GEOTECHNICAL REPORT
MERCY WELLNESS CENTER
REDDING, CALIFORNIA

Prepared For:

Dignity Health

CGI TECHNICAL SERVICES INC.

Dignity Health

CGI TECHNICAL SERVICES INC.

April 20, 2016
CGI: 15-1080.84

Jennifer Tanner
DIGNITY HEALTH – CRE
3400 Data Drive
Rancho Cordova, CA 95670

Subject: Geotechnical Report
Mercy Wellness Campus
City of Redding, California

Dear Ms. Tanner:

CGI Technical Services, Inc. (CGI), is pleased to submit this geotechnical report for the proposed Mercy Wellness Center in Redding, California. This report presents our findings, conclusions, and recommendations for design of the proposed development.

We appreciate the opportunity to perform this study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact Jim Bianchin at (530) 244-6277 at your earliest convenience.

Regards,

CGI TECHNICAL SERVICES, INC.

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1 GENERAL

This report presents the results of our geotechnical study for the proposed medical campus, located in Redding, California. CGI Technical Services, Inc. (CGI), has prepared this report for Dignity Health at the request of Nichols Melburg Rossetto (NMR), the project architect. The project location is shown on Plate 1 – Site Location Map. The following sections present our understanding of the project, the purpose of our study, and the findings, conclusions, and recommendations of this study. Our services were performed in general compliance with our proposal dated May 27, 2015.

1.1 PROJECT LOCATION

The project site is located at 855 Cypress Avenue (APNs 107-400-008; 107-500-16, -17, -18, -19, -020, -024, -025, -026; and 107-430-033, -034, -057, -059) in the City of Redding, Shasta County, California, as shown on Plate 1. Latitude and longitude for the approximate center of the property are as follows:

- Latitude: 40° 34’ 12.8” (40.570211°)
- Longitude: -122° 22’ 0.2” (-122.366719°)

1.2 PROJECT UNDERSTANDING

The project, as we understand it, consists of the design of a new medical campus located south of Cypress Avenue, west of Hartnell Avenue, and situated east of the Sacramento River. Based on a preliminary site map prepared by NMR, we understand that the proposed campus will contain three new structures and one relocated structure. According to NMR (2016), those improvements consist of:

- Building A, which is 5-story, encompassing about 100,000 ft^2. Finish floor elevation for Building A is 477.0 feet above MSL;
- Building B, which is 2- to 3-story encompassing 20,000 to 30,000 ft^2. Finish floor elevation at this site is 482.0 feet above Mean Sea Level;
- Building C, which is 2- to 3-story encompassing 20,000 to 30,000 ft^2. Finish floor elevation of this structure is 487.0 feet above MSL; and
- The relocated Dubrowsky House, moved from the proposed new Shasta County courthouse site.

Associated with the proposed structures will be design of new access roads, roundabouts, a trail, and 609 new parking spaces.

It is anticipated that the proposed structures will be founded on shallow foundations, for the relatively smaller structures and possibly deep foundations for the proposed 5-story structure. Foundation loads are unknown at this time.

1.3 STUDY PURPOSE

The purpose of our geotechnical study was to explore and evaluate selected site surface and subsurface conditions in order to provide geotechnical engineering recommendations related to the design and construction of the project, and to identify potential geologic hazards that could impact the project. The subsurface characterization was primarily intended to estimate the depth, profile, consistency, strength, and grain-size distribution of the soils that might be encountered during project construction, along with the general depth to groundwater.

1.4 PREVIOUS WORK PERFORMED

We know of no previous geotechnical studies that have been performed at the project site. Regional geologic and groundwater information was referred to during preparation of this report. Those regional data are cited within the report text and within references section of this report.

Our review and research of the site found no existing geotechnical work performed for the site. However, we obtained and reviewed the geotechnical study that was performed for the adjacent Cypress Avenue Bridge Replacement project (Kleinfelder, 2006).

1.5 SCOPE OF SERVICES

Services performed for this study are in general conformance with the proposed scope of services presented in our May 27, 2015 proposal. Our scope of services included:

- Reconnaissance of the site surface conditions, topography, and existing drainage features;
- Attempted acquisition of existing, available geotechnical data for the project site;
- Review of pertinent, selected regional geological data;
- Exploration of the subsurface conditions within the project site using exploratory drill holes. Exploration locations are shown on Plate 3 – Geotechnical Map. Exploration procedures and exploration logs are presented in Appendix A – Subsurface Exploration;
- Performance of laboratory testing on selected samples obtained during our field investigation. Laboratory test procedures and results of those tests are presented in Appendix B – Laboratory Testing;
- Preparation of this report, which includes:
    - A description of the proposed project;
    - A summary of our field exploration and laboratory testing programs;
    - A description of site surface and subsurface conditions encountered during our field investigation;
    - A description of ground shaking conditions expected at the site, including CBC seismic design criteria;
    - Recommendations for:
        - Site preparation, engineered fill, site drainage, and subgrades;
        - Suitability of on-site materials for use as engineered fill;
        - 2013 CBC seismic design criteria;

    - Concrete slabs on-grade;
    - Temporary excavations, shoring, and trench backfill;
    - Allowable bearing capacities and class of soil type for deep and shallow foundation design and construction;
    - Cement type based on soil chemistry
    - Retaining walls; and
    - Pavement design.

- Appendices that present a summary of our field investigation procedures and laboratory testing programs.

2 FINDINGS

2.1 FIELD INVESTIGATION

CGI conducted a geotechnical field investigation to evaluate subsurface soil conditions, and to provide subsurface data for evaluation of the proposed development. Our field geotechnical investigation was limited to reconnaissance-level geologic mapping of the project site and subsurface exploration through excavation of five drill holes. The drill holes, designated DH-1 through DH-5, were advanced between September 8 and 11, 2013. Drill hole locations are shown on Plate 2. Detailed descriptions of soils encountered are presented on the drill hole logs included in Appendix A. The soils encountered within the excavations were logged in general accordance with the Unified Soil Classification System (USCS). Surficial and subsurface soil samples were collected and transported to our laboratory for testing. Laboratory test results are included with this report.

2.2 SITE CONDITIONS

2.2.1 Site History

The development area has had a variety of land uses that have modified grades and resulting in the placement of fill materials and subsurface disturbance. According to Enplan (2015), much of the southwesterly project area was utilized as a concrete batch plant from before 1943 through 1993. This development likely accounts for the concrete slabs, foundations and walls due west of the Cobblestone Shopping Center. In addition, a number of aggregate mining areas were present on site and visible in 1943 and 1955 aerial photographs. Most of those ponds are west of the proposed development area; however, one apparent pond looks to have been located north of and possibly beneath a portion of proposed Building A.

In addition to the batch plan, a number of structures have been constructed and demolished across the site (Enplan, 2015). The most recent of those structures was located west of and possibly beneath the proposed footprint for Building C. In addition, up until at least 1955, a roadway bisected the site and crossed the Sacramento River, the foundations for which are still visible at the river crossing.

2.2.2 Surface Conditions

The project is located on a relatively flat site with grade separations present at a number of locations on site. Those grade separations are retaining walls that are up to about 8 feet tall and were constructed as improvements for past land uses at the site. As such, a number of concrete walls, slabs, foundation and improvements, and asphaltic concrete paving are present throughout the project parcels. In addition, two structures are present on site that house Pet Care Naturally and Augie's Enterprise Mower. Those structures underlie the footprint of proposed Building A.

Aside from the grade separations, slope inclinations at the site range up to about 10 degrees except west of Henderson Road and the Cobblestone Shopping Center, where slopes are inclined as steep as about 26 degrees.

Drainage at the site occurs as sheetflow towards the west and the Sacramento River. The project elevations range from about 470 to 490 feet above mean sea level (MSL).

2.2.3 Subsurface Conditions

The project site is underlain by artificial fill and alluvial soils. Those soils are composed predominately of granular soils consisting of silty sand, silty gravel and gravel with varying amounts and sizes of cobbles and boulders. Near the northwest and central portions of the project, gravelly clay and sandy clay were encountered within the upper 12 to 14 feet of the soil profile.

Groundwater was observed initially at depths of approximately 10 to 22 feet; however, the method of drilling may have obstructed the direct observation of groundwater depths.

The logs in Appendix A present specific soil and rock descriptions encountered within each drill hole.

2.3 GEOLOGIC CONDITIONS

2.3.1 Regional Geology

The project site is located in the northern Sacramento Valley near the northern margin of the Great Valley Physiographic province. The Great Valley province is bordered to the north by the Klamath and Cascade Physiographic provinces, to the east by the Cascade and Sierra Nevada Physiographic provinces, to the west by the Klamath and Coast Ranges Physiographic provinces, and to the south by the Transverse Ranges Physiographic province.

The Great Valley Physiographic province is about 50 miles wide and 400 miles long. The Sacramento Valley, which forms the northern portion of the province, is about 150 miles long and 40 miles wide (Hinds, 1952). According to Hackel (1966), "The Great Valley is a large elongate northwest-trending asymmetric structural trough that has been filled with a tremendously thick sequence of sediments ranging from Jurassic to recent." Sediment thicknesses of up to 10 miles are reported within the Sacramento Valley; however, in the project area, being at the northern margin of the valley, those thicknesses have been projected to be less than one mile (Hackel, 1966). Sediments within the Great Valley consist of both marine and continental deposits, with most of the sediments underlying the project area consisting of continental deposits.

2.3.2 Local Geologic Setting

The site is underlain by artificial fill and alluvial sediments. Artificial fill materials are present locally across the project site, as shown on Plate 3. They are present behind retaining walls, in embankment areas, and in areas where former ponds were once present. The artificial fill materials range in thickness up to at least 12.5 feet (DH-2) and could be locally deeper.

The alluvial materials consist of granular soils located beneath where artificial fill materials are present. The full thickness of these alluvial deposits was not fully penetrated during our exploration and is at least 110 feet thick (Kleinfelder 2006).

2.3.3 Groundwater

Groundwater was encountered in three drill holes advanced for this study. Groundwater was observed initially at depths of approximately 10 to 22 feet; however, the method of drilling may have obstructed the direct observation of groundwater depths. The following table summarizes the approximate depth to groundwater for this study and Kleinfelder (2006).

SUMMARY OF GROUNDWATER INFORMATION				
Study	**Exploration No.**	**Date Measured**	**Depth to Water (ft)**	**Water Surface Elevation**
CGI – This Study	DH-2	9/9/15	22	459
	DH-4	9/10/15	10	465
	DH-5	9/11/15	14	461
Kleinfelder (2006)	D	11/21/03	25	464
	E	11/20/03	24	465
	F	11/24/03	27	463
	I	11/19/03	35	455
	J	11/20/03	34	456

Groundwater elevations will fluctuate over time. The depth to groundwater can vary throughout the year and from year to year. Intense and long duration precipitation, modification of topography, and cultural land uses, such as irrigation, water well usage, on site waste disposal systems, and water diversions can contribute to fluctuations in groundwater levels. Localized saturated conditions or perched groundwater conditions near the ground surface should be anticipated during and following periods of heavy precipitation and snowmelt. If groundwater is encountered during construction, it is the Contractor's responsibility to install mitigation measures for adverse impacts caused by groundwater encountered in excavations.

3 GEOLOGICAL HAZARDS

Geologic hazards are addressed, herein, in general accordance with the California Geological Survey Note 48, which applies to hospitals, schools, and critical facilities.

3.1 GEOLOGIC HAZARD ZONES

No mapped geologic hazards zones are known for the project region.

3.2 FAULTING & SEISMICITY

3.2.1 Seismic Setting

The State of California designates faults as active, potentially active, and inactive depending on the recency of movement that can be substantiated for a fault. Fault activity is rated as follows:

FAULT ACTIVITY RATINGS		
Fault Activity Rating	**Geologic Period of Last Rupture**	**Time Interval (Years)**
Active	Holocene	Within last 11,000 Years
Potentially Active	Quaternary	>11,000 to 1.6 Million Years
Inactive	Pre-Quaternary	Greater than 1.6 Million Years

The California Geologic Survey (CGS) evaluates the activity rating of a fault in fault evaluation reports (FER). FERs compile available geologic and seismologic data and evaluate if a fault should be zoned as active, potentially active, or inactive. If an FER evaluates a fault as active, then it is typically incorporated into a Special Studies Zone in accordance with the Alquist-Priolo Earthquake Hazards Act (AP). AP Special Studies Zones require site-specific evaluation of fault location and require a structure setback if the fault is found traversing a project site.

The site is not located within an Alquist-Priolo Earthquake Fault Zone and no active faults are known to pass through the project site (Jennings, 1994; Hart & Bryant, 1997). However, a number of regional and local faults traverse the project region. The most significant of these faults is the potentially active Battle Creek fault, located about 16 miles south of the site (Jennings, 1994). The closest fault mapped to the site is the inactive Bear Creek fault, located about 13 miles to the southwest (Jennings, 1994). The closest active fault, as zoned by the State, is the Hat Creek-McCarthur Fault System, located about 48 miles east of the site.

In addition to the continental faulting noted above, the project area rests above the Cascadia subduction zone. West of the site, off the coast of California, the oceanic crust of the Gorda plate is being subducted beneath the continental crust of the Pacific Plate, in an area known as the Gorda Escarpment. The descending ramp caused by that subduction, called the Cascadia Subduction zone, extends beneath the project area at a depth of about 20 to 25 miles. That ramp is capable of storing elastic stress that periodically causes earthquakes that could affect the project area.

Historically over the last approximately 200 years, 25 earthquakes with local magnitudes (ML) equal or greater than 5.5 have occurred within approximately 50 kilometers of the site, based on a search of selected earthquake catalogs (Toppozada and Branum, 2002). The most recent significant earthquake to affect the project area was an earthquake (Vacaville-Winters) with a moment magnitude (Mw) of 6.4 that occurred on April 19, 1892 approximately 150 miles from the site.

Local earthquakes can also be expected from Lassen Peak if it enters a phase nearing eruption or if subsurface migration of magma occurs. Those earthquakes, similar to earthquakes experienced prior to eruption of Mt. St. Helens or at Mammoth Mountain (without eruption), typically occur as swarms with earthquake magnitudes of low to moderate intensity.

3.2.2 Deterministic Estimates of Strong Ground Motions

Peak horizontal ground accelerations were estimated for the project site using attenuation relations from Campbell & Bozorgnia (1994), Boore et al (1997), and Sadigh et al. (1997) and the computer program EQFAULT (Blake, 1999a). The results of those deterministic estimates were averaged and are shown in the following table. For Cascadia Subduction Zone events, attenuation relations of Youngs et al. (1993/1997) were used.

Soil conditions modeled in the deterministic studies consisted of stiff/dense soils. Based on these evaluations, the site could be subjected to horizontal ground accelerations of at least 0.16g from the rupture of continental faults. Based on those evaluations, the causative fault that is responsible for that peak horizontal ground acceleration is the Battle Creek Fault, located about 16 miles south of the project site. The relatively infrequent (although in this area, probably imminent) Cascadia Subduction Zone events are estimated to produce a peak horizontal ground acceleration of up to 0.5g. It should be noted that probability and exposure periods are not considered during deterministic evaluations and that, typically, deterministic estimates of strong ground motion for a site generate relatively conservative horizontal ground acceleration values.

DETERMINISTIC GROUND MOTION DATA

Fault Name	Maximum Credible Magnitude (M_W)	Distance From Site (km)	Fault Data		Deterministically Estimated Peak Ground Acceleration (g)	
			Length (km)	Slip Rate (mm/yr) [A]	M [B]	M+*S* [B]
Battle Creek	6.5	25	29	0.50±0.40	0.16	0.26
Foothills Fault System	6.5	39	360	0.05±0.03	0.12	0.20
Hat Creek-McArthur	7.0	77	96	1.5±1.0	0.10	0.15
Cedar Mtn-Mahogany Mtn	6.9	119	78	1 0±50	0.07	0.10

[A] – From Peterson et al. (1996). [B] – M = indicates estimated mean peak horizontal ground acceleration. M+*S* = peak horizontal ground acceleration utilizing mean plus at least one standard deviation (84th percentile) for seismicity data. Values from attenuation relations of Boore et al (1997), NEHRP "D".

3.2.3 Probabilistic Estimates of Strong Ground Motion and Peak Ground Acceleration

Probabilistic evaluations of horizontal strong ground motion that could affect the site were performed using the computer software FRISKSP (Blake, 1999b). The evaluations were performed using attenuation relations of Boore et al. (1997) using NEHRP class "D" condition. The horizontal ground accelerations were used to estimate the upper-bound (UBE) and design-basis (DBE) earthquake ground motions that the site might experience. The upper-bound event corresponds to horizontal ground accelerations having a 10 percent probability of exceedance in a 100-year exposure period. The DBE corresponds to horizontal ground accelerations having a 10 percent probability of exceedance in a 50-year time period.

Soil conditions modeled in the probabilistic studies consisted of dense and deep soils. The results of these evaluations are presented in the following table:

PROBABILISTIC GROUND MOTION DATA				
Earthquake Level	**Probabilistic Estimate Exposure Period (years)**	**Probability of Exceedance (%)**	**Return Period (years)**	**Estimated Peak Horizontal Ground Acceleration (g)**
Upper-Bound Ground-Motion	100	10	949	0.27
Design-Basis Ground-Motion	50	10	475	0.20

It should be noted that although the seismic hazard models used for this study predict the probability of exceedance for various levels of acceleration in a given exposure period, the models are not able to account for the effect that the passage of time since past earthquakes has on future earthquake probability. Thus, while time may affect the incipient risk of earthquakes occurring, the UBE and DBE values are based on any 100-year and 50-year exposure period, respectively, regardless of how recently earthquakes have occurred.

3.2.4 CBC Design Recommendations

At a minimum, structures should be designed in accordance with the 2013 California Building Code (CBC) seismic design criteria. CBC-based design requires the definition of the following seismic parameters: Site Class Designation; Site Coefficients (F_a and F_v); Mapped spectral accelerations for short periods (S_s); and Mapped spectral accelerations for a 1-second period (S_1).

CBC SEISMIC DESIGN PARAMETERS		
California Building Code	**Parameter**	**CBC Designation**
Site Coordinates	Latitude	40.570211°
	Longitude	-122.366719°
Section 1613.3.3 Table 1613.3.3(1)	Site Coefficient, F_a	1.240
Section 1613.3.3 Table 1613.3.3(2)	Site Coefficient, F_v	1.765
Section 1613.3.1 Figure 1613.3	Site Class Designation	D
	Seismic Factor, Site Class B at 0.2 Seconds, S_s	0.700g
	Seismic Factor, Site Class B at 1.0 Seconds, S_1	0.318g
Section 1613.3.3	Site Specific Response Parameter for Site Class D at 0.2 Seconds, S_{MS}	0.868g
	Site Specific Response Parameter for Site Class D at 1.0 Seconds, S_{M1}	0.561g
Section 1613.3.4	$S_{DS}=2/3S_{MS}$	0.579g
	$S_{D1}=2/3S_{M1}$	0.374g

The site is not located within 10 kilometers of an active fault, thus, a site specific analysis is not required.

3.3 LIQUEFACTION, LATERAL SPREADING, & COSEISMIC DEFORMATION

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures caused by cyclic loading from a seismic event. In simple terms, it means that a liquefied soil acts more like a fluid than a solid when shaken during an earthquake. In order for liquefaction to occur, the following are needed:

- Granular soils (sand, silty sand, sandy silt, and some gravels);
- A high groundwater table; and
- A low density in the granular soils underlying the site.

If those criteria are present, then there is a potential that the soils could liquefy during a seismic event.

The adverse effects of liquefaction include local and regional ground settlement, ground cracking and expulsion of water and sand, the partial or complete loss of bearing and confining forces used to support loads, amplification of seismic shaking, and lateral spreading. In general, the effects of liquefaction on the proposed project could include:

- Lateral spreading;

- Vertical settlement; and/or
- The soils surrounding lifelines can lose their strength and those lifelines can become damaged or severed.

Lateral spreading is defined as lateral earth movement of liquefied soils, or soil riding on a liquefied soil layer, down slope toward an unsupported slope face, such as a creek bank, or an inclined slope face. In general, lateral spreading has been observed on low to moderate gradient slopes, but has been noted on slopes inclined as flat as one degree.

The site is underlain by dense to very dense granular soils. Because of the grain size characteristics and relative density of the sediments, these materials are considered to have a low potential for liquefaction during a seismic event.

Another potentially adverse secondary seismic effect is co-seismic compaction of moderately consolidated, sandy, relatively cohesionless soils above or below groundwater. Co-seismic compaction is soil densification resulting from dynamic loading of relatively loose, non-cohesive soil materials. That is, shaking or vibration can densify loose to moderately consolidated granular soils, resulting in settlement of the ground surface.

Soils encountered during this investigation are estimated to have a low potential for seismic induced settlement under the anticipated seismic ground motions at the site. The soils were dense to very dense granular soils.

3.4 EXPANSIVE POTENTIAL

There is a direct relationship between plasticity of a soil and the potential for expansive behavior, with expansive soil generally having a high plasticity. Thus, granular soils typically have a low potential to be expansive, where as, clay-rich soils can have a low to high potential to be expansive. Atterberg limit testing performed on two selected samples found plasticity index (PI) ranging from nonplastic to approximately 11. A PI value of 11 is associated with soils having a very low to low expansion potential (Day, 1999)), as noted in the following table:

EXPANSION POTENTIAL – PLASTICITY INDEX CORRELATION

Plasticity Index	Correlated Expansion Potential
0 – 10	Very Low
10 – 15	Low
15 – 25	Medium
25 – 35	High
35+	Very High

Taken from Day (1999)

3.5 SOIL CHEMISTRY

Two selected samples of the near-surface soil encountered at the site was subjected to chemical analysis for the purpose of assessment of corrosion and reactivity with concrete. The sample was tested for soluble sulfates and soluble chlorides. Testing was conducted by HDR of Claremont and results are presented below, as well as included in the appendix of laboratory results.

SOIL CHEMISTRY RESULTS					
Sample	**Depth**	**Sulfates (ppm)**	**Chlorides (ppm)**	**pH**	**Resistivity (ohms-cm)**
DH-1	0'-5'	41	13	7.0	11,600
DH-2	4'	45	14	7.6	3,650

According to the ACI-318, a sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible. Tested soil samples are below that threshold and considered negligible.

A chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete. Tested soils are below that threshold.

Minimum resistivity testing performed on the soil sample indicated the soils are considered to be mildly corrosive to buried metal objects. A commonly accepted correlation between soil resistivity and corrosivity towards ferrous metals (NACE Corrosion Basics, 1984) is provided below:

RESISTIVITY & CORROSION CORRELATION	
Minimum Resistivity (ohm-cm)	**Corrosion Potential**
0 to 1000	Severely Corrosive
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderately Corrosive
Over 10,000	Mildly Corrosive

Also, based on our test results, soils at the site are non-corrosive for structural elements according to Caltrans corrosion guidelines (Caltrans, 2012).

3.6 LANDSLIDES & REGIONAL SUBSIDENCE

The project site is relatively flat. It is our opinion that naturally occurring landslides pose a low risk to the project.

Regional subsidence typically occurs due to sustained withdrawal of subsurface fluids or gas, leading to consolidation of the subsurface reservoirs and surface settlement. No regional subsidence is known to be occurring in the project area.

3.7 TSUNAMI AND SEICHE POTENTIAL

A tsunami, or seismically generated sea wave, is generally created by a large, distant earthquake occurring near a deep ocean trough. A seiche is an earthquake-induced wave in a confined body of water, such as a lake or reservoir. Damage from tsunamis is confined to coastal areas that are 20 feet or less above sea level. Since the project is not located near the coast or any confined bodies of water, the risk of inundation from a tsunami or seiche is considered negligible.

3.8 VOLCANIC HAZARDS

The project site lies within an area subject to potential hazards from future eruptions of Mount Shasta and Lassen Peak. The potential hazards would likely be in the form of ash fall.

3.9 FLOOD HAZARDS

We understand from maps provided by NMR (2015) that the proposed structure development sites will be above Special Flood Hazard Areas (1% annual chance floodplain or 100-year flood).

3.10 NONTECTONIC FAULTING AND HYDROCOLLAPSE POTENTIAL

There is probably little risk that nontectonic-related faulting might adversely affect the project site. That type of faulting is typically associated with ground fissures due to regional or local subsidence (typically caused by subsurface fluid or gas extraction), or from surface deformation due to heaving or subsidence from subsurface magma movement. In addition, nontectonic-related faulting can be caused by differential settlement or collapse in areas overlying sharp, steep, buried contacts between earth materials having significantly different settlement or collapse potentials. None of these conditions are known to exist at the project site.

Hydrocollapse occurs when sediments having weak argillic or other types of cementation which collapse when introduced to water. That collapse results in a volumetric decrease in pore space leading to significant settlement at the ground surface. Typically, hydrocollapse occurs in alluvial fans and other sediments in relatively arid environments. There is no known hydrocollapse occurring within the project region and the risk of hydrocollapse at the site is probably low. This conclusion is based on the presence of shallow groundwater throughout the project area, indicating that if collapse were to occur in underlying sediments, it likely has previously occurred and should not reoccur in the future.

3.11 GEOENVIRONMENTAL CONSIDERATIONS

A Phase I was performed for the project site by Enplan (2015). That study should be referred to regarding geoenvironmental considerations for the project site.

3.12 ASBESTOS, RADON, AND OTHER GEOLOGIC HAZARDS

Asbestos-bearing geologic materials are not known to be present at the project site. Therefore, it is our opinion that the geologic risks posed to the project and project area is low. Assessment of asbestos related hazards for existing structures on site is not within our scope fo services.

Similarly, radon-222 has not been identified as a significant geologic hazard in the project area. No other geologic hazards are known to exist in the project area.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that the site is suitable for the proposed improvements provided recommendations presented, herein, are utilized during design and construction of the project. Specific comments and recommendations regarding the geotechnical aspects of project design and construction are presented in the following sections of this report.

Recommendations presented, herein, are based upon the proposed site development plans prepared by NMR (2016), along with stated assumptions. Changes in the configuration from those studied during this investigation may require supplemental recommendations.

4.2 FAULTING

No known faults pass through the project site. Several faults have been mapped in the vicinity of the project area. The site does not lie within the boundaries of an Alquist-Priolo Earthquake Fault Zone; therefore, it is our opinion that surface rupture potential is low.

4.3 LIQUEFACTION POTENTIAL

Based on our observations and material exposed during the investigation, it our opinion that liquefaction and lateral spreading have a relatively low risk of adversely affecting the proposed improvements.

4.4 EXPANSIVE POTENTIAL

Atterberg limit testing performed on select surficial sample recorded a PI of approximately 11 for the materials that will be encountered on site This material correlates to material having a very low to low expansion potential (Day, 1999).

4.5 LANDSLIDING & OTHER GEOHAZARDS

The project site is relatively flat. It is our opinion that naturally occurring landslides pose a low risk to the project. See Sections 4.6.8 and 4.7 of this report regarding temporary and man-made slope stability issues.

In our opinion, geologic hazards discussed in sections 3.4 through 3.12 have a low risk of adversely impacting the proposed project.

4.6 SITE PREPARATION AND GRADING

4.6.1 Stripping

Prior to general site grading and/or construction of planned improvements, existing pavement, vegetation, trees, organic topsoil, slabs, walls, foundations, debris, and deleterious materials should be stripped and disposed of off-site or outside the construction limits. Concrete debris, AC and aggregate base (AB) materials can be ground and reused within engineered fill materials. It is anticipated that stripping depths will extend approximately 2 to 3 inches deep. Any tree or shrub root balls encountered during stripping could extend deep below grade and should be removed during stripping. CGI should be allowed to observe stripped areas to confirm that adequate removal of organic, deleterious, and unsuitable materials have been properly stripped and removed from the site.

4.6.2 Existing Utilities, Wells, and/or Foundations

Because the site was once occupied by a concrete batch plant and that numerous structures have had been present have been demolished, there is a possibility that construction debris, utility lines, foundations, former ponds, and unsuitable material may exist. Below-grade utility lines, septic tanks, cesspools, wells, on-site waste disposal fields and tanks, irrigation ponds and/or foundations that are encountered during construction should be removed and disposed of off-site. Buried tanks, if present, should be removed in compliance with applicable regulatory agency requirements. Existing, below-grade utility pipelines (if any) that extend beyond the limits of the proposed construction and will be abandoned in-place should be plugged with lean concrete or grout to prevent migration of soil and/or water. All excavations resulting from removal and demolition activities should be cleaned of loose or disturbed material prior to placing any fill or backfill.

4.6.3 Scarification and Compaction

Following site stripping and overexcavation, areas to receive engineered fill should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined using standard test method ASTM D1557[1].

4.6.4 Keying and Benching

The proposed development area is located on relatively flat ground. Therefore, keying and benching are not anticipated to be required for this project.

4.6.5 Wet/Unstable Soil Conditions

If site preparation or grading is performed in the winter, spring, or shortly after significant precipitation, near-surface on-site soils may be significantly over optimum moisture content. Also, water may migrate through trenches and pipelines from prior development that have previously been abandoned (if any). These conditions could hinder equipment access as well as efforts to compact site soils to a specified level of compaction. In addition, perched water can be present in subsurface layers throughout the year and contribute to wet soil conditions. If over optimum soil moisture content conditions are encountered during construction, disking to aerate, replacement with imported material, chemical treatment, stabilization with a geotextile fabric or grid, and/or other methods will likely be required to facilitate earthwork operations. The applicable method of stabilization is the contractor's responsibility and will depend on the contractor's capabilities and experience, as well as other project-related factors beyond the scope of this investigation. Therefore, if over-optimum moisture within the soil is encountered during construction, CGI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

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[1] This test procedure applies wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

4.6.6 Site Drainage

Finished grading should be performed in such a manner that provides a minimum of 10 horizontal feet of positive surface gradient away from all structures. The ponding of water should not be allowed adjacent to structures, retaining walls, or the top of fill sections. Water should be prevented from flowing over cut and fill slopes. Surface runoff should be directed toward engineered collection systems or suitable discharge areas and not allowed to flow onto or over slopes. Discharge from roof downspouts should also be collected, conveyed in solid (unperforated) pipelines, and discharged away from all structures and into engineered systems, such as storm drains. Landscape plantings around structures should be avoided or be dry climate tolerant and require minimal irrigation. Care should be taken to avoid overwatering all landscaping.

4.6.7 Excavation Characteristics

Explorations for this project were advanced using a Mobile Drill B-59 drill rig using 8.25-inch diameter hollowstem-augers. Underlying soils were excavated with moderate to great difficulty where numerous gravels, cobbles, and possibly boulders were encountered. Those areas could pose moderate to difficult excavation characteristics and will pose difficult pile driving conditions.

Based on those observations, it is our opinion that on-site soils should be excavatable using conventional heavy grading equipment. Conventional rubber-tired backhoes could have difficulty penetrating to depths below 5 feet. Larger excavators, such as a Caterpillar 320 or larger (or equivalent) could be needed to penetrate deeper soils. Large cobbles, boulders, and hardpan layers, if encountered within those soils, could pose difficult excavation conditions. Rippability and excavation does not imply that clasts will be reduced to acceptable dimensions for use in engineered fill materials.

4.6.8 Temporary & Permanent Slopes

This section explicitly excludes trench slopes for buried utilities. Temporary trench excavations are discussed in Section 4.7.1 of this report.

Temporary construction slopes should be constructed no steeper than 1:1. Permanent slopes should be constructed at inclinations of 2:1 or flatter. In isolated areas where a cut slope is less than 8 feet tall, is adequately protected from erosion, and is not intended to support structures or surcharges, then the cut slope can be constructed at inclinations of 1.5:1 or flatter, per Section J106 of the 2013 CBC.

In order to comply with CBC regulations, minimum setbacks for proposed structures, over slopes with inclination steeper than 3:1, should be equivalent to the height of the slope divided by 3, but need not exceed 40 feet. If the desired setbacks are less than these requirements, then the foundations of the structures should be deepened or opt for alternate setbacks in accordance with requirements of section 1808.7.5 of 2013 CBC.

4.6.9 Overexcavation

Artificial fill materials were encountered during exploration at the site. Because these fills were uncertified and the nature of the fill and level of compaction are unknown, it is our recommendation to remove and replace those fill materials. Also, if during grading operations, loose or disturbed material resulting from the removal of buried structures are encountered, it is recommended that these materials be overexcavated and replaced with engineered fill materials. The overexcavation should extend a minimum of 5 horizontal feet outside of the building perimeter. A CGI engineer or geologist should observe and approve the overexcavated areas to confirm that those materials have been fully removed prior to placement of engineered fill materials. Overexcavated materials containing organics, debris, or deleterious materials should be removed from the project site and disposed of at an approved location.

Areas that are overexcavated should be backfilled with engineered fill materials, in accordance with recommendations presented in Section 4.6.13 of this report.

4.6.10 On-Site Soil Materials

It is our opinion that most of the near-surface soils encountered at the site can be used for general engineered fill provided it is free of organics, debris, oversized particles (>3”) and deleterious materials. If highly plastic clayey materials (materials having a plasticity index exceeding 30 and a liquid limit in excess of 50) are encountered during grading, those materials should be segregated and excluded from engineered fill, where possible, or thoroughly mixed with granular materials to reduce the plasticity of the soil. The existing artificial fill materials encountered during exploration can also be re-used as engineered fill provided those materials are screened of organics, woody debris, refuse, and deleterious materials. If potentially unsuitable soil is considered for use as engineered fill, CGI should observe, test, and provide recommendations as to the suitability of the material prior to placement as engineered fill.

4.6.11 Imported Fill Materials - General

If imported fill materials are used for this project, they should consist of soil and/or soil-aggregate mixtures generally less than 3 inches in maximum dimension, nearly free of organic or other deleterious debris, and essentially non-plastic. Typically, well-graded mixtures of gravel, sand, non-plastic silt, and small quantities of clay are acceptable for use as imported engineered fill within foundation areas. Imported fill materials should be sampled and tested prior to importation to the project site to verify that those materials meet recommended material criteria noted below. Specific requirements for imported fill materials, as well as applicable test procedures to verify material suitability are as follows:

IMPORTED FILL RECOMMENDATIONS				
GRADATION				
Sieve Size	**General Fill**	**Granular Fill**	**Test Procedures**	
	Percent Passing		**ASTM**	**AASHTO**
3-inch	100	100	D422	T88
¾-inch	70 – 100	70 – 100	D422	T88
No. 200	0 - 30	<5	D422	T88
PLASTICITY				
Liquid Limit	<30	NA	D4318	T89
Plasticity Index	<12	Nonplastic	D4318	T90
ORGANIC CONTENT	<3%	<3%	D2974	NA
Soil chemistry tests are recommended on imported soils to evaluate corrosivity to buried improvements.				

4.6.12 Materials - Granular

All granular fill should consist of imported soil mixtures generally less than 3 inches in maximum dimension, nearly free of organic or other deleterious debris, and essentially non-plastic. Specific requirements for granular fill, as well as applicable test procedures to verify material suitability are presented in Section 4.6.11 of this report.

4.6.13 Placement & Compaction

Soil and/or soil-aggregate mixtures used for fill should be uniformly moisture-conditioned to within 3 percent of optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction[2]. Testing should be performed to verify that the relative compactions are being obtained as recommended herein. Compaction testing, at a minimum, should consist of one test per every 500 cubic yards of soil being placed or at every 1.5-foot vertical fill interval, whichever comes first. We recommend that CGI be retained to perform compaction testing to verify compliance with our recommendations.

In general, a "sheep's foot" or "wedge foot" compactor should be used to compact fine-grained fill materials. A vibrating smooth drum roller could be used to compact granular fill materials and final fill surfaces.

4.7 UTILITY TRENCHS AND TRENCH BACKFILL

4.7.1 Trenches and Dewatering

Utility trenches greater than 5 feet deep should be braced or shored in accordance with good construction practices and all applicable safety ordinances. In general, soils having a tendency to run or flow were observed during our study; thus, there is a potential that shallow un-shored trenches excavated with sidewalls steeper than 1:1 could locally cave. The actual construction of the trench walls and worker safety is the sole responsibility of the contractor.

[2] This test procedure applies wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 (horizontal to vertical) projection from the toe of the trench excavation to the ground surface. Where the stability of adjoining buildings, walls, buried utilities within the trench sidewalls, or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

4.7.2 Materials

Pipe zone backfill (i.e., material placed from the trench bottom to a minimum of 6 inches over the pipeline crown) should consist of imported soil having a Sand Equivalent (SE) of no less than 30 and having a particle size no greater than ½-inch in maximum dimension. On site soils will likely not meet this recommendation. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of on-site soil that meets the material requirements previously provided for engineered fill with 100% passing the ¾-inch sieve.

If imported material is used for pipe or trench zone backfill, we recommend it consist of fine-grained sand. In general, use of coarse-grained sand, crushed rock, and/or gravel is not recommended due to the potential for soil migration into and water seepage along trenches backfilled with this type of material.

Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local codes and/or bedding requirements for specific types of pipe. We recommend the project Civil Engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

4.7.3 Placement and Compaction

Trench backfill should be placed and compacted in accordance with recommendations previously provided for engineered fill. Mechanical compaction is strongly recommended; *ponding, flooding, and jetting should not be allowed during construction.* It should be noted that if in rare instances, ponding, flooding, or jetting are allowed, the pipe zone backfill materials should have an SE of 50 or greater and should be less than ½-inch in maximum dimension. In addition, a number of additional conditions for collection and removal of excess ponded, flooded, or jetted water will be required if those methods are utilized during construction. Special care should be given to ensuring that adequate compaction is made beneath the haunches of the pipeline (that area from the pipe springline to the pipe invert) and that no voids remain in this space.

4.7.4 Trench Subgrade Stabilization

Soft and yielding trench subgrade could be encountered along the bottom of trench excavations. It is recommended that the bottom of trenches be stabilized prior to placement of the pipeline bedding so that, in the judgment of the geotechnical engineer, the trench subgrade is firm and unyielding. The Contractor should have the sole responsibility for design and implementation of trench subgrade stabilization techniques. Some methods that we have observed used to stabilize trench subgrades include the following:

- Use of ¾–inch to 1½-inch floatrock worked into the trench bottom and covered with a geotextile fabric such as Mirafi 500X;
- Placement of a geotextile fabric, such as Mirafi 500X, on the trench bottom and covered with at least one foot of compacted processed miscellaneous base (PMB) conforming to the requirements of Section 200-2.5 of the Greenbook, latest edition;
- Overexcavation of trench subgrade and placement of two-sack sand-cement slurry; and
- In extreme conditions, injection grouting along the trench alignment.

If floatrock is used, typically sand with an SE of 50 or more should be used to fill the voids in the rock prior to placement of pipe bedding materials.

4.7.5 Erosion Protection

The on site soil materials are relatively erodable. Maintained, drought-resistant vegetation, riprap, or similar protective material should cover all permanent cut and fill slopes (if any). All drainage channels should be paved or lined with rip-rap.

4.8 SHALLOW FOUNDATIONS

4.8.1 Minimum Footing Embedment and Dimensions

Minimum embedment depths, widths, and thicknesses should conform to Table 1809.7 of the CBC, but should be determined by the Structural Engineer. Transition lot construction, where structures span across both native cut materials and engineered fills, can lead to differential settlement issues. **Foundations should not span both cuts and fills**.

As proposed in the design plans, buildings A & C will span cuts and fills and may experience differential settlements. Artificial fills in those areas are composed of uncertified fill.

Where proposed foundations span both cuts and fills, we recommend that:

- The area of cuts supporting the proposed foundations should be overexcavated below the planned bottom of footings to a depth of at least 3 times the width of the foundation. CGI should observe and approve the overexcavated area once exposed. Overexcavation limits should extend throughout the cut area and to a minimum of five horizontal feet past the perimeter foundations of the structure. The overexcavated area should then be backfilled in accordance with recommendations presented in Section 4.6.13 of this report;

OR

- Proposed foundations should be deepened to extend through engineered fill materials and extending to undisturbed native soils, so that the entire foundation system for the structure rests on undisturbed native soils. If this depth is less than 5 feet below the planned bottom of the foundation, then a two-sack sand-cement slurry can be used as backfill in lieu of structural concrete, from the excavation bottom up to the planned bottom of the proposed foundation. CGI should observe and approve the deepened foundation excavation prior to placement of slurry or structural concrete.

If foundations do not span both cuts and fills, then neither of the two alternatives recommended noted above should be necessary.

Frost penetration depths typically do affect soil within the area of the proposed project. Therefore, foundations should not require specific design recommendations to reduce the potential adverse affects of frost on structure foundations.

4.8.2 Allowable Bearing Capacity

It is assumed that all structure foundations for the proposed buildings will rest entirely on cut or entirely on engineered fill. The foundations must not be constructed partially on fill and partially on cut. Isolated and continuous footing elements should be proportioned for dead loads plus probable maximum live load, and a maximum allowable bearing pressure of the following:

MAXIMUM ALLOWABLE BEARING PRESSURES

Material	Allowable Bearing Capacity (psf)	Increase per Foot of Embedment (psf)	Maximum Allowable Bearing Capacity (psf)
Alluvium/Clay Material (0-10')	1,500	200	2,500
Alluvium/Coarse Material (0-10')	2,000	250	5,000
Alluvium (below 10')	3,000	300	5,000
Engineered Fill	1,500	150	2,500

The allowable bearing pressures provided are net values. Therefore, the weight of the foundation (which extends below finished subgrade) may be neglected when computing dead loads. The allowable bearing pressure applies to dead plus live loads and includes a calculated factor of safety of at least 3. An increase of allowable bearing pressure by one-third for short-term loading due to wind or seismic forces should NOT be incorporated unless an alternative load combination, as described in Section 1605.3.2 of the 2013 CBC, is applied. The allowable bearing value is for vertical loads only; eccentric loads may require adjustment to the values recommended above.

4.8.3 Lateral Earth Pressures

It is our understanding that retaining structures are not required for this project. However, if changes are made to the project and retaining structures are incorporated in the design, they should be designed to resist the earth pressure exerted by the retained, compacted backfill plus any additional lateral force that will be applied due to surface loads placed at or near the wall or below-grade structure. Recommended design criteria for subsurface structures are presented below:

The recommended equivalent fluid weights presented below are for static (non-earthquake) conditions with the ground level or inclined at 2:1 behind the shoring system.

LATERAL EARTH PRESSURES UNDER STATIC CONDITIONS		
Lateral Earth Pressure Condition	**Slope Inclination Above Retaining Structure**	**Equivalent Fluid Weight (pcf)**
		Drained
At-Rest	Flat	60
Active	Flat	40
At-Rest	2:1	80
Active	2:1	60

The resultant force of the static lateral force prism should be applied at a distance of 33 percent of the wall height above the soil elevation on the toe side of the wall. The tabulated values are based on a non-plastic, drained soil with a unit weight of 125 pounds per cubic foot (pcf), and do not provide for surcharge conditions resulting from construction materials, equipment, or vehicle traffic. Loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the shoring. If surcharges are expected, CGI should be advised so that we can provide additional recommendations as needed.

4.8.4 Minimum Footing Reinforcement

Footing reinforcement should be designed by a Structural Engineer and should conform to pertinent structural code requirements. Minimum footing reinforcement should not be less than that required for shrinkage, temperature control, and structural integrity.

4.8.5 Estimated Settlements

The proposed structures should not rest partially on fill and partially on cut. All foundations are anticipated to rest on native soils. Anticipated total settlement for the proposed structure foundations, if construction occurs as recommended within this report, should be less than one inch. Differential settlement for the structure foundations is anticipated to be less than ½ -inch in 20 feet.

4.8.6 Construction Considerations

Granular Material and shallow Groundwater may create difficult site conditions during construction. Temporary dewatering should be anticipated if bottom of excavations are within or below groundwater elevation. The type of dewatering (sump pump, well points, and deep wells) will depend on the depth, the type of material (loose, cohesioneless …) and the extend of the excavation. In order to maintain stability of the excavation, groundwater elevation should be lowered a minimum of 18 inches below the bottom of excavation. The applicable method of dewatering is the contractor's responsibility and will depend on the contractor's capabilities and experience, as well as other project-related factors beyond the scope of this investigation.

Prior to placing steel or concrete, foundation excavations should be cleaned of all debris, loose or disturbed soil, and any water. A representative of CGI should observe all foundation excavations prior to concrete placement.

4.9 SLIDING AND PASSIVE RESISTANCE

4.9.1 Sliding Resistance

Ultimate sliding resistance generated through a compacted soil/concrete interface can be computed by multiplying the total dead weight structural loads by the friction coefficient of 0.30 and 0.35 for artificial fill/native soils and imported granular engineered fill, respectively.

4.9.2 Passive Resistance

Ultimate passive resistance developed from lateral bearing of shallow foundation elements bearing against compacted soil surfaces for that portion of the foundation element extending below a depth of 1 foot below the lowest adjacent grade can be estimated using an equivalent fluid weight of 200 pcf, up to 10 feet below ground and 300 pcf, from 10 feet and below. Passive resistance of the upper one foot of the soil column should be neglected.

4.9.3 Safety Factors

Sliding resistance and passive pressure may be used together without reduction in conjunction with recommended safety factors outlined below. A minimum factor of safety of 1.5 is recommended for foundation sliding.

4.10 DEEP FOUNDATIONS

If uplift & lateral forces, differential settlement considerations, and/or other structural or economic considerations preclude the use of spread foundations for support, deep foundations systems will be required to support the proposed structure. Based on the subsurface soil conditions encountered during this study, it is anticipated that the building foundations can be supported on driven piles. For driven piles, we have assumed that H-piles will be utilized. Concrete, pipe, or other pile types can also be assessed for use on this project, if desired.

4.10.1 Vertical Capacity

The granular component of the site material should provide adequate support for deep foundations. Thus, in CGi's opinion, the proposed structure can be supported on HP 10x57 piles. We have recommended specified shaft lengths on the basis of estimated frictional resistances for HP 10x57 piles driven into undisturbed onsite materials.

The capacity of HP piles was estimated on the basis of frictional resistance only (i.e., end bearing was neglected). The actual analyses were facilitated by use of the computer program APILE, Version 7.3 (Ensoft, 2015). That program is specifically used for driven piles and incorporates commonly-accepted design procedures for multiple types of geo-materials, such as the sand and gravel within the alluvium. The capacities of the piles, and, thus, the recommended lengths for specific loading conditions, were estimated assuming that: 1) the upper 5 feet provides no resistance and is neglected; 2) the recommended supporting pile lengths are entirely within undisturbed onsite materials; 3) end bearing does not provide resistance; and 4) good construction practices are used. It is expected that HP piles constructed to the specified lengths (presented below) will provide for either 400- or 625-class loading conditions. The allowable pile capacities were calculated using a factor of safety of 2 for static loading conditions.

CGI's recommend shaft lengths are based on attached pile capacity versus depth charts. For HP 10x57, 45- and 70-ton capacity pipe piles, the estimated shaft lengths are presented below.

HP PILE DESIGN RECOMMENDATIONS			
Pile Type	Compressive Pile Capacity (Tons)		Minimum Specified Length (feet)
	Allowable	Ultimate	
HP 10x57	45	90	35
HP 10x57	70	140	44

Note that for the above stated compressive capacities, the recommended supporting pile lengths are assumed to be entirely within undisturbed materials. Therefore, the specified lengths are relative to the top of onsite materials, and should discount any thickness of overlying alluvial and fill materials.

4.10.2 Negative Skin Friction

Negative skin friction or downdrag may occur when sediments located directly adjacent to piles move downward (i.e., compress) and induce additional forces on the pile. The downward movement of sediments usually is the result of additional fill materials being placed above compressible, fine-grained sediments that are located either below the tip or along the sides of a pile. More specifically, at the project site, most of the encountered sediments were associated with the granular Alluvium. Field and laboratory data suggest that the upper 5 to 12 feet of onsite materials near building A & C are either fine grained or loose soils and have a moderate compressibility. It is CGi's opinion that the onsite materials have a medium potential for inducing negative skin friction on installed piles if thick fill is placed around the piles. This potential condition is only applicable to areas where fill is not removed. It is recommended that the potential for negative skin friction within those materials be compensated by adding an extra 3 feet to the recommended pile lengths. As an alternative, if the driveability of the extra length of the piles at deeper depth is difficult, piles should be driven in holes predrilled through the new embankment. The hole should have a diameter of not less than the greatest dimension of the pile cross section plus 6 inches. After driving the pile, the space around the pile should be filled to the ground surface with dry sand or pea gravel.

4.10.3 Uplift Capacity

Analyses for determining uplift capacity of piles rely on the frictional resistance and the weight of the pile. However, we estimate the maximum allowable uplift capacity of piles to be about 0.7 times the allowable frictional resistance within the granular alluvium materials. The upper 5 feet of the piles, and any length of pile that penetrates through fill, should be neglected when estimating uplift capacity. We recommend that CGi be provided the opportunity to review and potentially modify any uplift capacities that may be estimated based on the above stated numbers.

4.10.4 Lateral Deflection

Lateral movement of the piles due to wind & seismic forces can be substantial and could affect the integrity of the structures. If deep foundations are selected for this project, the analyses of lateral deflection should be performed once the lateral forces and moments applied near the top of the piles are provided by the structural engineer.

4.10.5 Settlement

We estimate that settlement of piles under vertical static loading conditions, when installed using good construction techniques, should not exceed 0.5-inch total and 0.25-inch differential between adjacent piles.

4.10.6 Spacing and Group Effects

The recommended shafts lengths (noted above for certain loads) are for piles that have a minimum center-to-center spacing of 3 times the diameter of the pile. We note that actual spacing may be controlled by construction conditions and requirements to limit disturbance of adjacent piles.

The ultimate capacity of a group of piles can be estimated by multiplying the sum of the capacities of all the piles in the group by a group efficiency factor. The group efficiency factor is defined as the ratio of the ultimate load capacity of the group to the sum of the ultimate capacities of the individual piles. Group efficiency factor of 1.0 is recommended for center-to-center spacing of 3 times the pile diameter or higher. We note that, because of the presence of over-sized materials (e.g., cobbles and boulders) beneath the project site, center-to-center spacing should be maximized to reduce the potential for disturbance to adjacent piles during drilling, driving, and concreting activities.

4.10.7 Construction Considerations

The methods of analyses used to estimate pile capacities inherently assume that excellent construction procedures have been employed.

As previously noted, alluvium consist predominantly of granular sediments that contain abundant over-sized material (e.g., cobbles and boulders), and the ground water level can be relatively shallow. Those conditions may produce spalling or caving for deep excavation. The presence of cobble and boulders could pose difficult driving conditions for the HP-Piles.

If the contractor elects to use an alternative type of pile for the foundation, CGI can provide supplemental pile capacity analyses, if requested.

4.11 INTERIOR CONCRETE FLOOR SLABS SUPPORTED ON-GRADE

4.11.1 General

All ground-supported slabs should be designed by a Civil Engineer to support the anticipated loading conditions but, as a minimum, should be at least 4 inches thick. Reinforcement for floor slabs should be designed by a Civil Engineer to maintain structural integrity, and should not be less than that required to meet pertinent code, shrinkage, and temperature requirements. Reinforcement should be placed at mid-thickness in the slab with provisions to ensure it stays in that position during construction and concrete placement.

The mat slab can be designed using a flat slab on an elastic half-space analog. A modulus of subgrade reaction (ks_1) of 100 kcf is recommended for design of mat-type foundations. That modulus of subgrade reaction value represents a presumptive value based on soil classification. No plate-load tests were performed as part of this study. The modulus value is for a 1-foot-square plate and must be corrected for mat size and shape, assuming a cohesionless subgrade.

4.11.2 Subgrade Preparation

Subgrade soils supporting interior concrete floor slabs should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near the optimum moisture content, and compacted to at least 90 percent relative compaction.

4.11.3 Rock Capillary Break/Vapor Barrier

Interior concrete floor slabs supported-on-grade should be underlain by a capillary break consisting of a blanket of compacted, free-draining, durable rock at least 4 inches thick, graded such that 100 percent passes the 1-inch sieve and less than 5 percent passes the No. 4 sieve.[3] Furthermore, a vapor barrier should be placed beneath all interior concrete floor slabs supported-on-grade that will be covered with moisture-sensitive floor coverings. This barrier may consist of a plastic or vinyl membrane placed directly over the rock capillary break. The vapor barrier should be sealed around all penetrations, including utilities. If a vapor barrier is not installed, there is a risk of moisture vapors and salts penetrating the slab-on-grade. For this project, flooring materials on slabs-on-grade are unknown. It is our recommendation that American Concrete Institute (ACI) guidelines ACI 302 and ACI 360 be referred to regarding installation of vapor barriers based on the anticipated flooring materials to be installed.

A capillary break and/or vapor barrier may not be required for some types of construction (such as equipment buildings, warehouses, garages, and other uninhabited structures insensitive to water intrusion and/or vapor transmission through the slab). For these types of structures, the gravel capillary break and/or vapor barrier recommended above may be omitted and the slab placed directly on the prepared subgrade or other approved surface. In the event a capillary break and/or vapor barrier is not to be used, CGI should review the planned structure in order to assess the applicability of the approach and provide (if necessary) additional recommendations regarding subgrade preparation and/or support.

4.12 EXTERIOR CONCRETE SLABS SUPPORTED-ON-GRADE

Subgrade soils supporting exterior concrete slabs[4] should be scarified to a minimum depth of 1-foot, uniformly moisture-conditioned to near the optimum moisture content and compacted to at least 90

[3] In general, Caltrans Class 2 aggregate base (or similar material) does not meet the requirements provided above for a capillary break. Therefore, we recommend this material <u>not</u> be used for a capillary break beneath interior concrete slabs supported-on-grade.

[4] Within this report, exterior concrete slabs supported-on-grade refers to walkways, patios, etc. and specifically excludes roadway pavements.

percent relative compaction. In the event the exposed subgrade is dense and uniformly compacted, scarification and compaction may be omitted if approved by CGI during construction.

4.13 RETAINING WALLS

4.13.1 Lateral Earth Pressures

It is our understanding that retaining walls will not be constructed in this project. In case a change in the project is made and retaining wall are incorporated, the retaining structures should be designed to resist earth pressures exerted by the retained, compacted backfill plus any additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. The recommended equivalent fluid weights are presented in section 4.8.3 of this report.

The resultant force of the static lateral force prism should be applied at a distance of 30 percent of the wall height above the bottom of the foundation on the back of the wall.

The tabulated values are based on a drained, non-plastic soil with unit weight of 125 pounds per cubic foot (pcf), and do not provide for surcharge conditions resulting from foundations, vehicle traffic, or compaction equipment. The drained values do not provide for hydrostatic forces (for example, standing water in the backfill materials). Foundation loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the wall. If conditions such as surcharge resulting from footings or hydrostatic forces are expected, CGI should be advised so that we can provide additional recommendations as needed.

Surcharge loads induce additional pressures on earth retaining structures. An additional lateral load on non-yielding walls equal to 0.5 times the applied surcharge pressure should be included in the design for uniform area surcharge pressures. Lateral pressures for other surcharge loading conditions can be provided, if required.

4.13.2 Drainage Measures

Drainage measures should be constructed behind the proposed retaining walls to reduce the potential for groundwater accumulation. To help reduce the potential for the buildup of hydrostatic forces behind walls, a granular free-draining backfill, at least 2 feet thick, should be placed behind the wall, as shown on Plate 5 – Retaining Wall Details. The two-foot thick layer can be decreased to one foot in thickness if wrapped with a geosynthetic filter fabric, as discussed on Plate 5; however, the structural engineer should be consulted to confirm that the retaining wall is design to withstand potential increased stresses due to compaction closer to the wall. The free-draining backfill should consist of clean, coarse-grained material with no more than 5 percent passing the No. 200 sieve. Acceptable backfill would be:

- Pervious Backfill conforming to Item 300-3.5.2 of the *Standard Specifications for Public Works Construction* (Greenbook), most current edition;
- Permeable Material (Class 2) conforming to Item 68-1.025 if the *Caltrans Standard Specifications*, most current edition;
- Pea gravel having a nominal diameter or ¼-inch; or

- Crushed stone sized between ¼-inch and ½-inch.

In lieu of free-draining backfill materials of the types suggested above, manufactured (geosynthetic) drainage systems (for example MiraDrain manufactured by TC Mirafi, Inc., or equivalent) can be used against retaining or below-grade walls. Manufacturer recommendations for the installation and maintenance of these products should generally be followed, although they should be reviewed by CGI for approval. In addition, manufactured drainage systems should be attached to the retaining wall face as opposed to the excavated slope face. This implies that provisions to protect the integrity of the drainage panels will need to be made while fill materials are placed behind the walls.

A perforated drainpipe system should be installed at the base of the wall to collect water from the free-draining material and/or geosynthetic drainage system. The drainpipe system should allow gravity drainage of the collected water away from the buried wall or, as a less preferred option, should be tied into a sump and pump system to remove the water to an acceptable outlet facility.

Finish surface grades should be sloped away from the retaining walls and designed to channel water to an acceptable collection and offsite disposal system. Provisions should be included for removal of surface runoff that may tend to collect behind the backs of walls and for drainage of water away from the fronts of walls. Also, provisions should be included to mitigate the infiltration of surface water into the below-ground, free-draining backfill/geosynthetic drainage system by placing a minimum of 18-inches of low permeability compacted soil over the top of those materials.

4.13.3 Dynamic Earth Pressures

For unrestrained walls, the increase in lateral earth pressure acting on the wall resulting from earthquake loading can be estimated using the approach of Seed and Whitman (1970). That theory is based on the assumption that sufficient wall movement occurs during seismic shaking to allow active earth pressure conditions to develop. For restrained walls, the increase in lateral earth pressure resulting from earthquake loading also can be estimated using these relations. Because that theory is based on the assumption that sufficient movement occurs so that active earth pressure conditions develop during seismic shaking, the applicability of the theory to restrained or basement walls is not direct; however, there have been studies (Nadim and Whitman, 1992) that suggest the theory can be used for such walls.

In the Seed and Whitman (1970) approach, the total dynamic pressure can be divided into static and dynamic components. The estimated dynamic lateral force increase (based on seismic loading conditions) for either unrestrained or restrained walls, could be taken as the following:

$$P_E=3/8*k_h*Y_t*H^2$$

Where:

P_E	=	Seismically-induced horizontal force (lbs per lineal foot of wall)
Kh	=	PGA/g
Y_t	=	Total unit weight of backfill (pcf)
H	=	Height of the wall below the ground surface (ft)

Peak ground acceleration (pga) parameters for the site are provided in Section 3.2.2 of this report. The centroid of the dynamic lateral force increment should be applied at a distance of 0.6*H above the base of the wall.

To estimate the total lateral force, the dynamic lateral force increase should be added to the static earth pressure force computed using recommendations for active lateral earth pressures presented above. That recommendation is based on the concept that during shaking, earth pressures recommended for permanent conditions will be reduced to those more closely approximating active conditions.

4.13.4 Compaction Adjacent to Walls

Backfill within 5 feet, measured horizontally, behind retaining walls should be compacted with relatively lightweight, hand-operated compaction equipment to reduce the potential for creation of relatively large compaction-induced stresses. If large or heavy compaction equipment is used, compaction-induced stresses could result in increased lateral earth pressures on the retaining walls in addition to those presented in this report.

Backfill material should be brought up uniformly behind retaining walls (in other words, the backfill should be at about the same elevation behind the retaining wall as the backfill is placed and compacted). The elevation difference of the backfill surface behind the wall should not be greater than about 2 feet, unless the walls are designed for those differences.

4.13.5 Retaining Wall Differential Settlement

Retaining walls that span across cut-fill lines have the potential to experience differential settlement much like structures, as discussed in Section 4.8.1 of this report. Differential settlement of walls can result in cracking and deformation of the walls, whether they consist of concrete cantilever, segmental block, or other retaining wall systems. Where proposed retaining wall foundations span both cuts and fills, we recommend that either: 1) recommendations made in Section 4.8.1 be performed; 2) control joints be established in the retaining walls at the cut-fill daylight line location; or 3) the retaining wall be designed by a structural engineer to be sufficiently rigid to resist stresses induced by anticipated differential settlement along the retaining wall.

4.14 PAVEMENT DESIGN

4.14.1 R-Values

An R-value test was performed on a selected sample of on-site soils obtained during subsurface exploration at the site. The R-value test was performed in accordance with Caltrans test method CT-301 and is presented in Appendix B. A laboratory R-value of 59 was obtained from the testing. If the actual subgrade material that will be present at finish subgrade is different than the tested native soil, we recommend that confirmatory R-value tests be obtained during construction. If construction R-values are significantly different than the R-value reported above, then the pavement design can be modified at that time to reflect the constructed conditions.

4.14.2 Subgrade Preparation

All subgrade soils should be scarified to a minimum depth of 1-foot, moisture conditioned as necessary to near optimum moisture conditions and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO (American Association of State Highway and Transportation Officials) Test Method T-180. The subgrade should be smooth and unyielding prior to the placement of aggregate base rock. Density testing and proof rolling of the subgrade using a loaded water truck should be performed with satisfactory results prior to placement of the aggregate base rock. Concrete curbs and landscape planters that border pavement sections should be embedded into the subgrade soils a minimum of 2 inches to reduce the migration of meteoric and irrigation water into the pavement section.

Because of the size of the project site and its previous use, soft and yielding areas may exist. In the event of the presence of such areas during construction, CGI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

4.14.3 Aggregate Base

The aggregate baserock (AB) should be of such quality as to meet or exceed Caltrans specifications for Class 2 AB and should have a minimum R-value of 78. The AB should be spread in thin lifts restricted to 8 inches in loose thickness or less, moisture conditioned as necessary to near optimum moisture content and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO T-180. Density testing and/or proof rolling should be performed prior to placement of the asphalt paving.

4.14.4 Asphalt Concrete Paving

An R-value obtained for this study had a value of 59. Traffic indices (TI) for proposed project access roads and parking areas were not available to us at the preparation time of this report. To provide recommendations for structural pavement sections, we evaluated design criteria for five TIs ranging from 5.5 to 10. Using those criteria, we have prepared AC structural pavement section recommendations based on the City of Redding's pavement standards. Recommendations for full depth AC, and AC and AB sections are provided in the following table:

MINIMUM RECOMMENDED STRUCTURAL PAVEMENT SECTIONS (1)

Section	Traffic Index	Type B AC Thickness (ft)	Class 2 AB Thickness (ft)
Full Depth AC	5.5	0.45	- - -
	6.0	0.50	- - -
	8.0	0.70	- - -
AC and AB	5.5	0.20	0.50
	6.0	0.20	0.55
	8.0	0.70	0.85

1 – City of Redding Constructions Standards

Asphalt paving materials and equipment should meet or exceed current City of Redding specifications.

5 REVIEW OF PLANS AND SPECIFICATIONS

We recommend CGI conduct a general review of final plans and specifications to evaluate that recommendations contained herein have been properly interpreted and implemented during design. In the event that CGI is not retained to perform this recommended review, we will assume no responsibility for misinterpretation of our recommendations.

6 LIMITATIONS

This report has been prepared in substantial accordance with the generally accepted geotechnical engineering practice, as it existed in the site area at the time our services were rendered. No other warranty, either express or implied, is made. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations will be conducted by CGI during the construction phase in order to evaluate compliance with our recommendations.

Conclusions and recommendations contained in this report were based on the conditions encountered during our field investigation and are applicable only to those project features described herein (see Section 1.2 – Project Understanding). Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions can also vary seasonally and for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the project site. The conclusions and recommendations presented in this report are based upon the findings at the point of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction. If conditions encountered during construction differ from those described in this report, or if the scope or nature of the proposed construction changes, we should be notified immediately in order to review and, if deemed necessary, conduct additional studies and/or provide supplemental recommendations.

The scope of services provided by CGI for this project did not include the investigation and/or evaluation of toxic substances, or soil or groundwater contamination of any type. If such conditions are encountered during site development, additional studies may be required. Further, services provided by CGI for this project did not include the evaluation of the presence of critical environmental habitats or culturally sensitive areas.

This report may be used only by our client and their agents and only for the purposes stated herein, within a reasonable time from its issuance. Land use, site conditions, and other factors may change over time that may require additional studies. In the event significant time elapses between the issuance date of this report and construction, CGI shall be notified of such occurrence in order to review current conditions. Depending on that review, CGI may require that additional studies be conducted and that an updated or revised report is issued.

Any party other than our client who wishes to use all or any portion of this report shall notify CGI of such intended use. Based on the intended use as well as other site-related factors, CGI may require that additional studies be conducted and that an updated or revised report be issued. Failure to comply with any of the requirements outlined above by the client or any other party shall release CGI from any liability arising from the unauthorized use of this report.

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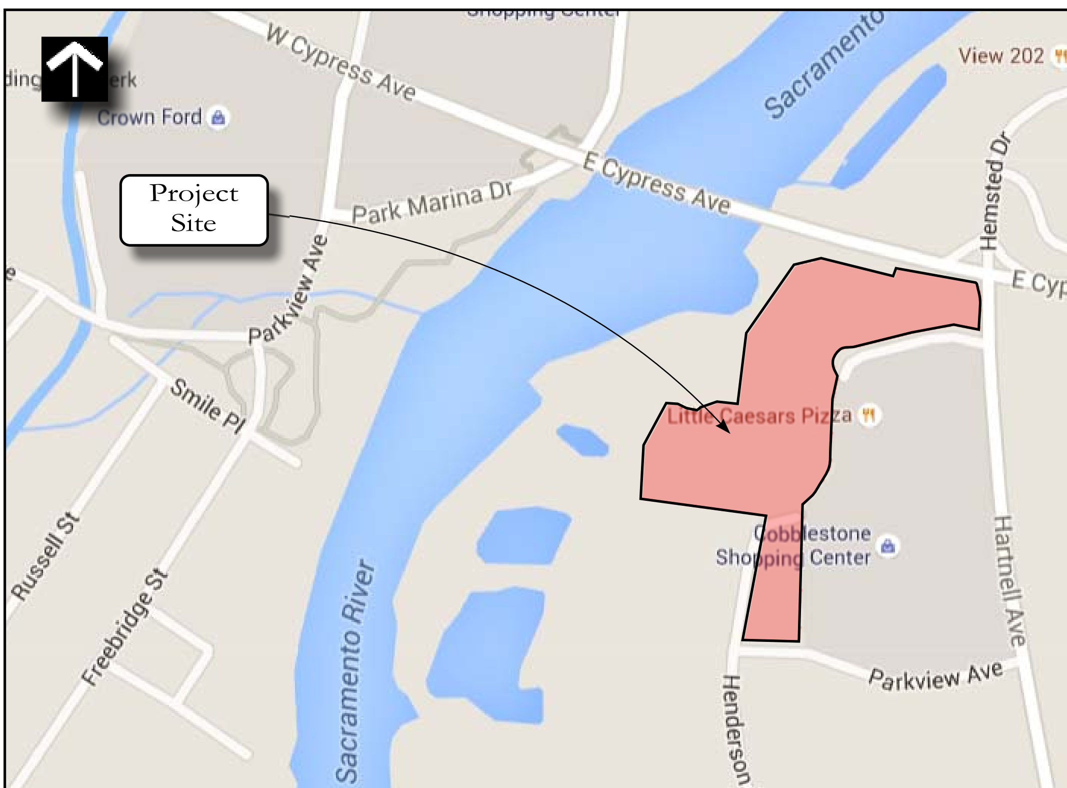


Base maps from Google Maps

Scale not determined

CGI Technical Services Inc.

Project No.:15-1080.84

SITE LOCATION MAP
MERCY WELLNESS CENTER
DIGNITY HEALTH
REDDING, CALIFORNIA

Plate

1

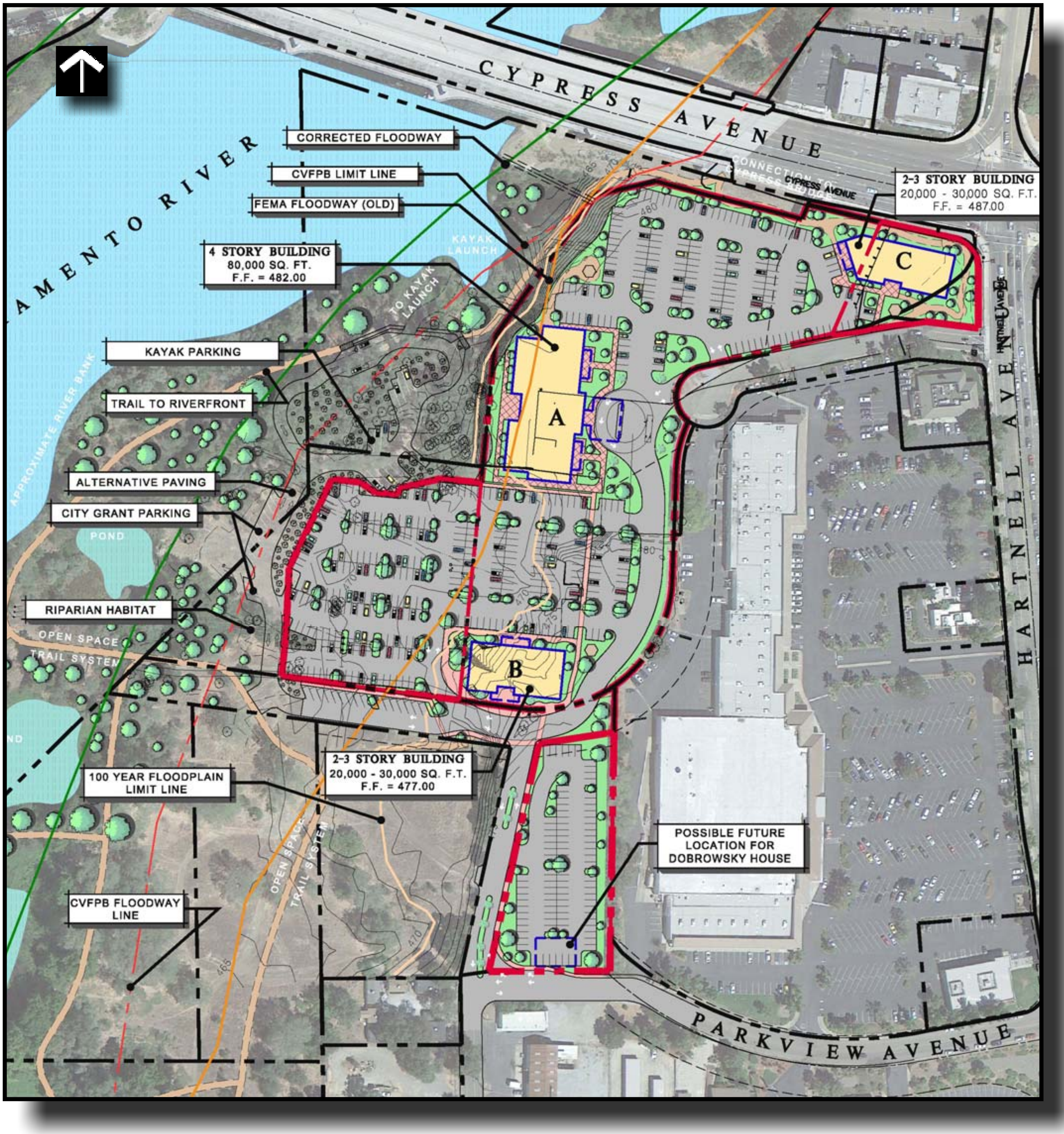
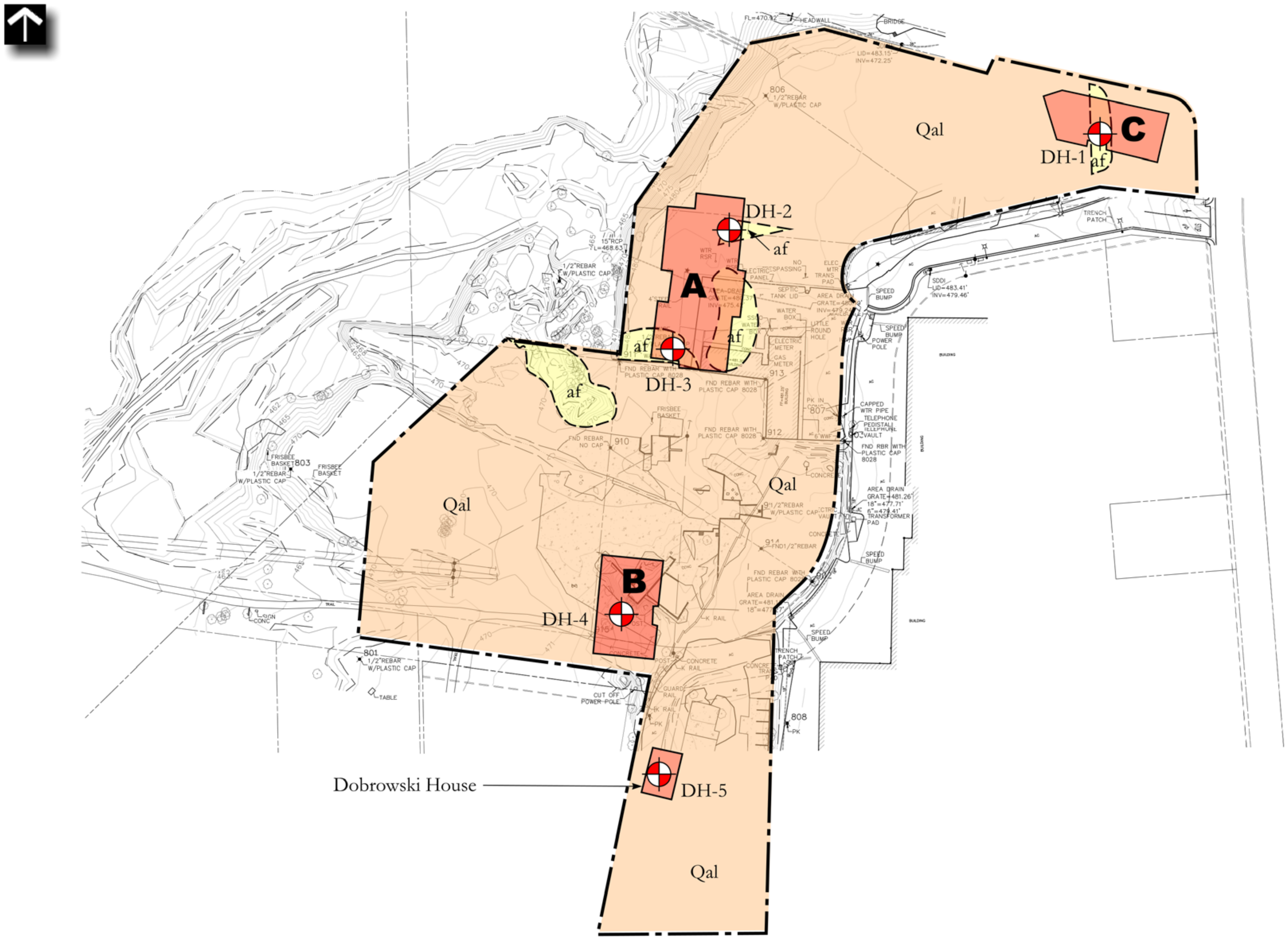


Illustration from NMR (2016)

Scale Undetermined

CGI Technical Services Inc.	PROJECT ELEMENTS MERCY WELLNESS CENTER DIGNITY HEALTH REDDING, CALIFORNIA	Plate 2
Project No.: 15-1080.84		



af — Artificial Fill

Qal — Alluvium

Geologic Contact: dashed where approximate, dotted where covered, queried where uncertain

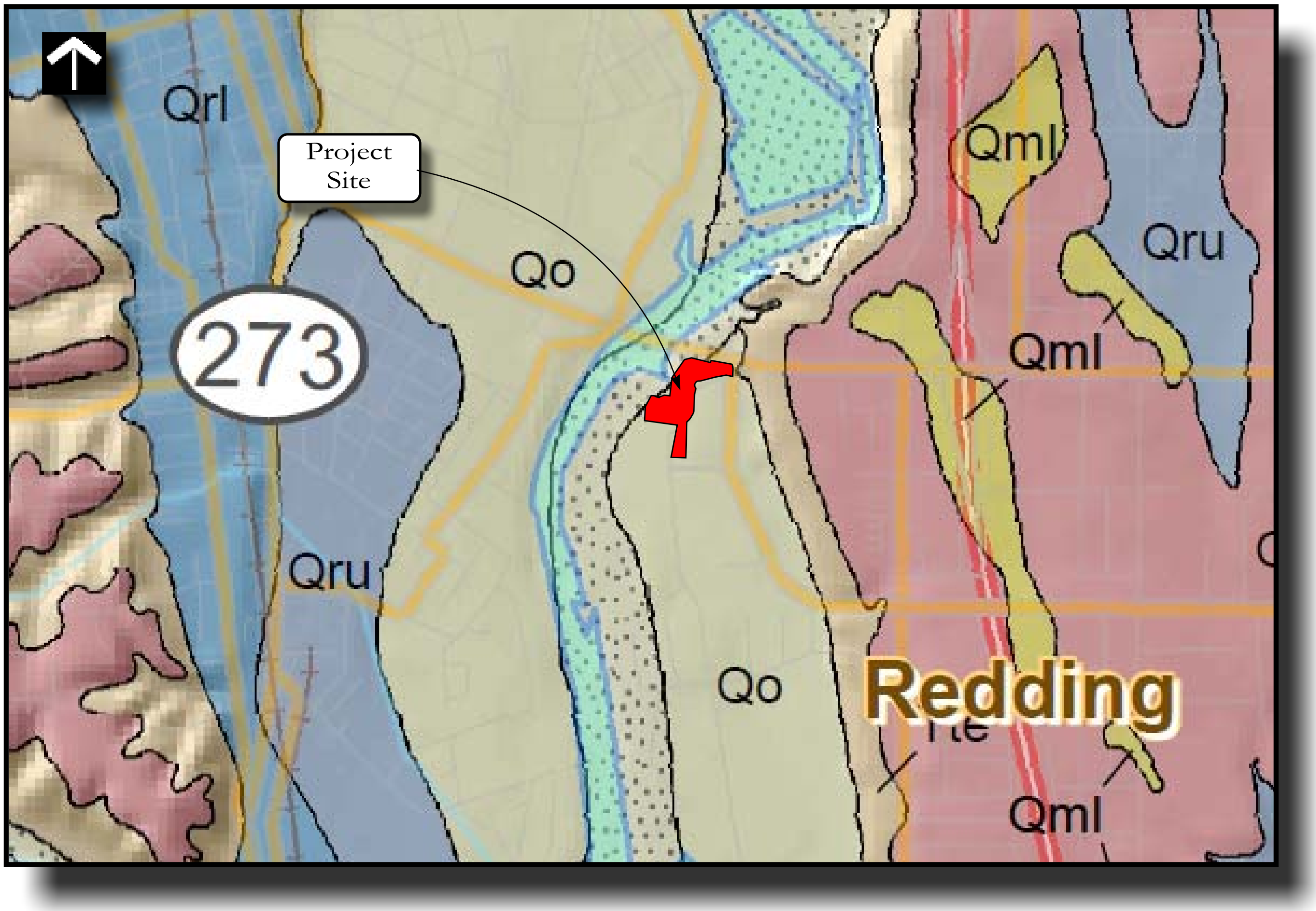
DH-5 — CGI Drill Hole Location

C — Proposed Structure

Base topographic map from OmniMeans, Ltd (2016). Building layout from NMR (2016)

Scale not determined

CGI Technical Services Inc.	GEOTECHNICAL MAP MERCY WELLNESS CENTER DIGNITY HEALTH REDDING, CALIFORNIA	Plate 3
Project No.:15-1080.84		



Qsc	Stream Channel Deposits
Qo	Overbank Deposits
Qml	Modesto Formation Lower Unit
Qrl	Riverbank Formation Lower Unit
Qru	Riverbank Formation Lower Unit
Qrb	Red Bluff Formation
Tte	Tehama Formation

?

Geologic Contact: dashed where approximate, dotted where covered, queried where uncertain

63 84 ?

Fault: showing dip of fault and and trend of striae on fault surface (arrow); bar and ball on downthrown side; dashed where approximate, dotted where concealed; queried where uncertain

Basemap from Helley & Harwood (1985)

Scale Undetermined

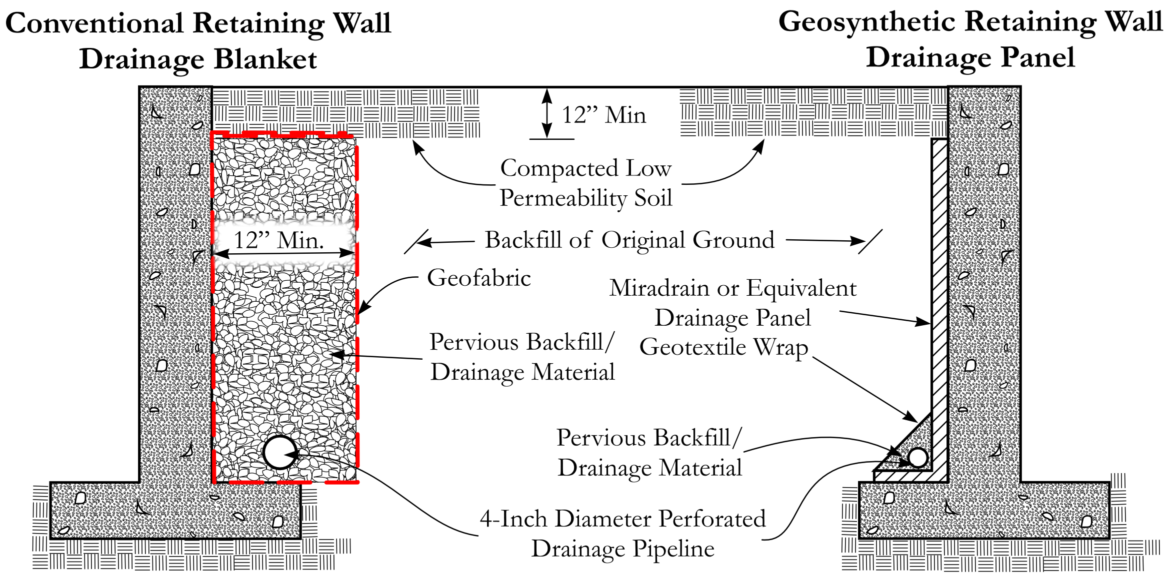
CGI TECHNICAL SERVICES INC.

Project No.: 15-1080.84

REGTIONAL GEOLOGIC MAP
MERCY WELLNESS CENTER
DIGNITY HEALTH
REDDING, CALIFORNIA

Plate

4



General Notes

Pervious backfill/drainage material should conform to Pervious Backfill per Greenbook specifications, Class 2 Permeable Material per Caltrans Standard Specifications, pea gravel having a nominal 1/4-inch diameter, or crushed stone sized between 1/4-inch and 1/2-inch.

Geosynthetic wrapping material should conform to Caltrans Standard Specifications Section 88, placed per manufacturer's specifications.

Performated drain pipe should ocnsist of 4-inch diameter Schedule 40 PVC, with two sets of 1/4-inch (maximum) diameter performations drilled axially at 90 degrees to each other, with at least one perforation per line spaced at 12 inches, and the perforations facing downward.

Drainage should be collected in a solid conduit and diverted to a proper, approved drainage facility.

CGI Technical Services Inc.	RETAINING WALL DETAILS MERCY WELLNESS CENTER DIGNITY HEALTH REDDING, CALIFORNIA	Plate 5
Project No.: 15-1080.84		

APPENDIX A
SUBSURFACE EXPLORATION

The subsurface exploration program for this study consisted of the advancement of five drill holes. The locations of the drill holes advanced for this study are shown on Plate 3. The drill holes were advanced between September 8 and 11, 2015 using a Mobil Drill B-59 drill rig provided by Diamond Core Drilling of the City of Shasta Lake, California. The drill holes were advanced using 8.25-inch diameter hollowstem augers and casing advancer.

Select samples of soils were collected from selected depth increments in each drill hole using California modified split-spoon and/or Standard Penetration Test (SPT) samplers. Samplers were driven by a 140-pound hammer situated on the drill rig, in accordance with standard test method ASTM D1586-11 Bulk samples were also obtained at selected depth intervals. Sample types and depths are presented on Plates A-2.1 through A-2.5. All samples were returned to CGI's Redding, California laboratory for testing. The results of the testing procedures are attached within Appendix B.

The exploration logs describe the earth materials encountered. The logs also show the location, exploration number, date of exploration, and the names of the logger and equipment used. A CGI geologist/engineer, using ASTM 2488 for visual soil classification, logged the explorations. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual and may change with time. The drill holes were backfilled with cement grout. Soils generated by drilling operations were disposed of on-site.

The drill hole logs are presented as Plates A-2.1 through A-2.5. A legend to the drill hole logs is presented as Plate A-1.1.

LOG OF EXPLORATION: Expl. No.

PROJECT:	CGI's Project Name	**EXPL. VENDOR:**	Expl. Subcontractor	**SURFACE ELEVATION:**	Expl. Elevation
PROJECT NO.:	CGI's Project No.	**EXPL. METHOD:**	Method of Expl.	**TOTAL DEPTH OF HOLE:**	Total Depth of Expl.
LOCATION:	General Location	**LOGGED BY:**	CGI's Logger	**DEPTH TO WATER:**	Depth to Water
START DATE:	Date Started	**CHECKED BY:**	CGI's Reviewer	**BACKFILLED WITH:**	Backfill Materials
END DATE:	Date Finished	**HAMMER TYPE:**	Type of Sample Hammer		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0			1			SAMPLES/BLOW COUNT SYMBOLS KEY Bulk Soils Sample							
			2	(24)		California modified split spoon sampler (CMSS) Brackets on blow counts indicates CMSS sample							CMSS: 2-3/8" ID, 3" OD, Driven
5			3	50:5"		Standard penetration test (SPT) sample and blow count							SPT: 1-3/8" ID, 2" OD, Driven
						No sample recovery							Blow counts are recorded as the number of blows required for one foot of sampler penetration using a 140-lb hammer falling 30 inches. Typically, sampler is driven 18" and the initial 6" discarded.
						LITHOLOGIC GRAPHICS DESCRIPTIONS FOR SOILS MATERIALS (per ASTM D2487 & D2488)							
10					GW	well graded GRAVEL							
					GP	poorly graded GRAVEL							
					GM	silty GRAVEL							
					GC	clayey GRAVEL							
15					SW	well graded SAND							Initial water level measurement
					SP	poorly graded SAND							Water level after initial measurement (may not represent stabilized water levels)
					SM	silty SAND							
					SC	clayey SAND							
20					ML	low plasticity SILT							
					MH	high plasticity SILT							
					CL	lean CLAY							Lab Abbreviations DS-direct shear; C-consolidation; GS-sieve; EI-Expansion Index; PI-Plasticity; UC-Unconfined; SC-soil chem.; SE-sand equiv.; R-R value; P-curve; PP-pocket penetrometer.
					CH	fat CLAY							
25					PT	organic soils or peat							
					OL	organic SILTS or CLAYS with low plasticity							
					OH	organic SILTS or CLAYS with high plasticity							
30					RX	ROCK							

LOG OF EXPLORATION: DH-1

PROJECT:	Dignity Wellness Center	**EXPL. VENDOR:**	Diamond Core Drilling	**SURFACE ELEVATION:**	~490 Feet
PROJECT NO.:	15-1080.84	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	23 Feet
LOCATION:	Redding, CA	**LOGGED BY:**	E. Cortez	**DEPTH TO WATER:**	Not Encountered
START DATE:	September 8, 2015	**CHECKED BY:**	J. Bianchin	**BACKFILLED WITH:**	Bentonite Chips
END DATE:	September 8, 2015	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0			B1		GM	ARTIFICIAL FILL (Qal) Silty GRAVEL with Sand, reddish brown, moist, fine to coarse grained with fine to coarse subangular to subrounded gravel, cobbles, and possibly boulders.						NP	Curve, R-value, PI
5			1	4	GC	Clayey GRAVEL with Sand, dark reddish brown, moist, loose, medium to coarse grained with fine to coarse subangular to subrounded gravel, cobbles, and possibly boulders.			10.1				
						At 7 feet: rounded to subrounded gravel.							
10			2	39		At 10 feet: dense with less silt and clay.			5.2				
					GP/ GC	ALLUVIUM (Qal) GRAVEL with Clay, reddish brown, moist, dense, medium to coarse grained with medium to coarse subrounded gravel, cobbles, and boulders.							
15			3	50:1.5"		At 14 feet: becomes brown, very dense, with fine to coarse subangular to subrounded gravel.			4.6				
20			4	50:3"		At 19 feet: becomes fine to medium grained.			5.3				
25						Bottom of Drill Hole at a Depth of 23 Feet.							

CGI TECHNICAL SERVICES INC.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

PLATE NO.: A-2.1

LOG OF EXPLORATION: DH-2

PROJECT:	Dignity Wellness Center	**EXPL. VENDOR:**	Diamond Core Drilling	**SURFACE ELEVATION:**	481 Feet
PROJECT NO.:	15-1080.84	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	50.25 Feet
LOCATION:	Redding, CA	**LOGGED BY:**	E. Cortez	**DEPTH TO WATER:**	22 Feet
START DATE:	September 9, 2015	**CHECKED BY:**	J. Bianchin	**BACKFILLED WITH:**	Bentonite Chips
END DATE:	September 9, 2015	**HAMMER TYPE:**	140-Lb		

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					CL	ARTIFICIAL FILL (Qal) Gravelly CLAY, reddish brown, moist, hard, slightly plastic, coarse grained with fine to medium angular gravel.							
5			1	55		At 5 feet: with thin (~2") layer of silty Sand.			12.6 14		30.5	11	PI, SoilChem
					CL	Sandy CLAY with Gravel, dark greyish brown, moist, hard, coarse grained with fine to medium subangular to angular gravel.							
10			2	30					15.5	35.9			GS
					SM	ALLUVIUM (Qal) Silty SAND, dark brown, moist, medium dense, very fine to fine grained.							
15			3	(25)						40.2			GS
					GM	Silty GRAVEL, dark brown, moist, very dense, very fine to fine grained with fine to medium subrounded to rounded gravel, cobbles, and possibly boulders.							
20			4	52					8.4	10.9			GS
					GW	GRAVEL, dark brown, wet, very dense, fine to coarse, subrounded to rounded and likely contains cobbles and boulders.	▼						
25			5	(50:2")									GS
					GM	Silty GRAVEL, dark brown, wet, very dense, very fine to fine grained, with fine to medium subrounded to subangular gravel, cobbles, and boulders, and withthin interbeds of Silty SAND, dark brown, wet, very dense, very fine to fine grained.							
30			6	50:4.5"					15.1	14.5			GS

CGI Technical Services Inc.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

PLATE NO.: A-2.2

LOG OF EXPLORATION: DH-2

PROJECT:	Dignity Wellness Center	**EXPL. VENDOR:**	Diamond Core Drilling	**SURFACE ELEVATION:**	481 Feet
PROJECT NO.:	15-1080.84	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	50.25 Feet
LOCATION:	Redding, CA	**LOGGED BY:**	E. Cortez	**DEPTH TO WATER:**	22 Feet
START DATE:	September 9, 2015	**CHECKED BY:**	J. Bianchin	**BACKFILLED WITH:**	Bentonite Chips
END DATE:	September 9, 2015	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
35				50:0"		At 31.5 feet: becomes coarser grained possibly indicating increased gravel, cobbles, and boulders.							
40			8	50:4"	GW/ GM	GRAVEL with Silt, dark brown, wet, very dense, fine to coarse grained, with fine to coarse subrounded to rounded gravel, and likely contains cobbles and boulders.							
45				50:0"	GW	GRAVEL, dark brown, wet, very dense, fine to coarse, subrounded to rounded and likely contains cobbles and boulders.							
50			10	50:3"									GS
						Bottom of Drill Hole at a Depth of 50.25 Feet.							

CGI TECHNICAL SERVICES INC.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

PLATE NO.: A-2.2

LOG OF EXPLORATION: DH-3

PROJECT: Dignity Wellness Center
PROJECT NO.: 15-1080.84
LOCATION: Redding, CA
START DATE: September 8, 2015
END DATE: September 8, 2015

EXPL. VENDOR: Diamond Core Drilling
EXPL. METHOD: 8.25" HSA
LOGGED BY: E. Cortez
CHECKED BY: J. Bianchin
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 480 Feet
DEPTH OF HOLE: 27.75 Feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Bentonite Chips

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					GC	ARTIFICIAL FILL (Qal) Clayey GRAVEL, brown, moist, very dense, with medium to coarse sand and medium to coarse angular gravel and cobbles.							
			1	67					10.3				
5					CL	Gravelly CLAY, reddish brown, moist, very stiff, with medium to coarse sand and fine to coarse angular to rounded gravel, cobbles, and possibly boulders.							
			2	23					15.5				
10					SM	ALLUVIUM (Qal) Silty SAND, dark brown, moist, medium dense, fine grained.							
			3	(20)					17.9				
						At 14 feet: becomes black.							
15													
			4	7					17.0				
20						At 20.5: becomes dark brown, loose At 21 feet: becomes medium dense with fine to medium rounded to subrounded gravel.							
			5	15					29.1				
25					GC	Clayey GRAVEL with Sand, grey, moist, very dense, with coarse sand and medium to coarse subangular ro rounded gravel, cobbles, and boulders.							
			6	20:3"					17.5				
						Bottom of Drill Hole at a Depth of 27.75 Feet.							

LOG OF EXPLORATION: DH-4

PROJECT: Dignity Wellness Center
PROJECT NO.: 15-1080.84
LOCATION: Redding, CA
START DATE: September 10, 2015
END DATE: September 10, 2015

EXPL. VENDOR: Diamond Core Drilling
EXPL. METHOD: 8.25" HSA
LOGGED BY: E. Cortez
CHECKED BY: J. Bianchin
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 470 Feet
DEPTH OF HOLE: 25.5 Feet
DEPTH TO WATER: 10 Feet
BACKFILLED WITH: Bentonite Chips

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					GM	ARTIFICIAL FILL (Qal) Silty GRAVEL with Sand, reddish brown, moist, with medium to coarse sand and medium to coarse angular gravel and cobbles.							
					GC	Clayey GRAVEL, dark reddish brown, moist, dense with medium to coarse sand and fine to coarse subrounded to rounded gravel, cobbles and possible boulders.							
5			1	43					8.0				
					GC	ALLUVIUM (Qal) Clayey GRAVEL, dark reddish brown, moist, dense with medium to coarse sand and fine to coarse subrounded to rounded gravel, cobbles and possible boulders.							
			2	50:4"					8.3				
10					GM	Silty GRAVEL, dark grey, wet, with medium to coarse sand and medium to coarse angular to subrounded gravel, cobbles, and boulders.	▼						
			3	50:3"									
15													
						At 16.5 Feet: becomes dense.							
			4	40					11.2				
20													
					SM	Silty SAND, grey, wet, very dense, medium to coarse grained with coarse subrounded gravel.							
			5	50:5"	GC	Clayey GRAVEL, grey, wet, very dense with fine to medium sand and fine to coarse subrounded to rounded gravel, cobbles and possible boulders.			17.2				
25						Bottom of Drill Hole at a Depth of 25.5 Feet							

LOG OF EXPLORATION: DH-5

PROJECT:	Dignity Wellness Center	**EXPL. VENDOR:**	Diamond Core Drilling	**SURFACE ELEVATION:**	~475 Feet
PROJECT NO.:	15-1080.84	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	23.6 Feet
LOCATION:	Redding, CA	**LOGGED BY:**	E. Cortez	**DEPTH TO WATER:**	14 Feet
START DATE:	September 11, 2015	**CHECKED BY:**	J. Bianchin	**BACKFILLED WITH:**	Bentonite Chips
END DATE:	September 11, 2015	**HAMMER TYPE:**	140-Lb		

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0					af GM	ARTIFICIAL FILL (Qal) Asphaltic Concrete (5")							
						Aggregate Base (10")							
5			1	12:0"		Silty GRAVEL, dark grey to black, moist, very dense, with medium to coarse sand and fine to coarse subangular to subrounded gravel, cobbles, and possibly boulders.							
					GM	ALLUVIUM (Qal) Silty GRAVEL, dark grey to black, wet, dense, with medium to coarse sand and fine to coarse subangular to subrounded gravel, cobbles, and possibly boulders.							
10			2	40					7.2				
						At 11.5 feet: becomes very dense.							
15			3	50:4"			▼ (14 ft)		8.4				
					GW/ GM	GRAVEL with Silt, dark grey, wet, very dense, with coarse sand and fine to coarse subangular to rounded gravel, cobbles, and boulders.							
20			4	50:4"									
			5	60:2"	GW	GRAVEL, dark grey, wet, very dense, fine to coarse, subrounded to rounded, with possible cobbles and boulders.			1.5				
25						Bottom of Drill Hole at a Depth of 23.6 Feet							

APPENDIX B
LABORATORY TESTING

Laboratory Analyses
Laboratory tests were performed on selected bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed under procedures described in one of the following references:

- ASTM Standards for Soil Testing, latest revision;
- Lambe, T. William, Soil Testing for Engineers, Wiley, New York, 1951;
- Laboratory Soils Testing, U.S. Army, Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906, November 30, 1970.

Plasticity Index Tests
Atterberg Limits (plastic limit, liquid limit, and plasticity index) tests were performed on two selected samples in accordance with standard test method ASTM D4318. Results of the Atterberg Limits tests are presented in the report text and on the attached plate labeled Atterberg Limits Tests.

Corrosion Testing
Soil chemistry tests were performed to evaluate the pH, resistivity, chloride, and sulfate concentrations within two samples of on-site soils tested. The results of the test are attached to this appendix.

Grain Size Distribution
Grain size distribution was determined for selected soil samples in accordance with standard test method ASTM D1140. The grain size distribution data are shown on the attached plate labeled *Laboratory Sieve Analysis*.

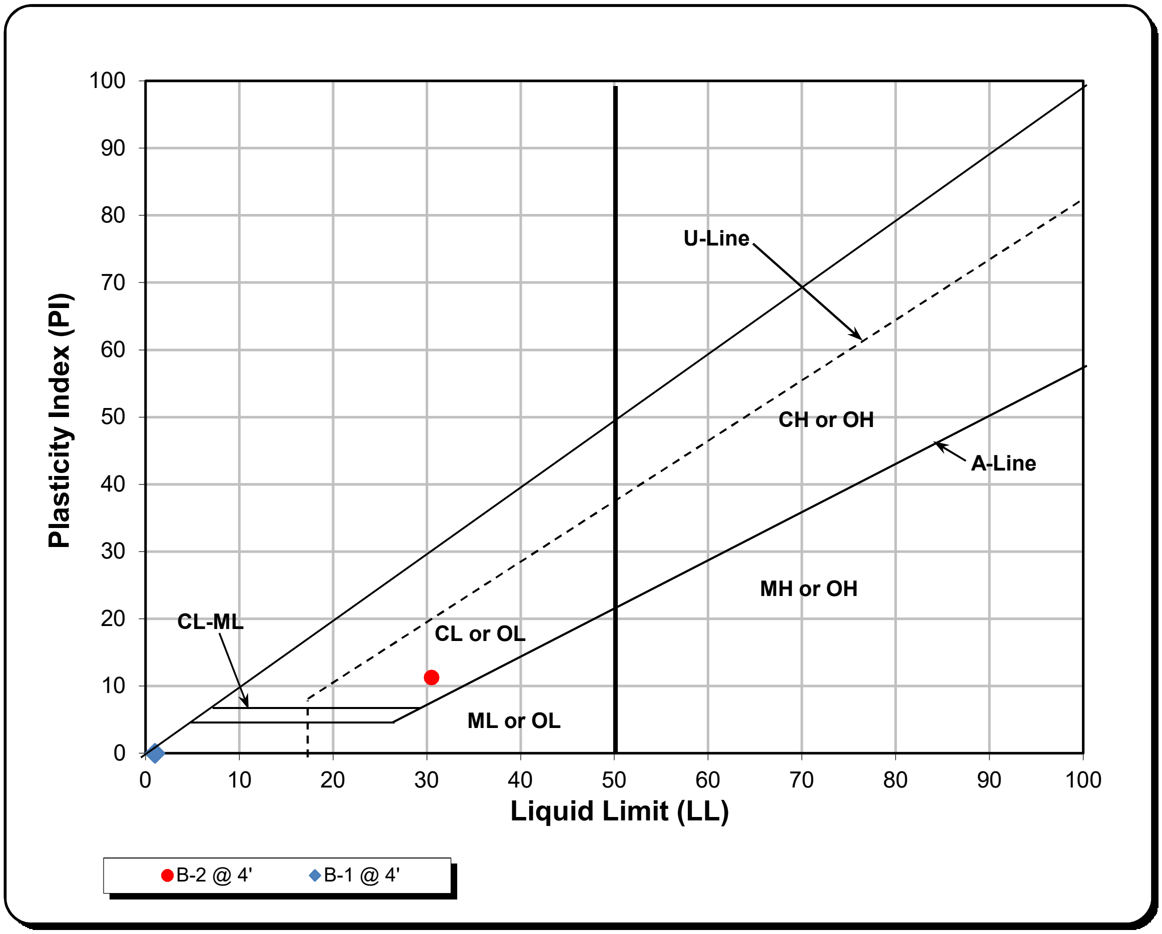
Moisture Density Relations
The compaction characteristics of a selected bulk soil sample were estimated in accordance with standard test method ASTM D1557. The results of the compaction test are shown on the attached plate labeled *Moisture Density Relationship*.

Resistance R-Value Test
One R-value test was performed on a selected relatively undisturbed sample using standard test method California Test Method 301. The results of the test are presented on the attached plate labeled *R-Value*.

CGI Technical Services Inc.

ATTERBERG LIMITS TESTS

Client:	Dignity Health	**Job No.:**	15-1080.84
Project:	Mercy Wellnes Campus	**Lab No.:**	8382
Location:	Redding, California		
Sampled By:	AB	**Date Sampled:**	8-Sep-15
Received By:	AE	**Date Received:**	8-Sep-15
Tested By:	AE	**Date Tested:**	23-Nov-15
Reviewed By:	AB	**Date Reviewed:**	23-Nov-15



LEGEND			CLASSIFICATION	ATTERBERG LIMITS TEST RESULTS		
Location	Depth, ft	Sample No.		Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
B-2	4'	1B	Gravelly Clay	31	19	11
B-1	4'	1	Silty Gravel	-	-	NP

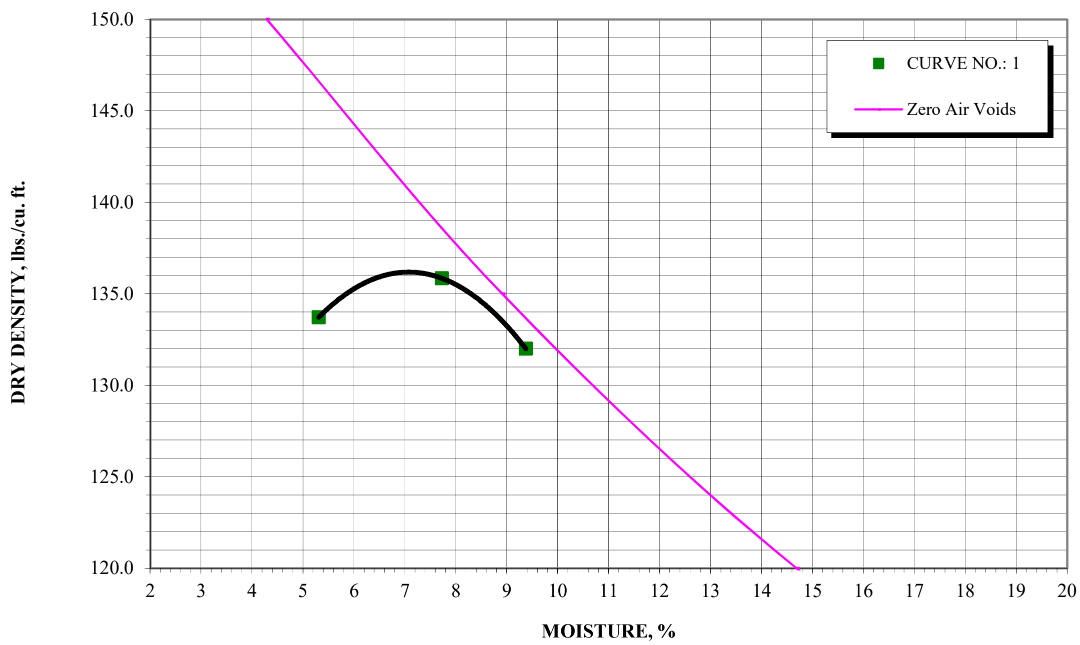
ASTM D4318 & D2487

CGI TECHNICAL SERVICES INC.

MOISTURE DENSITY RELATIONSHIP

Client: Dignity Health **Job No.:** 15-1080.84
Project: Mercy Wellnes Campus **Lab No.:** 8382
Location: Redding, California
Material Description: Silty Sand with Gravel **CURVE NO.:** 1
Material Supplier: On-Site
Material Type: Native
Sample Location: B-1 @ 3'
Sampled By: EC **Date Sampled:** 8-Sep-15
Received By: AE **Date Received:** 8-Sep-15
Tested By: JFB **Date Tested:** 9-Dec-15
Reviewed By: AB **Date Reviewed:** 14-Dec-15
Test Procedure: ASTM **Method:** D-1557

Oversized Material (%), +3/4": 9.2% **Correction Required:** [X] yes [ ] no



SPECIMEN	A	B	C	D
MOISTURE AT TEST, %	5.3	7.7	9.4	
DRY DENSITY	133.7	135.9	132.0	

Maximum Dry Density, PCF 136.2 **@ Optimum Moisture, %** 7.1

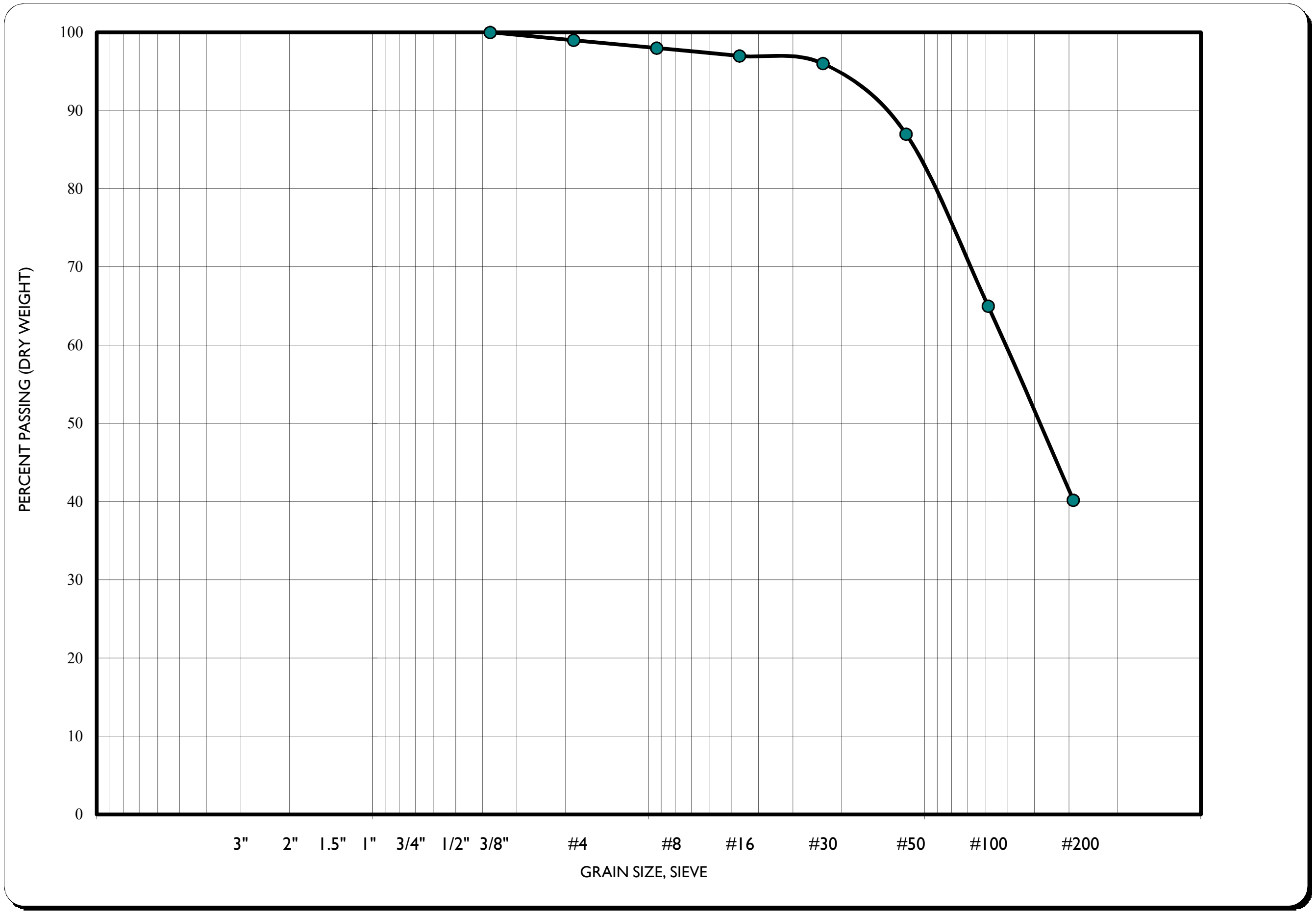
CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client: Dignity Health
Project: Mercy Wellnes Campus
Material Type: Silty Sand to sandy Silt
USCS: SM-ML

Sample Location: B-2 @ 19'
Sampled By: EC
Tested By: JFB

Job No.: 15-1080.84
Lab No.: 8382
Date Received: 8-Sep-15
Date Tested: 5-Oct-15
Date Reviewed: 5-Oct-15



SIEVE ANALYSIS		
Sieve Size Standard	Grain Size (mm)	Percent Passing
6	150.00	
2	50.00	
1.5	37.50	
1"	25.00	
3/4"	19.00	
1/2"	12.50	
3/8"	9.50	100
#4	4.75	99
#8	2.36	98
#16	1.18	97
#30	600um	96
#50	300um	87
#100	150um	65
#200	75um	40.2

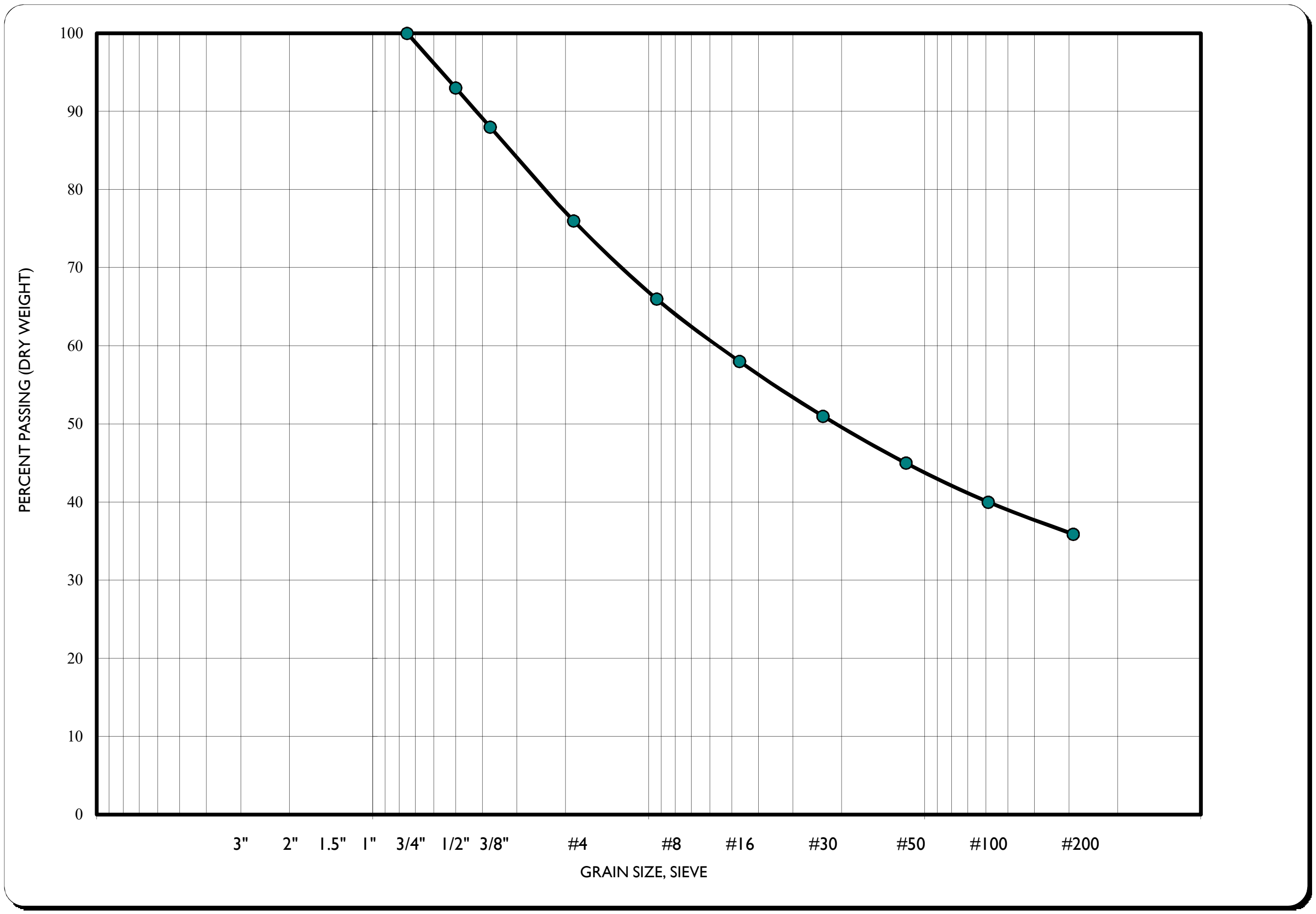
CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client: Dignity Health
Project: Mercy Wellnes Campus
Material Type: Silty Sand to sandy Silt
USCS: SM

Sample Location: B-2 @ 9'
Sampled By: EC
Tested By: JFB

Job No.: 15-1080.84
Lab No.: 8382
Date Received: 8-Sep-15
Date Tested: 5-Oct-15
Date Reviewed: 5-Oct-15

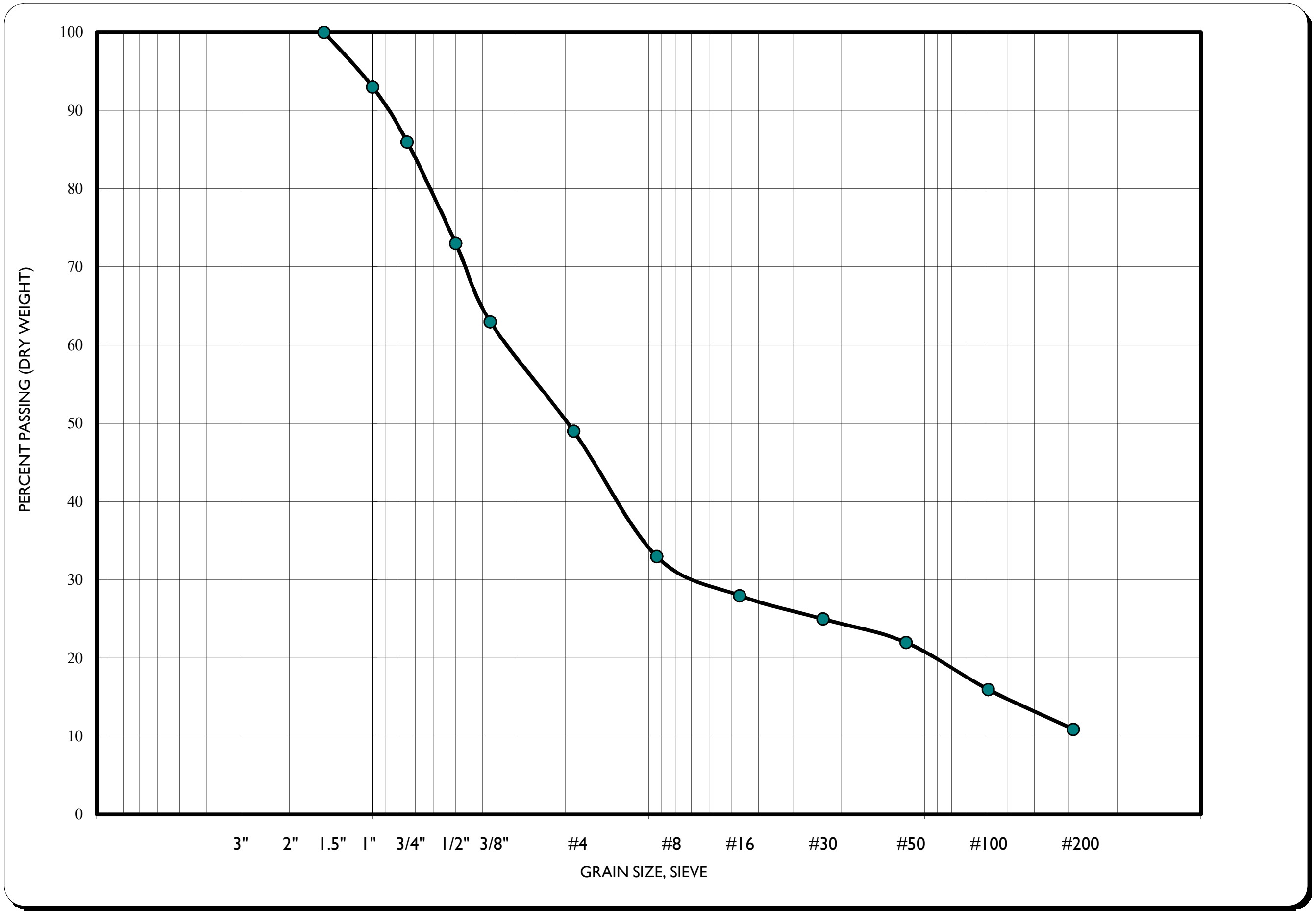


SIEVE ANALYSIS		
Sieve Size Standard	Grain Size (mm)	Percent Passing
6	150.00	
2	50.00	
1.5	37.50	
1"	25.00	
3/4"	19.00	100
1/2"	12.50	93
3/8"	9.50	88
#4	4.75	76
#8	2.36	66
#16	1.18	58
#30	600um	51
#50	300um	45
#100	150um	40
#200	75um	35.9

CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client:	Dignity Health			**Job No.:**	15-1080.84
Project:	Mercy Wellnes Campus			**Lab No.:**	8382
Material Type:	Silty Sand to sandy Silt	**Sample Location:**	B-2 @ 19.5'	**Date Received:**	8-Sep-15
USCS:	Gravelly Sand	**Sampled By:**	EC	**Date Tested:**	5-Oct-15
		Tested By:	JFB	**Date Reviewed:**	5-Oct-15



SIEVE ANALYSIS

Sieve Size Standard	Grain Size (mm)	Percent Passing
6	150.00	
2	50.00	
1.5	37.50	100
1"	25.00	93
3/4"	19.00	86
1/2"	12.50	73
3/8"	9.50	63
#4	4.75	49
#8	2.36	33
#16	1.18	28
#30	600um	25
#50	300um	22
#100	150um	16
#200	75um	10.9

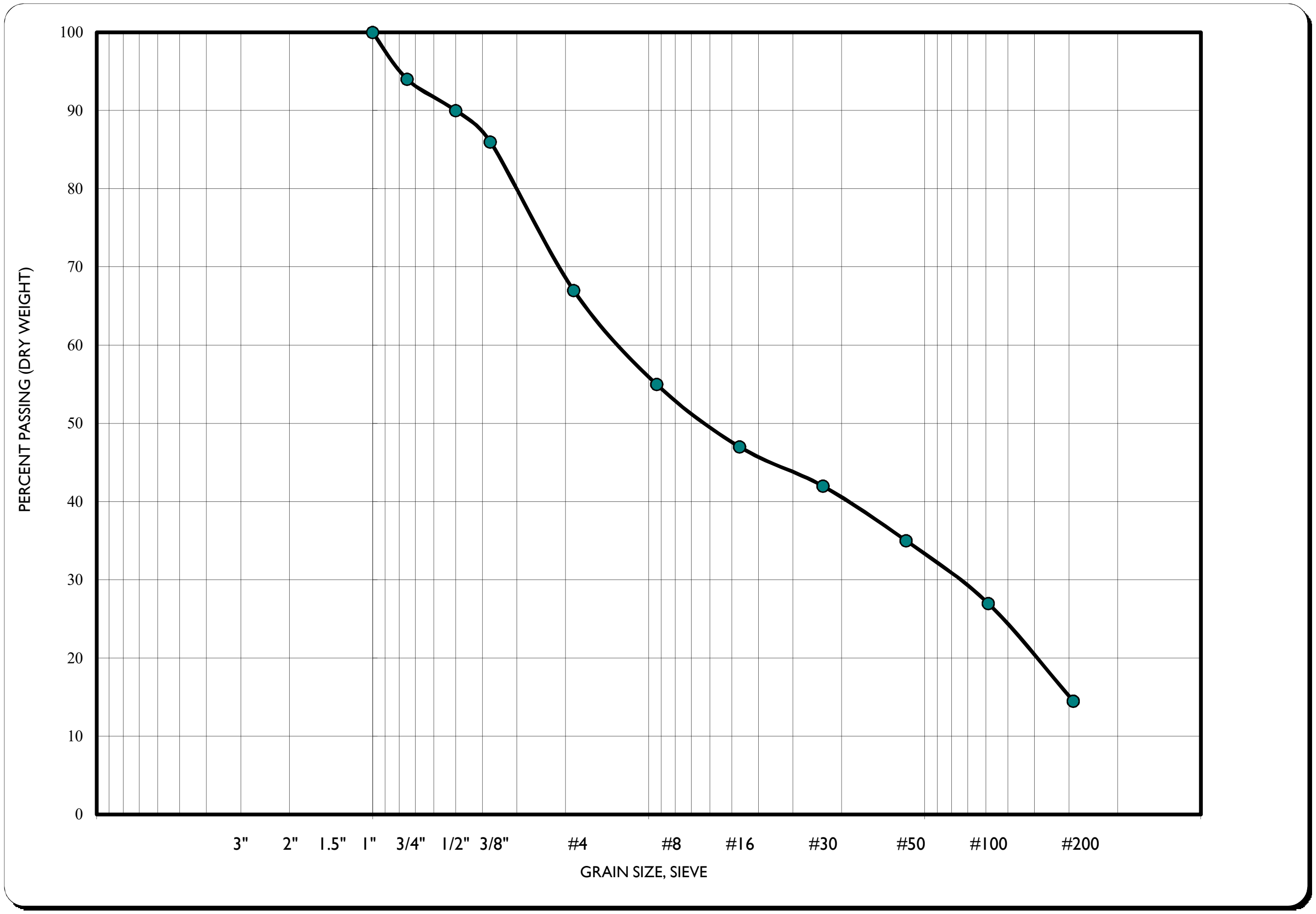
CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client: Dignity Health
Project: Mercy Wellnes Campus
Material Type: Silty Sand to sandy Silt
USCS: Gravelly Sand

Sample Location: B-2 @ 29'
Sampled By: EC
Tested By: JFB

Job No.: 15-1080.84
Lab No.: 8382
Date Received: 8-Sep-15
Date Tested: 5-Oct-15
Date Reviewed: 5-Oct-15



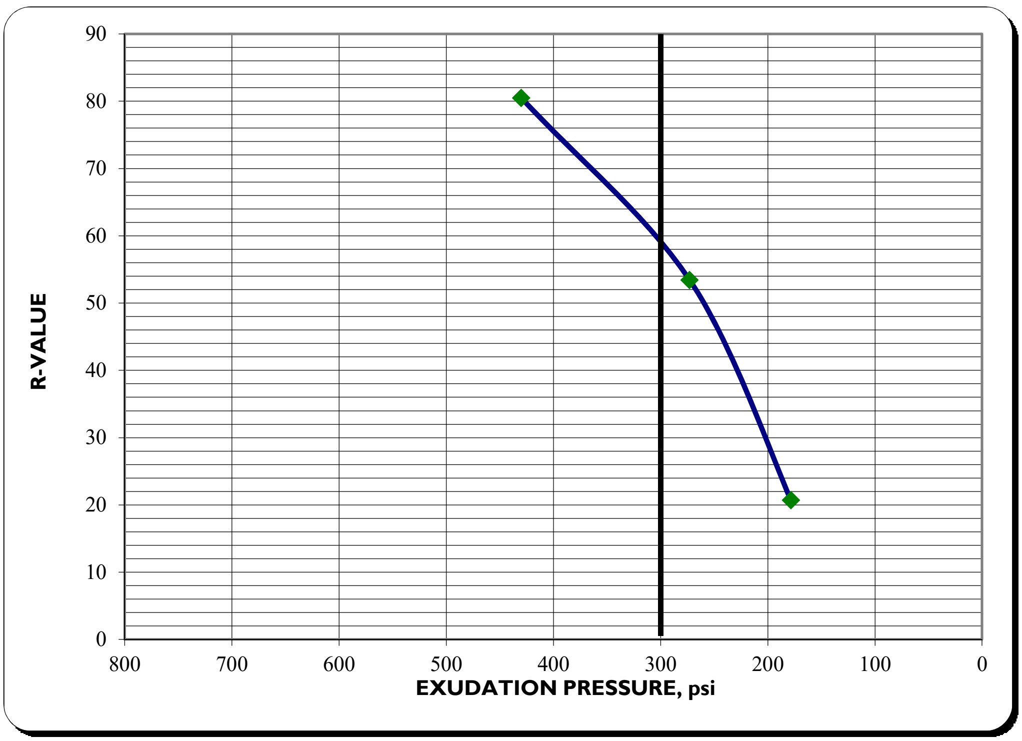
SIEVE ANALYSIS		
Sieve Size Standard	Grain Size (mm)	Percent Passing
6	150.00	
2	50.00	
1.5	37.50	
1"	25.00	100
3/4"	19.00	94
1/2"	12.50	90
3/8"	9.50	86
#4	4.75	67
#8	2.36	55
#16	1.18	47
#30	600um	42
#50	300um	35
#100	150um	27
#200	75um	14.5

CGI TECHNICAL SERVICES INC.

Resistance Value

Client:	Dignity Health	**Job No.:**	15-1080.84
Project:	Mercy Wellnes Campus	**Lab No.:**	8382
Location:	Redding, California		
Material Type:	Silty Sand with Gravel		
Material Supplier:	On-Site	**Date Sampled:**	9/8/2015
Material Source:	Native	**Date Received:**	9/8/2015
Sample Location:	B-1 @ 0-4'	**Date Tested:**	9/10/2015
Sampled By:	EC	**Date Reviewed:**	9/10/2015

Test Procedure: Caltrans **Method:** 301



	A	B	C
Moisture (%):	3.9	7.1	7.6
Dry Density (pcf):	135.0	139.9	144.9
Expansion Pressure (psf):	0.0143	0.0108	0.0143
Exudation Pressure (psi):	430	273	178
Resistance Value:	80	53	21

R - VALUE AT 300 PSI EXUDATION PRESSURE	59

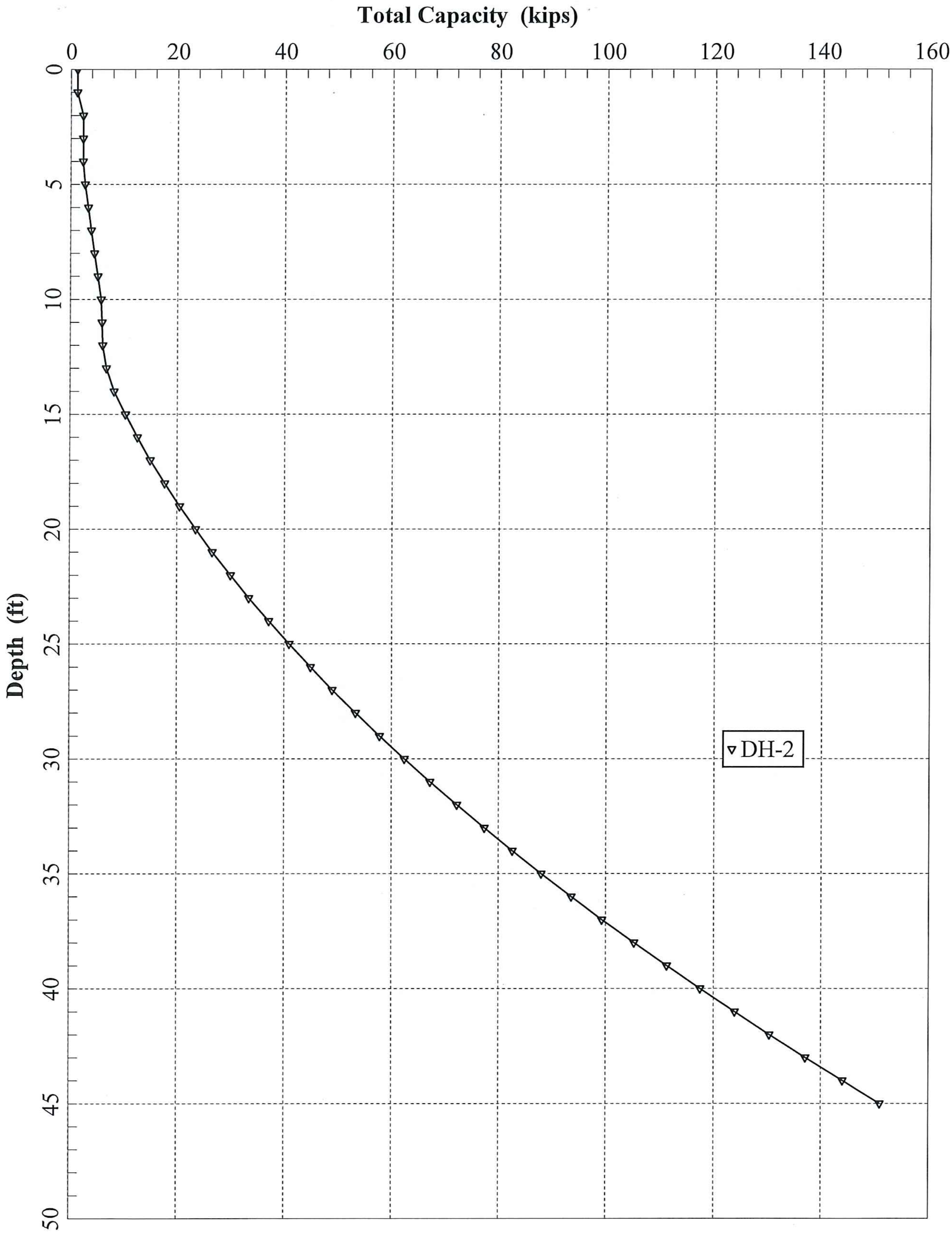
Table 1 - Laboratory Tests on Soil Samples

CGI Technical Services
Mercy Wellness Campus
Your #15-1080.84, HDR Lab #15-0948LAB
9-Dec-15

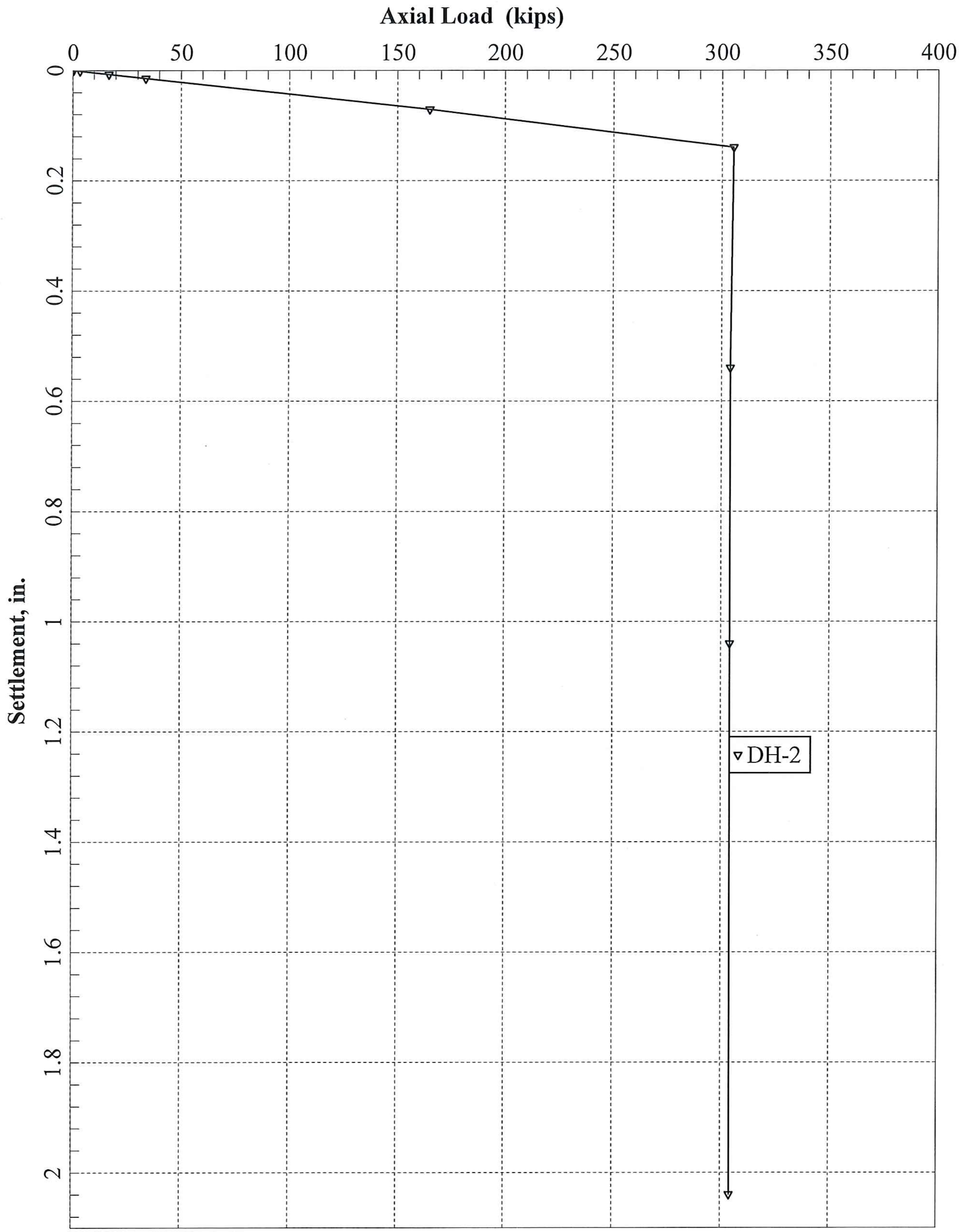
Sample ID			B-1 @ 0-5' Silty Gravel	B-2 @ 4' Silty Clay
Resistivity		**Units**		
as-received		ohm-cm	64,000	>4,400,000
saturated		ohm-cm	11,600	3,640
pH			7.0	7.6
Electrical Conductivity		mS/cm	0.07	0.08
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	43	24
magnesium	Mg^{2+}	mg/kg	7.7	18
sodium	Na^{1+}	mg/kg	22	47
potassium	K^{1+}	mg/kg	9.2	3.4
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	153	192
fluoride	F^{1-}	mg/kg	18	11
chloride	Cl^{1-}	mg/kg	13	14
sulfate	SO_4^{2-}	mg/kg	41	45
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	9.5	4.5
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

Pile Analysis



Total Capacity of pile vs. Depth



Settlement vs. Axial Load

===

APILE for Windows, Version 2015.7.3

Serial Number : 166868942

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.

===

This program is licensed to :

Curry Group, Inc.
Redding, WA

Path to file locations : C:\Users\abahloul\Desktop\Dignity Wellness Center\
Name of input data file : Dignity Wellness Center DH_2.ap7d
Name of output file : Dignity Wellness Center DH_2.ap7o
Name of plot output file : Dignity Wellness Center DH_2.ap7p

Time and Date of Analysis

Date: April 05, 2016 Time: 15:08:44

1

****
* INPUT INFORMATION *
****

Dignity Wellness Center

DESIGNER : AB

JOB NUMBER : 15-1080.84

METHOD FOR UNIT LOAD TRANSFERS :

- FHWA (Federal Highway Administration)
  Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)

TYPE OF LOADING :
- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.280E+08 PSI
- CROSS SECTION AREA = 100.00 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 45.00 FT.
- PILE STICKUP LENGTH, PSL = 0.00 FT.
- ZERO FRICTION LENGTH, ZFL = 5.00 FT.
- PERIMETER OF PILE = 40.00 IN.
- TIP AREA OF PILE = 100.00 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	CLAY	0.00	50.00	0.00	0.00
12.00	CLAY	0.00	50.00	0.00	0.00
12.00	SAND	0.00	60.00	0.00	0.00
17.00	SAND	0.00	60.00	0.00	0.00
17.00	SAND	0.00	60.00	0.00	0.00
50.00	SAND	0.00	60.00	0.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.20E+03	0.75	0.00	0.00	0.00	0.00
0.10E+08*	0.20E+03	0.75	0.00	0.00	0.00	0.00
0.20E+02	0.10E-06	0.00	0.00	20.00	0.00	0.00

0.20E+02	0.10E-06	0.00	0.00	20.00	0.00	0.00
0.20E+02	0.10E-06	0.00	0.00	40.00	0.00	0.00
0.20E+02	0.10E-06	0.00	0.00	40.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	0.500	0.500
12.00	0.500	0.500
12.00	0.500	0.500
17.00	0.500	0.500
17.00	0.500	0.500
50.00	0.500	0.500

1

****
* COMPUTATION RESULT *
****

****
* FED. HWY. METHOD *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	1.2	1.2
1.00	0.0	1.2	1.2
2.00	0.0	2.3	2.3
3.00	0.0	2.3	2.3
4.00	0.0	2.3	2.3
5.00	0.3	2.3	2.7
6.00	0.9	2.3	3.3
7.00	1.6	2.3	3.9
8.00	2.2	2.3	4.5
9.00	2.8	2.3	5.2
10.00	3.5	2.3	5.8
11.00	4.1	1.9	6.0
12.00	4.9	1.2	6.1
13.00	6.3	0.4	6.8
14.00	8.3	0.0	8.3
15.00	10.4	0.0	10.4
16.00	12.6	0.0	12.6
17.00	15.0	0.0	15.0

18.00	17.7	0.0	17.7
19.00	20.5	0.0	20.5
20.00	23.5	0.0	23.5
21.00	26.6	0.0	26.6
22.00	30.0	0.0	30.0
23.00	33.4	0.0	33.4
24.00	37.1	0.0	37.1
25.00	40.9	0.0	40.9
26.00	44.9	0.0	44.9
27.00	49.0	0.0	49.0
28.00	53.3	0.0	53.3
29.00	57.8	0.0	57.8
30.00	62.4	0.0	62.4
31.00	67.2	0.0	67.2
32.00	72.2	0.0	72.2
33.00	77.3	0.0	77.3
34.00	82.6	0.0	82.6
35.00	88.0	0.0	88.0
36.00	93.6	0.0	93.6
37.00	99.3	0.0	99.3
38.00	105.2	0.0	105.2
39.00	111.3	0.0	111.3
40.00	117.5	0.0	117.5
41.00	123.9	0.0	123.9
42.00	130.4	0.0	130.4
43.00	137.1	0.0	137.1
44.00	144.0	0.0	144.0
45.00	150.9	0.0	150.9

NOTES:
- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
  IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
  OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

```
**************************************************
* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING                      *
**************************************************
```

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.2037E-01
			0.0000E+00	0.3947E-01
			0.0000E+00	0.7257E-01
			0.0000E+00	0.1019E+00
			0.0000E+00	0.1273E+00
			0.0000E+00	0.2546E+00
			0.0000E+00	0.3820E+00

		0.0000E+00	0.6366E+00
		0.0000E+00	0.2546E+01
2	10	0.6025E+01	
		0.0000E+00	0.0000E+00
		0.7849E+00	0.2037E-01
		0.1308E+01	0.3947E-01
		0.1962E+01	0.7257E-01
		0.2355E+01	0.1019E+00
		0.2616E+01	0.1273E+00
		0.2355E+01	0.2546E+00
		0.2355E+01	0.3820E+00
		0.2355E+01	0.6366E+00
		0.2355E+01	0.2546E+01
3	10	0.1196E+02	
		0.0000E+00	0.0000E+00
		0.1397E+01	0.2037E-01
		0.2328E+01	0.3947E-01
		0.3492E+01	0.7257E-01
		0.4191E+01	0.1019E+00
		0.4656E+01	0.1273E+00
		0.4191E+01	0.2546E+00
		0.4191E+01	0.3820E+00
		0.4191E+01	0.6366E+00
		0.4191E+01	0.2546E+01
4	10	0.1200E+02	
		0.0000E+00	0.0000E+00
		0.7014E+00	0.1000E-01
		0.1403E+01	0.2000E-01
		0.2805E+01	0.4000E-01
		0.4208E+01	0.6000E-01
		0.5611E+01	0.8000E-01
		0.6312E+01	0.9000E-01
		0.7014E+01	0.1000E+00
		0.7014E+01	0.5000E+00
		0.7014E+01	0.2000E+01
5	10	0.1453E+02	
		0.0000E+00	0.0000E+00
		0.9095E+00	0.1000E-01
		0.1819E+01	0.2000E-01
		0.3638E+01	0.4000E-01
		0.5457E+01	0.6000E-01
		0.7276E+01	0.8000E-01
		0.8186E+01	0.9000E-01
		0.9095E+01	0.1000E+00
		0.9095E+01	0.5000E+00
		0.9095E+01	0.2000E+01
6	10	0.1696E+02	
		0.0000E+00	0.0000E+00
		0.1051E+01	0.1000E-01
		0.2101E+01	0.2000E-01
		0.4203E+01	0.4000E-01
		0.6304E+01	0.6000E-01
		0.8406E+01	0.8000E-01
		0.9457E+01	0.9000E-01
		0.1051E+02	0.1000E+00

		0.1051E+02	0.5000E+00
		0.1051E+02	0.2000E+01
7	10	0.1700E+02	
		0.0000E+00	0.0000E+00
		0.1134E+01	0.1000E-01
		0.2269E+01	0.2000E-01
		0.4537E+01	0.4000E-01
		0.6806E+01	0.6000E-01
		0.9075E+01	0.8000E-01
		0.1021E+02	0.9000E-01
		0.1134E+02	0.1000E+00
		0.1134E+02	0.5000E+00
		0.1134E+02	0.2000E+01
8	10	0.3353E+02	
		0.0000E+00	0.0000E+00
		0.2230E+01	0.1000E-01
		0.4460E+01	0.2000E-01
		0.8919E+01	0.4000E-01
		0.1338E+02	0.6000E-01
		0.1784E+02	0.8000E-01
		0.2007E+02	0.9000E-01
		0.2230E+02	0.1000E+00
		0.2230E+02	0.5000E+00
		0.2230E+02	0.2000E+01
9	10	0.4996E+02	
		0.0000E+00	0.0000E+00
		0.2913E+01	0.1000E-01
		0.5826E+01	0.2000E-01
		0.1165E+02	0.4000E-01
		0.1748E+02	0.6000E-01
		0.2331E+02	0.8000E-01
		0.2622E+02	0.9000E-01
		0.2913E+02	0.1000E+00
		0.2913E+02	0.5000E+00
		0.2913E+02	0.2000E+01

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.4340E-08	0.6366E-02
0.8681E-08	0.1273E-01
0.1736E-07	0.2546E-01
0.3472E-07	0.1655E+00
0.5208E-07	0.5348E+00
0.6250E-07	0.9295E+00
0.6944E-07	0.1273E+01
0.6944E-07	0.1910E+01
0.6944E-07	0.2546E+01

LOAD VERSUS SETTLEMENT CURVE

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TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.3361E+00	0.1419E-03	0.6818E-10	0.1000E-03
0.3361E+01	0.1419E-02	0.6818E-09	0.1000E-02
0.1681E+02	0.7097E-02	0.3409E-08	0.5000E-02
0.3361E+02	0.1419E-01	0.6818E-08	0.1000E-01
0.1649E+03	0.7089E-01	0.2040E-07	0.5000E-01
0.3054E+03	0.1398E+00	0.2660E-07	0.1000E+00
0.3041E+03	0.5397E+00	0.5045E-07	0.5000E+00
0.3041E+03	0.1040E+01	0.6392E-07	0.1000E+01
0.3041E+03	0.2040E+01	0.6944E-07	0.2000E+01