SIERRA GEOTECHNICAL SERVICES, INC.

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September 29, 2020 Project No. 3.31164.1

Lawrence and Associates
3590 Iron Court
Shasta Lake, Ca 96019

Attention: Mr. David Brown

Subject: **UPDATE GEOTECHNICAL INVESTIGATION**
Mammoth Disposal Waste Transfer Station
Mammoth Lakes, Mono County, California

Reference: **GEOTECHNICAL INVESTIGATION**
Proposed Mammoth Disposal Waste Transfer Station
Mammoth Lakes, California
SGSI Project No 3.31164; Dated December 14, 2012

In accordance with your request, we herein submit the results of our updated geotechnical investigation for the subject project. The purpose of this report was to update the foundation and earthwork recommendations and to update the site seismicity to conform to the current California Building Code (CBC). Our work consisted of a review of the above referenced report, engineering and geologic analyses, and the preparation of this report.

Future construction on the subject site is feasible from a geotechnical standpoint. The primary geologic and geotechnical constraint to development of the subject property is the potential seismic hazard associated with strong ground shaking.

The conclusions and recommendations presented herein are considered site specific and should not be extrapolated to other areas or used for other projects.

SGS
SIERRA GEOTECHNICAL SERVICES, INC.

We appreciate the opportunity to be of service to you. Should you have any questions regarding this report, please do not hesitate to contact us.

Respectfully,

SIERRA GEOTECHNICAL SERVICES, INC.

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UPDATE GEOTECHNICAL INVESTIGATION

FOR THE

MAMMOTH DISPOSAL WASTE TRANSFER STATION

MAMMOTH LAKES, CALIFORNIA

SEPTEMBER 29, 2020
PROJECT NO. 3.31164.1

Prepared By:

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1. PURPOSE AND SCOPE

This report presents the results of an updated geotechnical investigation for the proposed new transfer station, office building, weight scale, paved parking, and appurtenances to be constructed on the Mammoth Disposal sites at 59 and 85 Commerce Drive, in Mammoth Lakes, Mono County, California (Figures 1 and 2). The purpose of this study was to provide updated geotechnical and seismicity recommendations in conformance with the 2019 California Building Code (CBC).

The scope of this investigation included a review of a new draft Preliminary Grading Plan, prepared by Lawrence and Associates, dated August 25, 2020, our 2012 report, and preparation of this update report presenting the results of our findings, conclusions, and geotechnical recommendations for the proposed development.

2. SITE DESCRIPTION

The project site area is located south of Commerce Drive, approximately 0.2 miles southwest of the intersection of State Route 203 and Meridian Boulevard in the Town of Mammoth Lakes (Figures 1 and 2). The site area, 59 and 85 Commerce Drive (APN'S 037-200-049 and 050) is approximately 1.87-acres. Previously, the parcel to the south, 169 Commerce Drive, had been included as part of our 2012 study. That parcel however is no longer part of the project. In general, the site is in a similar condition to that observed for our 2012 study.

Figure 3 includes the locations of the proposed structures. The site area is slightly southwest to northeast sloping across both parcels. The elevation differential in the building pad area is approximately 5-feet. Coordinates are 37.6415, -118.9492.

3. PROPOSED DEVELOPMENT

The proposed construction is similar in nature to that proposed in our 2012 study, therefore additional subsurface investigation is not required. New construction will consist of a steel-framed warehouse and a steel or wood-framed office structure with loading docks, weight scale, paved parking, and other appurtenances. Grading is expected to be relatively minor with buildings set at or near existing grades.

As previously noted, detailed plans for construction are currently not available. SGSI should review foundation plans prior to construction to assure that they will conform with our recommendations

4. PREVIOUS FIELD WORK

Included as part of our 2012 report was a field investigation performed on November 20, 2012, that included the excavation and detailed logging of three exploratory test pits in the proposed construction areas. Logs of those test pits are presented in Appendix A. Approximate locations of the test pits are presented on Figure 3. Details of the laboratory testing performed as part of our 2012 are presented in Appendix B.

5. GEOLOGIC AND GEOTECHNICAL SITE CONSTRAINTS

Geotechnical constraints to development include the potential for moderate ground shaking along the nearby Hilton Creek fault (M_W~6.7) located approximately 1.26 mi east of the subject site.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The project site is located within the Sierra Nevada province, a generally north to northwesterly trending, asymmetric, and tilted fault-block, bordered on the east by the Sierra Nevada frontal-fault system. Predominant basement rock types of the Sierra Nevada include Cretaceous granitics with associated Paleozoic roof pendants along the west margin of Mono Basin, and to a lesser degree, Paleozoic meta-sedimentary formations mantled by Pleistocene glacial tills.

More specifically, the project site is located at the southwestern edge of the Long Valley caldera near the eastern flank of the Sierra Nevada. The caldera (collapsed volcano) is an east-west elongate, oval depression formed approximately 760,000 years ago with continued volcanic activity to the present (Bailey, 1989). The pre-volcanic basement rock in the Mammoth Lakes area is predominantly Mesozoic granitic rocks of the Sierra Nevada batholith. The batholith is a series of intrusions that displaced overlying ancient sedimentary sea floor rocks (roof pendants) during the Jurassic and Cretaceous Periods. Piedmont glaciation and more recent episodic volcanism occurred throughout the Pleistocene leaving a mantle of glacial till and pyroclastic deposits covering the basement rocks throughout the area now occupied by the Town of Mammoth Lakes.

Based upon our 2012 study, soils underlying the site will include: granular glacial deposits consisting of dark brown to grayish-brown and reddish-brown, moist, dense, silty, very fine to coarse SAND (Unified Soil Classification Symbols: SM) with few to abundant subangular rock fragments, cobbles and boulders to 36-inches diameter.

The thickness of the glacial deposits was not determined but based on research as well as seismic shear from other proximal sites, extends greater than 100-feet below the ground surface.

6.1 Groundwater

Based upon a review of the "Mammoth Basin Water Resources Environmental Study" prepared by the California Department of Water Resources (CDWR, 1973), and water well records from the Mammoth Community Water District for wells in the site vicinity, depth to permanent groundwater beneath the site is estimated at greater 250-feet.

Groundwater was not encountered during our 2012 study and is not anticipated to be encountered during site development. It should be noted that minor amounts of seepage from localized snowmelt percolation may be encountered if the site is graded during the peak snow-melt period between April and June. Since the prediction of the location of such conditions is difficult to determine, they are typically mitigated if or when they occur.

Subsurface strata which would retard the flow of water downward were not observed during the investigation. Therefore, drywells proposed for the site should function as designed.

7. FAULTING

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Geological Survey, an "active fault" is a fault that has had surface displacement within Holocene time (about the last 11,000 years); hence constituting a potential hazard to structures that might be located across it. This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972, which is detailed in the California Geological Survey Special

Publication SP-42 (Hart and Bryant, 1999). The intent of this act is to assure that unwise urban development does not occur across the traces of active faults.

Based on our review, the site is **not** located within any "Earthquake Fault Zones" or Alquist-Priolo Hazard Zones as identified in this document. Recent faulting (surface rupture less than 11,000 years ago) and historic faults (surface rupture less than 200 years ago) are located regionally near the site. The closest active fault to the site is the Hilton Creek fault zone. A brief description of this fault zone is included herein.

7.1 Hilton Creek Fault Zone

The nearest splay of the Hilton Creek fault is located 1.26 mi east of the subject site. The Hilton Creek fault is characterized by down-to-the-east normal displacement and it offsets late Tioga lateral moraines and outwash deposits. Surface-fault rupture was associated with four Mw 6+ earthquakes that occurred in May 1980 (Taylor and Bryant, 1980 #5586). Latest Pleistocene vertical slip rates range from 0.9 mm/yr to 4.2 mm/yr (Berry, 1990 #5582; Clark and Gilliespie, 1993 #5584).

8. CBC SEISMIC DESIGN PARAMETERS

Site coordinates of 37.6415, -118.9492 were obtained using the computer program **Google Earth**. Table I presents the Seismic Parameters for use in preparing a Design Response Spectra for the site.

TABLE I

SEISMIC PARAMETER (ASCE 7-16)	RECOMMENDED VALUE
Risk Category	II
Site Class	D – Stiff Soil
F_a	1.0 g
S_S	1.706 g
S_1	0.538 g
S_{MS}	1.706 g
S_{DS}	1.137 g
PGA/PGA_M	0.731/0.804 g

Conformance to the above criteria for strong ground shaking does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur during a large magnitude earthquake. Design of structures should comply with the requirements of the governing jurisdictions, building codes, and standard practices of the Structural Engineers Association of California.

9. SECONDARY EARTHQUAKE EFFECTS

Secondary effects that can be associated with severe ground shaking following a relatively large earthquake include shallow ground rupture, soil lurching, liquefaction, dynamic settlement, avalanches, and lateral spreading. These secondary effects of seismic shaking are discussed in the following sections.

9.1 Ground Rupture

Ground surface rupture results when the movement along a fault is sufficient to cause a gap or break of the fault zone on the surface. From our site reconnaissance, subsurface work, and a review of available geologic literature, we find no evidence to suggest that there are active, potentially active, or inactive faults that transect the subject site. Therefore the potential for ground rupture from an earthquake event is considered insignificant. The nearest known active regional fault is the Hilton Creek fault zone located approximately 1.26 mi east of the site.

9.2 Soil Lurching

Soil/ground lurching refers to the rolling motion of the ground surface as a result of seismic energy released during an earthquake. Effects of this nature are likely to cause severe damage to structures built on top of poorly consolidated sediments. In its present condition, the potential for lurching at the subject site is considered very low due to the presence of dense soils in the building areas.

9.3 Liquefaction

The project site is not located within any areas zoned for liquefaction hazards by local/state jurisdictions.

The potential for liquefaction to occur is not a design consideration, given the lack of a static or perched water table (See Section 6.1) and the dense nature of bearing soils on-site. Because the liquefaction potential is not a design consideration, the potential for ground failures associated with liquefaction, i.e post liquefaction reconsolidation, and sand boils are also considered not design considerations.

9.4 Dynamic Settlement

Portions of the shallow granular on-site soils may be loose and susceptible to dynamic settlement if strongly shaken by the design level earthquake. The potential for dynamic settlement will be greatly reduced if the loose and compressible soils near the surface (upper approximate 2-feet) are removed and properly compacted in accordance with recommendations in this report. The potential for dynamic settlement in the underlying glacial deposits is considered insignificant.

9.5 Avalanches

Avalanches can occur as a result of moderate to large earthquakes in Alpine terrain, which can cause rock and snow to move vertically and laterally downslope. These hazards typically affect structures which are located at the base of slopes or within proximity to the area of flow. The potential for rockfall or snow avalanches to occur at the subject site is not a design consideration, given the proximity of the site to a relatively steep slope area.

9.6 Lateral Spreading

Lateral spreading refers to landslides that form on gentle slopes as a result of seismic activity and have a fluid like movement. It differs from slope failures in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Soil types that are highly susceptible to lateral spread include silts and shale. Soils in the immediate vicinity of the building site consist of dense, sands with minor amounts of fines. Based on these findings, lateral spreading is not expected to occur on the site.

10. EXPANSIVE SOILS

Expansive soils are soils that shrink/swell when subjected to moisture. Shrink/swell potential is the relative change in volume to be expected with changes in moisture content; that is, the extent to which the soil shrinks as it dries or swells when it gets wet. The extent of shrinking and swelling is influenced by the amount and kind of clay in the soil. Shrinking and swelling of soils causes damage to building foundations, roads, and other structures.

Soils in the immediate vicinity of the building site consist of silty, fine to coarse sands. Based on these findings, there is a very low shrink/swell potential at the site.

11. VOLCANIC HAZARDS

The project site is in a highly active volcanic area. At least nineteen episodes of volcanism have occurred during the past approximately 3,000 (Kilbourne, Chesterman, and Wood, 1980). The most significant potential sources of volcanic activity are the Mono-Inyo Craters and the resurgent dome within the Long Valley caldera.

Explosive eruptions along the Inyo Craters volcanic chain occurred as recently as approximately 550 to 600 years ago (Miller, 1985). The most recent regional volcanic eruptions occurred between approximately 550 and 800 years ago along the Inyo Craters fracture zone (Rinehart and Huber, 1965; Miller, 1985; Sieh and Bursik, 1986). Historic non-eruptive volcanic activity occurred during the 1980 Mammoth Lakes earthquake sequence and during the 1989 Mammoth Mountain earthquake sequence (Sorey et al., 1999). Magmatic gas emissions associated with fumarolic activity have been documented on Mammoth Mountain and at Horseshoe Meadows (Sorey et al., 1999).

Future eruptions in the Mammoth Lakes area are certain to occur like those in the past, but they can be neither reliably predicted nor prevented. Future volcanic eruptions are more likely to occur along the Mono-Inyo Craters volcanic chain than from the resurgent dome or south moat area of the Long Valley caldera. The odds of an eruption occurring in any given year along the chain or in the caldera are very low (Miller, 1985; 1989).

12. ASBESTOS

Naturally occurring Asbestos is not present in the project area.

13. RADON

Radon gas is known to be present in the Mammoth Lakes area. However, the presence and amounts of the gas can be highly variable over short distances. So, while one site or structure may contain high concentrations of the gas, an adjacent building may contain limited amounts.

With respect to the site area, Radon levels are unknown. A passive mitigation system may need to be incorporated during construction. Therefore, a Radon specialist should be consulted.

14. SUBSIDENCE

The subject site is located not within an area known for past cases of substantial subsidence due to fluid removal. It is our opinion that the potential for significant subsidence due to the extraction of fluids is negligible. Soils subject to hydro-collapse, such as loose cemented silty and clayey soils were not noted in the test pits. Significant soil settlement associated with wetting of the subgrade materials is not anticipated.

15. FLOOD HAZARDS

Based upon a review of the FEMA Flood Insurance Rate Map, Mono County Panel 1389D, Map No. 06051C1389D, for the Mammoth Lakes area of Mono County (2011); the site is located in Zone X - outside the 0.2% annual chance flood plain.

16. CONCLUSIONS

Based upon the results of this study, it is our opinion that geologic hazards at the site area are minimal and any future construction within, is feasible from a geologic and geotechnical standpoint. The following more explicitly summarize our findings.

- There are no known active, potentially active, or inactive faults that transect the subject site. Evidence of past soil failures, or landslides on the site were not encountered.

- Seismic hazards at the site may be caused by ground shaking during seismic events on regional active faults. The nearest known active regional fault is the Hilton Creek fault located approximately 1.26 miles east of the site.

- A volcanic eruption could occur somewhere along Mono-Inyo Craters volcanic chain producing pyroclastic flows and surges, as well as volcanic ash and pumice fallout, which could impact the subject site. The odds however, of such an eruption are very low in a given year (Miller, 1985; 1989).

- Groundwater was not encountered in 2012. Minor amounts of seepage from localized snowmelt percolation may be encountered if the site is graded during the peak snow-melt period between April and May. Since the prediction of the location of such conditions is difficult to determine, they are typically mitigated if or when they occur.

- Site soils encountered during our field investigation consisted of dense, silty, fine to coarse-grained sands, with cobbles and boulders to 36-inches diameter. Clayey soils were not observed.

- Based upon findings from our 2012 investigation, the proposed building areas are situated on slightly sloping terrain underlain by up to approximately 2-feet of loose surficial soils considered "unsuitable" for the support of new fill or structural loads. Where these soils will be subjected to increased loads from new fills or structures, remedial grading consisting of over-excavation and compaction is recommended to improve the bearing capacity of those materials. Remedial grading recommendations are provided in this report.

- The depth of the unsuitable soils is based upon the areas observed during the field investigation. It should be anticipated that the overall depth and extent of the unsuitable materials exposed during construction may vary from that encountered.

- Reasonably continuous construction observation and review during site grading and foundation installation should be employed. This will allow for evaluation of the actual soil conditions and the ability to provide appropriate revisions where required during construction.

- Subsurface strata which would retard the flow of water downward were not observed during the investigation. Drywells should therefore function as designed.

- Due to the semi-cohesionless nature of the site soils, sloughing may occur in the utility trench excavations. Shoring or forming may be required.

- This study did not include an environmental review of the Site area. It is possible that some dump fill soils may exist on the site. Since the prediction of the location of such conditions is difficult to determine, they are typically mitigated if or when they occur.

17. RECOMMENDATIONS

The following recommendations should be adhered to during site development. These recommendations are based on empirical and analytical methods typical of the standard of practice in California. If these recommendations appear not to cover any specific feature of the project, please contact our office for additions or revisions to the recommendations.

17.1 Earthwork

Site grading should be observed by SGSI. Such observations are considered essential to identify field conditions that differ from those anticipated by the investigation, to adjust design to actual field conditions, and to determine that the grading is accomplished in general accordance with the recommendations of this report. Earthwork and grading recommendations which include guidelines for site preparation fill compaction, temporary excavations, and trench backfill are provided in Appendix C.

The recommendations contained in Appendix C are general grading specifications provided for typical grading projects. Some of the recommendations may not be strictly applicable to this project. The specific recommendations contained in the text of this report supersede the general recommendations in Appendix C. The contract between the developer and earthwork contractor should be worded such that it is the responsibility of the contractor to place the fill properly in accordance with the recommendations of this report and the specifications in Appendix C, notwithstanding the testing and observation of the geotechnical consultant.

17.1.1 Site Preparation

Prior to grading, the proposed structural improvement areas (i.e. all structural fill, pavements areas and structural building, etc.) of the site should be cleared of surface and subsurface obstructions, including vegetation. Vegetation and debris should be disposed of offsite. Holes resulting from removal of buried obstructions, which extend below the recommended removal depths described herein or below finished site grades (whichever is lower) should be filled with properly compacted soil. Should existing underground utilities be encountered they should be completely removed and properly backfilled.

17.1.2 Removals

Up to approximately 2-feet of loose surficial soils were observed during our 2012 investigation. These soils will need to be over-excavated and removed from within all structural areas. Excavations should extend to a minimum horizontal distance of at least 3-feet outside any building footprints. Removals and compaction recommendations are provided in Appendix C.

Cut/fill transitions shall not be allowed below foundation elements. If this will occur, we recommend that all footings be deepened to extend into uniform competent native soils, and that all soils below structural slabs be undercut/removed so that slabs will be supported on an at least a 2-foot thick compacted fill mat. As an alternative to the 2-foot fill mat, the slab may be designed to accommodate for differential settlements which conservatively speaking may be 1” static over 30’.

For any paved driveway, parking areas and other improvements a 1½-foot removal is recommended depending on site conditions (i.e. depth of root zone, and depth of disturbance which may have locally deeper removal depths). The removal should also extend a minimum horizontal distance of 2-feet beyond the back of curbs and pavement. Removals and compaction recommendations are provided in Appendix C.

Site soils are suitable for use as compacted fill if they are processed in accordance with the recommendations in Appendix C. Approved fill soils should be placed in thin lifts (8-inches loose thickness) and moisture conditioned to at least optimum moisture content. All fill should be compacted to a minimum of 90-percent of the laboratory maximum dry density per ASTM D 1557.

17.2 Foundations

Shallow, spread or continuous footings may be used to support the proposed structure provided they are founded entirely upon properly compacted fill, or competent native deposits. Continuous and isolated column foundations should be sized according to the allowable soil bearing pressures shown in Table II below. The pressures shown on Table II are for dead loads plus long-term live load.

TABLE II

Allowable Soil Bearing Pressure (psf) FS =3.0	Passive Resistance (psf/ft) FS =1.5
2,500	250

An allowable coefficient of friction of 0.25 may be used between the concrete and the underlying soil. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. In addition, when passive resistance is calculated, the upper one foot of soil should be neglected unless the ground surface is covered by pavement.

Footings may be constructed according to California Building Code requirements regarding width (minimum 12-inches). Exterior and interior foundations shall be founded within compacted fill or competent native soils. Exterior foundations shall have a minimum embedment depth of 24-inches below outside adjacent grade. Interior foundation depths shall also be a minimum of 12-inches below adjacent grade.

17.3 Lateral Earth Pressures

The recommended equivalent fluid pressure for each case for walls founded above the static ground water and backfilled with select soils is provided in Table III. Wall footings should be designed in accordance with structural considerations.

TABLE III

Slope of Backfill Behind Retaining Wall	Active Pressure Non-restrained (psf/ft)	At-Rest Pressure restrained walls (psf/ft)
Level	30	45
2:1 Slope	45	60

Passive Resistance – 250 psf/ft.
Coefficient of friction against sliding - 0.25.
Soil Unit Weight – 110.5 pcf

The passive resistance and coefficient of friction may be used in combination if there is a fixed structure, such as a floor slab over the toe of the retaining wall. If the two values are used in combination, the passive resistance value should be reduced by one-third.

The select backfill should have an expansion index (EI) of no greater than 50 and a sand equivalent (SE) greater than 15. The backfill soils should be tested by the soils engineer prior to backfill operations starting for the retaining wall structures.

Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third the anticipated surcharge load for unrestrained walls, and one-half the anticipated surcharge load for restrained walls. Surcharge loading effects from the adjacent structures should be evaluated by the structural engineer.

17.3.1 Seismic Lateral Earth Pressures

During an earthquake, an additional earth induced lateral earth pressure will be applied to the wall. Experience has shown that walls adequately designed for static loading have generally performed well during earthquake loading. However, if walls are to be designed for seismic loading, the magnitude of the seismic pressure can be evaluated using the procedures developed by Mononobe-Okabe which consider that the seismic pressure is approximated using a lateral pressure coefficient of 0.75x the effective ground acceleration. The effective ground acceleration is taken as equal to 2/3^{rds} the maximum expected ground acceleration.

For this project the site specific PGA_M is 0.804g. The effective ground surface acceleration is therefore 0.50g. Considering a soil unit weight of 110.5 pcf, we recommend an additional fluid pressure of 44 pcf (added to the pressures shown in Table III) be used to calculate the lateral seismic pressure. The resultant of the seismic pressure should be applied at a height of 0.6x the wall height above the base of the wall.

The pressure increment for cantilevered retaining walls should be taken as an inverted triangular distribution from the stem of the cantilevered retaining wall to the top of the cantilevered retaining wall. For resistive walls, i.e. basement walls, the pressure increment should be taken as a rectangular force applied from the stem of the basement wall to the top of the basement wall.

17.4 Wall Drainage

All retaining wall structures should be provided with appropriate drainage and waterproofing. Drainage should consist of continuous drains installed along the base of the wall out-letting to a storm drain system or the surface if grade allows. Waterproofing shall be designed by the project Architect but should consist of no less than placement of a flexible adhesive waterproofing membrane, overlain (Mel-Rol, Bituthene or eq) by dimpled drainboard. Additionally, all cold joints (especially at any footing/wall interfaces) should be appropriately sealed with a concrete joint sealer (WR Meadows SealTight or eq.) prior to placement of the adhesive waterproofing membrane.

The lateral earth pressures assume sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended for both restrained and unrestrained walls.

17.5 Anticipated Settlement

The total settlement of the conventional foundations bearing into competent native or shallow properly compacted fills is anticipated to be less than ½-inch. Differential settlement is anticipated to be less than ¼-inch between adjacent foundations.

17.6 Foundation Construction

The following preliminary recommendations assume very low expansive soils near finish pad grade.

- Footings should be designed in accordance with the structural engineer’s requirements. Exterior and interior foundations shall be founded within compacted fill or competent native soils.

- All footing excavations should be observed by a representative of SGSI prior to placement of reinforcing steel, to assure proper embedment into competent soils.

- Footing trenches should not have any rocks or boulders protruding into the trench bottom. Soft soil pockets created by rock removal during foundation excavation shall be replaced with approved fill material and compacted to 90-percent of the material's maximum dry density.

- Site soils are suitable for use as compacted fill as long as they are processed in accordance with the recommendations in Appendix C.

- Approved fill soils should be placed in thin lifts (max 8-inches loose thickness) and moisture conditioned to at least optimum moisture content. All fill should be compacted to a minimum of 90-percent of the laboratory maximum dry density per ASTM D1557.

- Any import soils shall be tested for suitability in advance by the project Geotechnical Engineer. Earth fill material shall not contain more than 1-percent of organic materials (by volume). Imported fill shall have a maximum plasticity index of ≤ 12, and a liquid limit less than 40 when measured in accordance with ASTM D 4318.

17.7 Foundation Setback

Utility trenches that parallel or nearly parallel structure footings should not encroach within a 1:1 plane extending downward and outward from the outside edge of the footing.

17.8 Concrete Slab-on-Grade Floors

Interior: Building slabs may be supported by compacted fill or competent native deposits. Cut/fill transitions below slabs should be avoided. Subgrade soils should have a very low expansion potential (EI < 20). Slabs should be designed for anticipated loading and thickness and shall meet the requirements of the Structural Engineer of record. Likewise, control joints and reinforcement should be designed by the Structural Engineer.

Structural fill and subgrade soils underlying concrete slabs shall be compacted to a minimum of 95-percent of the material's maximum dry density for the upper 12-inches. Concrete slabs should be underlain by a vapor barrier/retarder (Stego Wrap or equivalent - 15 mil minimum thickness), which is in turn, underlain by a 4-inch layer of ¾" crushed stone. All penetrations and laps in the moisture barrier should be appropriately sealed. The membrane should have a high puncture resistance and should be installed so that there are no openings or holes. All seams should be overlapped and sealed at the laps per the manufacturer's recommendations. Where pipes extend through the membrane, the barrier should be sealed to the pipes.

Moisture retarders can reduce, but not eliminate moisture vapor movement from the underlying soils up through the slab. We recommend that the floor coverings installer test the moisture vapor flux rate prior to attempting application of the flooring. "Breathable" floor coverings should be considered if the vapor flux rates are high. A slip-sheet should be used if crack sensitive floor coverings are planned.

The use of reinforcement in slabs and foundations will generally reduce the potential for drying and shrinkage cracking. However, some cracking may be expected as the concrete cures. Concrete cracking and/or spalling is often aggravated by a high cement ratio, high or low concrete temperature at the time of placement, small nominal aggregate size, rapid moisture loss, or the addition of water during placement. The use of low slump concrete (not exceeding 4-inches at the time of placement), a water-cement ratio no greater than 0.45 by weight, and proper curing methods can reduce the potential for shrinkage cracking.

Exterior Concrete Flatwork: Concrete flatwork should be a minimum 4-inches in thickness and should be supported by very low expansion subgrade soils (EI < 20). Flatwork should be reinforced with #3 rebar placed at slab mid-height on 24-inch centers, both ways. Crack control joints should be used and should have a maximum spacing of 5-foot on center each way for sidewalks, and 10-foot on center each way for slabs. Actual crack control joints should be designed by the project Civil Engineer. A vapor retarder is not needed.

18. FLEXIBLE PAVEMENT RECOMMENDATIONS

SGSI recommends the following pavement sections:

- 4-inches Asphalt Concrete / 6-inches Caltrans Class II Aggregate Base

The upper 12-inches of subgrade material along with the Caltrans specification for Class II Aggregate Base and the Asphaltic concrete shall be compacted to a minimum of 95-percent of the material's maximum density. If pavement areas are adjacent to landscape or snow storage areas, some deterioration of the subgrade load bearing capacity may result. We recommend some measures of moisture control (such as deepened curbs or other moisture barrier materials) be provided to prevent the subgrade soils from becoming saturated.

19. DRAINAGE

Water should not be allowed to pond adjacent to buildings, and drainage should not flow uncontrolled over the top of, or down the face of, any descending slopes. Positive site drainage should direct runoff away from foundations and pavement areas; Site drainage should be directed to an approved drainage facility. Positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 5-percent for earthen surfaces for a distance of at least 10-feet away from the face of wall. If 10-feet cannot be achieved, an alternative of a gradient of at least 5-percent to an area drain or swale having a gradient of 2-percent is acceptable.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made.

20. CONSTRUCTION CONSIDERATIONS

Excavations will be required to construct retaining walls, footings, install utilities, and to remove locally weak or unsuitable soils. All excavations that will be deeper than 4-feet and will be entered by workers should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards. For temporary excavations, we recommend that the following OSHA soil classifications be used:

Fill - Type C
Native - Type B

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant on a case-by-case basis in accordance with the OSHA regulations. For trench or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1.5:1 in engineered fill and slope wash, and 1:1 in native deposits. Temporary excavations that encounter seepage may be shored or stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

Excavation spoils should not be stockpiled adjacent to excavations as they can surcharge the soils and trigger failure. In addition, proper erosion protection, is recommended to reduce the possibility for erosion of slopes during grading and building construction. Ultimately, it is the contractor's responsibility to maintain safe working conditions for persons on-site.

21. GEOTECHNICAL OBSERVATION AND TESTING DURING CONSTRUCTION

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction. Geotechnical observation and testing are required per the California Building Code (CBC). Geotechnical observation and/or testing should be performed by SGSI at the following stages:

- During grading (removal bottoms, fill placement, etc);
- During backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Preparation of pavement subgrade and placement of aggregate base;
- After building and wall footing excavation and prior to placing concrete and/or reinforcement; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

22. LIMITATIONS

This report has been prepared for the sole use and benefit of our client. The conclusions of this report pertain only to the site investigated. It should be understood that the consulting provided, and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report, and/or any other geotechnical aspects of the project, should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive this advice. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Sierra Geotechnical Services Incorporated from and against any liability, which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Sierra Geotechnical Services Incorporated.

Conclusions and recommendations presented herein are based upon the evaluation of technical information gathered, experience, and professional judgment. Other consultants could arrive at different conclusions and recommendations. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings within this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

23. <u>REFERENCES</u>

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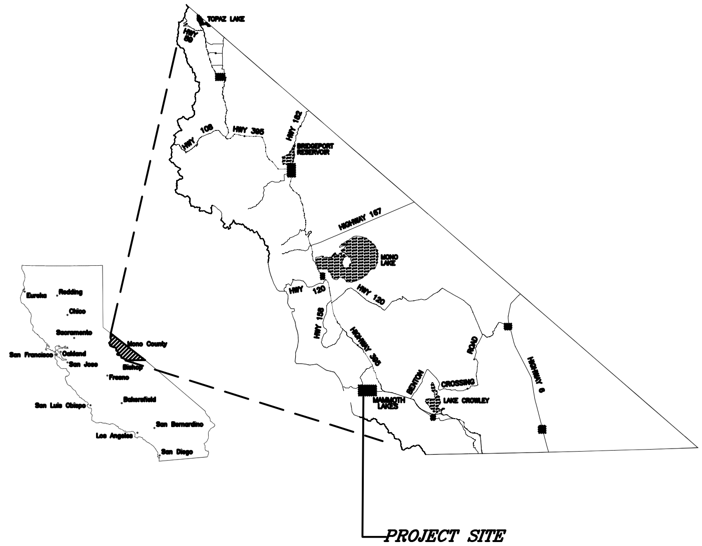
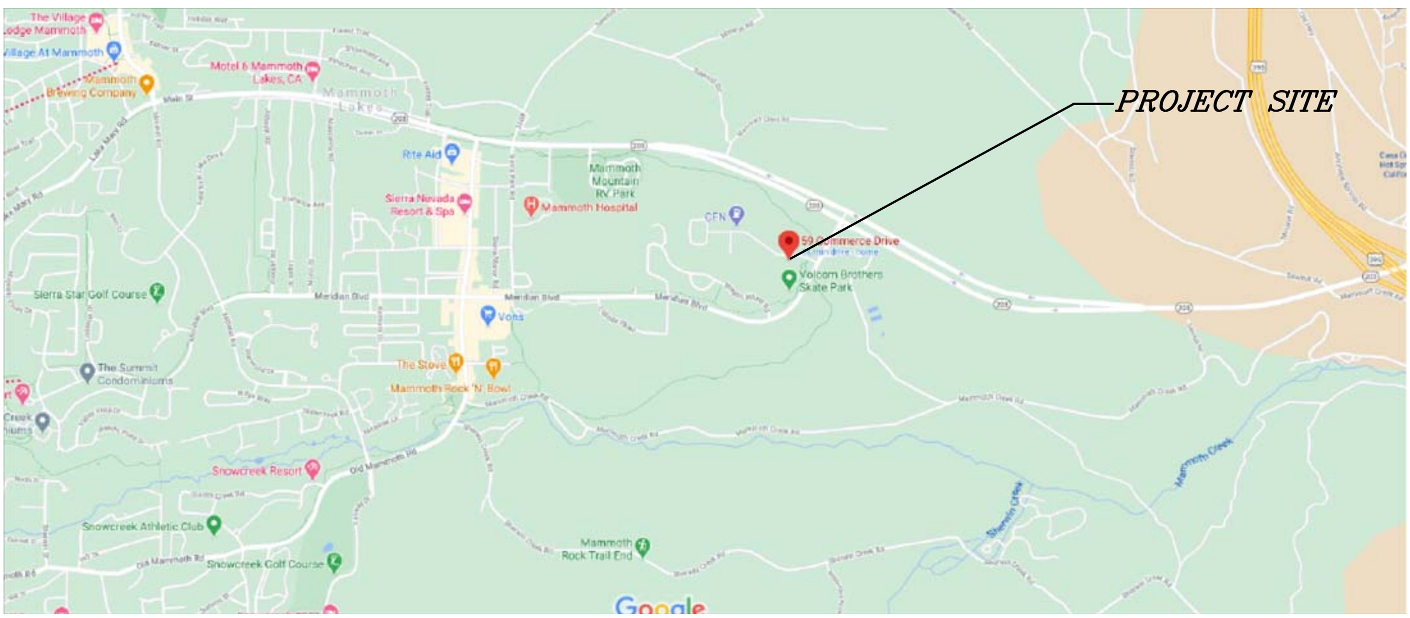




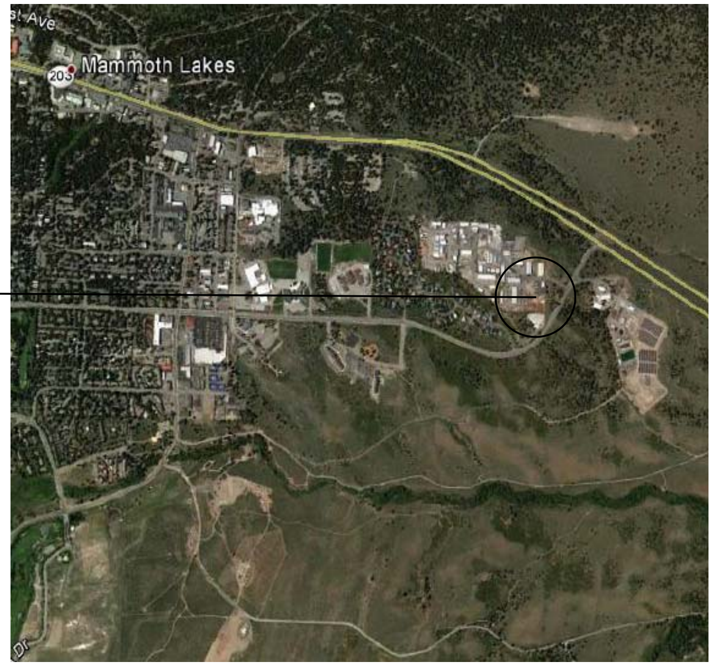
N

NOT TO SCALE

SIERRA GEOTECHNICAL SERVICES, INC.

PROJECT: REGIONAL MAP MAMMOTH DISPOSAL FACILITY	
SCALE: NTS	DATE: 9/2020
DRAWING: FIG1.DWG	DRAWN BY: JAA
JOB NO.: 3.31164.1	FIGURE: FIGURE 1

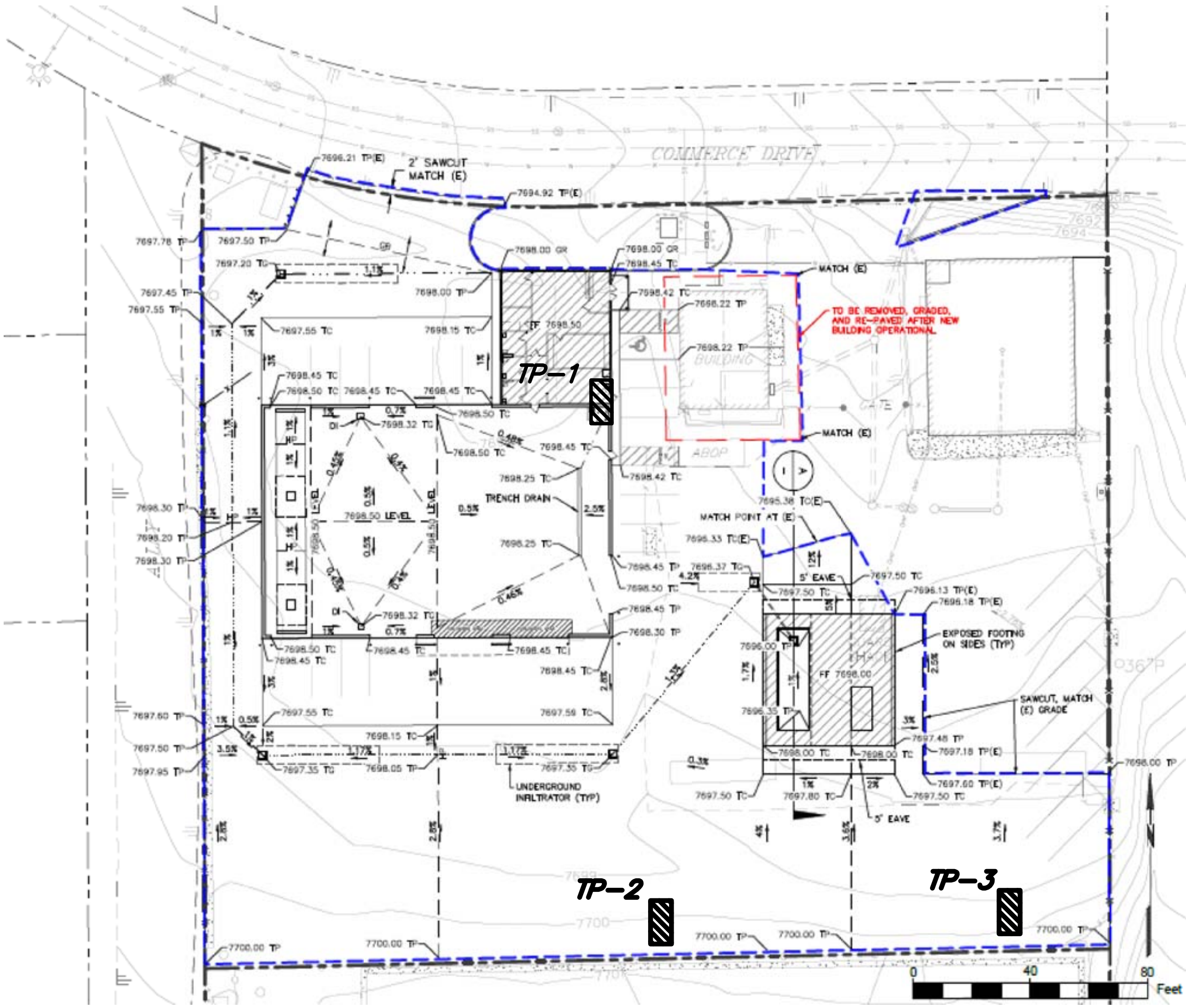
PROJECT
SITE



NOT TO SCALE

PROJECT: VICINITY MAP MAMMOTH DISPOSAL FACILITY	
SCALE: NTS	DATE: 9/2020
DRAWING: FIG2.DWG	DRAWN BY: JAA
JOB NO.: 3.31164.1	FIGURE: FIGURE 2

SGS
SIERRA GEOTECHNICAL SERVICES, INC.



LEGEND

TP-3

APPROXIMATE LOCATION OF TEST PIT

PROJECT: SUBSURFACE GEOTECHNICAL MAP MAMMOTH DISPOSAL FACILITY	
SCALE: NTS	DATE: 9/2020
DRAWING: FIG3.DWG	DRAWN BY: JAA
JOB NO.: 3.31164.1	FIGURE: FIGURE 3

APPENDIX A

EXPLORATORY TEST PIT LOGS

A subsurface field investigation was performed in November 20th, 2012 that included the excavation of three exploratory test pits with a Case Backhoe and 24-inch bucket. A geologist from our office logged the excavations as they were advanced.

In-place nuclear density tests and bulk samples of the soils encountered were obtained during the field investigation. Results of the in-place nuclear density tests are presented on the logs of the exploratory test pits. Details of the laboratory testing are presented in Appendix B.

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TEST PIT LOGS

JOB NO: <u>3.31164</u> **PROJECT:** <u>MAMMOTH DISPOSAL SITE</u>
DATE: <u>11/20/2012</u> **LOGGED BY:** <u>JA</u>
EQUIP: <u>CASE EXCAVATOR W/ 24" BUCKET</u>

TEST PIT	DEPTH (ft)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH (ft)	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
1	0 - 4	SM	2 - 3	7.0	91.1	**<u>GLACIAL DEPOSITS</u>** Dark brown to grayish-brown, moist, dense, silty, very fine to coarse SAND, few cobbles and boulders to 13" diameter.
	4 - 8	SM				Reddish-brown.
						-- *Total depth = 8-feet. No groundwater encountered. Backfilled 11/20/2012.*

TEST PIT LOGS

JOB NO: 3.31164
DATE: 11/20/2012
EQUIP: CASE BACKHOE W/ 24" BUCKET

PROJECT: MAMMOTH DISPOSAL SITE
LOGGED BY: JA

TEST PIT	DEPTH (ft)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH (ft)	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
2	0 - 4	SM	1 - 2	5.0	83.7	**GLACIAL DEPOSITS** Dark brown to dark reddish-brown, moist, medium dense to dense, silty very fine to coarse SAND with few cobbles.
	4 - 5	SM	4 - 5	6.5	95.0	Grayish-brown, moist, dense, silty, very fine to coarse SAND with abundant rock fragments and few sub angular boulders to 36" diameter. Rock content is approximately 10-15% of the deposit.
						Total depth = 5-feet. No groundwater encountered. Backfilled 11/20/2012.

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TEST PIT LOGS

JOB NO: **3.31164**
DATE: **11/20/2012**
EQUIP: **CASE BACKHOE W/ 24" BUCKET**

PROJECT: **MAMMOTH DISPOSAL SITE**
LOGGED BY: **JA**

TEST PIT	DEPTH (ft)	U.S.C.S. GROUP SYMBOL	SAMPLE DEPTH (ft)	PERCENT MOISTURE	DRY DENSITY (pcf)	DESCRIPTION
3	0 - 5	SM	1 - 2	5.5	93.0	**GLACIAL DEPOSITS** Grayish-brown, moist, dense, silty, very fine to coarse SAND with abundant rock fragments and few sub angular boulders to 36" diameter. Rock content is approximately 10-15% of the deposit. --- *Total depth = 5-feet. No groundwater encountered. Backfilled 11/20/2012.*

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed on the representative test samples to provide a basis for development of design parameters. Soil materials were visually classified in the field according to the Unified Soil Classification System (USCS). Selected samples were tested for the following parameters: Classification and grain size distribution, direct shear, in-situ moisture and density and maximum dry density (Proctor). Laboratory tests were performed in general accordance with the American Society of Testing and Materials (ASTM) procedures.

The results of our laboratory testing are presented herein. The results of the USCS classifications and in-situ moisture and density are presented on the test pit logs (Appendix A).

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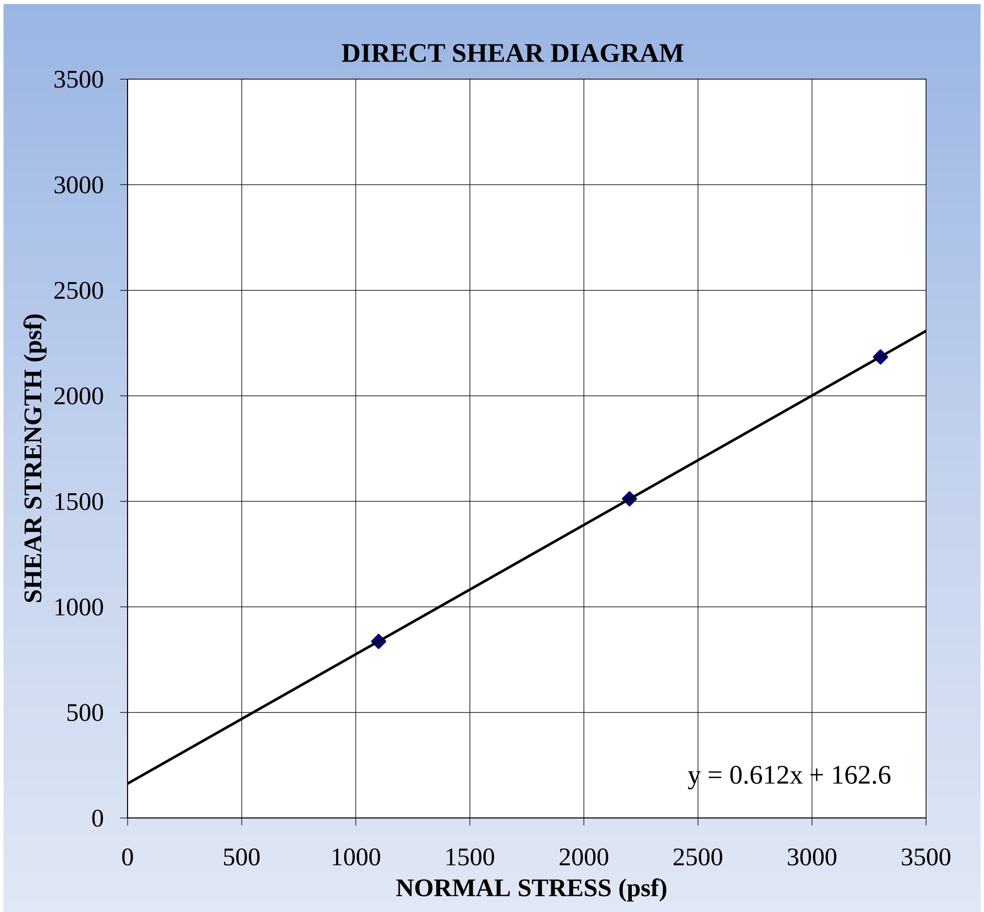

Boring No: TP-1

Sample Depth: 2-3 feet

Friction Angle: 31 degrees

Cohesion: 163 psf

Dry Density: 99.5 pcf

Remolded to 90%

PROJECT: MAMMOTH DISPOSAL TRANSFER FACILITY

3.31164

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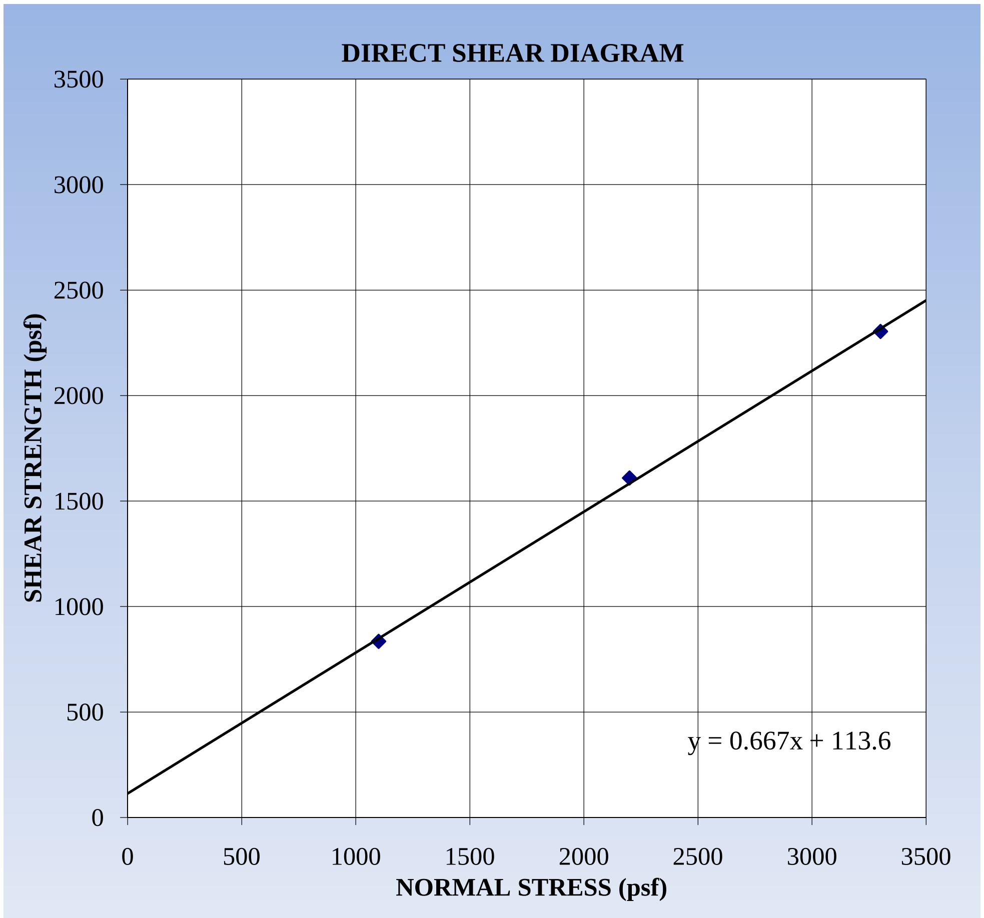

Boring No: TP-2 | Sample Depth: 4-5 feet

Friction Angle: 34 degrees | Cohesion: 114 psf

Dry Density: 101.7 pcf | Remolded to 90%

PROJECT: MAMMOTH DISPOSAL TRANSFER FACILITY

3.31164

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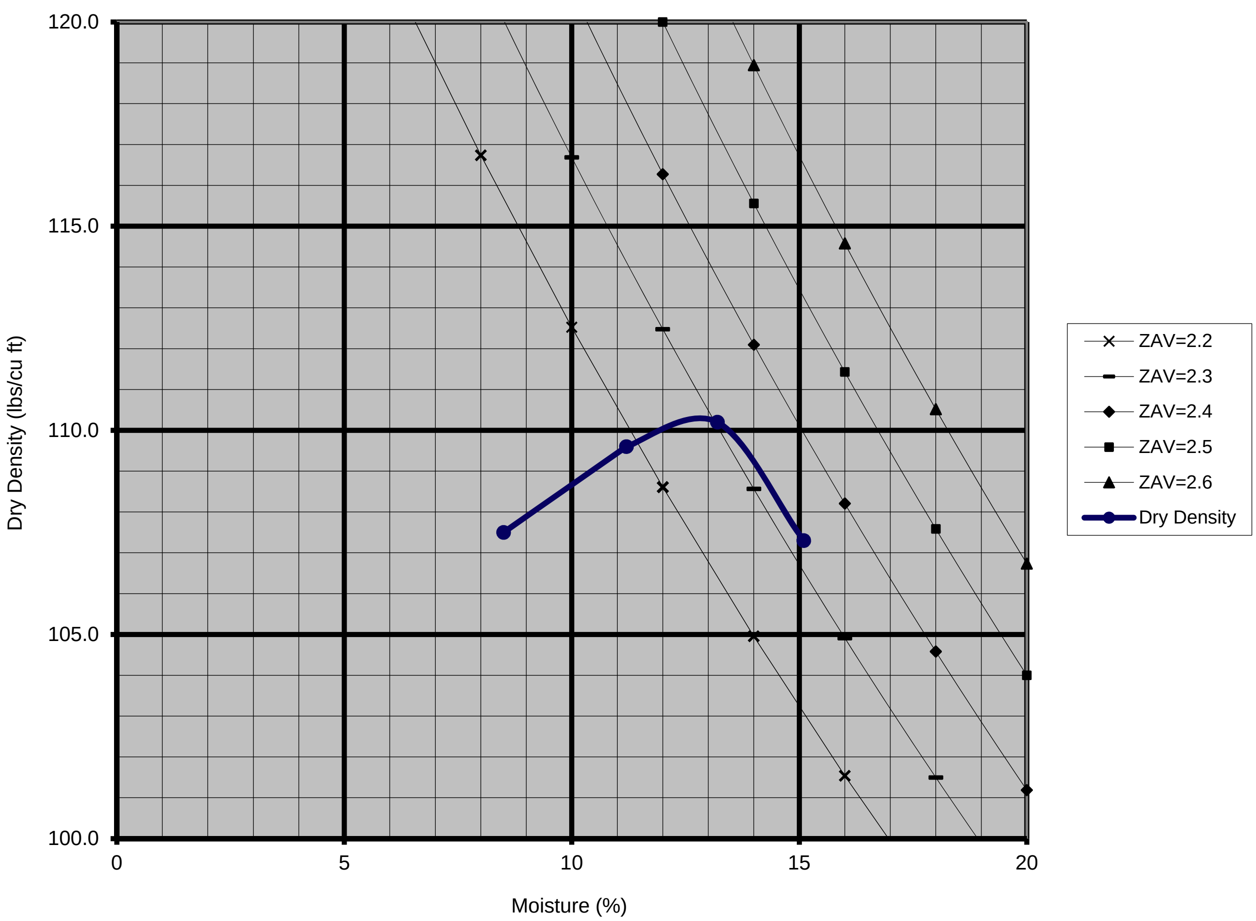
Caltrans Lab #214

AASHTO R18

MAXIMUM DENSITY-MOISTURE CURVE (PROCTOR)

Project Name								Project No.	
Geotechnical Investigation for Recovery/Waste Transfer Station - Mammoth Transfer MRF								3.31164	
Client								Material	
Waste Connections, Inc.								Native Spoils	
Source								Delivered By	Deliver Date
Test Pit TP-1 between 2' and 3' deep								JA	11/20/12
Proctor No	Test Date	Native	Belt Cut	Screen	Chute	Stockpile	Truck	Tested By	Report Date
2	12/7/12	X						BY	12/7/12



Laboratory Data:

Test #	Soil & Mold (lb)	Mold (lb)	Soil (lb)	Wet Density (pcf)	Percent Moisture	Dry Density (pcf)	Mold Volume (cf)	Max. Dry Density (pcf)	Optimum Moisture (%)
1	13.608	9.698	3.910	116.7	8.5	107.5	0.03350	110.5	13.0
2	13.779	9.698	4.081	121.8	11.2	109.6			
3	13.876	9.698	4.178	124.7	13.2	110.2		Rock	
4	13.835	9.698	4.137	123.5	15.1	107.3		Corr. (pcf)	
5									

Note: ZAV=Zero Air Voids per Specific Gravity of Soil Solids

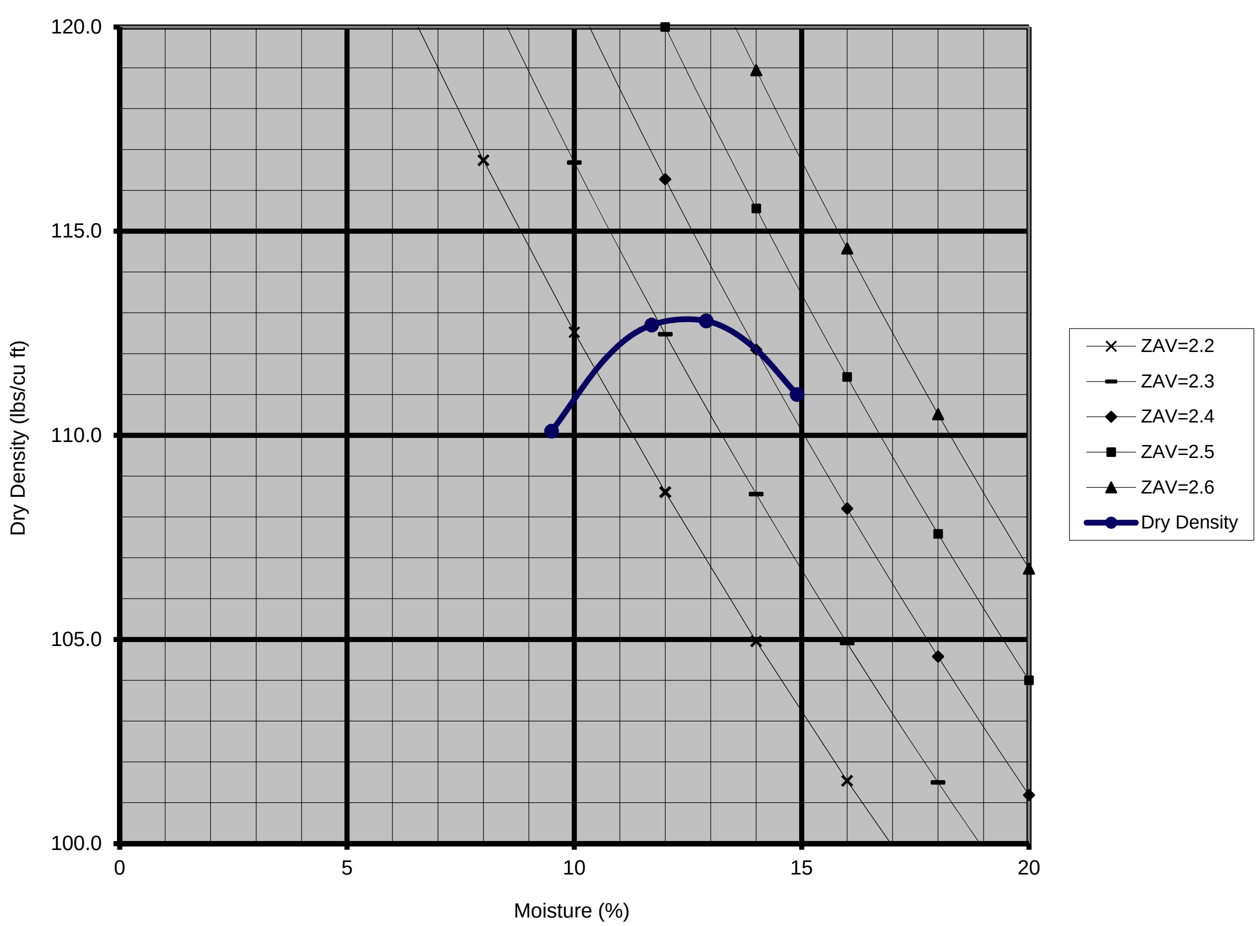
SIERRA GEOTECHNICAL SERVICES, INC.

ENVIRONMENTAL • GEOTECHNICAL • GEOLOGY • HYDROGEOLOGY • MATERIALS

Caltrans Lab #214

MAXIMUM DENSITY-MOISTURE CURVE (PROCTOR)

Project Name								Project No.	
Geotechnical Investigation for Recovery/Waste Transfer Station - Mammoth Transfer MRF								3.31164	
Client								Material	
Waste Connections, Inc.								Native Spoils	
Source								Delivered By	Deliver Date
Test Pit TP-2 between 4' and 5' deep								JA	11/20/12
Proctor No	Test Date	Native	Belt Cut	Screen	Chute	Stockpile	Truck	Tested By	Report Date
1	12/6/12	X						BY	12/7/12



Laboratory Data:

Test #	Soil & Mold (lb)	Mold (lb)	Soil (lb)	Wet Density (pcf)	Percent Moisture	Dry Density (pcf)	Mold Volume (cf)	Max. Dry Density (pcf)	Optimum Moisture (%)
1	13.738	9.698	4.040	120.6	9.5	110.1	0.03350	113.0	12.5
2	13.914	9.698	4.216	125.9	11.7	112.7			
3	13.965	9.698	4.267	127.4	12.9	112.8		Rock	
4	13.972	9.698	4.274	127.6	14.9	111.0		Corr. (pcf)	
5									

Note: ZAV=Zero Air Voids per Specific Gravity of Soil Solids

SIERRA GEOTECHNICAL SERVICES INC.

APPENDIX C

GENERAL EARTHWORK AND GRADING

These general earthwork and grading specifications are for the grading and earthwork shown on the approved grading or construction plan(s) and/or indicated in the geotechnical report(s). Earthwork and grading should be conducted in accordance with applicable grading ordinances, the current California Building Code, and the recommendations of this report. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on field conditions observed by the geotechnical consultant during grading.

Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record. The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of grading or construction.

During grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground, after it has been cleared for receiving fill but before it has been placed, bottoms of all "remedial removal areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the contractor on a routine and frequent basis.

The Earthwork Contractor

The Earthwork Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications. The Earthwork Contractor shall review and accept the plans, geotechnical report(s) and these Specifications prior to the commencement of grading. The Earthwork Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant unsatisfactory conditions, such as unstable soil, improper moisture condition, inadequate compaction, adverse weather, etc… are resulting in a quality of work less than required in these Specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

Site Preparation

General: Site preparation includes removal of deleterious materials, unsuitable materials, and existing improvements from areas where new improvements or new fills are planned. Deleterious materials, which include vegetation, trash, and debris, should be removed from the site and legally disposed of off-site. Unsuitable materials include loose or disturbed soils, undocumented fills, contaminated soils, or other unsuitable materials. The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1-percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant etc…) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fine and/or imprisonment and shall not be allowed.

Any existing subsurface utilities that are to be abandoned should be removed and the trenches backfilled and compacted. If necessary, abandoned pipelines may be filled with grout or slurry cement as recommended by, and under the observation of, the Geotechnical Consultant.

Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured, or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

Compaction

The onsite soils are suitable for placement as compacted fill provided the organics, oversized rock (greater than 6-inches in diameter) and deleterious materials are removed. Rocks greater than 6-inches and less than 2-feet in diameter can be placed in the bottom of deeper fills or approved areas provided they are selectively placed in such a manner that no large voids are created. All rocks shall be placed a minimum of 4-feet below finish grade elevation unless used for landscaping purposes. Any import soils shall be tested for suitability in advance by the project Geotechnical Engineer.

After making the recommended removals prior to fill placement, the exposed ground surface should be scarified to a depth of approximately 8-inches, moisture conditioned as necessary, and compacted to at least 90-percent of the maximum dry density obtained using ASTM D1557 as a guideline. Surfaces on which fill is to be placed which are steeper than 5:1 (Horizontal to vertical) should be benched so that the fill placement occurs on relatively level ground.

For the parking areas and other improvements a one-foot removal is recommended depending on site conditions (i.e. depth of root zone, and depth of disturbance which may have locally deeper removal depths). The removal bottom should be observed (tested as needed) by the geotechnical consultant prior to placing fill soils. The upper 12-inches of subgrade material along with the Class II Aggregate Base and the Asphaltic concrete shall be compacted to a minimum of 95-percent of the materials maximum dry density as determined by ASTM D1557. The subgrade and aggregate base shall be moisture-conditioned and compacted to 95-percent of the material's maximum dry density as determined by ASTM D-1557 to a depth of 12-inches.

All fill and backfill to be placed in association with the proposed construction should be accomplished slightly over optimum moisture content using equipment that is capable of producing a uniformly compacted product throughout the entire fill lift. Fill materials at less than optimum moisture should have water added and the fill mixed to result in material that is uniformly above optimum moisture content. Fill materials that are too wet can be aerated by blading or other satisfactory methods until the moisture content is as required. The wet soils may be mixed with drier materials in order to achieve acceptable moisture content.

The fill and backfill should be placed in horizontal lifts at a thickness appropriate for equipment spreading, mixing, and compacting the material, but generally should not exceed 8-inches in loose thickness. Retaining wall backfill shall be composed of a granular material (maximum ≤ 3-inch rock) with an expansion index (EI) of no greater than 50 and a sand equivalent (SE) greater than 30.

No fill soils shall be placed during unfavorable weather conditions. When work is interrupted by rains or snow, fill operations shall not be resumed until the field tests by the geotechnical engineer indicate that the moisture content and density of the fill are as previously specified.

Slopes

All slopes shall be compacted in a single continuous operation upon completion of grading by means of sheepsfoot or other suitable equipment, or all loose soils remaining on the slopes shall be trimmed back until a firm compacted surface is exposed. Slope compaction tests shall be made within one foot of slope surface.

Cut and fill slopes shall be a maximum of 2:1 (horizontal to vertical) unless approved by the Geotechnical Consultant.

Planting and irrigation of cut and fill slopes and/or installation of erosion control and drainage devices should be completed due to the erosion potential of the soil.

Temporary Excavations

Temporary excavation shall be made no steeper than 1:1 (horizontal to vertical). The recommended slope for temporary excavations does not preclude local raveling and sloughing. Where wet soils are exposed, flatter excavation of slopes and dewatering may be necessary. In areas of insufficient space for slope cuts, or where soils with little or no binder are encountered, shoring shall be used.

All large rocks exposed above temporary cuts shall be removed prior to foundation excavation. In addition any rocks exposed during development from raveling and sloughing should be removed immediately.

All excavations should comply with the requirements of the California Construction and General Industry Safety Orders and the Occupational Safety and Health Act and other public agencies having jurisdiction.

Trench Backfill

Exterior trenches, paralleling a footing and extending below a 1:1 plane projected from the outside bottom edge of the footing, shall be compacted to a minimum of 95-percent per ASTM D1557. All trenches in structural areas and under concrete flatwork shall be compacted to a minimum of 95-percent per ASTM D1557. All trenches in non-structural areas shall be compacted to a minimum of 85-percent per ASTM D1557.

All material used for trench backfill shall be approved by the Geotechnical Engineer prior to placement. All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1-foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 95-percent of maximum from 1-foot above the top of the conduit to the surface.

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical

Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

Regulations of the governing agency may supersede the above, and all trench excavations should conform to all applicable safety codes. The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.