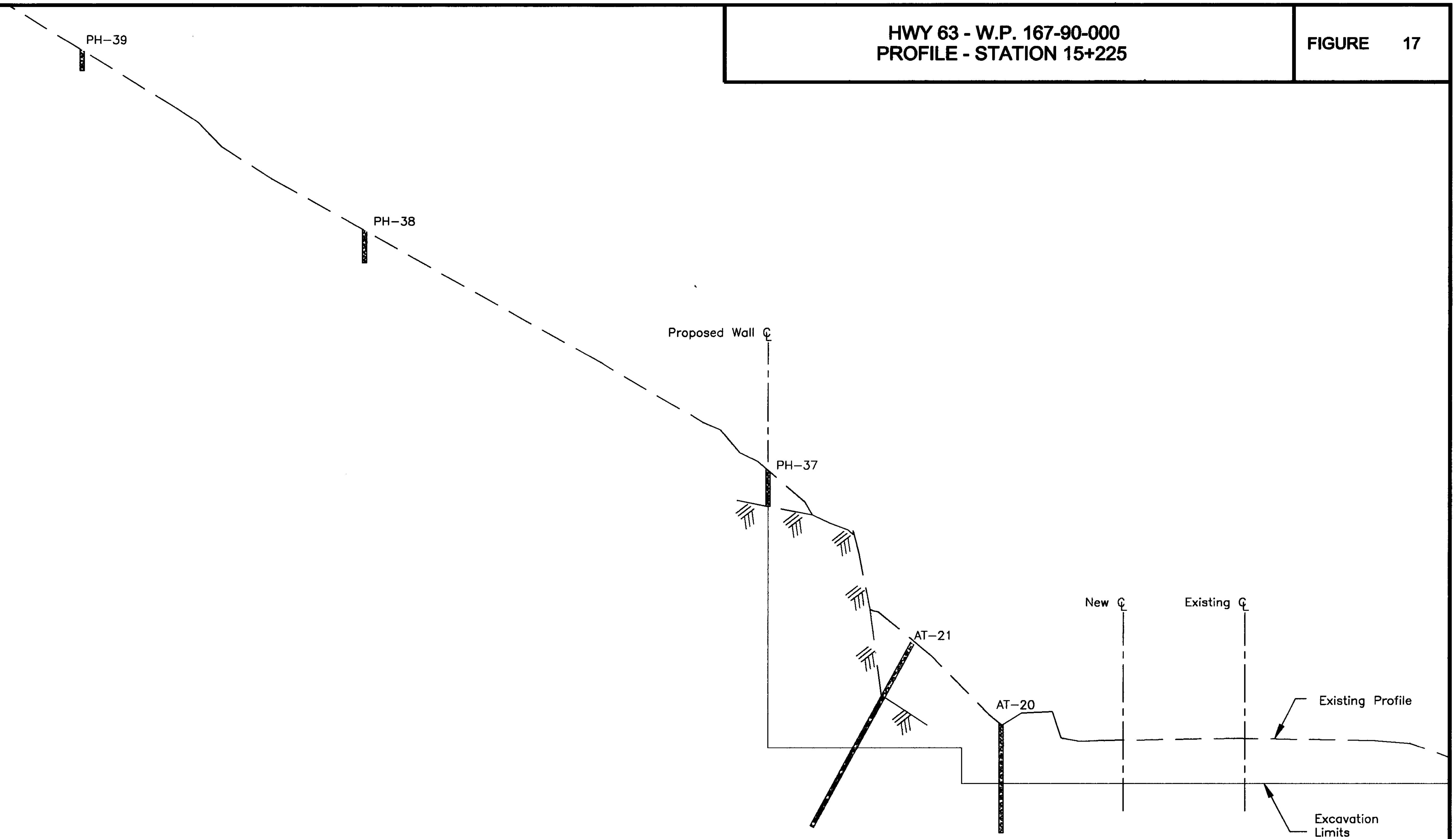
Scale 1:100 | Date DEC/98
File 9819109FIGS.dwg | Project 981-9109

Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+225

FIGURE 17



Sand & Gravel, Cobbles and/or Boulders
Bedrock
Estimated Bedrock Surface

Scale ....1:100........ Date ....DEC/98....

File 9819109FIGS.dwg Project 981-9109

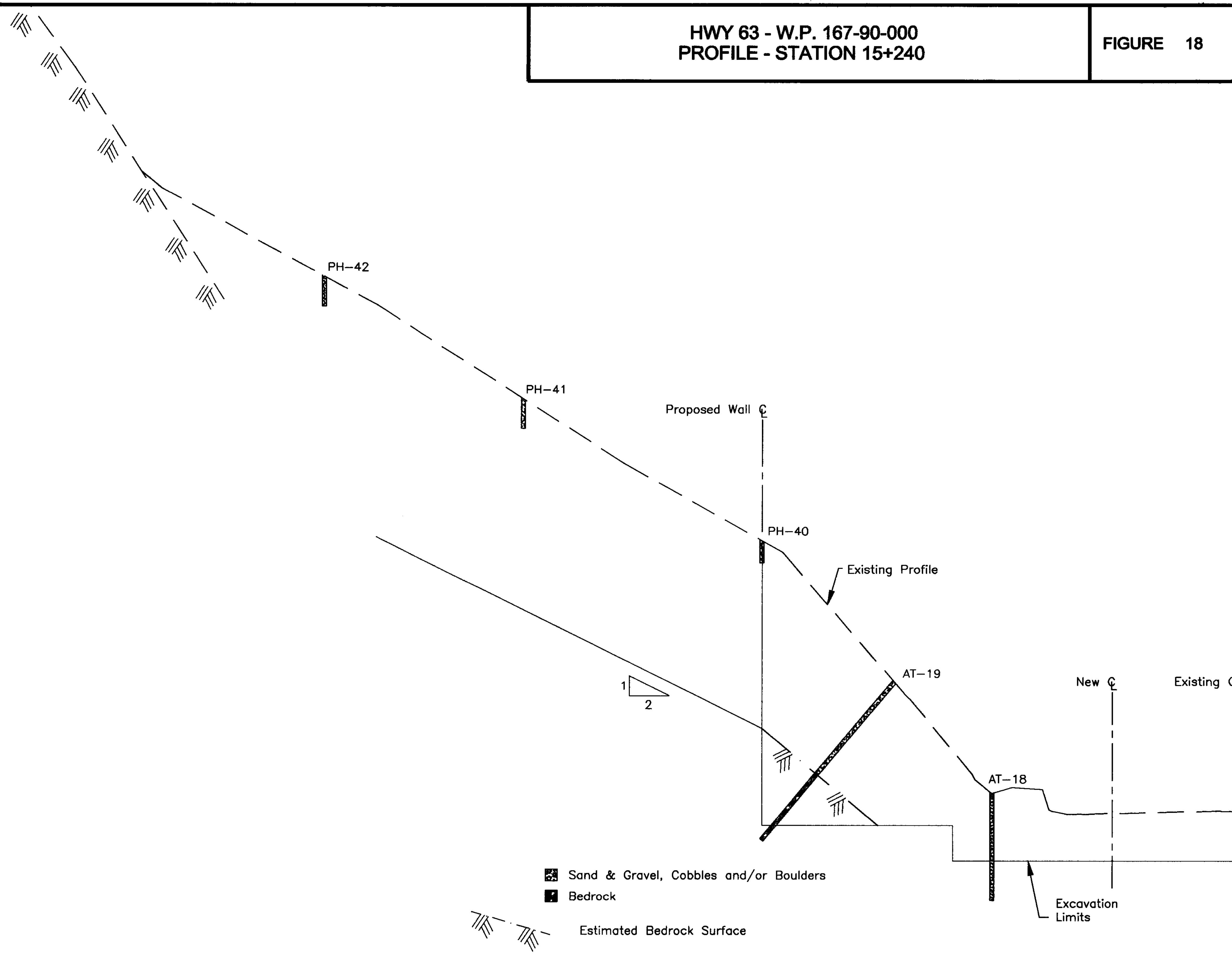
Golder Associates

Drawn ..KLS....

Chkd ......DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+240

FIGURE 18



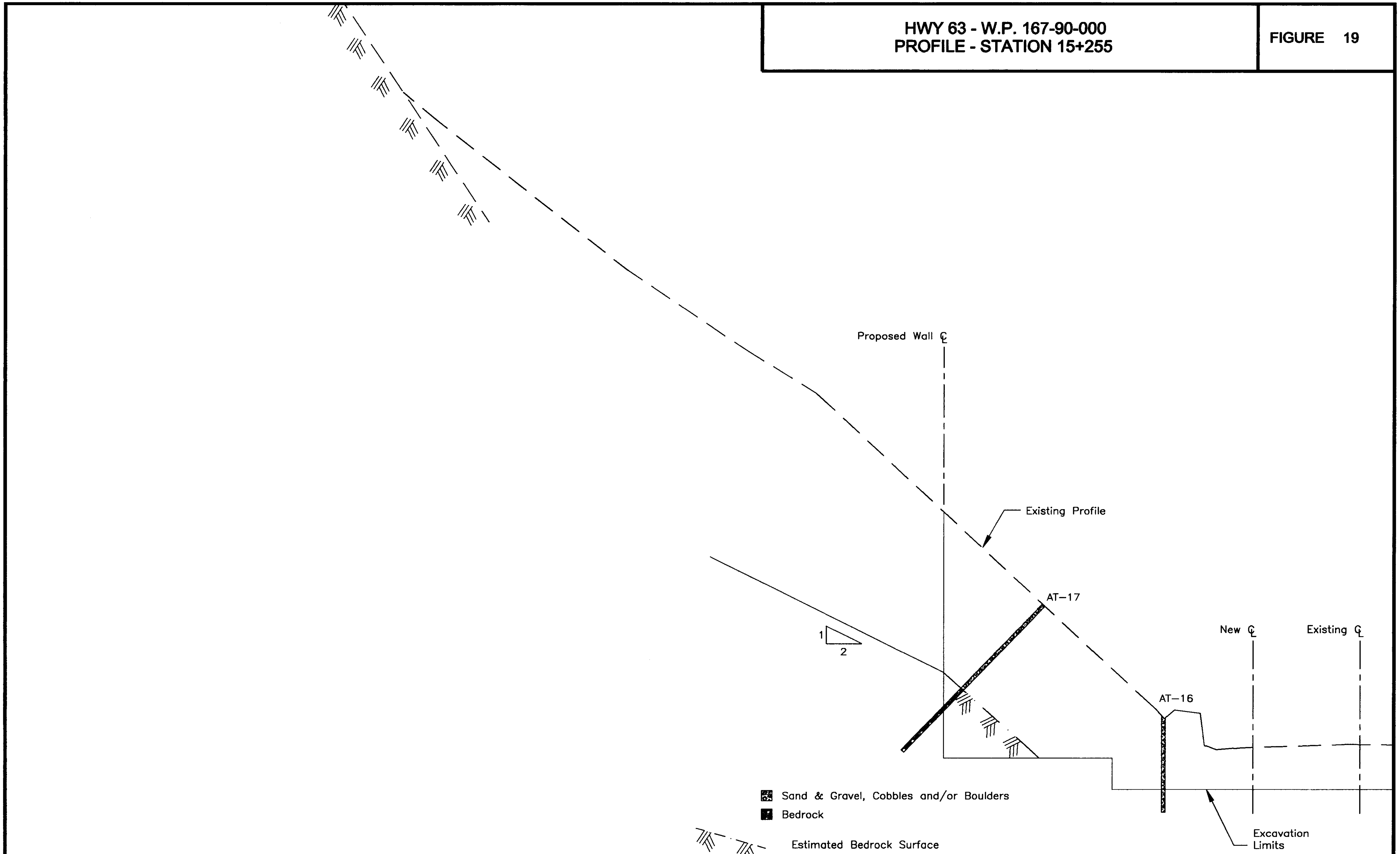
Scale 1:100 | Date DEC/98
File 9819109FIGS.dwg | Project 981-9109

Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+255

FIGURE 19



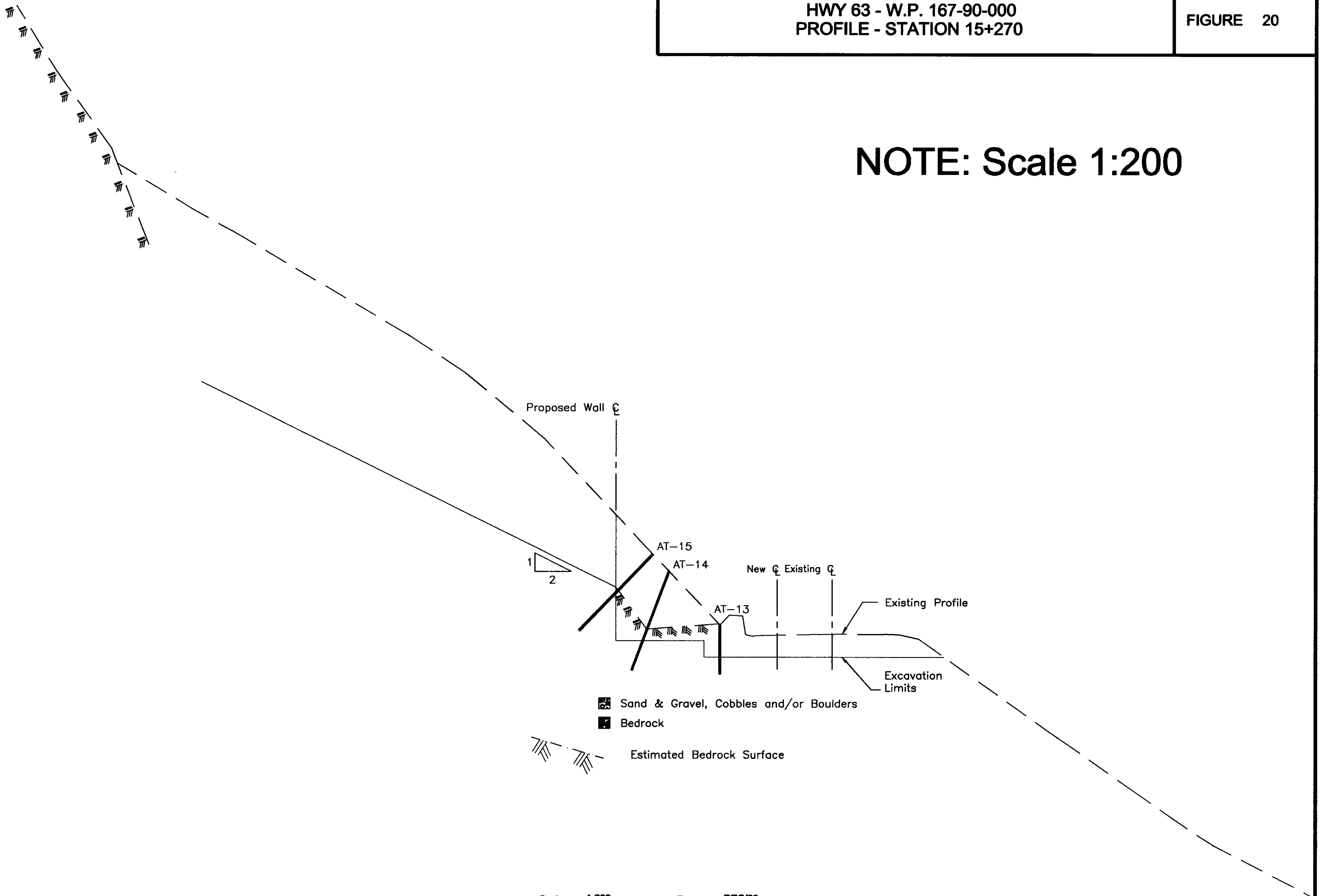
Scale 1:100 | Date DEC/98 | File 9819109FIGS.dwg | Project 981-9109

Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+270

FIGURE 20



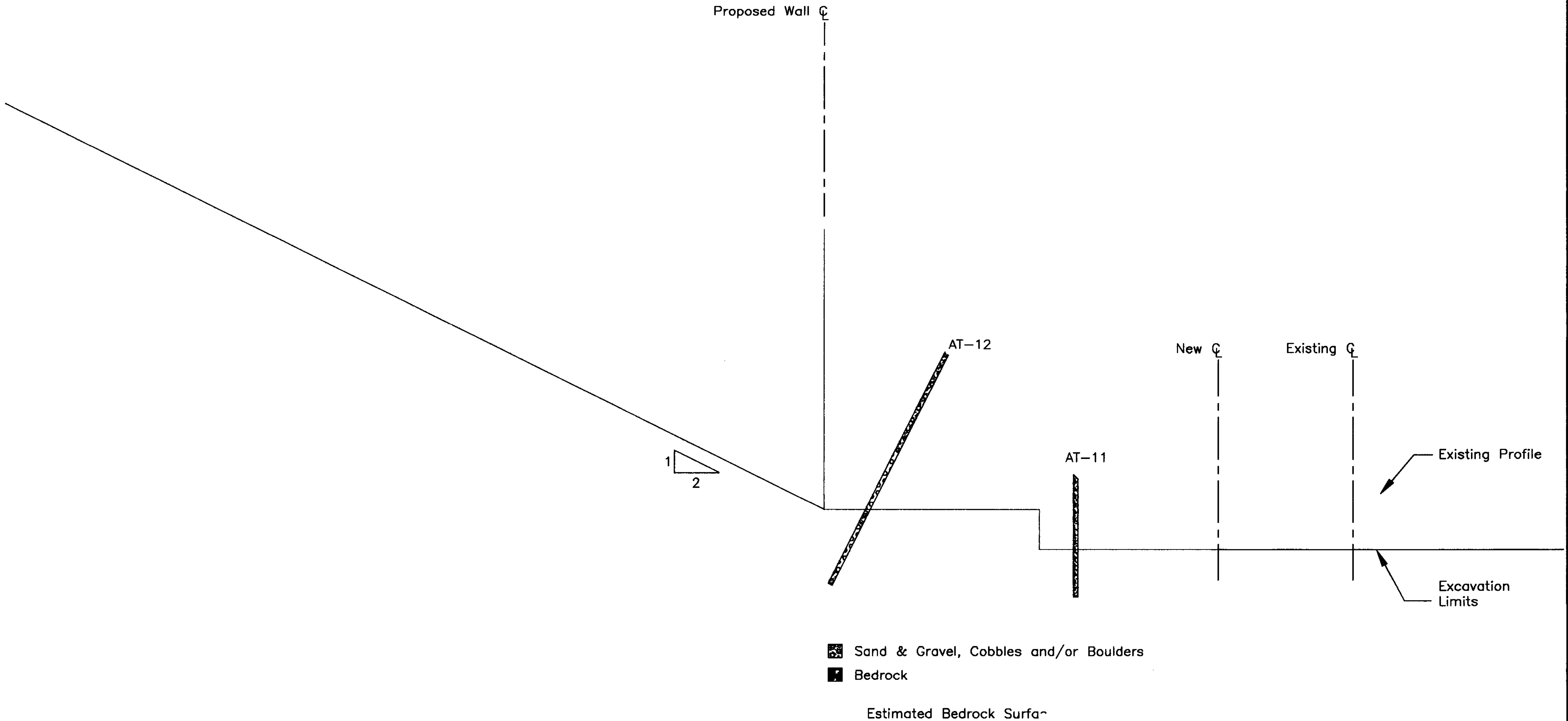
Scale 1:200
Date DEC/98
File 9819109FIGS.dwg
Project 981-9109

Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+285

FIGURE 21



Scale 1:100 | Date DEC/98

File 9819109FIGS.dwg | Project 981-9109

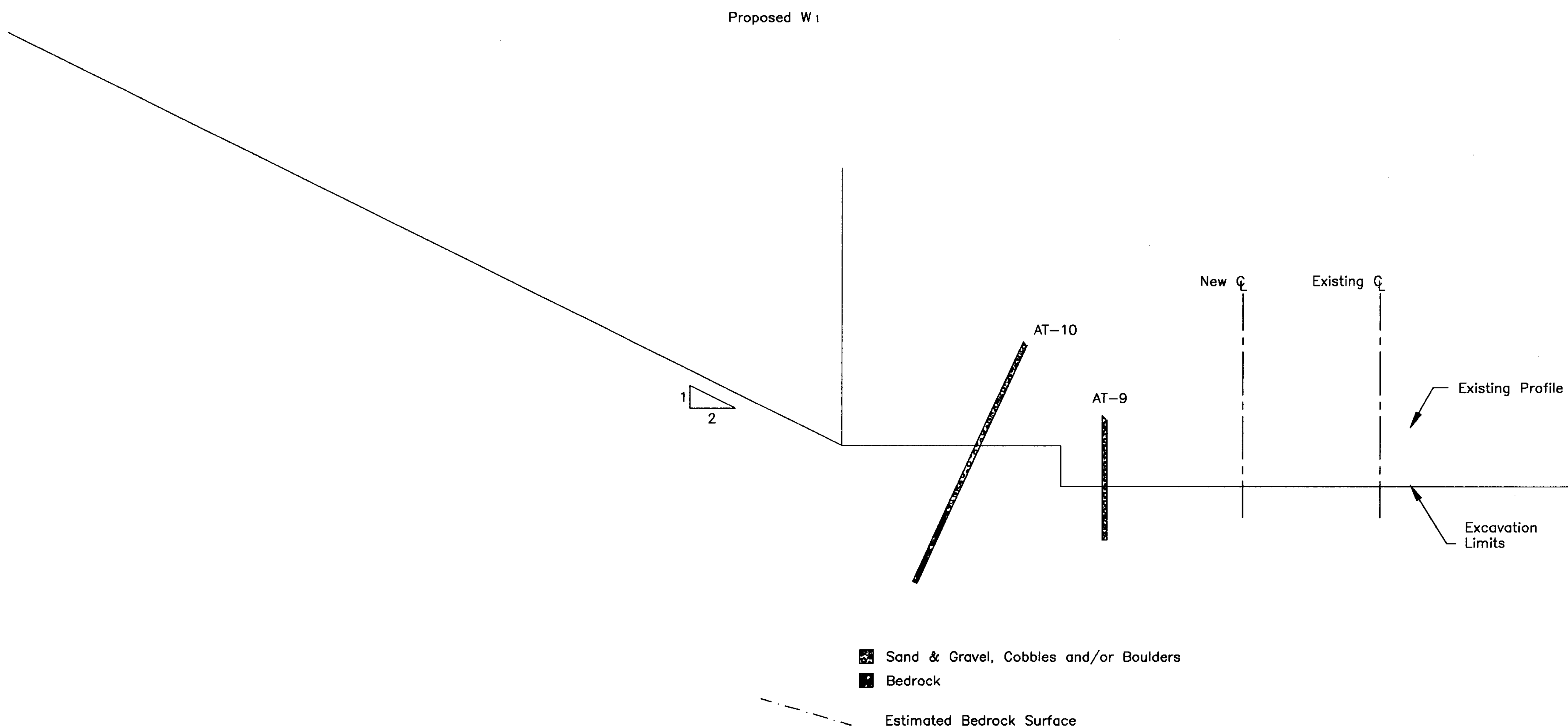
Golder Associates

Drawn KLS

Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+300

FIGURE 22



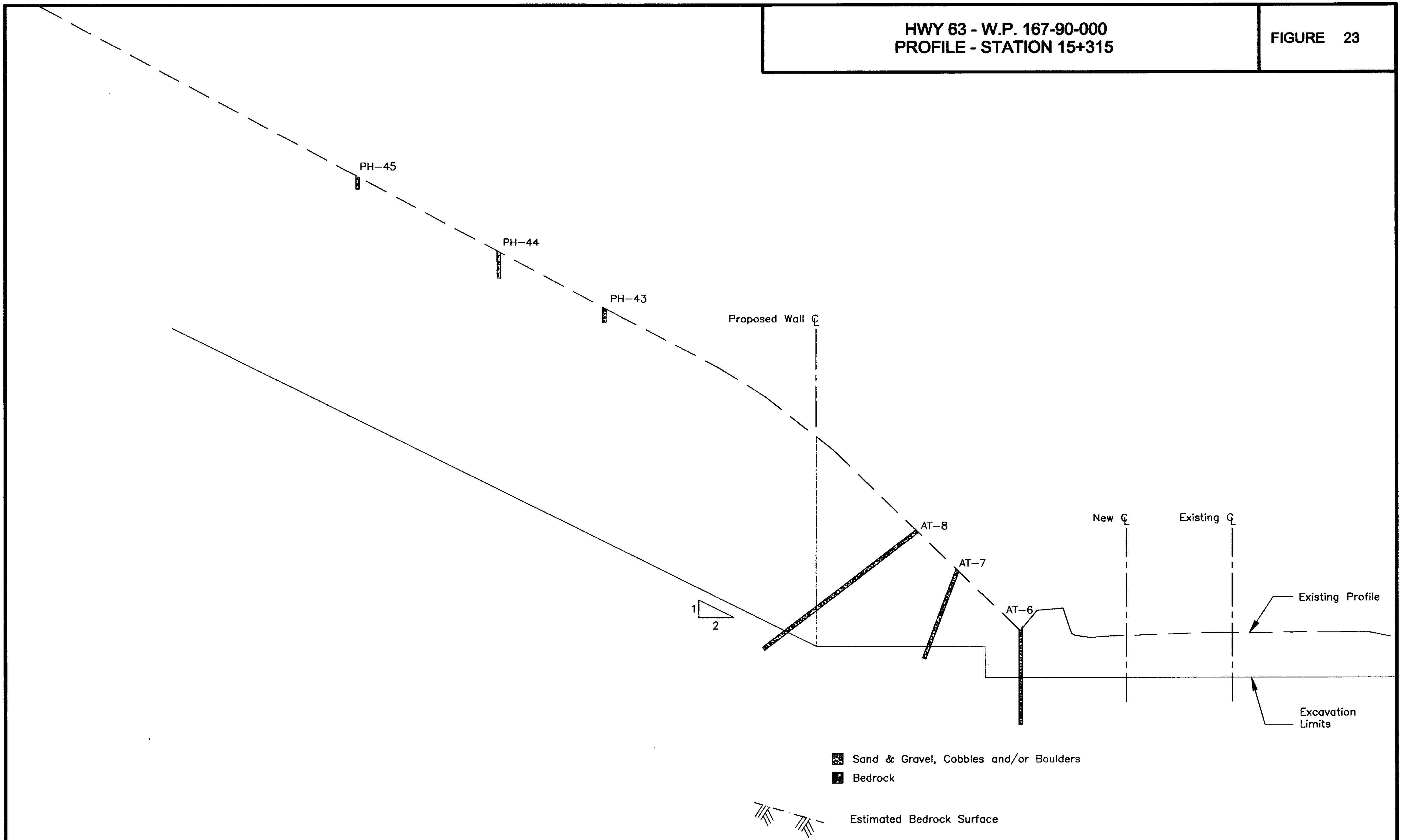
Scale 1:100
File 9819109FIGS.dwg
Date DEC/98
Project 981-9109

Golder Associates

Drawn KLS
Chkd DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+315

FIGURE 23



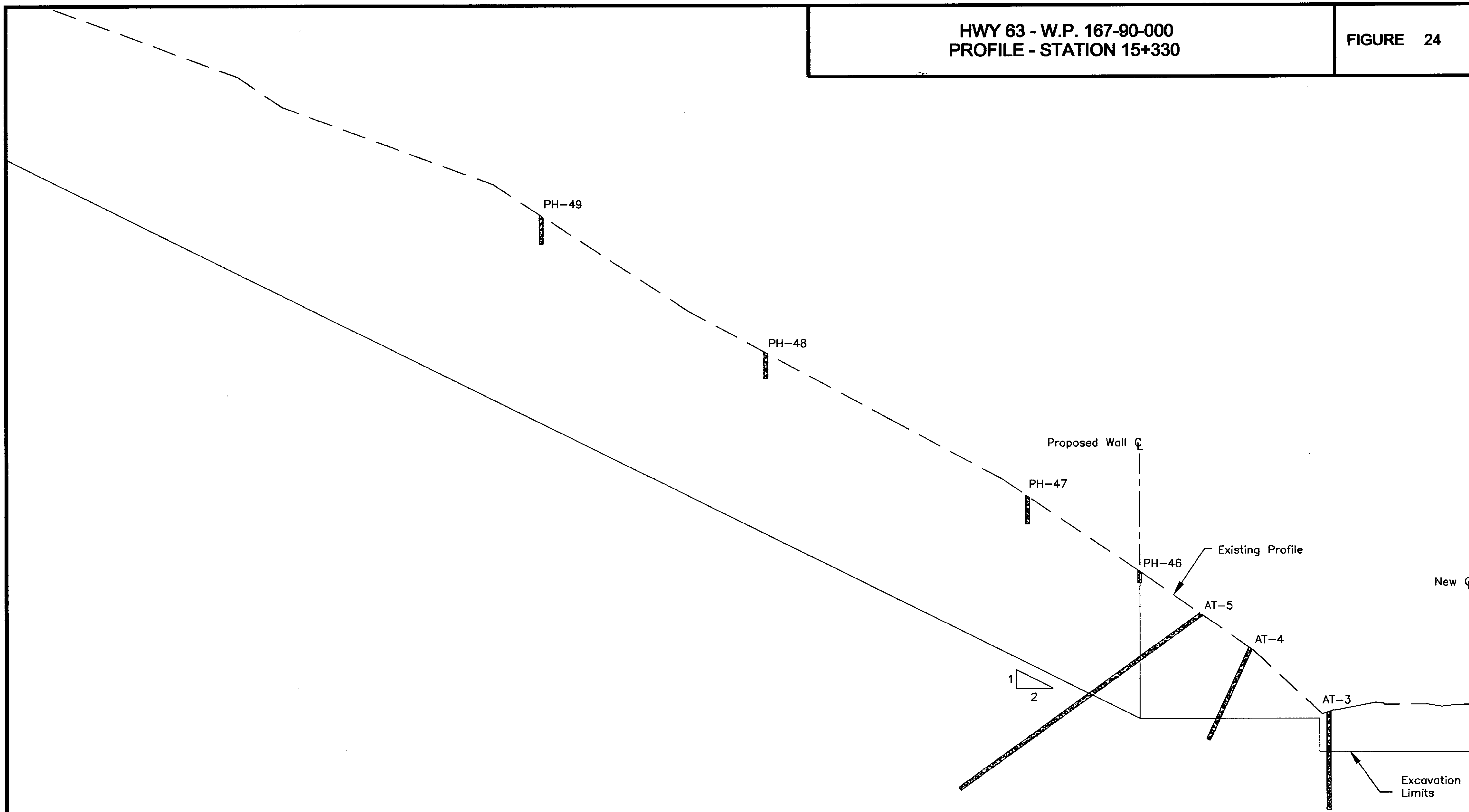
Scale ....1:100.........
Date ....DEC/98.....
File 9819109FIGS.dwg
Project 981-9109

Golder Associates

Drawn ...KLS....
Chkd ....DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+330

FIGURE 24



Sand & Gravel, Cobbles and/or Boulders

Bedrock

Estimated Bedrock Surface

Scale ......1:100........ Date ....DEC/98....

File 9819109FIGS.dwg Project 981-9109

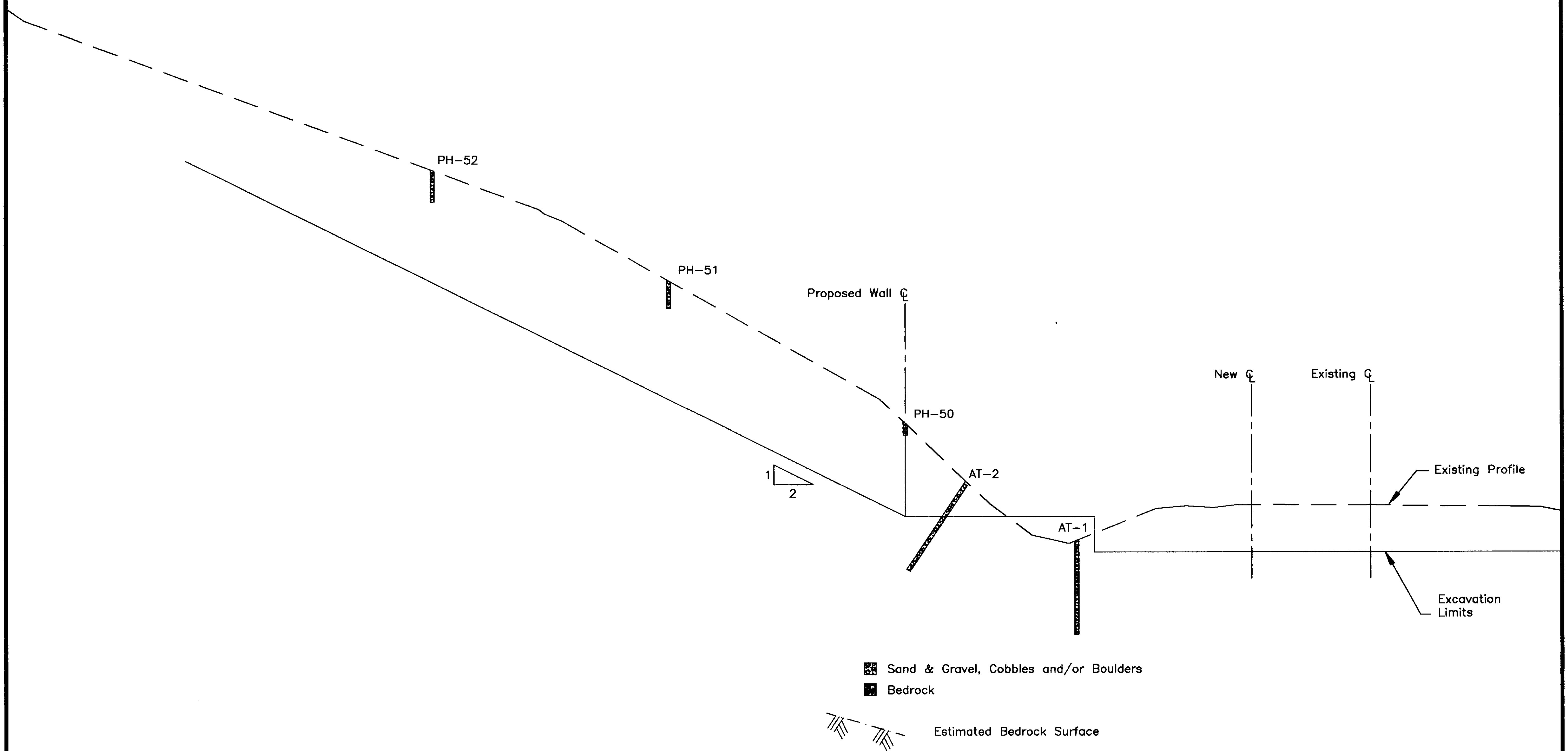
Golder Associates

Drawn ..KLS....

Chkd ......DMC

HWY 63 - W.P. 167-90-000
PROFILE - STATION 15+345

FIGURE 25



Scale 1:100
File 9819109FIGS.dwg
Date DEC/98
Project 981-9109

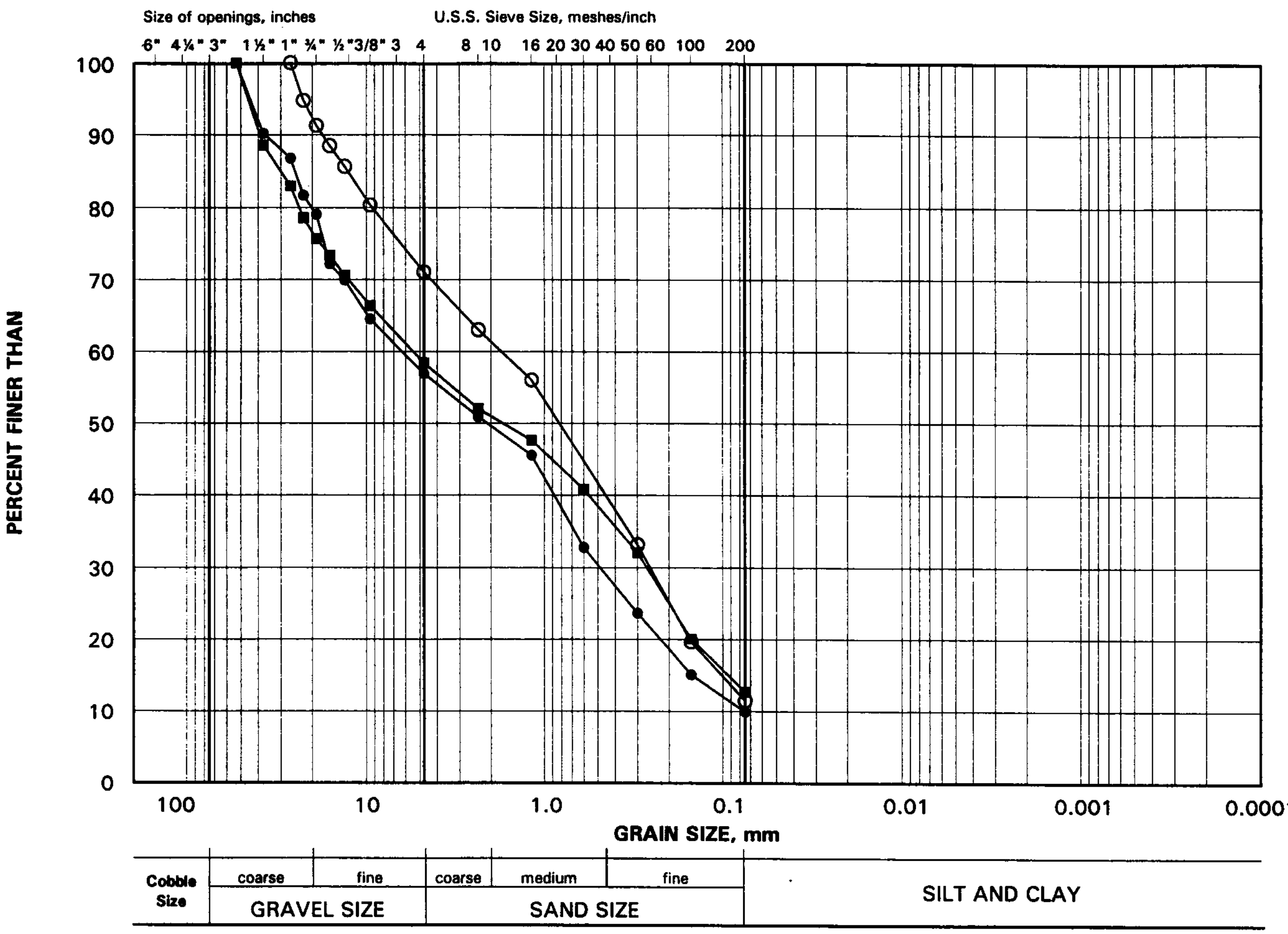
Golder Associates

Drawn KLS
Chkd DMC

GRAIN SIZE DISTRIBUTION

FIGURE 26

Hwy 63 Retaining Wall



LEGEND

SYMBOL	STATION	SAMPLE
●	15+152	1
■	15+241	2
○	15+316	3

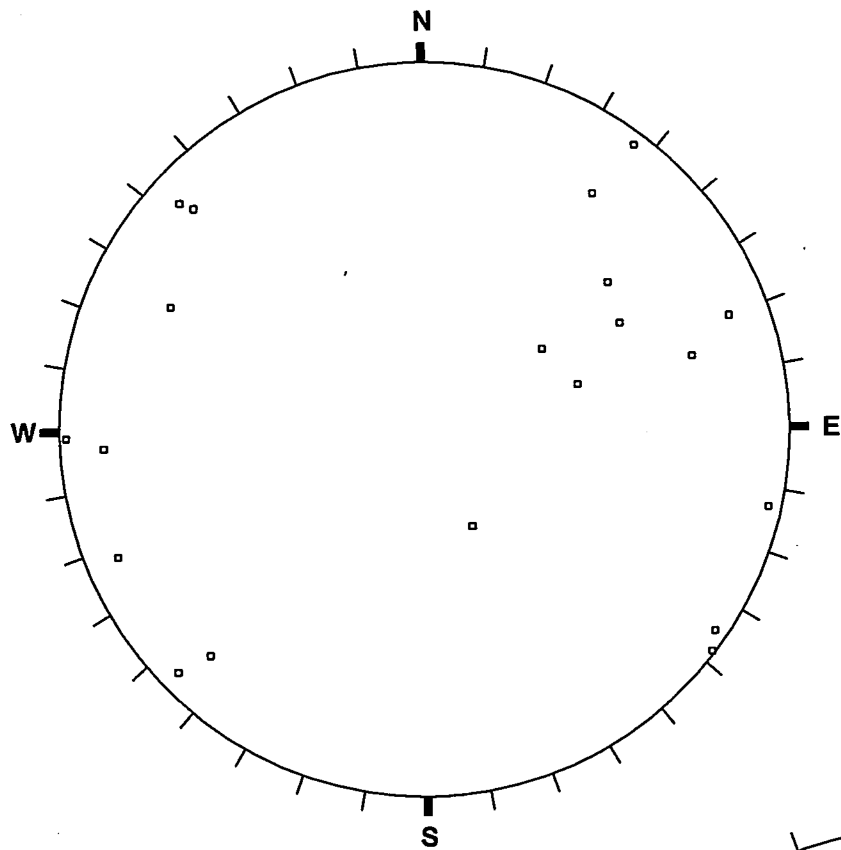
ProGrain ver. 2.01

Project 981-9109

Golder Associates

STEREOPLOT OF MAJOR STRUCTURE

FIGURE 27

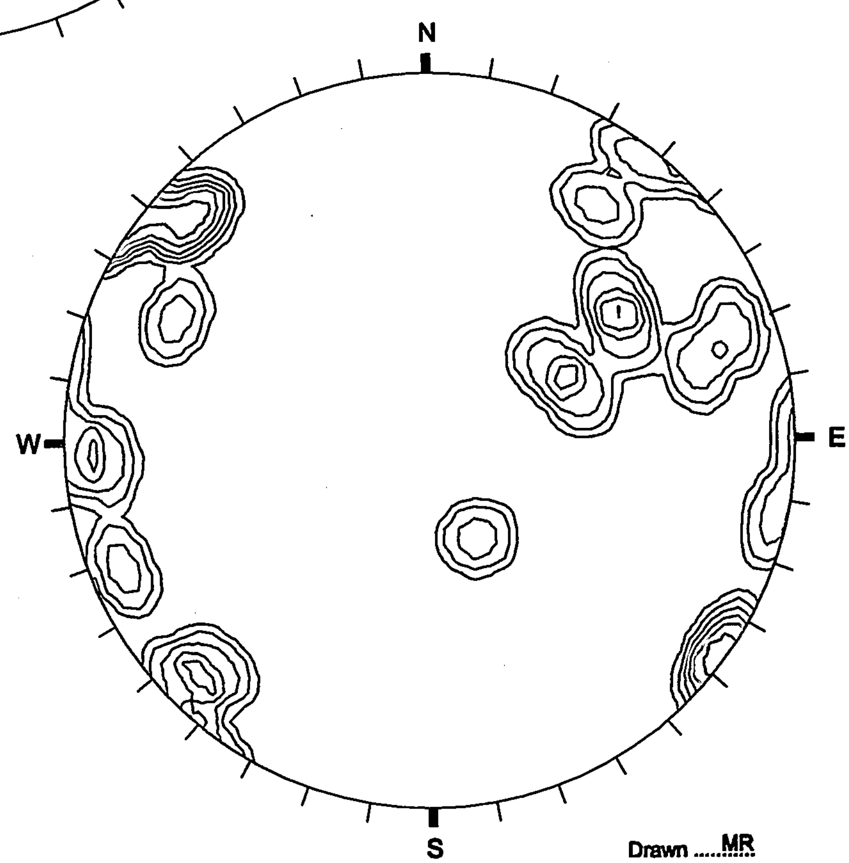


POLE PLOT

▫ POLES

EQUAL AREA
LOWER HEMISPHERE

20 Poles Plotted
20 Data Entries



CONTOUR PLOT

FISHER POLE CONCENTRATIONS
% of total per
1.0 % area

Minimum Contour	=	1.5
Contour Interval	=	1.5
Max.Concentration	=	9.84

Date ...October, 1998...

Project ...981-9109...

Golder Associates

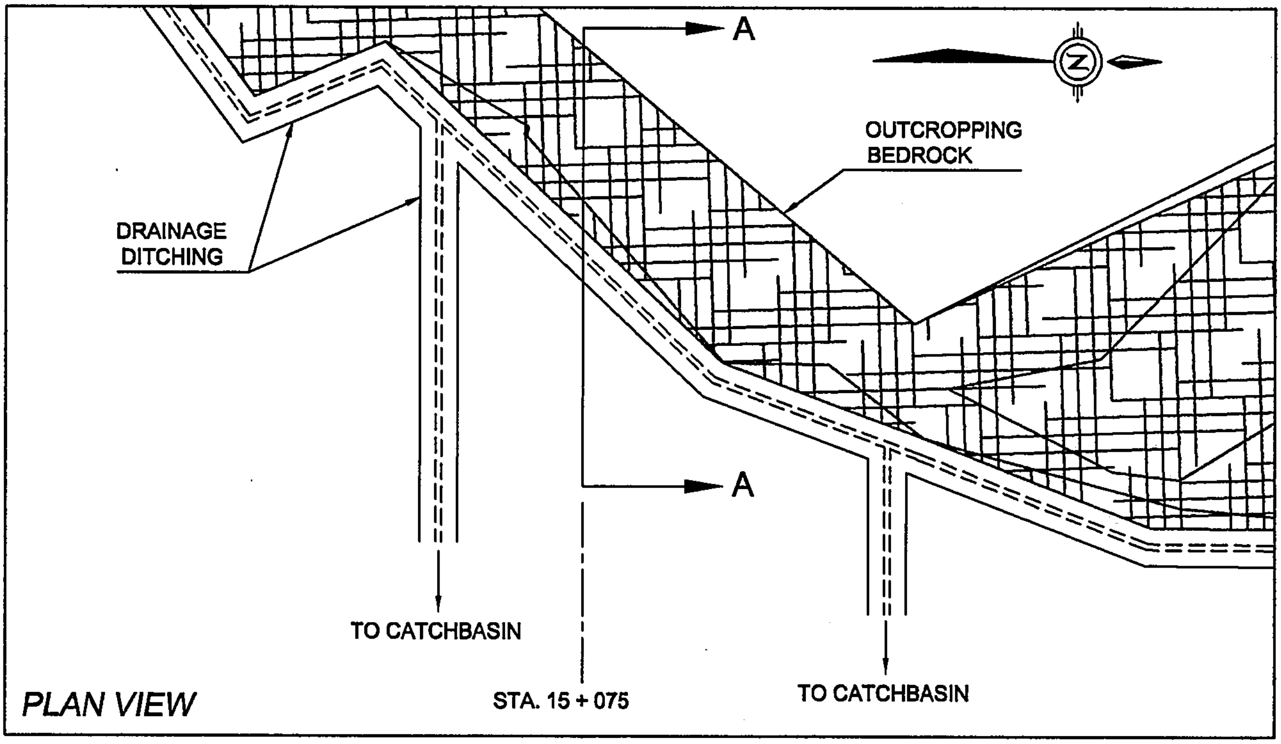
Drawn ...MR...

Chkd ....

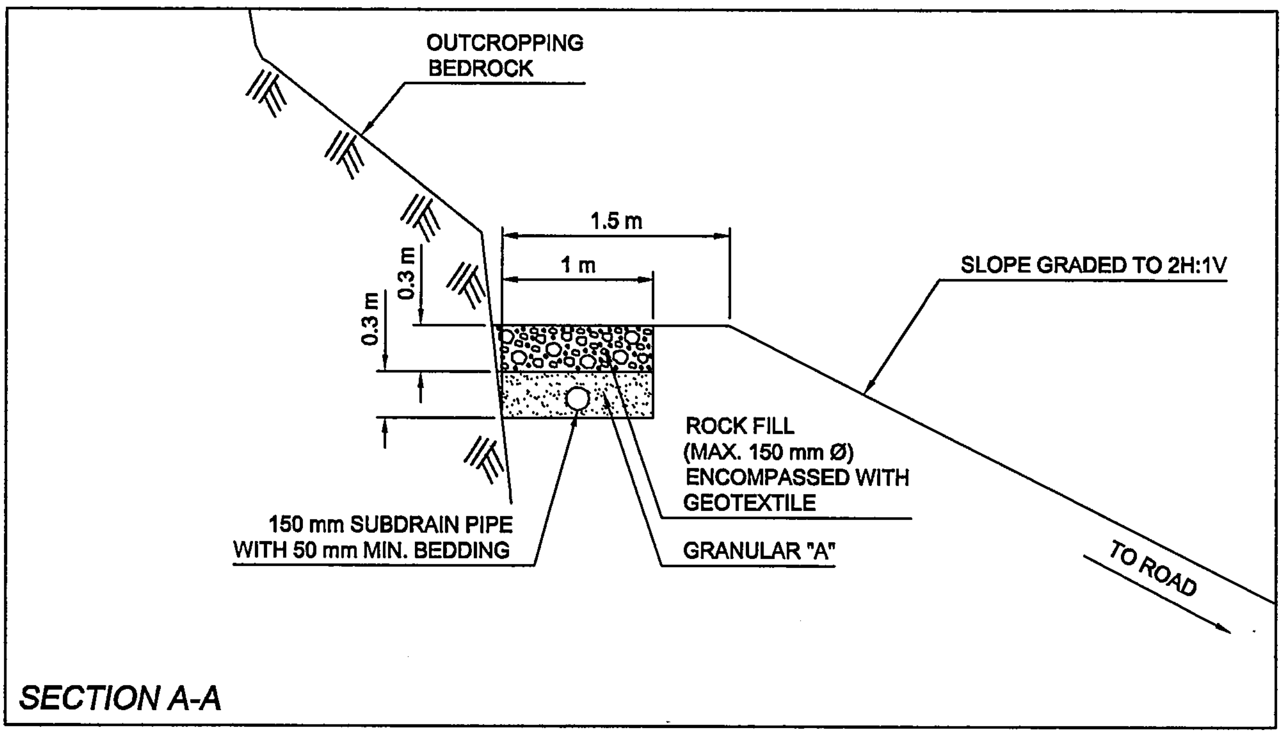
981-9109\structur\fig004.skd

TYPICAL DRAINAGE DETAILS
W.P. NO. 167-90-00
HIGHWAY 63

FIGURE 28



SCALE: 1:200



SCALE: 1:100

Date ...DECEMBER 1998

Project 981-9109

Golder Associates

Drawn ...G.K....

Chkd ....

SCALE: NOTED FILE: 9819109FIG28.DWG

APPENDIX A

SITE PHOTOGRAPHS

HWY 63 PHOTOGRAPHS OF AIR TRACK DRILL RIG	PHOTO 1

HWY 63 PHOTOGRAPHS OF AIR TRACK DRILL RIG	PHOTO 2

Date OCT/98

Project 981-9109

Golder Associates

Drawn PZ

Chkd

APPENDIX B

STEREOGRAPHIC TECHNIQUES

B.1 GEOLOGICAL DATA PRESENTATION

Field data derived from structural geological mapping essentially consists of descriptions of various types of geological discontinuities such as joints, faults and bedding planes. The most significant aspect of these features is their orientation in space. Whenever mapping projects involve the collection of large amounts of such orientation data, it is necessary to resort to some means of graphically displaying these orientations in order to identify sets of planes with similar orientation. The plotting techniques which are most widely used are called stereographic or equal area projections. These diagrams (stereonets) consist of plots of the collected orientations of individual field readings.

If each of the geological discontinuities is assumed to pass through the centre of an imaginary oriented sphere, then the intersection of the plane with the sphere defines a great circle (see Figure A-l) which uniquely orients the plane in space. A line drawn perpendicular to the plane through the centre of the sphere intersects the sphere at two points (poles) which also uniquely define the plane' s orientation. Because of symmetry, only half of the reference sphere is required to display all of the orientation data and by convention, the lower hemisphere is used. For convenience, the data on the lower hemisphere is projected towards the zenith through the equational plane (see Figure A-1) and this plane projection (called the stereographic project) is used to conveniently summarize the collection of orientation data.

Any inclined geological plane is defined by its inclination to the horizontal (its dip) and by its orientation with respect to north which may be defined by the strike or by the dip direction of the plane. The relationship between these terms is illustrated in Figure A-l. Note that dip direction is always measured clockwise from north and that the strike line is at 90 degrees to the dip direction of the plane.

A hypothetical set of discontinuity data has been plotted for illustrative purposes on Figure A-2. In total, 351 poles have been plotted directly onto a polar equal area stereonet. The resulting scatter diagram isolates certain families of discontinuities (bedding planes, joints) as well as individual features (such as the fault). Each point within the scatter diagram represents a single measurement of the orientation of one geological discontinuity in the field.

Such a graphical representation can still be somewhat confusing since individual planes with similar geological origin are rarely perfectly parallel. Lack of parallelism arises from two factors: one is the natural variability between similar geological features in situ, the second is measurement variability arising from the irregularity of surface shape of individual geological features. Hence, unless an extremely large number of data points are collected in the field, point concentrations which define sets of planes with similar orientation tend to be diffuse and difficult to identify.

In order to establish overall trends for sets of geological discontinuities, the density of poles as plotted on the scatter diagram are contoured.

Such contouring may be carried out by hand by counting the number of poles on the stereonet in order to accentuate any patterns of preferred orientation. (Alternatively the method can be readily carried out by computer techniques). The Schmidt method is the most common counting technique although other more sophisticated computational techniques are available. By hand, the counting is carried out by moving a circular counter of one per cent of the area of the scatter diagram to successive intersections on a rectangular grid. The number of poles falling within the counting circle is entered at that point. These numbers are then divided by the total number of poles to obtain the percentage of poles per one per cent area of the stereonet and the grid is contoured.

Many of the density diagrams contained in this report use a slightly different counting technique. Rather than arbitrarily giving each pole a full weight over a one per cent area of the diagram as in the Schmidt method, each pole on the reference sphere is assigned a probability distribution corresponding to variations in probable measurement accuracy (Figure A-3). Not only does this type of distribution have physical significance, but it produces a more continuous and realistic contoured diagram. Also, the geometric distortion caused by counting on the diagram itself is eliminated by carrying out the counting on the reference sphere.

The bell-shaped probability distribution used for such counting is known as a spherical normal or Fisher distribution (Figure A-3). The probability is greatest at the recorded position of the pole, tapering off to negligible size within an arbitrary distance (usually ten degrees) in any direction. The significance of this distribution is related to the variation inherent in any geological measurement; rarely will a measurement of discontinuity attitude be out by ten degrees, occasionally it may be out by five degrees, but differences of one or two degrees from the actual attitude are common. Such a probability distribution for representing each recorded pole therefore reflects the random error inherent in the measurement process, and replaces each discrete pole by a continuous statistical probability distribution centred about the recorded pole.

APPENDIX C

LIST OF ABBREVIATIONS AND SYMBOLS
&
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N <u>Blows/300 mm or Blows/ft.</u>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, S_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416

ln x, natural logarithm of x

$\log_{10}$ x or log x, logarithm of x to base 10

g acceleration due to gravity

t time

F factor of safety

V volume

W weight

II. STRESS AND STRAIN

γ shear strain

Δ change in, e.g. in stress: $\Delta \sigma$

ε linear strain

ε_v volumetric strain

η coefficient of viscosity

v Poisson's ratio

σ total stress

σ' effective stress ($\sigma' = \sigma - u$)

σ'_{vo} initial effective overburden stress

$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)

σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$

τ shear stress

u porewater pressure

E modulus of deformation

G shear modulus of deformation

K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)

$\rho_d(\gamma_d)$ dry density (dry unit weight)

$\rho_w(\gamma_w)$ density (unit weight) of water

$\rho_s(\gamma_s)$ density (unit weight) of solid particles

γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)

D_R relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)

e void ratio

n porosity

S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content

w_l liquid limit

w_p plastic limit

I_P plasticity Index = $(w_l - w_p)$

w_s shrinkage limit

I_L liquidity index = $(w - w_p) / I_p$

I_C consistency index = $(w_l - w) / I_p$

e_{max} void ratio in loosest state

e_{min} void ratio in densest state

I_D density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential

q rate of flow

v velocity of flow

i hydraulic gradient

k hydraulic conductivity (coefficient of permeability)

j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)

C_r recompression index (overconsolidated range)

C_S swelling index

C_α coefficient of secondary consolidation

m_v coefficient of volume change

c_v coefficient of consolidation

T_v time factor (vertical direction)

U degree of consolidation

σ'_p pre-consolidation pressure

OCR Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength

ϕ' effective angle of internal friction

δ angle of interface friction

μ coefficient of friction = $\tan \delta$

c' effective cohesion

c_u, S_u undrained shear strength ($\phi = 0$ analysis)

p mean total stress $(\sigma_1 + \sigma_3)/2$

p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$

q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$

q_u compressive strength $(\sigma_1 - \sigma_3)$

S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
II - Parallel To	
⊥ - Perpendicular To	

Golder Associates Ltd.
662 Falconbridge Road
Sudbury, Ontario, Canada P3A 4S4
Telephone (705) 524-6861
Fax (705) 524-1984

Golder Associates

January 19, 1999

981-9109

Proctor & Redfern Limited
222 McIntyre Street West, Suite 410
North Bay, Ontario
P1B 2Y8

Attention: Mr. A.E. Rose, C.S.T.

RE: ADDENDUM #2
GEOTECHNICAL/FOUNDATION INVESTIGATION AND DESIGN
HIGHWAY 63 - W.P. 167-90-00
MTO DISTRICT 54, SUDBURY
YOUR PROJECT EO 97447

Dear Sirs:

Further to the Technical Review Meeting of January 14, 1999, we provide this second addendum report to provide clarifications to concerns presented. This addendum report must be read with our original reports number 981-9109, dated December, 1998 and number 981-9110, dated August, 1998, along with our first addendum, letter report dated January 5, 1999.

The concerns presented during the Technical Review Meeting include the following:

- erosion control of soil slopes at the ELDEE hill;
- erosion control slope at the bedrock and soil interface at the ELDEE hill;
- the stability of the right hand slope along ELDEE hill; and
- the use of rock fill for road widening.

EROSION CONTROL OF SOIL SLOPES - ELDEE HILL

The reconstruction of the left side slope at the ELDEE hill to accommodate the proposed centre-line shift will have sections that comprise the grading of the native soils to a 2 horizontal to 1 vertical (2H:1V) configuration. At these locations, it has been proposed to replace the design of seeding and mulching the slopes (for erosion control) with a prefabricated erosion control blanket. Golder Associates is in agreement with this substitution.

EROSION CONTROL AT BEDROCK/SOIL INTERFACE - ELDEE HILL

Section 5.3, "Permanent Cut Slopes" of our report number 981-9109 discusses erosion control at the interface of exposed bedrock and the 2H:1V soil slopes. The original recommendation in our report (as shown on Figure 28) included the use of rock filled interceptor drains leading to catchbasins at the base of the slope. Proctor & Redfern has stated that an interceptor drain system would not be practical due to the proximity and anticipated slope of the bedrock surface, along with the variable thicknesses of overburden. As such, the use of such a system has been abandoned. Protection of the native soils from scouring due to surface water run-off from the exposed bedrock using erosion control blanks will most likely not suffice at the soil bedrock interface.

The protection of these soils from scouring remains a concern and they should therefore be protected with a rip-rap layer. The rip-rap layer should be 300 mm thick, with a geotextile separator between the rip-rap and the native soils. The rip-rap layer should extend for a vertical distance of at least 10 m from the bedrock face. This distance should be reviewed for each section and should it prove to be more practical, the rip-rap could extend the full distance (i.e., greater than 10 m) from the base of the slope to the bedrock face, to avoid small sections of erosion control blankets and to improve the seepage flow to the base f the slope.

STABILITY OF THE RIGHT HAND SLOPE - ELDEE HILL

As discussed in Section 5.6, "Stability of Lower (Right Hand) Slope" of our report number 981-9109, there is evidence of localized slope failure in the right side shoulder, through signs of the tilting of the guard rail, depressions in the asphalt shoulder, and lateral movement of the granular fill. From our cursory review of this slope (beyond the terms of reference of the RFP), it appears the road platform was constructed with granular fill over rock fill. The rock fill slope does not show obvious signs of slope failure. The Proctor & Redfern survey indicates that the right hand slope between about Station 15+090 and 15+255 is between 1.4H:1V and 1.5H:1V. To the north and south of this area, the slope is generally 1.75H:1V to 2H:1V.

It is considered that the shifting of the roadway by 3 m to the left and the lowering of the grade by about 1 m at the edge of the shoulder/top of slope will correct the shoulder granular fill movement. As discussed during the Technical Review Meeting, it would be prudent to cover the shoulder granulars with rip rap. This rip rap could extend from the shoulder rounding to the rock fill interface.

The stability of the lower portion of the slope, i.e., below the road platform, is difficult to determine with the soils information presently available. To properly evaluate this slope, it is necessary to determine slope construction and founding conditions. An investigation of this nature would be difficult to complete due to the presence of the rock fill. As such, a decision must be made to either accept the current stability of the overall slope, given the past performance or to rehabilitate the slope to current MTO standards. Rehabilitation of the slope could include flattening the slope to 2H:1V, with 2 m benches at 8 m vertical spacing. Excess material from the left hand cut and elsewhere on the project could be used for the rehabilitation. Because of the height of the slope, a large amount of land will be required at the base to accommodate the new slope. This will require clearing of the forested area, foundation preparation and possibly land acquisition. Alternatively, a reinforced earth type wall could be provided along the right side slope over much of the total length of the project under consideration. Maintaining traffic during construction of the wall would be difficult, as excavation of the shoulder and part or all of the right hand lane would be required to install the tie back strips. Golder Associates can provide additional consultation services with respect to stability of this slope, should they be required.

ROCK FILL FOR ROAD WIDENING

The use of rock fill for road widening is proposed at the following locations:

1. Stations 10 + 215 to 10 + 325, Lt., Clarkson Township;
2. Stations 10 + 075 to 10 + 125, Lt., Poitras Township;
3. Stations 10 + 060 to 10 + 080, Rt., Poitras Township; and
4. Stations 18 + 075 to 18 + 175, Rt., Poitras Township.

At locations 1 to 3 listed above, rock fill as per the O.P.S.S. 1004 series has been specified. The rock fill must be well graded and the surface "chinked" to ensure no voids are present to minimize the migration of the finer road granulars into the rock fill. The use of a geotextile separator in addition to the chinking would provide added security to prevent this migration.

At location 4, the MNR has specified that the rock fill placed along the bank of the adjacent Ottawa River has a minimum diameter of 300 mm. It will therefore be necessary to provide an intermediate layer between this large rock fill layer and the fine road granulars. This intermediate layer should comprise a well graded rock fill with a maximum diameter of 150 mm. Also, this layer should be chinked into the surface of the lower rock fill. This will facilitate the installation of the proposed geotextile separator located beneath the shoulder granulars and prevent the tearing of the geotextile over the large rock fill.

Should you have any questions regarding this brief addendum report #2, please do not hesitate to contact the undersigned at our office.

Yours truly,

GOLDER ASSOCIATES LTD.

Dan M. Cacciotti, P.Eng.
Geotechnical Engineer

P. Fintan J. Heffernan, P.Eng.
Designated MTO Contact

DMC:FJN:tl

Golder Associates Ltd.

662 Falconbridge Road
Sudbury, Ontario, Canada P3A 4S4
Telephone (705) 524-6861
Fax (705) 524-1984

Golder Associates

January 5, 1999 981-9109

RECEIVED JAN 11 1999

Proctor & Redfern Limited
222 McIntyre Street West
Suite #410
North Bay, Ontario
P1B 2Y8

ATTENTION: Mr. A.E. Rose, C.S.T.

RE: **ADDENDUM**
GEOTECHNICAL/FOUNDATION INVESTIGATION AND DESIGN
PROPOSED RETAINING WALL ALONG HIGHWAY 63
W.P. 167-90-00 - MTO DISTRICT 54, SUDBURY
YOUR PROJECT EO 97447

Dear Sirs:

This addendum letter must be read in conjunction with the Golder Associates Ltd. report No. 981-9109, dated December 1998. This addendum presents discussions that refer to "Section 5.3 Permanent Cut Slopes" in our report.

It is our understanding that the preferred option is to grade the overburden to a 2 Horizontal to 1 Vertical (2H:1V) configuration from the edge of the road realignment to the face of a bedrock outcrop or until the 2H:1V line "daylights" through the existing grade. The re-graded overburden slope will also incorporate 2 m wide berms at every 8 m of vertical height. It is understood that in the section between Stations 15+290 to 15+360, there is significant slope cutting required to achieve the "day lighting" to the existing grade. As such, Proctor & Redfern has requested consideration of steepening of the grade along the upper portion of this slope in this area.

The proposed steepening would involve maintaining the original design for the lower 24 m vertical height (i.e. with 3 benches for a horizontal distance of 54 m) and steepening the remaining upper portion of the slope to 1.5H:1V, to allow the new slope to "daylight" through the existing slope over a shorter distance.

This steepened slope of 1.5H:1V will be subject to more erosion and sloughing than the flatter 2H:1V, but is probably flatter than natural angle of repose for this material. Although, the lower 3 levels (at 2H:1V) will provide a catchment area for ravelling material, it will still be necessary to minimize the amount of mobilized material by providing surficial protection. This protection could be comprised of either a geosynthetic / geoweb material covering the slope to permit topsoiling and seeding or rock protection. The rock protection could consist of fine rock fill with maximum size of 300 mm.

Should you have any questions regarding this brief addendum report, please do not hesitate to contact the undersigned at our office.

Yours truly,

GOLDER ASSOCIATES LTD.

Dan M. Cacciotti, P.Eng.
Geotechnical Engineer

Fin J. Heffernan, P.Eng.
Consultant

DMC/FJH/tl