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Geotechnical Investigation of the LIGO Site

Prepared For:

CALIFORNIA INSTITUTE OF TECHNOLOGY
PASADENA, CALIFORNIA

January 1995

WCC File 93B107C

Woodward-Clyde

Engineering & sciences applied to the earth & its environment

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GEOTECHNICAL INVESTIGATION

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LIVINGSTON, LOUISIANA

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California Institute of Technology
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LIVINGSTON, LOUISIANA LIGO SITE

1.0 INTRODUCTION

We have completed the geologic and geotechnical investigation of the LIGO site located at Livingston, Louisiana. Please find below our findings and recommendations which are contained in the following sections. Detailed results of geotechnical tests performed are included as Appendix A. Appendix B contains the results of electronic Cone Penetrometer Tests (CPT), seismic piezocone tests (SCP) and crosshole seismic tests. Appendix C presents the results of seismicity research, and Appendix D contains idealized subgrade profile plots where both CPT and conventional geotechnical test results are summarized, and Appendix E includes the results of chemical and corrosion potential investigations.

2.0 SCOPE OF WORK

This investigation was authorized by the California Institute of Technology on February 18, 1993. The investigation was performed in general accordance with our proposal of January 8, 1993.

3.0 FACILITY DESCRIPTION

The LIGO facility is located in Livingston Parish north of Livingston, Louisiana (Figure A-1, Appendix A). The LIGO facility consists of a corner station, two end stations and beam tube housing constructed on an above grade berm and a service road founded on the berm. The top of the berm will be above the 100-year flood level at elevation of 65 feet (Mean Sea Level).

3.1 Corner Station

The corner station is located at the apex of the southeast and southwest arms of the facility. The corner station will consist of a 60,000 square foot steel frame structure. This building will include vacuum chambers and associated equipment, utility rooms and an office/shop area. The vacuum equipment is planned to be constructed atop a thick single monolithic reinforced concrete mat foundation. The building height over the vacuum equipment area will range from 35 to 55 feet. It is anticipated the single story office and shop area is planned to be constructed with conventional spread footings and concrete slab. The maximum column loads are on the order of 50 kips plus or minus twenty percent.

3.2 End Station

The end station will consist of an 8,000 square feet structure for housing the vacuum equipment area and it is anticipated that it will be supported on a thick single monolithic reinforced concrete mat foundation. The maximum column loads are on the order of 50 kips plus or minus twenty percent. The mat foundation will support equipment loads ranging from a nominal 700 psf to maximum of 1,500 psf for the vacuum equipment.

3.3 Beam Tube

The beam tube will be housed in a 16-foot diameter semi-circular reinforced concrete arch. This enclosure will provide sufficient room for service access to the beam tube and will exert a load of 2 kips per lineal foot on each side of the arch. The 48-inch diameter beam tube will be supported on steel frames spaced at 65 feet on centers and attached to a 19-foot- wide reinforced concrete slab. The distributed loads on the slab will be 2,500 psf. The maximum differential subgrade settlement after initial laser alignment can not exceed 1/4-inch over a 65-foot span, nor exceed $\pm$1-inch over each of the 2.5 mile arms. These settlements are only for that portion of the berm immediately beneath the beam tube.

3.4 Service Road

A road to service the beam tube and its peripherals will be constructed over the berm to carry low volume light traffic.

4.0 FIELD INVESTIGATION

The field geotechnical investigation included the drilling of twenty (20) conventional geotechnical borings, and performing forty nine (49) CPT soundings. The locations of borings and CPTs are shown in Figure A-2 of Appendix A. During the performance of borings the soil cores were visually classified and recorded on boring logs. Samples from these cores were preserved and transported to the geotechnical laboratory for testing of selected samples to determine their engineering properties. Upon completion of the laboratory testing the logs of borings were revised to more accurately reflect the results of the laboratory tests. These are included in Appendix A. Borings were tremie-grouted full depth with bentonite mix upon completion in accordance with the drilling permit.

The attached boring logs indicate the types of soils and strata encountered. Relatively undisturbed 3-inch diameter tube samples were generally obtained in the cohesive, fine grained soils and disturbed 2-inch-diameter split-spoon samples were obtained in the coarse grained soils. Standard Penetration Tests (SPT) were performed on the split-spoon samples. This test consists of dropping a 140-pound hammer 30 inches and recording the number of blows required to drive the sampler. The number of blows on the final 12 inches is recorded on the boring log under the "SPT" column. The depths at which the driven and pushed split-spoon samples were obtained are indicated as cross-hatched square symbols and as a "V" symbol in the "Sample" column on the boring logs respectively. The depth between which the tube samples were obtained are shown as shaded symbols under the "Sample" column of the boring log.

Forty nine (49) CPTs were performed to further identify the in situ properties of these soils and to assess the variability or uniformity of engineering properties of various soil strata of the area investigated. The results of these soundings are shown in Appendix B. Also included in the same appendix are the results of three (3) Seismic Cone (piezocone) Penetrometer (SCP) tests and one crosshole seismic test performed to determine the dynamic properties of the subgrade.

4.1 Groundwater Conditions

The groundwater information was developed during the geotechnical sampling. In addition, temporary polyvinylchloride (PVC) pipes with screens and removable caps were installed in two (2) of the boreholes for overnight water observations. These are discussed later in this report. They were later removed and the holes were grouted.

5.0 LABORATORY TESTING

Selected samples obtained from the conventional geotechnical borings were tested in the geotechnical laboratory to assess the physical properties of the subsoil. Strength tests consisted of sixteen (16) unconfined compression tests, and fifteen (15) undrained triaxial compression tests. The results of these tests are shown on Appendix A. The compressibility of the soils were determined by performing twelve (12) consolidation tests. Detailed and summary results are shown in Appendix A. Appropriate columns of the boring logs also contain the results of laboratory tests. Eighty four (84) natural moisture content and thirty two (32) density tests were performed. In addition forty nine (49) Atterberg Limit determinations and twelve (12) percent finer than the No.200 sieve and two (2) grain size analyses were made.

Three composite samples representing the top strata from 6 to 15 feet below the existing ground surface were prepared and submitted to other laboratories for testing. These samples were tested for corrosion potential, including pH, sulfates, sulfides, and chloride contents as well as resistivity. The summaries and copies of laboratory reports are included in Appendix E.

6.0 LIMITATIONS

Professional judgments and recommendations are presented in this report. They are based partly on evaluations of technical information gathered, partly on historical reports and partly on our general experience with subsurface conditions in the area. We do not guarantee the performance of the project in any respect other than that our engineering work and the judgment rendered meet the standards and care of our profession. If during construction soil conditions are encountered that vary from those discussed in this report or historical reports or if design loads and/or configurations change, Woodward-Clyde Consultants should be notified immediately in order that they may evaluate effects, if any, on foundation performance. It should be noted that the borings may not represent potentially unfavorable subsurface conditions between borings. If such conditions become evident, additional borings should be performed to characterize these conditions for design review. The recommendations presented in this report are applicable only to this specific site. This data should not be used for other purposes.

Included in Appendix A is a document entitled "Important Information About Your Geotechnical Engineering Report", which is published by ASFE, The Association of Engineering Firms Practicing In The Geosciences. This document should be considered as part of this report and should be furnished to all persons who receive part or all of the report.

7.0 SITE CONDITIONS AND GEOLOGY

The topography of the Livingston LIGO site, in general is featureless, and flat which results in parts of the site being poorly drained.

The site was originally used for tree farming with the majority of the remaining stands being composed of young Southern Pines with hardwood stands clustered in the lower and worse drained areas.

The site is located in the Coastal Plain Physiographic Province, which is an elevated sea bottom about two hundred miles wide following the shores of the Gulf of Mexico and extending North along the Atlantic coast to Cape Cod. In Louisiana the Coastal Plain is divided into a series of terraces which crudely follow the Gulf Of Mexico Shoreline. These terraces form low elevation "uplands" relative to the Mississippi River alluvial planes and coastal marshlands.

The alluvium at the Livingston site is estimated to be about 200 feet thick. The geologic conditions are sequential, and as seen from the geotechnical investigations show minor spatial variations.

The surface outcrops throughout the site are composed of; clays, silty clays, silts and sands. These deposits comprise the Prairie Terrace Formation of the Pleistocene Series which was deposited about 100,000 years ago. The beginning of the Pleistocene Series was approximately one million years ago. A thin veneer of Holocene Alluvial deposit (reworked Pleistocene) overlies the Prairie Terrace in the small creeks and branches. Due to the extensive timber logging operations in the area the top 1 to 5 feet of the Terrace Formation has been disturbed. Within the Prairie Terrace there are two prominent sand channel deposits. One is located at the apex of the LIGO and the other at the end of the southwest leg.

The remainder of the site consists primarily of clay and silty clay deposits interbeded with clayey silts and clayey sands with thin sand layers. Below the Prairie Terrace are deposits of the Intermediate and High Terrace Formations, forming the mid and basal Pleistocene and upper Pliocene Series. Below these deposits is the upper Miocene which occurs at approximately 2,100 to 2,500 feet below the existing ground elevation.

7.1 Seismicity

The State of Louisiana is located in a Seismic Risk Zone 1 (Appendix C, Figure C-1) (Algermissen 1969), with ground accelerations of less than 0.1 g and a Seismic Zone Factor "Z" of 0.075. The site coefficient for this location is S2, and the "S" factor is 1.2 (Uniform Building Code 1994).

Recent seismic history of the state (Appendix C, Table C-1) shows that there have been minor tremors reported near Baton Rouge and Donaldsonville. The Baton Rouge events of 1905, 1957 and 1958 have been reported to have had Modified Mercalli (MM) intensities of V (Newman 1954).

The Donaldsonville event of 1930 was reported to have a MM intensity of VI. The hypocenter of this event is located about 50 miles from the LIGO site.

It has been predicted that a repeat of the New Madrid seismic event of 1811-1812 will affect the area at an MM intensity of V to VI.

7.2 Faults

There are no surface or near surface faults within the site that indicate topographic evidence of displacement (Appendix C, Figure C-2). The deep faults below the site are in the Tuscaloosa Trend Oil and Gas Production Zone and are approximately 15,000 to 20,000 feet deep. The faults shown on the map were transposed from the references cited in Appendix C, Table C-2 and C-3, and their locations are approximate. The surface and near surface faults (Scotlandville-Denham Springs and the Baton Rouge Fault) were positioned based on observed structural damage or distress and prominent topographic escarpments, closely spaced contour lines, abrupt changes in direction of drainage features and geologic interpretation between known fault points. The closest of the surface faults to the site is the Scotlandville-Denham Springs Fault which is located about 5.5 to 8 miles south-southwest of the site. The two faults are part of the Tepetate-Baton Rouge Fault Zone which runs from Southwestern to Southeastern Louisiana. The Bancroft Fault Zone, which is parallel to and north of the Tepetate-Baton Rouge Fault Zone is not known to extend as far as the LIGO site.

The surface and near surface faults have shown some movement in recent historical times. At the present time this movement has been attributed to groundwater withdrawal rather than tectonic causes.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 Soil Types

A review of CPT and geotechnical boring and test results shows good agreement between two sets of data. Soil strata in general appears to show only gradual variations from one boring or CPT location to the other with some outcrops of sands/silts or clays showing between boreholes (Appendix D). CPT results indicate the presence of thin silt or sand layers in clays and clayey deposits. Some such thin layers could not be seen in conventionally obtained soil cores because of smearing of the surfaces inherent to Shelby tube type sampling procedure.

The top 2 to 5 feet of the soils are primarily composed of silty clays and some sandy clays. The consistencies of the top strata vary from very stiff to very soft, depending on the soils relative elevation, drainage, and disturbance caused by timber harvesting operations. The surficial soils are underlain by medium to very stiff silty clays and clays interspersed with dense to very dense sand layers followed by medium to very stiff clays. The unified classification of the soils show the site deposits to be composed of CL-ML, CL and CH type soils. No discernible deposits of organic soils with the exception of the thin veneer of top soil, were encountered at this site.

8.1.2 Soil Strength

Unconfined compression and triaxial test results show the soil strengths ranging from 1,900 pounds per square foot (psf) to 5,500 psf with two slickensided or jointed specimen producing lows of 750 psf and 850 psf.

8.1.3 Consolidation

Results of consolidation tests indicate soils at this site to be preconsolidated. The preconsolidation stresses vary from about 2 tsf to about 3 tsf.

The pleistocene clays of this area are generally slickensided and fissured due to desiccation during recent geologic times and due to deposition patterns. Some of the shallower pleistocene deposits are interlaced with thin (about 1/8-1/4 inch) lenses of silt, fine sand or ferrous oxide deposits. These lenses generally are not continuous and they do not contribute to the dissipation of excess hydrostatic pressures; thus, they do not significantly affect the consolidation process. However, they are responsible in developing fissures which may result in the exhibition of low strengths during some unconfined compressive strength tests.

Settlement calculations made for the worst case scenarios show that at the end stations the maximum expected primary consolidation near the center of the building site (1,000 x 900 feet) for embankment and building loads (this assumes building loads to be supported on a slab founded on the embankment) will be about 3 3/4 inches. At other locations where single story buildings founded on slabs supported on the embankment may be built, settlement will vary from 1 to 2 inches.

It should be noted that the excavation of borrow pits next to the embankement will change the groundwater regime. It is estimated that an additional 3/4- to 1 1/4-inch settlement will take place during 8 to 10 years following the construction. The magnitude and the period of the settlement caused by the latter will depend on the magnitude of the loads, depth of the original, and after borrow excavation, groundwater elevations and soil types at various locations along the embankment. It is expected that 90 percent of the primary consolidation will take place either during the construction or during the 6 months following the construction. Secondary settlement of these soils are negligible.

No subsidence or bearing capacity values for the embankment have been presented. They will depend on the properties of the compacted, or stabilized soils in the embankment. Uniform loads to be exerted along the top of the embankment by the beam tube assembly (tube, cover, slab) are not expected to cause subsidence within the embankment provided the embankment is properly compacted and stabilized. However, if designers choose to support the beam tube on individual footings placed at some intervals along the tube settlement and the bearing capacity need to be determined based on loads, footing geometry, and actual embankment material properties.

8.2 Groundwater

Groundwater, was encountered at varying depths at different locations; while, in general, the groundwater was encountered at an average depth below the existing ground of about 8 feet. At some locations the groundwater was either not encountered or encountered at 13 or 25 feet below. Some of the shallow "groundwaters" appear to be perched waters which is common to this area. These shallower "groundwaters" appeared to be under slight artesian pressure which may be the result of the tilt of silty and sandy water bearing layers.

It should be noted that the groundwater elevations of the area is largely dependent on precipitation and will fluctuate with seasons. They should be verified prior to initiating any construction operations, such as excavations, which it may affect.

8.3 Dynamic Properties

Three Seismic Cone Penetrometer (SCP) and one Crosshole test were performed to determine shear wave velocities of the natural deposits (Appendix B). The Crosshole test was performed to provide local verification of the SCP data. Test results show good agreement. There appears to be relative uniformity in the dynamic properties of soils at three different locations tested. Data also show that the shear wave velocities of in-situ deposits are confined within a range of 550 fps to 850 fps (Plates 5, 6, and 7, Appendix B).

It should be noted that in the case where shallower layers have higher wave velocities than lower layers, seismic test results will not give reliable indication of the layering (See SCP test results B-SW-01-SC and B-SW-35-SC). Also in homogeneous layers the presence of the water table will reflect wave patterns falsely indicating them to be layered; however, these tests can accurately predict the depth of shallow water tables.

8.4 Resistivity and Chemical Analysis

The pH, resistivity and sulfate, sulfide and chloride contents of the soils were determined using three composite soil specimens representing typical soils of surface deposits from 6 to 20 feet below the ground elevation (Appendix E, Table E-1).

Soils having similar engineering properties were composited and submitted for testing to Soil Testing Engineers, Inc. and to Benchmark Laboratories both of Baton Rouge, Louisiana. Test results are included in Appendix E.

Resistivities of the soils at their natural moisture content indicate that they are "virtually non-aggressive" as far as their corrosion potential is concerned (Table E-2). A brief description of the resistivity test method used by the laboratory is also included in

Appendix E. The pH tests of the composite specimens indicate the soil to be, practically, neutral.

8.5 Stress-Strain Properties of Soils

Stress-strain moduli E_s, Poisson's Ratio μ, and moduli of subgrade reactions k_s, for various types of soils found at this site and the dynamic modulus G' are shown below. These values were obtained from a review of the stress-strain properties of the materials as exhibited in triaxial testing and by correlating the site soil characteristics with those shown in the literature and local experience. The Dynamic modulus G' was computed using a shear wave velocity of 700 fps.

It should be noted that these elastic and dynamic properties are highly dependent on soil composition (i.e., ratio of silt to clay), density, moisture content at the time of testing, and stress-strain history of the deposits and testing methods. In other words, there are built-in uncertainties in these values.

The soils found in this area can be grouped into four categories;

Medium Clays	E_s=500-1,000 ksf
Stiff Clays	E_s=1, 000-2,000 ksf
Sandy/Silty Clays	E_s=500-3,000 ksf
Dense Sand	E_s=800-1,500 ksf

Typical values for Poisson's Ratio μ are given below:

Saturated Clays	0.3-0.5
Clays above the Water Table	0.2-0.3
Sandy/Silty Clays	0.2-0.3

We recommend the use of a Poisson's Ratio of 0.4 for computations involving the in-situ soils at this site.

Typical Moduli of Subgrade reactions k_s for the existing deposits located 4 to 10 feet (assumes that 2 to 5 feet of loose surface materials are removed) below the existing ground elevation are shown below.

Clays and Silty/Sandy Clays	50-100 kcf
Silty Sands	100-225 kcf
Clayey Sands	175-350 kcf

Dynamic modulus, M', for the tested locations is 14 ksi.

8.6 Hydraulic Conductivity and Leach Fields

Hydraulic conductivities of natural deposits of this area vary both with the type of soil and with the nature and extent of desiccation and rootlet fissures. The hydraulic conductivity of the soils with no fissures on this site are shown below:

Stiff clays and silty clays:	$1x10^{-6}$ cm/sec to $1x10^{-7}$ cm/sec
Medium silty and sandy clays:	$1x10^{-6}$ cm/sec
Silts and clayey silts:	$1x10^{-5}$ cm/sec to $1x10^{-6}$ cm/sec
Sands:	$1x10^{-4}$ cm/sec

Our earlier experience in these soils have shown that even though the hydraulic conductivity of the natural soil may be very low the in-situ conductivity of the massive soil structure varies by several orders of magnitude because of the presence of fissures. It should also be mentioned that the State of Louisiana's Department of Environmental Quality does not allow the installation of leach fields.

9.0 FOUNDATIONS

9.1 Engineering Analysis

Prior to performing various engineering analysis the results of conventional geotechnical tests and the CPT results were compared and correlated. Where needed, additional geotechnical laboratory tests were performed to verify earlier results.

Analyses were performed to determine stress distribution at various depths under the embankment. The results from the analyses were compared with unconfined compressive strength of soils at various elevations under the proposed embankment site. Results of Atterberg limits and natural moisture content tests for strata under the water table were used for preliminary determination of the compressibility of various soils. Consolidation test results were used to determine settlement properties of the subsoils. Settlement analyses were performed for worst case soil conditions for embankment and beam tube loads.

Pile capacity and bearing capacity analyses were also performed.

Pile capacities of subsoils at the apex were developed.

Drilled shaft capacities at the apex and at the tip of the southeast arm were computed. Slope stability analyses using Bishop method was performed for 10-foot embankment height with the beam tube load imposed upon it. The subgrade was assumed to have a cohesion of 750 psf from the ground surface to a depth of 18 feet. The adjacent borrow ditch was assumed to be 5 feet deep with embankment slopes of 1:2. Computations were also done using a 100 psf surcharge and an earthquake loading factor of 0.1, in both vertical and horizontal directions. The minimum cohesion of the embankment material was assumed to be 1,000 psf and the water table to be at the ground surface. The angle of internal friction for all soils were assumed to be 0 degrees.

In addition the Atterberg limits were used to determine the soil types, soil workability for embankment and other construction, and the suitability of soils for lime or portland cement stabilization.

10.0 RECOMMENDATIONS

10.1 Beam Tube Embankments

10.1.1 Loads

The beam tube for the LIGO facility and its associated equipment and the maintenance access road will be constructed over two connected embankments. At the time of the preparation of this report the final elevation of the crest of the embankment, exact position of loads to be imposed upon the embankment vis-a-vis the embankment center line, or exact dimensions were not available. Thus this and following sections of the report are based on information which accompanied the California Institute of Technology contract for this work as discussed earlier in this report.

We have used values derived from the above source for our computations. Engineers performing the final design need to critically review these values and make proper compensations for changes in design parameters. Two embankment heights have been considered; 5 and 10 feet.

10.1.2 Unit Weights

The average unit wet weight of all specimens is 124 pcf with an average unit dry weight of 100 pcf. For computational purposes we recommend using 120 pcf for wet unit weight.

10.1.3 Settlement

Our analysis indicates that the foundation materials at this site are adequate to support the embankments and the loads as outlined above. It is expected that there will be 3/4 to 1 1/4 inches of uniform settlement due to the embankment loads. The primary consolidation is expected to be completed during the construction of the embankments and during the nine month period following the completion of the construction. Ninety percent of the settlement is expected to be completed within six months. Present LIGO plans call for the embankment to be allowed to settle and be dimensionally stabilized for a period of nine months. At the end of this period the differential settlement of the embankment foundation is expected to be within the required tolerances.

The period required for the dimensional stabilization of the embankment can be greatly reduced by the use of a surcharge load to accelerate the subsidence and consolidation of the embankment and its foundation by using a surcharge equivalent to the 125 percent to 150 percent of the loads to be imposed. If this alternate is selected it is recommended that the surcharge be left in place for a minimum period of ninety days.

In either case it is recommended that the progress of the consolidation be monitored with the use of settlement plates installed within the embankment and on the natural grade, as well as with the use of piezometers installed in the foundation soils. The piezometers are to be used for monitoring the dissipation of excess hydrostatic pressures thus the progress of primary consolidation. It is recommended that piezometers be installed every 2,500 feet, along the centerline of the embankment and at the corner and end station locations. Settlement plates should be located near the piezometer installations.

10.1.4 Surcharging

In general the foundation soils are suitable to carry the embankment loads without preloading or surcharging as discussed above. Soils in the vicinity of boring Nos. SW-21-GT and SW-25-GT produced lower strengths due to apparent slickensided and jointed structure of the deposits about 15 feet below the ground elevation. It is recommended that if the embankment in its entirety is not surcharged this isolated area be surcharged with a load equivalent 1.5 times the total loads to be imposed at this location to avoid shear displacements.

10.1.5 Stability Analysis

The Bishop's method for stability analysis using worst soil conditions yielded safety factors of 2.3 for the condition of groundwater at the ground elevation, and 2.0 for saturated, flooded condition.

10.1.6 Site Preparation

It is recommended that about 2 to 5 feet of the top loose and disturbed material be removed and replaced with compacted engineered fill material. The amount of material to be removed should be determined in the field after the site is graded and proof rolled with fully loaded (50T) rubber tire (wobble wheel) roller(s) with minimum tire pressures of 100 psi to identify the location and the extent of soft and disturbed layers. The footprint of the LIGO embankment covers an area where a large number of trees were removed. Prior to the proof rolling the entire site should be inspected to assure that no stumps remain.

Based on soil borings performed for this report, borrow materials that may be obtained from within the LIGO ROW, in general, appear to be suitable for embankment construction. It should be noted, however, that fine sands and silts of this area generally are composed of well rounded particles and, in their pure form (i.e., all fine sand or all silt), they present stability problems unless they are mixed with finer materials or stabilized with portland cement.

Soils with plasticity indexes of 12 to 25 with liquid limit not exceeding 40 percent are suitable for the construction of these embankments. Soils with higher plasticities should be modified by mixing them with quick or hydrated lime. Soils with lower plasticities should be stabilized with either lime, portland cement, or a combination of Type C fly ash and lime. Modification of high plasticity soils should be performed to render them friable and workable. Stabilization is recommended for low plasticity soils to develop cementation and dimensional stability.

All backfill and embankment materials (with the exception of reinforced earth), if not stabilized with portland cement, should be compacted to obtain a minimum of 96% relative density as determined by the "Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort", ASTM D1557.

As indicated by the soil profiles of the site, the type of natural materials vary spatially and vertically. When on site borrow materials are used for the construction of the embankment it will be necessary to develop a family of moisture/density curves for use by trained soil technicians to direct and monitor the compaction of embankment materials.

An alternate approach to using embankment materials with varying compaction properties is to require the construction contractor to blend various borrow materials into uniform mixtures prior to placing them in the embankment or as backfill. This is an option however it will greatly increase the cost of earthwork. In cases where the borrow areas contain soils with plasticity indices of 20 or higher blending will be time consuming and uniformity may not be achieved.

It is recommended that on-site or off-site borrow materials be characterized prior to their use and, if needed, a family of curves be developed for use in the field. It should also be noted that the method of excavation of borrow materials will determine the final composition of the material placed in the embankment. Some spot blending as directed by the field engineer, may be necessary. The blending of large volumes of soil and stockpiling is not recommended.

For estimating purposes the use of compaction factor of 1.18 or 18% is recommended for borrow materials on this site. Actual compaction factor should be determined following the identification of borrow soils.

The liquid limits of tested soils are, in general, less than 50 indicating that the swelling potential is negligible. All borrow materials should be tested at their source for characterization. Suitability of borrow materials for this embankment should be determined prior the approval of the borrow source.

Embankment slopes should be blanketed with a minimum of 8 inches of clayey soil compacted to 95 percent of the maximum relative density as determined by the "Test Method of Laboratory Compaction Characteristics of Soils Using Standard Effort, ASTM D698". The clay blanket in turn should be covered with a minimum of 6 inches thick layer of top soil suitable to develop and sustain a protective cover of vegetation. No settlement or bearing capacity values for the embankment itself has been presented. They will depend on the properties of compacted, or stabilized soils of the embankment.

10.1.7 Service Road

A service road parallel to the beam tube is to be constructed over the embankment. This road is expected to carry light and low volume traffic. For light service use it is recommended that the pavement profile be composed of a minimum of 10 inches of portland cement stabilized soil and 3.5 inches of bituminous hot mix concrete. A rigid pavement structure consisting of a minimum of 6 inches of stabilized base and 10 inch thick portland cement concrete pavement may be substituted.

10.2 Beam Tube Foundation

The plans call for the beam tube to be founded on a concrete slab. It should be noted that if the concrete slab is expected to be subjected to extreme temperature changes on the surface the temperature gradients developed between the top and the bottom of the slab will induce thermal stresses which in turn may result in differential vertical movement of the slab along and across its centerline. During the summer heat, in this area, surface temperatures of concrete slabs may rise to 110 degrees Fahrenheit while the bottom of an 18-inch-thick slab remains at about 75 degrees fahrenheit.

In light of precise alignment requirements of the beam tube, if the support slab will be subjected to ambient temperature variations, it is recommended that consideration be given to either provide individual footings at support points or sufficient reinforcement be provided to overcome thermally induced movements of the slab.

10.3 Building Foundations

10.3.1 Bearing Capacity

The shear strengths of soils tested are, in general, in excess of 1.2 tsf. For these soils the bearing capacity is determined to be 3,450 psf (with a safety factor of 2.5). The bearing capacity of weaker soils were computed to be 2,680 psf. We recommend that a bearing capacity of 2,700 psf be used for general design purposes. Soils at the corner station of the facility yields a bearing capacity of 3,600 psf using a safety factor of 3.0.

10.4 End and Corner Stations

10.4.1 End Stations

It is recommended that the end stations be supported on foundations placed in the natural ground. The founding of settlement sensitive structures on the fill is not recommended. It is recommended that the end structures be supported on straight shafts founded 35 to 40 feet below the grade existing at the time the borings were performed (see Table 1).

The shafts should only be installed after the completion of the construction and subsidence of the embankment as discussed above. It should be noted that shaft installation deeper than 40 feet is not recommended for the Southwest end station.

TABLE 1

STRAIGHT-SIDED SHAFT CAPACITIES AT END STATIONS OF THE ARMS

Depth of Tip Embedment (feet)*	Allowable Single Shaft Compression Capacities (kips) Shaft Diameters (inches)				
	12	18	24	30	36
15	23	36	51	67	85
20	29	46	64	83	103
25	NR	58	79	102	126
30	NR	69	95	121	149
35	NR	71	110	141	172
40	NR	92	126	160	195
45	NR	104	181	179	218
50	NR	115	156	198	241

NOTES:

Shafts are not recommended below 40 feet
* Feet below existing grade
NR Not Recommended

10.4.2 Corner Station

It is recommended that the corner station be supported on foundations consisting of straight shafts or pile foundations installed 45 to 50 feet below the ground elevation existing at the time the soil borings were performed. Straight shaft capacities for the corner stations are shown in Table 2 below. It should be emphasized that the shaft or pile installation should be done after the completion of the placement and subsidence of the embankment is completed. The founding of the corner station on the compacted embankment is not recommended.

The shop and office portions of the corner station can be founded on the embankment as long as the foundations of the vacuum chamber building and the office/shop buildings are structurally isolated.

TABLE 2

STRAIGHT-SIDED SHAFT CAPACITIES AT CORNER STATION

Depth of Tip Embedment (feet)*	Allowable Single Shaft Compression Capacities (kips) Shaft Diameters (inches)				
	12	18	24	30	36
15	8	12	17	23	29
20	15	23	33	44	56
25	NR	33	47	61	77
30	NR	45	63	81	101
35	NR	56	78	100	123
40	NR	67	92	118	145
45	NR	83	113	145	179
50	NR	97	132	169	207

NOTES:

* Feet below existing grade
NR Not Recommended

10.4.3 Drilled Shafts

Drilled shaft capacities have been computed for the soils at the corner station and at the end stations. These capacities are listed in Tables 1 and 2. As can be seen, drilled shafts will provide high load capacities and they are recommended for use at the LIGO site. The shaft capacities as presented assume a cut-off of 2 feet below present grade. They will carry loads by both side friction and end bearing. We recommend that compression values be reduced by one half for shafts in tension. Reinforcing steel for shafts subject to uplift pressure should extend to within 6 inches of the bottom of the shaft.

Consideration should be given to the group effect of shafts installed in clusters of 4 or more. Shafts in clusters should be installed with a minimum center-to-center spacing of no less than 2 shaft diameters. Experience has shown that the group effect of large clusters of shafts is best accounted for in such materials through use of the "Perimeter Shear" formula. This formula assumes that the material enclosed within the shaft cluster tends to act as a large block and the forces resisting the movement of this block are compared with the total load on the block. If a safety factor of 2.5 or greater is obtained with this formula, no reduction in shaft capacity is necessary. Otherwise, the allowable foundation load is taken as 40 percent of the total supporting power of the block. The Perimeter Shear formula may be written as:

$$Q = P\left(\sum c_i L_i\right) + Aq$$

where:

Q = Ultimate supporting capacity of the soil block (kips)
P = Perimeter of shaft group (feet)
c_i = Cohesion of soil layer (i)(kips/square feet)
L_i = Length of shaft embedded in soil layer (i)(feet)
A = Horizontal area of shaft (square feet)
q = Bearing Capacity
i = Number of soil layers

For uplift loads the above equation should be used except that the second term (end bearing part) is not utilized.

The contractor installing shafts has to be made aware of the fact that the soil profile shows the presence of dense sand layers. Drilling of shafts through these layers may require the use of casing or bentonite slurry. We recommend that any casing used should penetrate at least 1 foot into the very stiff to hard clay layers to seal seepage and avoid sloughing of silts and sands. Concrete should be placed immediately after the excavation has been completed and inspected. In no event the excavation should remain open more than three hours. Should there be seepage in excess of one inch, the hole should be pumped dry and the concrete properly tremied in place. Casing should not be pulled above the concrete surface during the placement of concrete.

Shaft capacities for the tip of the southwest arm were not computed since the friction values for those soils are higher thus bearing capacities would be slightly higher than those for the southeast arm. We recommend the use of values computed for the southeast arm.

10.4.4 Pile Types

The use of timber piles are not recommended at this site. Driving of timber piles to full depth to develop frictional resistance will most probably destroy the piles since they have to penetrate stiff to very stiff clays and dense sands.

If pile foundations are selected we recommend the use of steel pipe or precast concrete piles. Considering the fact that the soil profile shows variations in type and properties we also recommend that a test pile program be developed to determine field capacities.

TABLE 3 PILE CAPACITIES AT END STATIONS							
Depth of Tip Embedment (feet)	Allowable Single Pile Capacities (kips)						
	Square Precast Concrete (inches)			Steel Pipe Diameter (inches)			
	12	14	16	10	12	14	16
30	38	45	52	24	29	35	41
35	45	54	62	29	35	42	48
40	53	62	71	33	41	49	56
45	63	75	86	40	49	58	68
50	71	83	96	45	55	65	76

NOTES:

* Feet below existing grade

10.4.4.1 Pile Foundation Settlements.

Analysis of pile foundation settlement of proposed structures is dependent on the column loads as well as the size (Table 3) and configuration of the pile groups. Since specific configurations or criteria are not available at this stage a detailed settlement analysis has not been performed. For piles driven in single rows widely spaced or used in small groups where the width of the pile cap is small relative to the pile length, the settlement of piles driven to 40 to 45 feet are estimated to be 1/4 to 1/2 inch. These movements are in addition of elastic shortening of piles, which depend on the type of the pile, actual applied load and the distribution of loads along the length of the pile.

10.4.4.2 Uplift. For piles subject to uplift forces, it is recommended that maximum tensile capacities not be greater than 30 percent of the maximum recommended single pile compression capacities.

10.4.4.3 Downdrag. Downdrag on piles or shafts at the corner station are estimated to be 1 tsf. However if the piles are installed after the completion of the embankment placement and subsidence, as recommended above, negative skin friction on piles or shafts should be minimal.

10.4.4.4 __Temporary Loads.__ The maximum recommended compression values may be increased by 30 percent for temporary wind loads.

10.4.4.5 __Group Efficiency.__ No reductions from the single pile capacities appear necessary for the effects of group action in the case of clusters of piles driven to tip embedments in the very stiff to hard clays.

Driving to grade through the dense sands may be difficult. The installation of piles through these soils may be accomplished by the use of pilot holes. Pilot holes should be no larger than 2 inches less than the outside diameter of the pile and no deeper than 5 feet less than the tip embedment. Care should be taken to avoid overwashing the holes. Piles may reach refusal in the very stiff to hard clays of the before reaching an embedment. Refusal driving resistances for steel and concrete piles may be estimated using the ENR formula. Higher blow counts may damage the piles. It is normally recommended that the maximum number of blows not exceed 100 and 75 blows per foot respectively for open-ended steel pipe and square precast concrete piles. However higher blow counts are sometimes allowed in order to get the piles to grade. This requires close inspection to reduce the chance of structurally damaging the piles.

10.4.4.6 __Heave.__ When groups of piles are driven through stiff clays such as are found here, some of the previously driven piles can heave or be displaced due to driving later adjacent piles. The piles should be driven from the center outward. It is recommended that the butt elevations of each pile be determined immediately after it is driven and again when the group is completed. If any pile is noted to have heaved more than 1/4-inch, it should be redriven to at least its original final resistance.

10.4.4.7 __Inspection.__ All pile driving operations should be inspected by a qualified geotechnical inspector and records of driving resistance versus depth, tip elevation, driving equipment, etc. should be permanently kept. The inspection services would determine when the desired embedments are attained and prevent overdriving to avoid structural damage of the piles.

10.4.4.8 __Spacing.__ It is recommended that the piles be driven on minimum center-to-center spacings of 3 pile diameters or 5 percent of pile length, whichever is greater.

10.4.4.9 **Driving.** It is recommended that the steel pipe piles and prestressed concrete piles be driven with a hammer that develops a minimum manufacturer's rated energy of 19,500 foot-pounds per blow suspended from fixed leads.

The above is a summary of the geotechnical investigation authorized by the California Institute of Technology for the LIGO facility and it is prepared to satisfy the requirements of our contract with the Institute.