HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

October 21, 2014
Project Number 3184-01-14

Muir Beach Community Services District
Attention: Leighton Hills, District Manager
16 Miller Avenue, Suite 203
Mill Valley, California 94941

RE: Report
Geotechnical Investigation
Sunset Way and Cove Lane Improvements
Muir Beach, California

Dear Mr. Hills:

This presents the results of our geotechnical investigation in connection with proposed water main, street and drainage improvements at Sunset Way and Cove Lane in Muir Beach, California. The scope of our investigation was to review our previous work in the vicinity, review selected geologic references, observe exposed site conditions, drill twelve test borings, perform engineering analyses, and develop geotechnical recommendations for the design and construction of the project. Our scope of work was outlined in our professional services agreement dated revised October 3, 2014.

PROJECT DESCRIPTION

As shown on the plan by ILS Associates, Inc. dated September 3, 2014, the project will consist of replacing the existing water main, upgrading pavements along Sunset Way, and installing retaining walls to support cuts and fills for a turnaround at the southern terminus of Sunset Way. In addition to the improvements depicted on the plans, we understand that the project will include an approximately 15 foot long retaining wall to accommodate widening Sunset Lane near the intersection with Highway 1, a retaining wall to accommodate widening of the southwestern side of Cove Lane at the intersection with Sunset Lane, and a new retaining wall to support fills for a turnaround at the southeastern terminus of Cove Lane.

WORK PERFORMED

We reviewed the following information as part of our work:

- Blake, M.C. Jr. et al, 1974, *Preliminary Geologic Map of Marin and San Francisco Counties and Parts of Alameda, Contra Costa, and Sonoma Counties, California,* U.S. Geologic Survey, BDC 64.

- Blake, M.C., Graymer. R.W. and Jones, D.L., 2000, *Geologic Map and Map Database of Parts of Marin, San Francisco, Alameda, Contra Costa, and Sonoma Counties, California.*

- Davenport, C.W., 1984, *An Analysis of Slope Failures in Eastern Marin County, California, Resulting From the January 3 & 4, 1982 Storm,* California Department of Conservation, Division of Mines and Geology DMG Open-File Report 84-22.

- Graymer, R.W., Moring, B.C., Saucedo, G.J., Wentworth, C.M., Brabb, E.E., and Knudsen K.L., 2006, *Geologic Map of the San Francisco Bay Area,* U.S. Geological Survey, Scientific Investigations Map 2918.

We explored the subsurface conditions within the project areas on October 2 and 7, 2014 to the extent of twelve test borings ranging between approximately 1 and 15 feet deep. The locations of the test borings are shown on the attached *Site Plan*, Plate 1.

Our personnel observed the drilling, logged the subsurface conditions encountered, and collected soil samples for visual examination and laboratory testing. Samples were retrieved using Sprague and Henwood and Standard Penetration Test samplers driven with a 70-pound hammer. Penetration resistance blow counts were obtained by dropping the hammer through a 30-inch free fall. The number of blows was recorded for each 6 inches of sampler penetration. These blow counts were then correlated to equivalent standard penetration resistance blow counts. The blows per foot recorded on the boring logs represent the accumulated number of correlated standard penetration blows that were required to drive the sampler the last 12 inches or fraction thereof.

Logs of the test borings are presented on Plates 2 through 13. The soils encountered are described in accordance with the criteria presented on Plate 14. Bedrock is described in accordance with the *Engineering Geology Rock Terms* presented on Plate 15. The logs depict our interpretation of subsurface conditions on the date and at the depths indicated. The stratification lines on the logs represent the approximate boundaries between soil types; the actual transitions may be gradational.

Selected samples were laboratory tested to determine their moisture content, dry density, plasticity and resistance (R-) value. Laboratory test results are posted on the boring logs in the manner described on the *Key to Test Data*, Plate 14. The results of the Atterberg Limits plasticity testing are presented on Plate 16. The results of R-value testing are presented on Plate 17.

FINDINGS

Site Conditions

Sunset Way traverses a hillside which generally extends down towards the south at inclinations ranging between approximately 3:1 and 1-1/2:1 (horizontal:vertical). The roadway was created by excavating into the hillside on the upslope side and by placing fill beneath the downslope portion. Portions of the cuts are retained by concrete and wood walls ranging to about 10 feet in height. Unretained cut banks along the upslope side of the roadway generally range to about 8 feet high, and are inclined between approximately 1:1 and near-vertical. The cuts generally expose varying thicknesses of colluvium (slopewash) and landslide debris overlying moderately to highly weathered sandstone and shale bedrock. Fill banks along the downslope edges of the roadway generally range to about10-feet high, and are inclined between approximately 1:1 and 1-1/2:1. This is steeper than allowed by current engineering practice. Portions of the fill banks have experienced yielding, resulting in settlement and cracking of the outboard roadway pavement. Portions of the outboard roadway fills are retained by yielding walls and bulkheads constructed in connection with roadway widening and the creation of turnouts and parking areas.

Cove Lane is accessed on the northwest end from Sunset Way, and extends down a prominent south-trending swale towards the Pacific Ocean. The western side of the upper portion of the roadway is situated at the top of a drainage ravine. The ravine banks range to about 8 feet high and are inclined at between about 1:1 and 1-1/2:1. The southern terminus of Cove Lane is a graded pad which slopes gently down to a steep bank which steps down to the beach. During our investigation we noted extensive seepage emerging from this bank.

Subsurface Conditions

The site is within the Coast Range Geomorphic Province which includes San Francisco Bay and the northwest-trending mountains that parallel the coast of California. These features were formed by tectonic forces resulting in extensive folding and faulting of the area. Previous geologic mapping by Blake (2000) indicates the area to be underlain by bedrock of the Franciscan Melange. The Melange unit is Jurassic to Cretaceous in age, and typically consists of a heterogeneous mixture of sandstone, sheared shale, metavolcanic rock, serpentinite and chert. The mapping indicates the prominent south-trending swale in the vicinity of Cove Lane to be blanketed with Quaternary aged alluvial deposits which washed down from upslope areas.

Our test borings encountered fill, colluvium (slopewash), landslide deposits, residual soil and bedrock. The fill encountered generally consists of loose to medium dense silty and clayey sand and gravel and of soft gravelly clay. The colluvium encountered consists of loose to medium dense silty sand, soft gravelly silt, medium stiff to stiff sandy and gravelly clay, and of medium dense clayey gravel. Portions of these materials may be slide deposits. The residual soils encountered consist of medium dense clayey sand derived from the in-place weathering of the

underlying parent bedrock. The fill and upper portions of the native soils encountered are relatively weak and compressible, and are subject to downslope creep on hillsides. Portions of the soils at the site are slightly expansive. Expansive soils undergo changes in volume with changes in moisture content, and can cause heaving cracking of pavements. Bedrock encountered in the borings consist of firm to moderately hard sandstone and shale.

The approximate test boring locations are shown on the *Site Plan* (Plate 1). The test borings encountered the following profiles:

TABLE 1

Boring	Depth (feet) Fill	Colluvium/Residual Soil	Bedrock	Depth to Supporting Material (feet)
B-1	---	0-1.5+	1.5-3.0+	3.0
B-2	0-3.2	---	3.2-3.5+	3.2
B-3	0-2.0	2.0-7.2	7.2-8.0+	7.2
B-4	0-3.0	3.0-4.5	4.5-6.5+	4.5
B-5	0-4.5	4.5-10.5+	---	8.5
B-6	0-4.5	4.5-15.0+	---	12.0
B-7	0-3.5	3.5-9.0	9.0-10.0+	9.0
B-8	0-2.5	2.5-5.0	5.0-5.5+	5.0
B-9	0-5.0	5.0-7.0	7.0-7.5+	7.0
B-10[1]	---	---	---	---
B-11	0-5.5	5.5-7.0	7.0-7.5+	7.0
B-12	0-1.5	1.5-3.5	3.5-4.0+	3.5

Descriptions of the subsurface conditions encountered are presented on the boring logs.

Groundwater

Free groundwater was encountered in Boring 6 at a depth of approximately 9-1/2 feet, but did not develop in the remaining borings at the time of our investigation. Groundwater levels at the site are expected to fluctuate over time due to variations in rainfall and other factors. Rainwater percolates through the relatively porous surface soils. On hillsides, the water typically migrates downslope in the form of seepage within the porous soils, at the interface of the soil/bedrock contact, and within the upper portions of the weathered and fractured bedrock.

[1] Drilling refusal in asphalt pavement.

Landsliding

The Blake (1971, 2000) mapping does not indicate the presence of previous landsliding along the project alignment. Most of Sunset Way lies within Category 4 as defined in *"Relative Slope Stability and Land Use Planning"* (USGS, 1979). Category 4 includes slopes greater than 15 percent that are underlain by bedrock units susceptible to landsliding, but that are not underlain by landslide deposits. The swale containing Cove Lane lies within Category 3, which includes generally stable to marginally stable areas that are not underlain by landslide deposits or bedrock units susceptible to landsliding. The categories range from 1 to 5, with Category 5 indicating least stable.

We observed that Sunset Way extends across several areas displaying topographic features indicative of old landslide deposits. In addition, we encountered possible slide deposits within several of the borings. We understand that slide mitigation measures are not to be addressed as part of the current project, and that the risk of potential roadway yielding and/or instability in areas beyond those being retained will be acceptable to the District.

CONCLUSIONS

Roadway Pavements

Our investigation and laboratory testing indicate that subgrade soils within the project area are relatively weak and compressible. In order to reduce pavement distress, existing weak subgrade soils beneath pavements should be overexcavated as necessary to expose firmer soils, and the expansive soils segregated. Improved performance may be achieved by increasing the depth of overexcavation, but this work would necessitate deep excavations, possible disruption of utilities, and possibly the need for shoring. The depth and extent of required overexcavation should be evaluated in the field by the geotechnical engineer prior to placement of backfill. In order to reduce expansive soil heave, fill material located within 18 inches of pavement subgrade should consist of non-expansive material. Expansive soils exposed by overexcavation should be moisture conditioned to a high moisture content prior to recompacting to cause a portion of the expansion to occur prior to backfilling. Expansive soils may be used greater than 18 inches below pavement subgrade provided the materials are moisture conditioned to a high moisture content prior to compaction.

If it will not be economically feasible to perform overexcavation at the site, it will be necessary to at least scarify and recompact the existing subgrade surface to create a uniform subgrade, to install a geotextile stabilization fabric on the recompacted subgrade, and to then install the design aggregate baserock and asphaltic concrete section. It will still be necessary to overexcavate soft and yielding areas, and to place stabilization fabric and compacted baserock in overexcavated areas. It must be noted and accepted by the District that if the upper subgrade soils are not

overexcavated and replaced as properly compacted fill, the pavements will have a shorter design life, and will require increased maintenance and repairs.

Seasonal high moisture contents of near surface soils may cause soft "pumping" conditions during construction which may require additional overexcavation, geotextile reinforcement, and imported granular fill. To reduce the risks of such costly special construction methods, it would be prudent to perform site grading during the late summer and fall months, to perform the excavation from unexcavated areas using an excavator, and to restrict the contractor from operating trucks or equipment directly on the subgrade soils.

Retaining Walls

It will be necessary to support retaining walls for new cuts and fills on drilled piers extending into bedrock or approved competent material. The new walls should be provided with adequate backdrainage to prevent hydrostatic buildup. Our investigation indicates the depth to competent supporting material in the vicinity of the Cove Lane turnaround may require the use of tiebacks or drilled pier deadman anchors in order to derive additional lateral support.

Although not a part of our current work, the existing roadway cuts are steeper than permitted by current engineering standards, and typically expose weak colluvial soils and highly weathered bedrock which have experienced varying degrees of sloughing and sliding. In areas where the risk of future sloughing or sliding will not be acceptable, it will be necessary to retain the cuts with engineered retaining walls.

Outboard Road Stabilization

Our investigation indicates that the outboard edges of the roadway are generally underlain by varying thicknesses of relatively poorly compacted and overly steep fills which are underlain by weak native soils. These materials have experienced downslope creep, and are subject to continued yielding and possible instability. In areas where this risk will not be acceptable, it will be necessary to support the outboard edges of the roadways with engineered retaining walls supported on drilled, cast-in-place, reinforced concrete piers extending into bedrock or approved competent soils, and designed to resist lateral creep loads imposed by the overlying materials. It will be necessary to closely space the piers in order to retain upslope materials. Due to the relatively large depth to supporting materials, it may be necessary to generate additional lateral resistance for the walls utilizing drilled tieback anchors extending into bedrock. Although the walls will enhance stability of upslope areas, steep fills and non-engineered walls downslope of the engineered walls will continue to be subject to creep and possible instability. If future movement of downslope soils exposes retaining wall piers, lagging should be immediately provided between the exposed piers to prevent soil loss.

Surface Drainage

It is important that surface and subsurface water be controlled to reduce future moisture variations in the weak and expansive subgrade soils. Positive drainage should be provided away from walls and slopes. Surface drains should be connected to non-perforated conduits which discharge into a storm drain or at an approved erosion resistant outlet well away from walls or slopes.

RECOMMENDATIONS

Site Preparation and Grading

Temporary slopes should be laid back or shored in conformance with OSHA standards. All temporary slopes and shoring should be contractually established as solely the responsibility of the Contractor.

Pavements, abandoned pipes and other objects encountered should be removed. The resultant voids should be cleaned and backfilled as outlined below. Existing soils beneath planned pavements should be overexcavated with an excavator operating from unexcavated perimeter areas to a depth of at least 18 inches below proposed pavement subgrade, or to a depth of 18 inches below existing grade, whichever is deeper. Additional overexcavation may be required depending on conditions observed by our representative in the field during construction. The depth and extent of required overexcavation should be reviewed in the field by Herzog Geotechnical. Expansive on-site soils should be segregated during overexcavation and not used within 18 inches of pavement subgrade.

Heavy trucks or construction equipment can cause "pumping" of the weak subgrade soils which can cause substantial damage to the site and significantly increase the amount of overexcavation required. The contractor should not operate trucks or equipment on these soils. In areas where yielding is encountered, additional overexcavation, geotextile reinforcement and aggregate base material may be required. The Contractor should provide unit prices for overexcavation, placement and compaction of baserock, and installation of additional layers of geotextile reinforcing.

Soils exposed by required excavations should be scarified to a depth of at least 8 inches, moisture conditioned to near optimum moisture content (or at least 3 percent above optimum moisture content where expansive soils are encountered), and recompacted with light equipment to between 90 and 93 percent relative compaction. Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 test procedure. Optimum moisture content is the water content of the soil (percentage by dry weight) corresponding to the maximum dry density.

All fill material should be free of organic matter. The fill material should not contain rocks or lumps larger than 4 inches in greatest dimension, and no more than 15 percent should be larger than 2 inches. Fill material within 18 inches of pavement subgrade should consist of clean well-graded soil with little or no potential for expansion. The non-expansive material should have a plasticity index of 15 percent or less, and a maximum liquid limit of 40 percent. Herzog Geotechnical should approve all imported fill prior to it being brought to the site.

Fill should be placed in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned to within 3 percent of optimum moisture content, and compacted to at least 90 percent relative compaction to establish subgrade level for the pavement section. The upper 6 inches of the subgrade should be compacted to a minimum of 95 percent relative compaction, and should be smooth and unyielding.

If overexcavation and replacement will not be economically feasible for the District, and the District is willing to accept degraded performance and increased maintenance, subgrade soils should be scarified to a depth of at least 8 inches, moisture conditioned to near optimum moisture content, and recompacted to at least 95 percent relative compaction. The compacted subgrade surface should be smooth and unyielding. Where yielding areas are noted, overexcavation should be performed with an excavator as recommended by our representative in the field during construction. An approved geotextile reinforcement Mirafi 600X, or equivalent) should be placed over the subgrade (and over any localized overexcavated areas), and Caltrans Class 2 Aggregate Baserock placed and compacted as outlined above.

Asphalt Pavement

Based on our discussions with ILS Associates, Inc., we understand that a design Traffic Index (TI) of 5.5 is applicable to the project. Based on this value and an a measured subgrade R-value of 16, we calculate a design pavement section of 3 inches of asphalt concrete over 10 inches of Class 2 Aggregate Base placed over the prepared subgrade. Increasing asphalt concrete thickness would increase the life and durability of pavements. Where pavements will be subjected to increased traffic, self-loading garbage trucks or other concentrated loads, the pavement section should be increased to accommodate these loads or reinforced concrete slabs should be used as specified by the Project Civil Engineer. Drainage swales in the pavement should be constructed with reinforced concrete.

The upper 6 inches of subgrade beneath the design pavement section should be moisture conditioned and compacted to at least 95 percent relative compaction, and should be smooth and unyielding. Aggregate baserock should be compacted to at least 95 percent relative compaction to provide a smooth unyielding surface. Characteristics and placement of asphalt concrete and aggregate base, and subgrade preparation should conform to the *California Department of Transportation Standard Specifications*, latest edition, except that the test method for compaction should be determined by ASTM D1557.

Utility Trenches

Utility trenches should be backfilled and compacted prior to subgrade compaction. Trenches should be backfilled with material that is mechanically compacted to at least 90 percent relative compaction (or at least 95 percent relative compaction within the upper 12 inches of pavement subgrade). Lift thicknesses should not exceed 8 inches in uncompacted thickness. Compaction by jetting should not be permitted. Sand backfill is subject to piping, and should not be used. Utility trenches should be backfilled before pavement subgrade preparation. Governmental or public utility requirements may exceed those listed above and should govern where applicable.

Retaining Walls

Lateral Pressures

Retaining walls should be supported in rock or approved supporting material on foundations designed in accordance with the recommendations presented in this report. Free-standing retaining walls should be designed to resist active lateral earth pressures equivalent to those exerted by a fluid weighing 45 pounds per cubic foot (pcf) where the backslope is level, and 60 pcf for backfill at a 2:1 slope. Retaining walls restrained with deadman or tieback anchors should be designed to resist an "at-rest" equivalent fluid pressure of 60 pcf for level backfill and 75 pcf for backfill at a 2:1 slope. For intermediate slopes, interpolate between these values. Where wall backfill will be subject to vehicular loading, a traffic surcharge equivalent to 2 feet of additional backfill should also be added to walls. A minimum factor of safety against instability of 1.5 should be used to evaluate static stability of retaining walls.

Drilled Piers

Drilled piers should be at least 18 inches in diameter and should extend at least 6 feet into bedrock or at least 10 feet into approved competent soils. The depth to bedrock and competent soils may be estimated from Table 1 in the *Subsurface Conditions* section of this report. Design pier depths and diameters should be calculated by the Project Structural Engineer using the criteria presented below. The materials encountered in the pier excavations should be evaluated by our representative in the field during drilling.

The portion of the piers extending into bedrock can impose a passive equivalent fluid pressure of 450 pounds per cubic foot (pcf) acting over 2 pier diameters, and vertical dead plus real live loads of 1000 pounds per square foot (psf) in skin friction. The portion of the piers extending into approved competent soils can impose a passive equivalent fluid pressure of 300 pcf acting over 2 pier diameters, and vertical dead plus real live loads of 500 psf in skin friction. These values may be increased by 1/3 for seismic and wind loads, but should be decreased by 1/3 for determining uplift resistance. The portion of piers designed to impose passive pressures should

have at least 7 feet of horizontal confinement from the face of the nearest slope or wall. End bearing should be neglected due to the uncertainty of mobilizing end bearing and skin friction simultaneously.

Groundwater may be encountered, in which case it will be necessary to dewater the holes and/or to place concrete by the tremie method. If caving soils are encountered, it may be necessary to case the holes. Hard drilling or coring will be required to achieve required bedrock penetrations.

Tiebacks

Tiebacks may be installed to generate lateral resistance. Tiebacks should be located so as to not interfere with upslope foundations or utilities. It will be necessary to obtain appropriate easements if tiebacks will extend onto neighboring properties. Tiebacks should be inclined downward at an angle of at least 15 degrees from the horizontal. Tiebacks should have minimum unbonded lengths of 10 and 15 feet for bars and strands, respectively. Tiebacks should have minimum bonded lengths of 12 feet in bedrock. The allowable skin friction for tiebacks will depend upon drilling method, grout installation pressure, and workmanship. For estimating purposes, the portion of tiebacks grouted into bedrock may be assumed to impose a skin friction value of 2000 pounds per square foot (psf). The contractor should be responsible for determining the actual length of tiebacks necessary to resist design loads based on their familiarity with the installation method utilized. Our field engineer should be present to observe conditions during drilling.

Tieback materials, installation, corrosion protection and testing should conform to *Recommendations for Prestressed Rock and Soil Anchors* (Post-Tensioning Institute, latest edition). The tieback bars or strands should be double corrosion protected. The bars or strands should be positioned in the center of the holes, and the bonded length grouted in place from the bottom. If a frictionless sleeve is used over the unbonded length, the bars or strands may be initially grouted over their entire length. When the grout has attained the required compressive strength, the anchors should be proof tested to 1.33 times the design load as outlined by the Post-Tensioning Institute. Proof test loads should be held for 10 minutes, and the deflection at test load between the 1 and 10 minute readings should not exceed 0.04 inches. After testing, the tension in the anchor should be reduced to the design load and locked off. Replacement tiebacks should be installed for tiebacks that fail the load testing.

Deadman Anchors

Additional resistance for retaining walls may be derived from buried grade beams or double corrosion protected rods which are restrained by drilled piers designed in accordance with the recommendations presented previously. Deadman piers should be located at least 15 horizontal feet away from the upslope face of walls and beyond a 2:1 place projected up from the base of

the walls, whichever is greater. In order to avoid future disruption of the deadman system, the piers and rods/grade beams should be shown on appropriate as-built and title documents.

Wall Backdrains

The retaining walls should be fully backdrained. The backdrains should consist of 4-inch diameter, rigid perforated pipe surrounded by a drainage blanket. The top of the drain pipe should be at least 8 inches below lowest adjacent downslope grade. The pipe should be PVC Schedule 40 or ABS with an SDR of 35 or better, and the pipe should be sloped to drain at least 1 percent by gravity to an approved outlet. Accessible subdrain cleanouts should be provided, and should be maintained on a routine basis. The drainage blanket should consist of clean, free-draining crushed rock or gravel wrapped in a filter fabric such as Mirafi 140N. Alternatively, the drainage blanket could consist of Caltrans Class 2 "Permeable Material", in which case the filter fabric may be omitted. A prefabricated drainage structure such as Mirafi Miradrain may also be used provided that the backdrain pipe is embedded in permeable material or fabric-wrapped crushed rock. The drainage blanket should be continuous, at least 1 horizontal foot thick, and should extend to within 1 foot of the surface. The uppermost 1 foot should be backfilled with compacted soil to exclude surface water.

Wall Backfill

Wall backfill should conform to the requirements for non-expansive engineered fill outlined previously. Wall backfill should be placed in level lifts not exceeding 8 inches in loose thickness. Each lift should be brought to within 3 percent of optimum moisture content and compacted to at least 90 percent relative compaction. Backfilling should be performed only with hand operated equipment to avoid over-stressing the walls. Wall backfill slopes should be constructed at an inclination no steeper than 2:1. Fill slopes should be overbuilt, and trimmed back as necessary to expose a well-compacted surface. Routine maintenance of slopes should be anticipated. Fill slopes and areas disturbed during construction should be planted with vegetation to resist erosion. Erosion that occurs must be repaired promptly before it can enlarge.

Outboard Stabilization Walls

Lateral Pressures

Outboard roadway wall systems should be designed to resist lateral earth pressures equivalent to those exerted by a fluid weighing 60 pounds per cubic foot (pcf). Earth pressures should be assumed to act on the length of the wall plus 3 feet at each end, and from the top of the walls down to the top of bedrock or approved supporting material. For planning purposes, the depth to supporting bedrock and supporting material may be estimated based on Table 1 of this report. The actual depths to supporting bedrock or supporting material should be evaluated by Herzog Geotechnical during pier drilling. A traffic surcharge equivalent to 2 feet of additional backfill

should also be included in the wall design. Facing for the walls should extend at least 18 inches below the finished ground surface on the downslope side of the wall.

Drilled Piers

Drilled, cast-in-place, reinforced concrete piers should be spaced no more than 3 pier diameters (measured center-to-center) and should have a clear spacing of no more than 3 feet in order to reduce soil movement between the piers. Piers should be designed based on the criteria presented in the *Retaining Walls* section of this report.

Tiebacks

Tiebacks should be designed as outlined in the *Retaining Walls* section of this report.

Wall Backdrains

Wall facing and grade beams greater than 30 inches in height should be fully backdrained. The backdrain should consist of a 4-inch diameter, rigid perforated pipe surrounded by a drainage blanket. The pipe should be located at the base of the wall, should be PVC Schedule 40 or ABS with an SDR of 35 or better, and should be sloped to drain at least 1 percent by gravity to an approved outlet. Accessible subdrain cleanouts should be provided, and should be maintained on a routine basis. The drainage blanket should consist of clean, free-draining crushed rock or gravel wrapped in a filter fabric such as Mirafi 140N. Alternatively, the drainage blanket could consist of Caltrans Class 2 "Permeable Material", in which case the filter fabric may be omitted. A prefabricated drainage structure such as Mirafi Miradrain may also be used provided that the backdrain pipe is embedded in at least 1 cubic foot of permeable material per lineal foot of pipe. Drainage blankets should be continuous, at least 1 horizontal foot in width, and should extend from the base of the wall facing to within 1 foot of the surface. The uppermost 1 foot should be backfilled with compacted soil to exclude surface water. Surface drainage must be maintained entirely separate from retaining wall backdrain.

Site Drainage

Positive drainage should be provided away from walls and slopes. Runoff from areas upslope of the road should be intercepted at the top of retaining walls using lined ditches, and runoff from the road should be intercepted with curbs or swales which discharge at an approved outlet. Conduit pipes should be PVC or ABS, and should be Schedule 40 or SDR 35, or equivalent. Surface drains should be maintained entirely separate from subsurface drains. Drains and conduits should be periodically checked for blockage, and cleaned and maintained as necessary to provide adequate drainage.

Maintenance

Surface drains and wall backdrains should be periodically checked for blockage and cleared as necessary. As future movement of downslope soils exposes piers, additional lagging should be provided between the exposed soldier piers to prevent loss of soil between the piers. Lagging may consist of either treated timber or pre-stressed concrete panels. Voids behind the lagging should be filled with Caltrans Class 2 Permeable Material, and vertical spacers should be provided between the lagging to allow seepage through the face of the walls.

SUPPLEMENTAL SERVICES

Our conclusions and recommendations are contingent upon Herzog Geotechnical being retained to review the project plans and specifications to evaluate if they are consistent with our recommendations, and being retained to provide intermittent observation and testing during subgrade overexcavation, expansive soil segregation,, scarifying and recompaction, backfill placement and compaction, utility trench backfilling and compaction, subgrade preparation and compaction, baserock placement and compaction, pier drilling, tieback drilling and load testing, retaining wall backdrain installation, and wall backfilling to evaluate if subsurface conditions are as anticipated and to check for conformance with our recommendations. If concrete is not placed immediately following pier drilling, we should be contacted to re-inspect pier holes immediately prior to concrete placement. Alignment, steel, concrete, asphalt pavement, shoring, temporary slopes, surface drainage and/or waterproofing should be inspected by the appropriate party, and are not part of our scope of work.

If during construction subsurface conditions different from those described in this report are observed, or appear to be present beneath excavations, we should be advised at once so that these conditions may be reviewed and our recommendations reconsidered. The recommendations made in this report are contingent upon our being notified to review changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, the recommendations of this report may no longer be valid or appropriate. In such case, we recommend that we review this report to determine the applicability of the conclusions and recommendations considering the time elapsed or changed conditions. The recommendations made in this report are contingent upon such a review.

We should be notified at least 48 hours before the beginning of each phase of work requiring our observation, and upon resumption after interruptions. These services are performed on an as-requested basis and are in addition to this geotechnical reconnaissance. We cannot provide comment on conditions, situations or stages of construction that we are not notified to observe.

LIMITATIONS

This report has been prepared for the exclusive use of Muir Beach Community Services District And their consultants for the project described in this report. Our services consist of professional opinions and conclusions developed in accordance with generally-accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided us regarding the proposed construction, the results of our field exploration and laboratory testing programs, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The test boring logs represent subsurface conditions at the locations and on the dates indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration and may not necessarily be the same or comparable at other times. The locations of the test borings were established in the field by reference to existing features, and should be considered approximate only.

Our investigation did not include an environmental assessment or an investigation of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, ground water or air, on or below, or around the site, nor did it include an evaluation or investigation of the presence or absence of wetlands.

We appreciate the opportunity to be of service to you. If you have any questions, please call us at (415) 388-8355.

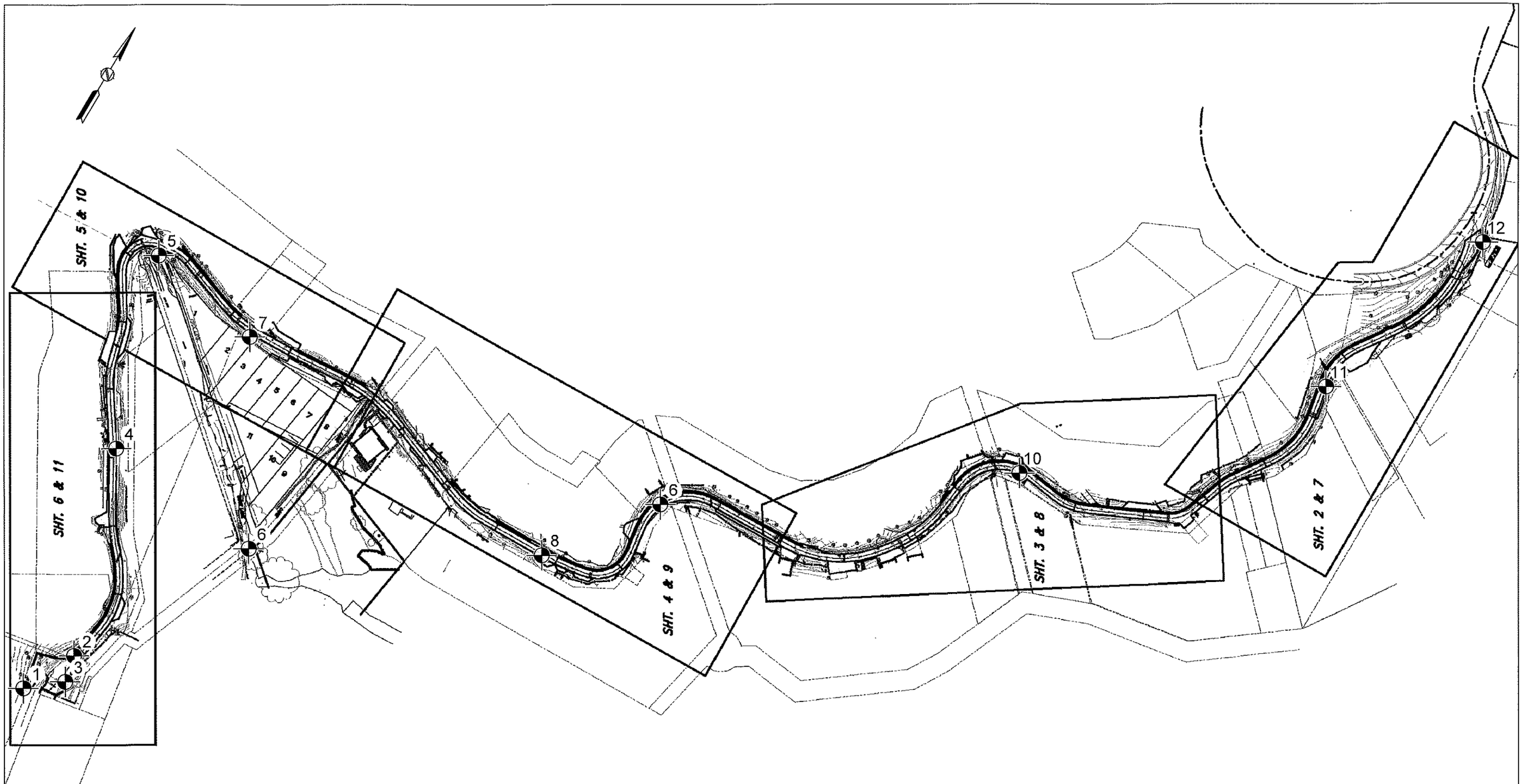
Sincerely,
HERZOG GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
CRAIG W. HERZOG
No. 002383
GEOTECHNICAL
STATE OF CALIFORNIA

Craig Herzog, G.E.
Principal Engineer

Attachments: Plates 1 - 17

cc. ILS Associates, Inc.
Attention: Mr. Michael Evans, P.E.
79 Galli Drive, Suite A
Novato, California 94949-5717



Reference: Title Sheet by ILS Associates, Inc., dated 10/10/14.

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

Job. No: 3184-01-04
Appr:
Drwn: LPDD
Date: OCT 2014

SITE PLAN
Sunset Way/Cove Lane
Improvements
Muir Beach, California

PLATE
1

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	GRAY-BROWN SILTY SAND (SM), loose, dry, with roots
							1	
					51		2	MOTTLED ORANGE-GRAY SANDSTONE, moderately hard, weak, highly weathered
							3	BOTTOM OF BORING 1 @ 3.0 FEET No Free Water Encountered

EQUIPMENT: **4" Flight Auger**
LOGGED BY: **C.H.**
ELEVATION: **
START DATE: **10-2-14**
FINISH DATE: **10-2-14**

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr: CH
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 1

Sunset Way/Cove Lane

Muir Beach, California

PLATE **2**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	LIGHT GRAY-BROWN SILTY SAND (SM), loose, dry, with roots (Fill)
							1	
					8			
							2	DARK GRAY-YELLOW-BROWN SANDY SILT WITH GRAVEL (ML), soft, moist, with abundant wood debris (Fill)
					33/6"		3	ORANGE-BROWN SANDSTONE, moderately hard, weak, highly weathered

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-2-14
FINISH DATE: 10-2-14

BOTTOM OF BORING 2 @ 3.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr: [illegible]
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 2

Sunset Way/Cove Lane

Muir Beach, California

PLATE 3

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-2-14
FINISH DATE: 10-2-14

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	LIGHT BROWN SANDY GRAVEL WITH SILT (GM/GP), medium dense, dry (Fill)
							1	
							2	DARK GRAY-BROWN SILTY SAND (SM), medium dense to dense, dry, with decomposed sandstone fragments
		10.4	100		29		3	
							4	
							5	
							6	ORANGE-BROWN CLAYEY SAND (SC), medium dense, moist (Residual Soil)
					44		7	MOTTLED ORANGE-GRAY-BROWN SANDSTONE, firm, friable to weak, highly weathered
							8	BOTTOM OF BORING 3 @ 8.0 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr: CH
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 3

Sunset Way/Cove Lane

Muir Beach, California

PLATE 4

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-2-14
FINISH DATE: 10-2-14

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	3" ASPHALT CONCRETE
								YELLOW-BROWN SILTY SAND WITH GRAVEL (SM), loose, moist (Fill)
							1	
		10.4	109		9		2	
							3	DARK BROWN SILTY SAND (SM), loose to medium dense, dry
							4	
								GRAY-BROWN SANDSTONE WITH INTERBEDDED SHALE, firm, friable to weak, highly weathered
					24		5	
							6	

BOTTOM OF BORING 4 @ 6.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr:
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 4

Sunset Way/Cove Lane

Muir Beach, California

PLATE 5

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-2-14
FINISH DATE: 10-2-14

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	GRAY-BROWN SILTY GRAVEL WITH SAND (GM), loose, dry, with wood and glass debris (Fill)
							1	
							2	
							3	
		19.3	96		15		4	
							5	DARK GRAY GRAVELLY SILT WITH SAND (ML/MH), soft, dry to moist, with decomposed sandstone fragments
							6	
	1.5				14		7	
	2.0				29		8	
							9	BROWN SANDY CLAY (CL), stiff, moist
					27		10	

BOTTOM OF BORING 5 @ 10.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr: [illegible]
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 5

Sunset Way/Cove Lane

Muir Beach, California

PLATE 6

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-7-14
FINISH DATE: 10-7-14

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	DARK BROWN GRAVELLY SILT WITH SAND (GM), soft, dry, with roots (Fill)
							1	
							2	
		12.9	107		17			becomes medium stiff at 2-1/2'
							3	
							4	
								DARK BROWN CLAYEY SAND (SC), loose, dry
		13.6	105		23		5	
								YELLOW-DARK BROWN CLAYEY GRAVEL (GC), dense,moist, with decomposed sandstone fragments
							6	
							7	
								DARK BROWN GRAVELLY CLAY (CL), medium stiff, wet
							8	
							9	
								free water encountered at 9-1/2'
		14.1	117		17		10	
								becomes softer at 10-1/2'
					12		11	
							12	becomes stiffer at 12'
					18			
							13	
					21		14	
							15	BOTTOM OF BORING 6 @ 15.0 FEET

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr:
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 6

Sunset Way/Cove Lane

Muir Beach, California

PLATE 7

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-7-14
FINISH DATE: 10-7-14

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	GRAY-BROWN SILTY GRAVEL WITH SAND (GM), loose, dry (Fill)
							1	
							2	
							3	
LL=43, PI=22, see Plate 16		13.9	104		21		4	BLACK SANDY CLAY (CL), stiff, dry to moist, with decomposed sandstone fragments
							5	
					15		6	
							7	GRAY GRAVELLY CLAY (CL), soft to medium stiff, moist
					13		8	
					33		9	GRAY-BROWN SHEARED SHALE, firm, friable, highly weathered
							10	BOTTOM OF BORING 7 @ 10.0 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

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Job No: 3184-01-14
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LOG OF BORING 7

Sunset Way/Cove Lane

Muir Beach, California

PLATE 8

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	9" ASPHALT CONCRETE
							1	BROWN SILTY SAND (SM), loose, dry (Fill)
					19		2	
							3	GRAY GRAVELLY CLAY (CL), soft, moist
							4	
					33/6"		5	YELLOW-BROWN SANDSTONE, moderately hard, moderately strong, highly weathered

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-7-14
FINISH DATE: 10-7-14

BOTTOM OF BORING 7 @ 5.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr:
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 8

Sunset Way/Cove Lane

Muir Beach, California

PLATE 9

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	12" ASPHALT CONCRETE
							1	BROWN SILTY GRAVEL WITH SAND (GM), loose to medium dense, dry, with cobbles (Fill)
					21/6"		2	
							3	
							4	
							5	OLIVE-BROWN CLAYEY GRAVEL WITH SAND (GC), medium dense, moist
					36		6	
							7	BROWN SANDSTONE, firm, friable, highly weathered

BOTTOM OF BORING 9 @ 7.5 FEET
No Free Water Encountered

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-7-14
FINISH DATE: 10-7-14

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

Job No: 3184-01-14
Appr:
Drwn: LPDD
Date: OCT 2014

LOG OF BORING 9

Sunset Way/Cove Lane

Muir Beach, California

PLATE 10

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	ASPHALT CONCRETE
							1	drilling refusal at 1'

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-7-14
FINISH DATE: 10-7-14

BOTTOM OF BORING 10 @ 1 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG
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Job No: 3184-01-14
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Drwn: LPDD
Date: OCT 2014

LOG OF BORING 10

Sunset Way/Cove Lane

Muir Beach, California

PLATE 11

EQUIPMENT: 4" Flight Auger
LOGGED BY: C.H.
ELEVATION: **
START DATE: 10-7-14
FINISH DATE: 10-7-14

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	7" ASPHALT CONCRETE
							1	YELLOW-GRAY-BROWN CLAYEY GRAVEL WITH SAND (GC), loose, moist (Fill)
							2	BLACK SANDY CLAY (CL), soft to medium stiff, moist (Fill)
		18.3	98		10		3	
							4	
							5	
							6	DARK BROWN SILTY SAND (SM), loose to medium dense, moist
		19.0	112		17		7	MOTTLED YELLOW-GRAY GRAVELLY CLAY (CL), medium stiff, moist, sheared texture

BOTTOM OF BORING 11 @ 7.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.
** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14
Appr: CK
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Date: OCT 2014

LOG OF BORING 11

Sunset Way/Cove Lane

Muir Beach, California

PLATE 12

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	
							0	BROWN SILTY GRAVEL WITH SAND (GM), loose, dry, with roots (Fill)
							1	
								LIGHT BROWN SILTY SAND WITH GRAVEL (SM), medium dense, dry
							2	
		15.1	106		24		3	
								YELLOW-BROWN SANDSTONE,, firm, friable, highly weathered
							4	BOTTOM OF BORING 12 @ 4.0 FEET No Free Water Encountered

EQUIPMENT: 4" Flight Auger

LOGGED BY: C.H.

ELEVATION: **

START DATE: 10-7-14

FINISH DATE: 10-7-14

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3184-01-14

Appr: [illegible]

Drwn: LPDD

Date: OCT 2014

LOG OF BORING 12

Sunset Way/Cove Lane

Muir Beach, California

PLATE 13

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

			Shear Strength, psf (Confining Pressure, psf)	
Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Unconsolidated Undrained Direct Shear
PI	Plasticity Index	TV	1320	Torvane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample (2.5-inch ID)	FS	Free Swell	
◪	2-inch-ID Sample	EI	Expansion Index	
◩	Standard Penetration Test	Perm	Permeability	
⊠	Bulk Sample	SE	Sand Equivalent	

KEY TO TEST DATA

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

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SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

Sunset Way/Cove Lane

Muir Beach, California

PLATE 14

ROCK SYMBOLS

SHALE OR CLAYSTONE	CHERT	SERPENTINITE
SILTSTONE	PYROCLASTIC	METAMORPHIC ROCKS
SANDSTONE	VOLCANIC	DIATOMITE
CONGLOMERATE	PLUTONIC	SHEARED ROCKS

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINNLY BEDDED	2-1/2 to 8 inches
VERY THINNLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2-1/2 to 8 inches
VERY CLOSELY SPACED	3/4 to 2-1/2 inches
EXTREMELY CLOSELY SPACED	Less than 3/4 inch

HARDNESS

SOFT - Pliable; can be dug by hand

FIRM - Can be gouged deeply or carved with a pocket knife

MODERATELY HARD - Can be readily scrached by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visable

VERY HARD - Cannot be scratched with pocket knife; leaves a metallic streak

STRENGTH

PLASTIC - Capable of being molded by hand

FRIABLE - Crumbles by rubbing with fingers

WEAK - An unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - Specimen will withstand a few heavy hammer blows before breaking

STRONG - Specimem will withstand a few heavy ringing hammer blows and usually yields large fragments

VERY STRONG - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

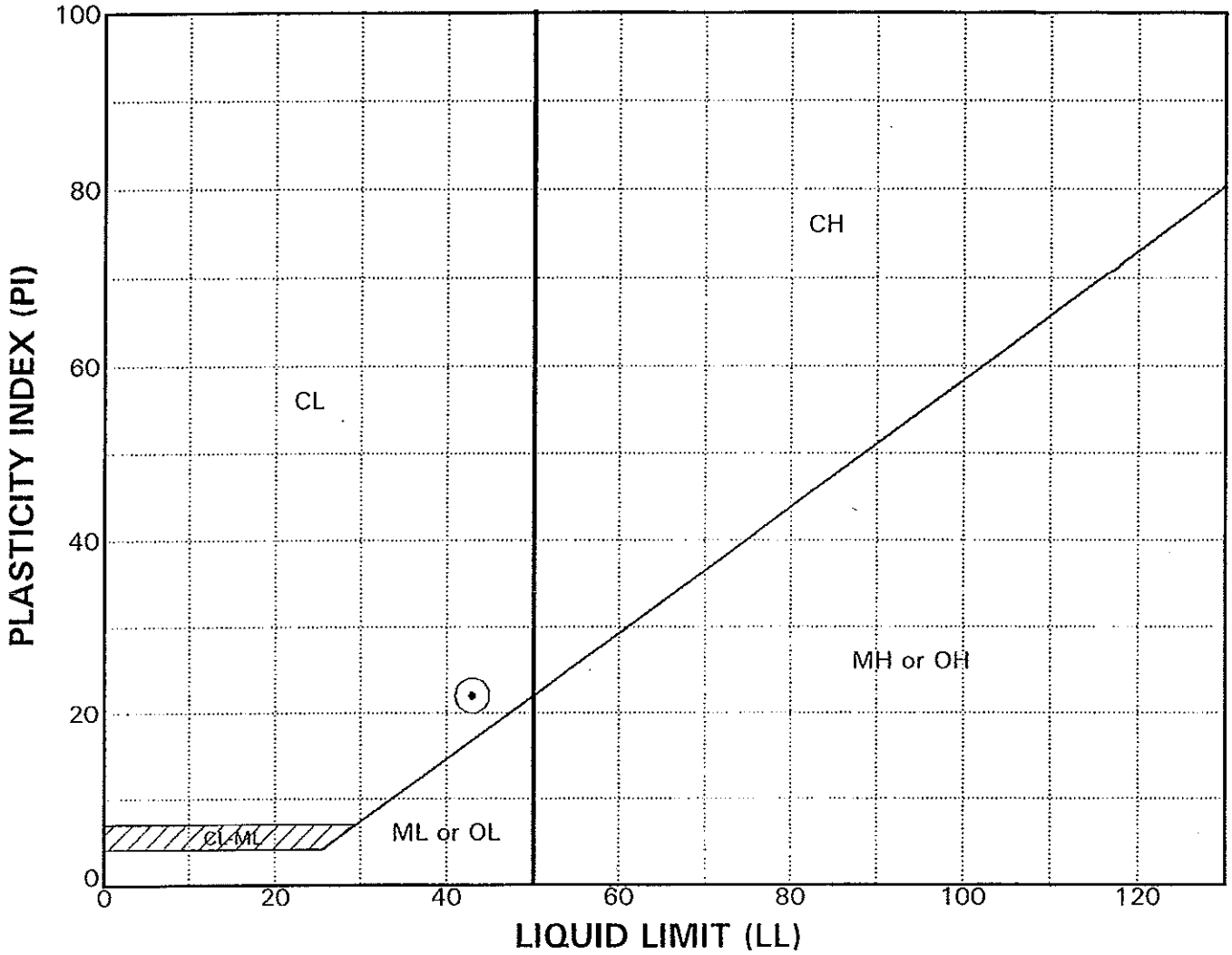
DEGREE OF WEATHERING

HIGHLY WEATHERED - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thourough discoloration, rock disintegration, mineral decomposition

MODERATELY WEATHERED - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

SLIGHTLY WEATHERED - A few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

FRESH - Unaffected by weathering agents, no appreciable change with depth



SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
⊙ Bor. 7 @ 3.5'	Black Sandy Clay (CL)	43	21	22	

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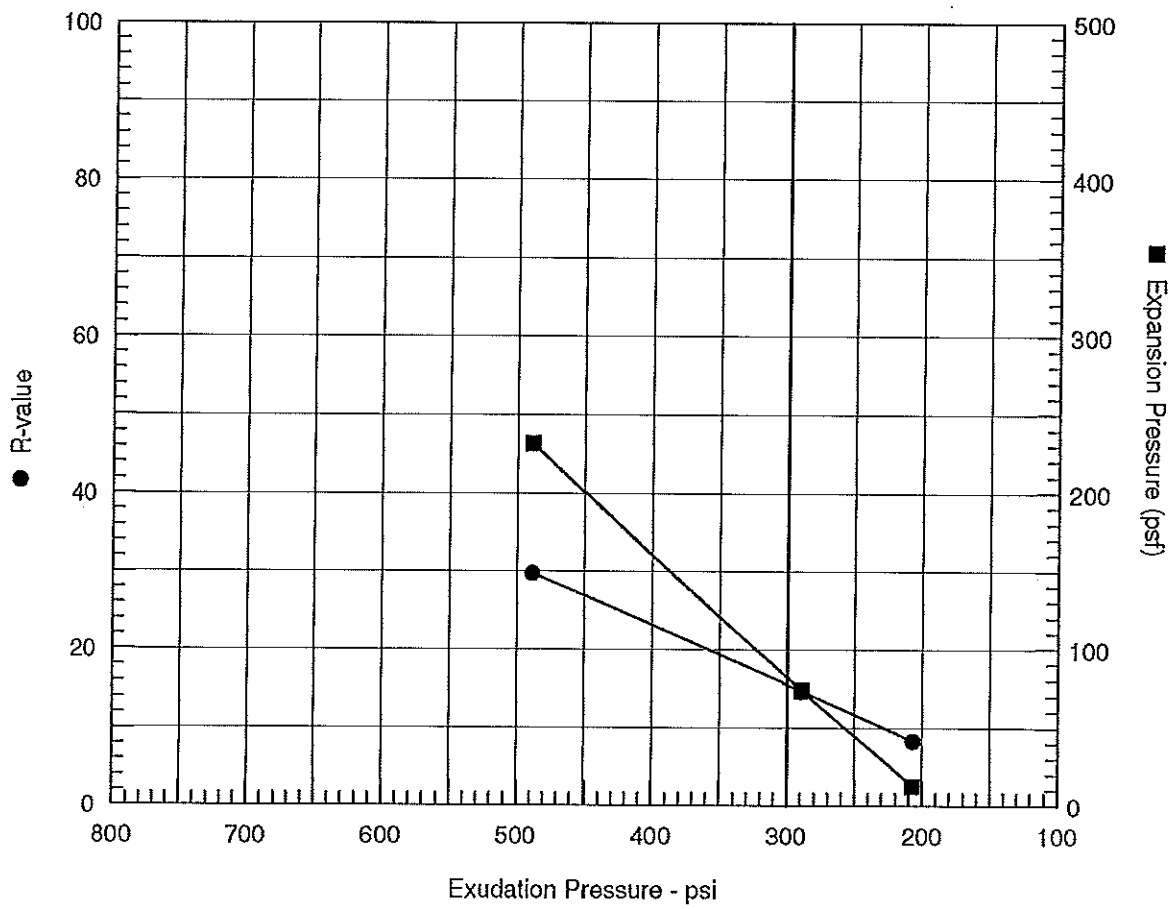
PLASTICITY CHART

Sunset Way/Cove Lane
Muir Beach, California

PLATE
16

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	170	113.7	16.6	74	122	2.52	289	15	15
2	75	108.8	18.5	13	133	2.53	207	8	8
3	350	120.1	14.4	231	94	2.45	489	30	30

Test Results	Material Description
R-value at 300 psi exudation pressure = 16 **Exp. pressure at 300 psi exudation pressure** = 82 psf	Dk Brn Clay W/ Sand (CL)

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R-VALUE TEST DATA
Sunset Way/Cove Lane
Muir Beach, California

PLATE
17