FOR CONTRACT NO.: 04-4S6504

INFORMATION HANDOUT

MATERIALS INFORMATION

UNDERGROUND CLASSIFICATION
Potentially Gassy with Special Conditions
CLASSIFICATION NO. C056-097-11T thru C057-097-11T

GEOTECHNICAL DESIGN REPORT AND RECOMMENDATION

ROUTE: 04-Son-101-R52.1/R52.5

STATE OF CALIFORNIA ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF INDUSTRIAL RELATIONS
DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT
2211 Park Towne Circle, Suite 2
Sacramento, California 95825

Telephone (916) 574-2540
FAX (916) 574-2542

December 16, 2010

Department of Transportation
2015 East Shields Avenue, Suite 100
Fresno, CA 93726

Attention: Geo Leyva (via e-mail: Geo_Leyva@dot.ca.gov)

Subject: Underground Classification #'s.: C056-097-11T thru C057-097-11T

Route 101 Retaining Walls - Cloverdale

Mr. Leyva:

The information provided to this office relative to the above project has been reviewed. On the basis of this analysis, Underground Classification of "Potentially Gassy with Special Conditions" has been assigned to the shaft(s) identified on your submittal. Please retain the original Classification for your records and deliver a true and correct copy of the Classification to the shaft contractor(s) for posting at the job site.

When the contractor who will be performing the work is selected, please advise them to notify this office to determine if a mandated Prejob Conference with the Division is required prior to commencing any activity associated with drilling of the shaft(s).

Should you have another bore under construction that is not required to have an Underground Classification (i.e.: less than 30 inches in diameter), please contact the Mining and Tunneling Unit prior to any employee entry of such a space.

If you have any questions on this subject, please contact this office at your earliest convenience.

Sincerely,

John R Leahy

John R. Leahy
Senior Engineer

cc: R. Brockman
File

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

Underground Classification

C056-097-11T

DEPARTMENT OF TRANSPORTATION

(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of 2015 East Shields Avenue, Suite 100, Fresno, CA 93726

(MAILING ADDRESS)

at ROUTE 101 RETAINING WALLS - CLOVERDALE

(LOCATION)

has been classified as *** POTENTIALLY GASSY with Special Conditions***

(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

SPECIAL CONDITIONS

1. A Certified Gas Tester shall perform pre-entry and continuous monitoring of the underground environment to measure Oxygen and detect explosive, flammable, and toxic gasses whenever an employee is working in the underground environment.
2. Mechanical ventilation shall provide for continuous exhaust of fumes and air at any time an employee is working in the underground environment. The primary ventilation fans must be located outside of the underground environment and shall be reversible by a single switch near the fan location.
3. The Division shall be notified immediately if any **Flammable Gas** or **Petroleum Vapor** exceeds 5% of the Lower Explosive Limit.
4. All utilities that may be in conflict with the project shall be identified and physically located (potholed) prior to the start of project operations.

The forty 30-inch diameter by 40 feet deep drilled shafts located on the side of Route 101, approximately 0.2 miles north of the intersection of Route 101 and the First Street Underpass, Cloverdale, Sonoma County.

This classification shall be conspicuously posted at the place of employment.

Date December 16, 2010

John R. Leahy

(SENIOR ENGINEER)

John R. Leahy

03 77089

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MS. OFELIA ALCANTARA
Supervising Bridge Engineer
Bridge Design West
Structures Design

Date: March 17, 2010

File: 4-SON-101-PM 52.1/52.5
04–4S6500
Cloverdale Slide

From: M. ZABOLZADEH/A. KADDOURA
Associate Materials and Research Engineers
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

H. Nikoui
HOSHMAND NIKOUI
Chief, Branch A
Office of Geotechnical Design - West
Geotechnical Services
Division of Engineering Services

Subject: Storm Damage Restoration and Protection of Cloverdale City Waterline

This memorandum presents our geotechnical recommendations for the proposed extension of the existing segmented CIDH soldier pile, proposed new tieback wall, and protection of Cloverdale City waterline from an existing landslide in the above referenced project.

I. BACKGROUND

Cloverdale slide has been moving for years and several small/large projects and contracts have been administered in order to address the slide movements. The slide is about 300 ft long along Route 101 and its head scarp is about 130 ft high above Route 101 in a private property driveway (Mr. Bray). The depth of the failure plane is estimated to be about 23 ft below ground surface at the location of the existing Slope Indicator (SI 17 installed in May 2001) as shown on the attached Exhibits A and B.

In order to protect Mr. Bray's driveway at the top of the hill, a temporary segmented CIDH soldier pile wall was constructed in 2003 near the head scarp of the slide. This wall was constructed under EA 04-27890K as phase I of the storm damage repair strategy.

In Phase II, to convert the above-mentioned temporary soldier pile wall to a permanent tieback wall (as expected for a second stage) anchors, lagging, and waler were added to the wall to achieve adequate protection of the Bray's private road.

Also, as part of Phase II, to address the entire landslide, a 20 ft wide, 66 ft high, and 350 ft long reinforced embankment buttress fill was contracted at the toe of the slide to stop the slide movement below the above-mentioned tieback wall. Both of these projects were constructed under Contract No. 04-2S1003 in 2004.

In 2008, the above-mentioned buttress showed some distress and minor movement at its toe along the shoulder for a distance of about 70 ft. The concrete lined ditch at the top of the buttress had cracked allowing the surface water to seep into the buttress fill worsening the situation. The cause of the movement was due to leakage of the existing DI, connecting RCP, and the metal culvert over the buttress fill side slope, saturating the buttress fill. As a result, reconstruction of the concrete lined ditch, removal of one DI, removal of metal culvert over the buttress fill side slope, and minor grading on the top bench within the limits of slide were necessary to temporarily address and slow down the buttress movement under emergency Maintenance Contract 04-930322.

In early 2009, the buttress fill showed some more movements and bulging at its toe within the same 70 ft long distance mentioned above. To stop the movement, a 160± ft long, 3 ft diameter segmented CIDH pile wall was constructed at the toe of the slide about 23 ft from the toe of the buttress into the side slope under emergency Contract 04-0F7704 in 2009. This has stopped the slide movement up to date. See attached Exhibit A.

There is an existing 16 inches water line at approximate Station 110+50 that supplies water to the entire City of Cloverdale. This water line crosses under Route 101 from a treatment facility located adjacent to the Russian River, runs up the cut slope, and turns north along King Ridge Heights to the existing water tanks. The tanks are located just above the southern end of the existing soil nail wall. The waterline is buried about 5 ft (this should be verified by the City of Cloverdale) below the cut slope with concrete cover around it. The scarp of an adjacent landslide is propagating towards the waterline. See attached Exhibit A.

The above-performed work in 2009 addressed the 70 ft long moving part of the buttress. This Geotechnical Report is prepared to provide foundation recommendations to stabilize the landslide, to prevent the remainder of the existing buttress fill from getting pushed out by the landslide and to protect the existing City waterline from the adjacent landslide movement.

SCOPE OF WORK

Work performed for this investigation includes field mapping, reviewing background information on file, available on site geology, and sub-surface soil/rock conditions from the available boring logs.

III. REGIONAL AND SITE GEOLOGY

The project site sits atop a ridge that trends roughly N40W and loses elevation to the southeast. Oat Valley Creek to the north and Cloverdale Creek to the south have worked to erode away portions of the surrounding terrain leaving only this ridge. The city of Cloverdale lies between this ridge and the taller mountains to the west. The cut slope averages approximately 130 feet high and is currently at a slope of 1.4:1. The roadway elevation is 350 feet, and the highest point at the top of the cut is 490 feet in elevation.

The ridge is composed primarily of Franciscan graywacke and tan sandstone blocks floating in a sheared argillite matrix. Red-brown cherts have also been observed, but are not common. Blocks range in size from <3 feet to >30 feet in diameter. Their shapes are generally spherical to oblate as they were rounded and shaped during the tectonic processes that placed them there. The blocks, as well as the surrounding argillite, are pervasively sheared, with fracture surfaces running in all directions. The morphology of the ridge is typical of Franciscan terrain, appearing humocky as more resistant blocks become exposed while the softer argillite erodes around them. This may explain the pronounced high at the southeastern tip of the ridge. Lab tests on a mixture of five random samples from the highly weathered and failed portion of this material classify it as a clayey-sand (SC). The argillite is composed of micaceous minerals that easily weather to clay with the addition of water. Cores recovered from the ridge showed thin fractures within both the argillite and graywacke filled with fat, gray clay. Most likely, this clay was the result of weathered argillite.

IV. PROJECT SITE SEISMICITY

The project site is located within a seismically active region dominated by the northwest trending San Andreas Fault (see Figure 3, Regional Fault Map). Several other faults that parallel the San Andreas make up the larger San Andreas Fault system and separate the Pacific plate on the west from the North American plate to the east. The San Andreas Fault system can be thought of as a diffuse plate boundary at which strain is spread across a wide region. There are larger, well-known faults within the system that tend to be the most active, however, there are other unnamed faults that are not mapped that may produce moderate earthquakes. An example of this comes from the Yountville earthquake in September 2000. Originally, the USGS determined that the earthquake occurred along the West Napa Fault, but later the hypocenter (location of the earthquake at depth) was determined to be on an unnamed fault. That earthquake registered 5.2 on the Richter scale and caused widespread damage to parts of Napa County.

Few large earthquakes have occurred in the vicinity of Cloverdale during historic times. Thirty seven (37) miles to the south, in the city of Santa Rosa, a magnitude 5.9 earthquake occurred in 1969 along the Rodger's Creek Fault, a probable northern extension of the Hayward Fault. Activity along the Maacama Fault - north of the Rodger's Creek Fault and closer to Cloverdale – is dominated by smaller quakes along its three segments. Quakes may also be smaller in magnitude because of the crustal conditions in the area. The geothermal fields east of Cloverdale act to warm the crust in the region, making it less brittle and less susceptible to large ruptures. In January 2000, three separate quakes occurred a short distance from Cloverdale. These quakes measured 4.2 to 4.3 on the Richter scale on unnamed branches of the Maacama Fault (Earthquake Hazards Program – Northern California, USGS, 2000).

Table 1.
Predicted Maximum Credible Earthquake and Ground Accelerations

FAULT*	Distance from Project*	Maximum Credible Earthquake**	Peak Ground Acceleration
San Andreas (North)	21.6 mi	8.0	0.30 g
Rodger's Creek/ Healdsburg	9.0 mi	7.0	0.34 g
Maacama	3.8 mi	7.25	0.19 g

* *Jennings (1994)* ** *Mualchin (1996)*

Foundation soil at the location of the proposed Extension of the Existing Segmented CIDH Soldier Pile Wall

Based on borings SH-1 and SH-2 drilled (dated 9/10/2002 under Contract # 04-2S1003) in the shoulder area within the limits of the embankment buttress, the foundation soil/rock is described as gray-green slightly weathered to fresh Graywacke gravel to the depth of 10 ft (3m) below ground surface. This overlay gray to black slightly weathered and intensely fractured Argillite to the depths of the borings at 40 ft (12 m) and 31 ft (9.5 m) below the original ground, respectively. The SPT blow count for Graywacke gravel ranges between 17 and greater than 50 blows per foot (0.3 m) of depth. The SPT blow count for Argillite rock ranges between 33 and greater than 50 blows per foot (0.3 m) of depth. Hard drilling was encountered between the depths of 19 ft (5.8 m) and 39 ft (11.9 m) below the ground surface.

Groundwater was not encountered at the time of drilling (9/10/2002).

Foundation soil at the location of the proposed tieback wall

A single boring (SI-17) was installed in May 2001, under EA 04-27890K during the preliminary investigation of the Cloverdale Landslide. Since the drilling of this boring, significant work has been done to the area including grading and construction of a geogrid reinforced embankment buttress. The top 0-10 ft will be medium stiff, moist, gravelly clay containing layers of geogrid reinforcement. Below 10 ft, the subsurface consists of moderately weathered argillite, moderately soft to moderately hard, and intensely fractured to locally sheared. Within this argillite are blocks of hard to very hard graywacke and metachert, fresh to slightly weathered, and moderately fractured. These blocks vary in size from coarse gravel to cobble to boulder. Drilling conditions for the proposed tie-back wall will vary from soft ground to very hard rock. Holes drilled for CIDH piles shall not be left open overnight. Drilling for tie-back anchors may require temporary casing.

Based on the existing piezometers installed on November 23, 2008 over the bench within the slide limits, the depth of the groundwater fluctuates between 4 ft and 11 ft.

Foundation soil at the location of the existing City water line.

Because of the lack of access, borings were not drilled for the proposed CIDH piles at this location, however, many borings have been recovered in the vicinity, and foundation materials to be encountered are similar throughout the job site. The top 0-10 ft will be deeply weathered argillite, brown to gray to black, soft, and intensely fractured, often containing coarse gravel or cobbles of harder graywacke. Below 10 ft, the rock becomes less weathered argillite, moderately soft to moderately hard, and intensely fractured. Within this argillite are blocks of hard to very hard graywacke and metachert, fresh to slightly weathered, and moderately fractured. These blocks vary in size from coarse gravel to cobble to boulder. Drilling conditions for the CIDH pile wall will vary from soft ground to very hard rock. Temporary casing may be necessary in the upper 10 ft. The ground encountered near the toe of the slope is more deeply weathered and softer than that found upslope; therefore as the drilling operation moves upslope, the rock condition changes to moderately hard to hard greywacke at the surface.

VI. RECOMMENDATIONS

Extension of the Existing Segmented CIDH Soldier Pile Wall

As mentioned above, in 2009, a 165 ft long, 3 ft diameter segmented CIDH soldier pile wall without lagging was constructed near the toe of the embankment buttress between approximate Stations 105+35 and 107+00 to address the buttress movement. The laggings were eliminated because this wall was designed to be buried inside the embankment fill and the embankment buttress was constructed with geosynthetic reinforcement. The reinforcements are assumed to intensify the arching effect of the embankment soil and act as laggings. Only the top 2 ft of the wall that is exposed has laggings to catch any future localized failures/debris, etc.

The existing 3 ft diameter CIDH soldier pile wall was designed to act as a 15 ft (max.) cantilever wall without wood lagging. As shown on the attached Exhibit A, this wall addresses about half of the entire slide. In order to stop the rest of the slide from any future movement, we recommend extending the existing segmented CIDH soldier pile wall for a distance of 200 ft towards north along the same alignment. The approximate wall extension limits will be between Stations 107+00 and 109+00. We recommend the design of the wall extension be the same as the existing wall. Contact David Romero of Office of Structures Design for as-built plans and complete design details.

Proposed tieback wall

Because the landslide is relatively large, to assist the existing segmented CIDH pile wall and its extension to withstand the slide mass, we recommend constructing a 260 ft long high tieback soldier pile wall with wood lagging at the existing bench level with design height of 10 ft and one row of anchors. The tieback wall should be constructed about 12 ft from the hinge point between Stations 105+55 and 108+15 as shown on Exhibits A and B.

We recommend the following requirements for the proposed tieback wall:

- The proposed one row of tiebacks should be installed at 8 ft below the bench level. The anchors should be installed at a minimum angle of 15 degrees below the horizontal plane.

- No Laggings should be installed below the waler to minimize the amount of excavation.

- The unbounded length of the tieback anchors should be a minimum of 65 ft long as shown on Exhibit B.

- Based on our slope stability analysis using SlopeW computer program, the total force (P_{TOTAL}) is estimated to be 12,500 pounds /foot of wall length for a safety factor of 1.5.

- The design of the anchor type and any anchor length in excess of the minimum length specified herein should be left up to the contractor. The contractor is responsible for providing tieback anchors that satisfy the contract specifications.

Earth Pressures

The wall should be designed for the following:

For *active pressure* against the wall, use the following:

- An equivalent fluid pressure of 57 pcf ($\phi = 25°$, $\gamma_{moist} = 140$ pcf).

- Earth pressure distribution shall be in accordance with "Memo to Designers 5-12" dated August 1990.

- The tieback wall shall be capable of resisting an additional seismic uniform earth pressure estimated to be equal to 50H psf.

For *passive pressure* against the soldier piles, use the following input:

1. $\phi = 36°$, $c = 200$ psf, and $\gamma = 130$ pcf.
2. Friction Factor (δ) = 2/3 ϕ.

Vertical CIDH Pile Capacities and Penetration Depth

Soldier piles should be embedded a minimum of 40 feet below the bench level.

Pile spacing should be limited to not more than 8 ft.

The ultimate vertical compression and tension capacities of piles may be calculated using the following design parameters:

- Use a unit pile shaft friction of 2.5 kips per unit surface area of the pile length below the dredge line of the wall.
- Use 60 percent of the compression shaft resistance values mentioned above to calculate the ultimate tension (uplift) resistance of the pile.
- Use an ultimate pile tip compression bearing pressure of 120 kips per unit tip area of the CIDH piles.

The above recommendations are based on parameters established by our field exploration, engineering judgment, and submitted wall cross-sections.

Existing City Water Line

To protect the existing water line mentioned above from propagation of the failure scarp of the adjacent landslide, we recommend constructing 15 ft deep segmented spiral caged reinforced CIDH piles adjacent and along the waterline for a distance of 120 ft. The distance between the face of the existing concrete of the waterline and the face of the concrete of the proposed piles should be 2 ft as shown on the attached Exhibit B. The pile spacing varies between 4.5 ft on center to 8 ft on center as shown on the attached Exhibit A.

Because these piles are intended to block the propagation of the slide scarp, the piles are not acting as a wall, thus no special design is warranted.

CONSTRUCTION CONSIDERATIONS AND REQUIREMENTS

The following construction considerations and requirements should be included in the design and construction specifications for the proposed walls and mitigation measures. The contractor may encounter difficulties during drilling for the proposed CIDH piles. This is due to the hard drilling condition experienced during drilling.

As mentioned above groundwater was encountered in the existing piezometers located over the existing bench. Thus, use of casing and placing concrete in wet condition may be required. For displacement of groundwater, the contractor may choose to use a closed system using a concrete pump or a tremie tube to place concrete at the bottom of the holes for CIDH soldier piles. A positive head should be maintained at all times to reduce potential for concrete segregation.

Piles

Installation of CIDH piles should be performed in accordance with Section 49-4 of the Standard Specifications.

The drilling and concrete placement for CIDH pile construction shall be staggered. No open holes shall be adjacent.

Use of casing may be required during drilling in unstable fill to keep the drilled hole walls from collapsing and reduce the amount of dewatering required (if encountered). The casing should be removed with the help of continuous vibration to reduce the potential for the concrete to "hang up" on casing.

VII. CORROSION

The Department considers the site to be corrosive to foundation elements if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

The following table provides our corrosion test summary:

Boring	***SIC Number***	***Sample Depth***	***Resistivity (Ohm-Cm)***	***pH***	***Chloride Content (ppm)***	***Sulfate Content (ppm)***
P-1	537826	15'-25'	1997	7.2	N/A	N/A
P-2	537827	15'-20'	2303	7.3	N/A	N/A
P-2	538129	5'-10'	2370	7.9	N/A	N/A

Note: *Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.*

* * * * * *

If you have any questions or need additional information, please call us at (510) 286-4676/4831 or Hooshmand Nikoui, Branch Chief at (510) 286-4811.

Attachment

c: TPokrywka, HNikoui, CCashin, MZabolzadeh, AKaddoura, Project File, Daily File

Zabolzadeh/mm/SON-101-PM 52.2 Cloverdale 2010

REGISTERED PROFESSIONAL ENGINEER
MOHAMMAD ZABOLZADEH
No. C 052953
Exp. 12/31/2010
CIVIL
STATE OF CALIFORNIA

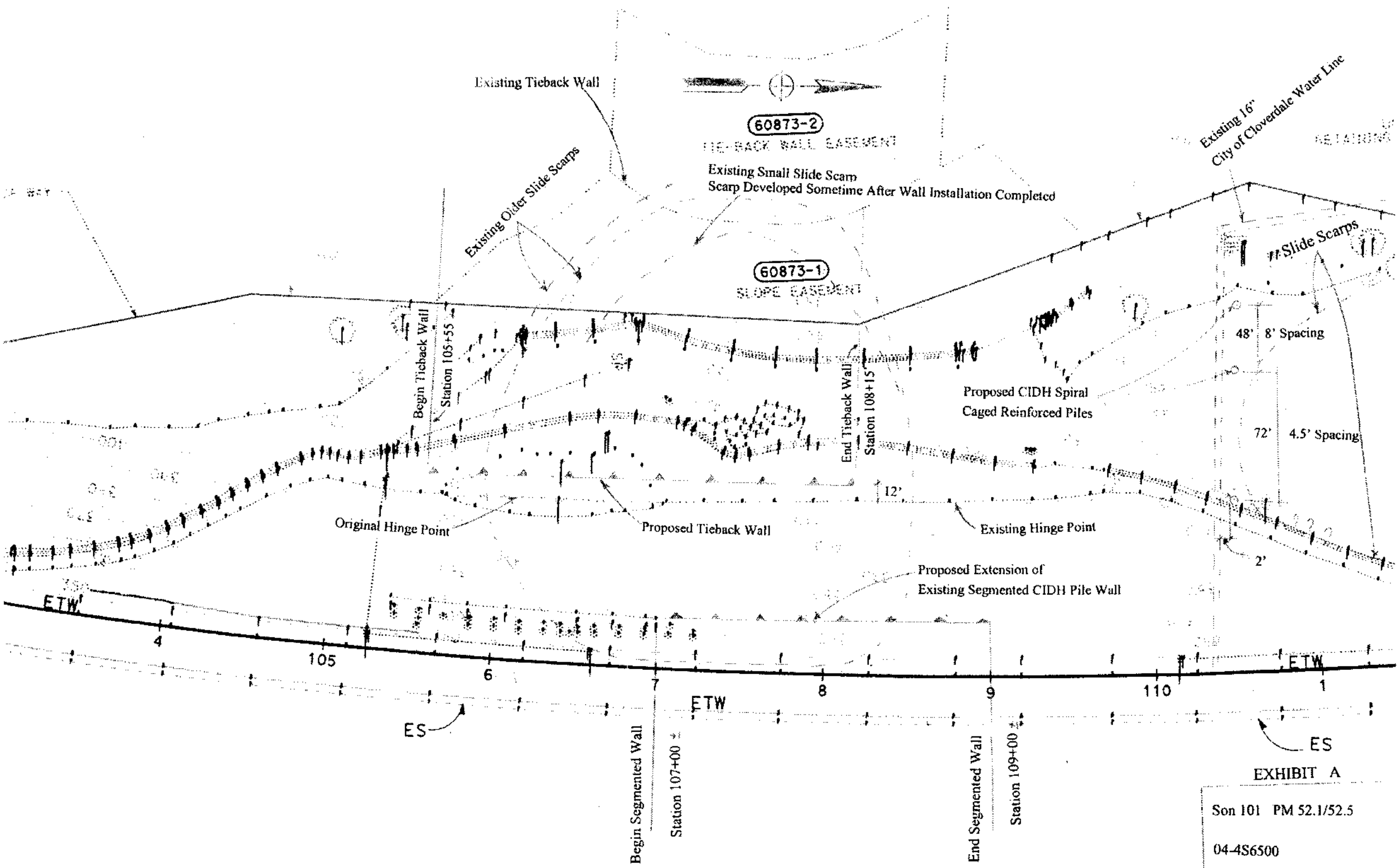


EXHIBIT A

Son 101 PM 52.1/52.5

04-4S6500

3/18/2010

Not to Scale

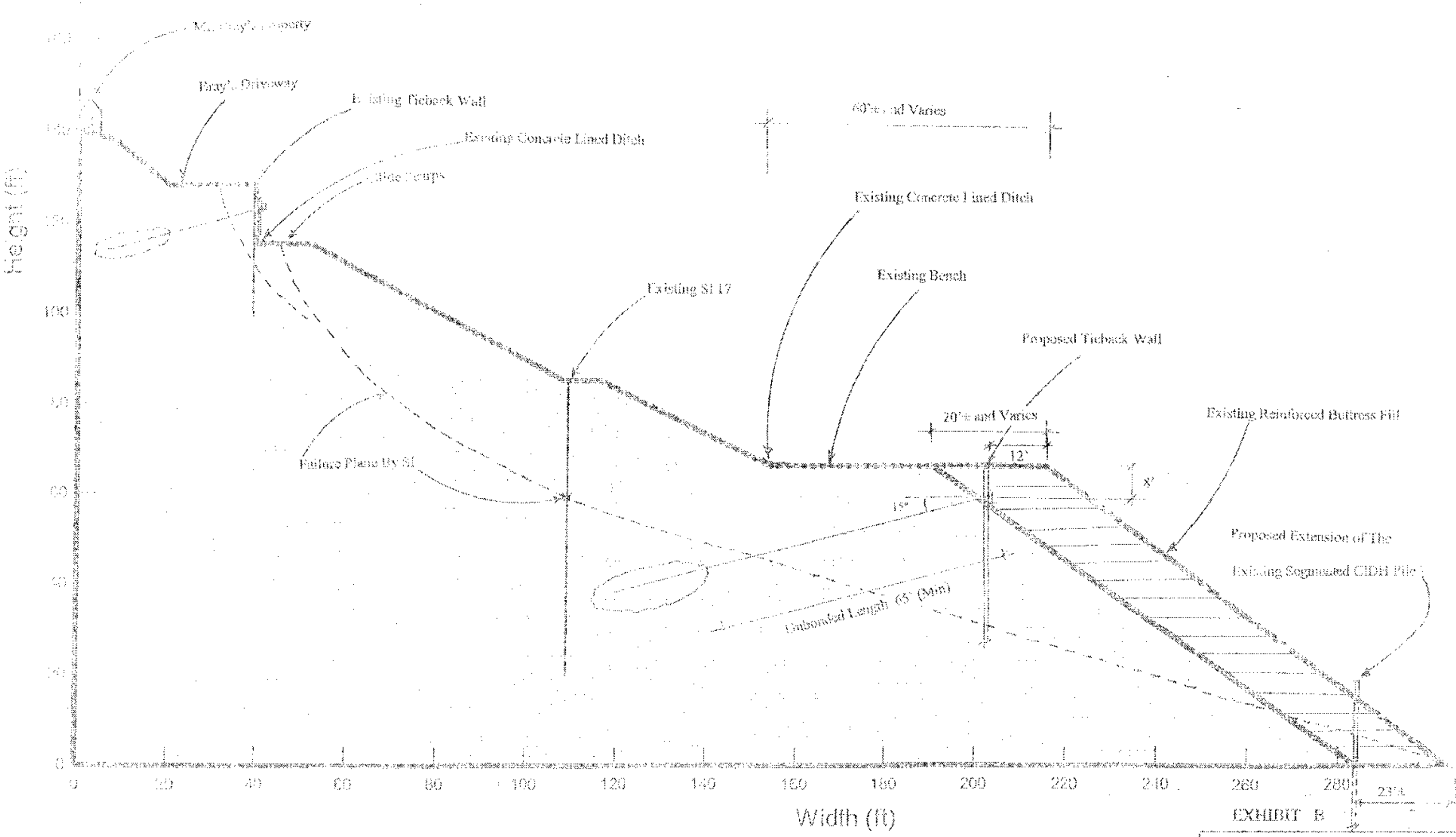


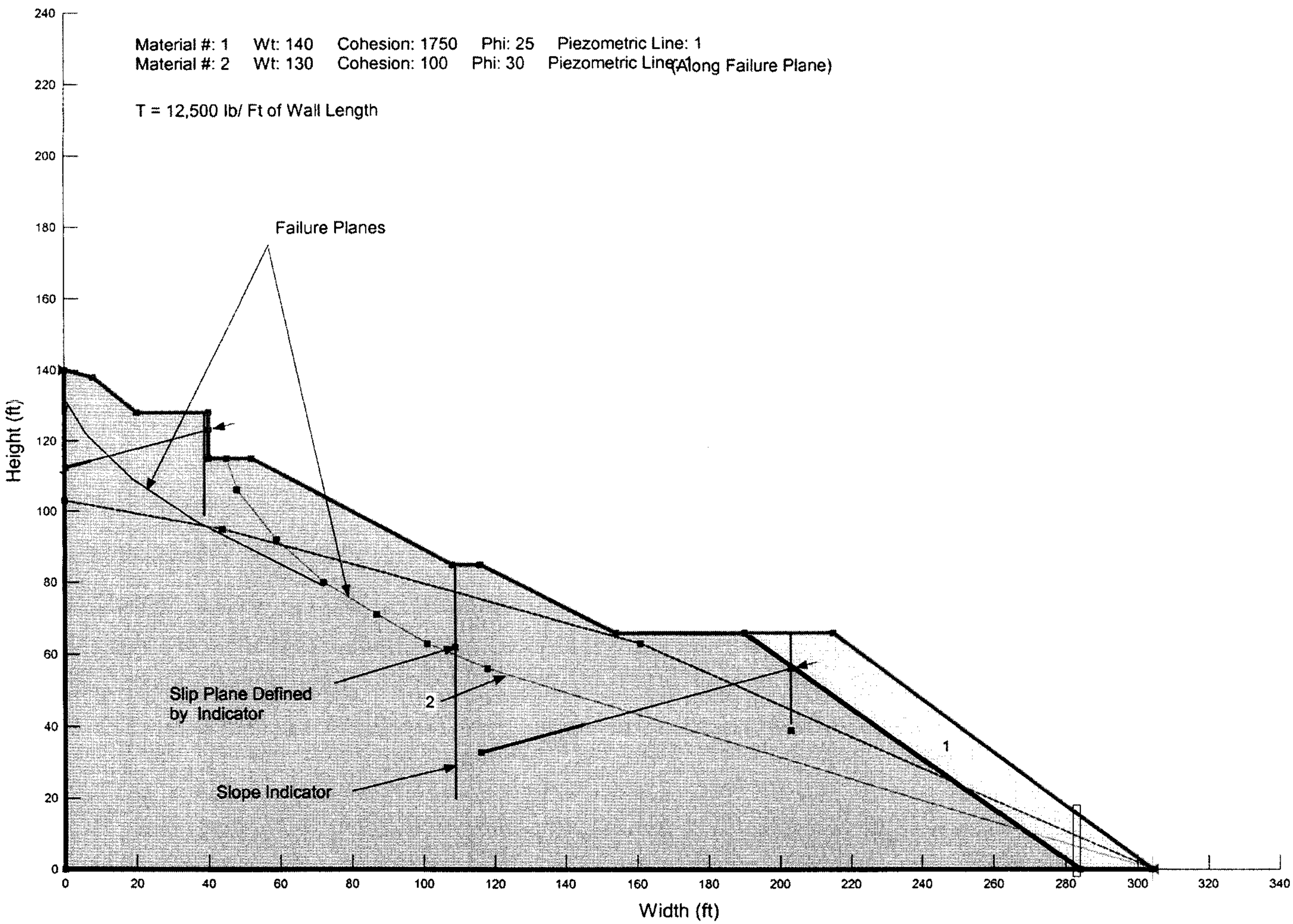
EXHIBIT B

Not to Scale

Sec 101 PM 52.1/52.5

04-486500

3/18/2010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Material #: 2 Wt: 130 Cohesion: 100 Phi: 30 Piezometric Line: 1 (Along Failure Plane)
T = 12,500 lb/ Ft of Wall Length
Failure Planes
Slip Plane Defined by Indicator
Slope Indicator
1
2
Height (ft)
Width (ft)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340