REPORT OF GEOTECHNICAL INVESTIGATION
CITY OF INGLEWOOD CIVIC CENTER SEISMIC UPGRADES
CITY HALL, MAIN LIBRARY AND POLICE DEPARTMENT BUILDINGS
INGLEWOOD, CALIFORNIA

Prepared for

KPFF CONSULTING ENGINEERS, INC.
700 S Flower Street, Suite 2100
Los Angeles, California 90017

Prepared by

GROUP DELTA CONSULTANTS, INC.
370 Amapola Avenue, Suite 212
Torrance, California 90501

Project No. LA-1474
March 3, 2021 (Rev. 1 April 22, 2021)

KPFF Consulting Engineers, Inc.
700 S Flower Street, Suite 2100
Los Angeles, California 90017

March 3, 2021 (Rev. 1 April 22, 2021)
Group Delta Project No. LA1474

Attention: Dr. Maikol Del Carpio, Project Manager

SUBJECT: REPORT OF GEOTECHNICAL INVESTIGATION
City of Inglewood Civic Center Seismic Upgrades
City Hall, Main Library, and Police Department Buildings
1 W. Manchester Boulevard
Inglewood, California

Dear Dr. Del Carpio:

Group Delta Consultants (Group Delta) is submitting this geotechnical investigation report for the planned City of Inglewood Civic Center Seismic Upgrades. The project consists of the evaluation, design and construction of voluntary seismic retrofits for the existing City Hall, Main Library, and Police Department buildings. Group Delta prepared this report to provide geotechnical information to support the voluntary seismic retrofit in accordance with the referenced Request for Proposals (City of Inglewood, 2020).

For this Revision 1 report, belled caisson tensile capacity curves have been included. The remaining recommendations are unchanged from our March 3, 2021 report and remain valid.

We appreciate this opportunity to be of continued professional service. Please contact us with questions or comments, or if you need anything else.

GROUP DELTA CONSULTANTS

L. Keykhosropour
Lohrasb Keykhosropour, Ph.D.
Staff Engineer

Ethan Tsai, P.E., G.E.
Associate Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER ETHAN TSAI GE 3004 Exp 12/31/21 GEOTECHNICAL STATE OF CALIFORNIA

Kristen Chang, P.E. 70959
Associate Engineer

REGISTERED PROFESSIONAL ENGINEER KRISTEN E. CHANG C 70959 EXP. 6-30-2021 CIVIL STATE OF CALIFORNIA

Distribution: Addressee, Dr. Maikol Del Carpio (maikol.delcarpio@kpff.com)

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation by Group Delta Consultants (Group Delta) to support the seismic evaluation and retrofit of three City of Inglewood Civic Center buildings (City Hall, Main Library, and Police Department) located Inglewood, California. The location of the site is shown in Figure 1, Site Vicinity Map.

The purpose of this report is to provide geotechnical information to support the seismic evaluation and retrofit design for the project. This report provides interpretations of the geologic and geotechnical conditions observed and recommendations to support the seismic retrofit. Group Delta developed the recommendations from reviewing the previous studies referenced in this report, recent supplemental subsurface exploration, geologic and geotechnical engineering interpretation and analyses, and our previous experience with similar geologic conditions.

1.1 Scope of Services

Group Delta provided the following scope of services:

- Review of available background information, including geologic maps, fault maps, hazard maps, topographic and aerial photographs, as well as available existing geotechnical reports prepared by Amec (2017) and their legacy company, LeRoy Crandall and Associates, as-built plans, and documents pertaining to the site conditions. Figure 2 shows the approximate locations of the relevant historical borings. Appendix B contains relevant Previous Boring Records and pertinent laboratory test data.
- A geotechnical supplemental field investigation to obtain supplemental geotechnical data consisting of one cone penetration tests (CPT) and two seismic cone penetration tests (SCPT) to a maximum depth of about 87 feet below ground surface. Figure 2 shows the approximate locations of these explorations. Appendix A provides current field exploration results.
- Development of site-specific ARS for two seismic hazard levels (BSE-2E and BSE-1E) for the site.
- Development of site-specific ground motion time histories for use in nonlinear dynamic analyses the City Hall structure for the two seismic hazard levels (Appendix C).
- Engineering analysis of the field and laboratory data to develop geotechnical parameters for evaluation of the existing foundations, as well as design of new foundation elements.
- Preparation of this report with our findings, conclusions and recommendations.

1.2 Site Description

The project site is L-shaped and bounded by Manchester Boulevard to the South, South La Brea Avenue to the East, East Regent Street to the North and, North Grevillea Avenue, West Queen Street, and South Fir Avenue to the West. The approximate centroid of the site is at a latitude of 33.9631º and longitude of -118.3549º. The site gently slopes from northeast of the site down to the southwest toward S Fir Avenue, with the elevation ranging approximately from 130 feet above Mean Sea Level (MSL) at the northeast to about 115 feet above MSL at the southwest.

The Police Department, City Hall, and the Main Library Buildings, all connected through pedestrian bridges, occupy the site. The City Hall Building, a nine-story structure, is located in the middle of the site as shown in Figure 2. The Police Department building is located in the northeastern side of the site, and the Main Library is located in the southeastern side of the site. Additional improvements include, walkways, paved parking lots, landscaped areas with grass, shrubs and trees. Various subsurface utilities exist on the site.

1.3 Proposed Seismic Upgrades

The project consists of the design and construction of seismic retrofits for the three Civic Center Buildings. The primary purpose of this voluntary seismic retrofit is to improve the seismic performance of the existing structures by correcting identified structural deficiencies against the selected hazard levels in ASCE 41-17 for ground shaking.

The proposed development for the City Hall Building includes expanding and thickening of existing shallow foundations, strengthening of structural elements with fiber reinforced polymer (FRP) and adding rebars (longitudinal and shear), and adding new concrete beams and walls. The seismic retrofits for the Police Department Building includes enlargement of concrete columns and footings, adding rebars to structural elements, addition of new concrete walls, and strengthening of existing concrete walls, beams and slabs with FRP. The proposed development for the Main Library Building includes enlargement of existing footings, adding grade-beams, addition of new concrete walls, adding new rebars to existing beams and footings, and widening of existing beams.

1.4 As-Built Information

The City of Inglewood Civic Center buildings, including City Hall, the Police Department and the Main Library, were constructed in 1971. City Hall is a 9-story reinforced concrete structure with two subterranean levels. There is a 12-inch thick concrete retaining wall extended along the East, South, and West edges of the building, supported by shallow continuous footings. The first basement level of City Hall is shared with basement of the Police Department building for parking. A smaller second basement extends below a portion of the City Hall building on the northern side for the Emergency Operations Center. The basement foundations are supported by shallow continuous wall footings and spread footings. The Police Department building is a 2-story reinforced concrete structure that is partially supported by the basement level, and with the extension supported by a combination of deepened spread footings and belled caissons.

The Main Library building consists of four stories above grade with additional mechanical penthouse at roof level. The first level shares a larger floor with an outdoor plaza area and lecture hall. The lecture hall is its own structural system that is different from the main library building portion, but they share the common first level diaphragm above grade. The library is supported by shallow spread footings.

AMEC (2017) reported that their legacy company, LeRoy Crandall and Associates performed geotechnical inspection and testing services during the construction of the civic center buildings in the 1970’s, and observed that the foundations are properly supported on competent, native alluvial soils, as recommended in their reports.

2.0 FIELD AND LABORATORY INVESTIGATION

2.1 Previous Investigation

The geotechnical investigations for the original construction of the Civic Center development were performed by Leroy Crandall and Associates in 1964 and 1970. The geotechnical information (boring logs and laboratory test data) from these investigations, along with the recent geotechnical report by AMEC (2017) that was prepared for the previous Tier 2 seismic evaluations of the Civic Center buildings, were reviewed in support the current investigation. We used the boring logs and pertinent laboratory data to support geotechnical analyses and conclusions.

In the original geotechnical investigations, LeRoy Crandall and Associates drilled 22 borings to depths ranging 20 to 77 feet below the grade. Laboratory tests, including moisture content and dry density determination, consolidation, direct shear, maximum dry density and optimum moisture determination, and expansion tests, were performed. Figure 2 shows the approximate location of these borings. Appendix B provides relevant boring logs along with their pertinent laboratory test results.

2.2 Current Field Investigation

The field investigation included geologic reconnaissance and subsurface exploration consisting of two (2) seismic cone penetration tests (SCPT) and one (1) cone penetration test (CPT). The locations of the explorations are shown in Figure 2. The CPTs were advanced to depths ranging from 70 to 87 feet below existing grades. Shear wave velocity measurements were obtained within the SCPTs at 5-foot intervals to the depths explored. No groundwater was encountered during drilling. Details of the field exploration program, including CPT logs and CPT interpretations are presented in Appendix A.

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

3.1 Geologic and Seismic Setting

Regionally, the site is located within the seismically active Los Angeles Basin area of the southern California Peninsular Ranges geomorphic province. The Peninsular Ranges are characterized by a series of northwest trending mountain ranges separated by valleys, with a coastal plain of subdued landforms. The Los Angeles Basin is filled with sediments thousands of feet thick, structurally influenced by thrusting fault blocks and strike-slip faults dividing the basin into northwest trending valleys and ridges.

Numerous faults are located in close proximity to the site which are sources of strong ground shaking, ground deformation, and surface fault rupture. The State of California define active faults as Holocene-active faults that have ruptured in the last approximately 11,000 years. The closest active fault is the Newport-Inglewood fault zone (NIFZ), which is mapped as crossing the Civic Center property. A list of nearby active faults and further discussion of the site seismicity is included in Appendix C.

3.2 Local Geology

Locally, the site is situated within an elevated alluvial plain at the southern edge of Baldwin Hills. Baldwin Hills are the result of faulting along the Newport-Inglewood fault zone. The entire site is underlain by Holocene to Pleistocene-aged Old Alluvial Valley Deposits (Qoa), which locally include dense sand, silty and clay. The geologic conditions in the site vicinity are depicted on the Regional Geologic Map, Figure 3.

3.3 Subsurface Conditions

Existing fill is present at the site, mantling the underlying Old Alluvial Valley Deposits (Qoa). AMEC (2017) noted that during the original site development, up to eight feet of existing fill was encountered, but it was removed during construction activities for the basement. Some fill is still present at the site, as backfill for walls, subgrade for pavements, etc. During our subsurface investigation, fill was undifferentiated from the underlying alluvial soils.

Subsurface soils encountered consisted of interbedded silty sand (SM), sandy silt (ML), sandy lean clay (CL), and poorly-graded sand (SP). Sands are interpreted to be dense to very dense, and fine-grained soils such as silt and clay are interpreted to be stiff to very stiff based on penetration resistances and descriptions of the soils encountered in previous investigations.

3.4 Groundwater

No groundwater was encountered in our geotechnical explorations nor in previous investigations in the 1960's and 1970's. Our previous experience in the site vicinity suggests that the local groundwater table is relatively deep.

4.0 GEOLOGIC HAZARDS

The publicly available USGS and CGS resources along with the Seismic Hazard Zone Report for the Inglewood 7.5-Minute Quadrangle (CDMG, 1998) and the City of Inglewood, 1995, Safety Element were reviewed for the evaluation of geologic hazards at the project site. A summary of our limited geologic hazard evaluation is presented in this section.

4.1 Strong Ground Motion

The primary geologic hazard at the site is the potential for strong ground shaking due to nearby or distant seismic events. The site could be subject to moderate to strong ground shaking from nearby or more distant, large magnitude earthquakes occurring during the expected continued life span of these buildings. This hazard is being managed through the seismic retrofit process following structural evaluation in accordance with ASCE 41-17. Seismic parameters and ground motion recommendations are provided in the *Recommendations* section of this report (Section 6.2).

4.2 Earthquake Surface Fault-Rupture Hazard

4.2.1 General

Surface rupture is the result of movement on an active fault reaching the ground surface. The site is located within an Alquist-Priolo Earthquake Fault Zone associated with the Newport-Inglewood fault. The Newport-Inglewood fault zone (NIFZ) is a zone of discontinuities, folds and faults which stretches across the Los Angeles basin in a northwest/southeastern orientation from Beverly Hills to Newport Beach. The mapped location of the fault traverses the project site through the Police Department and parking structure to the south of the Police Department (Figure 4). Therefore, there is a potential for surface rupture at the site from movement of Newport-Inglewood fault reaching the ground surface.

An Alquist-Priolo Earthquake Fault Zone requires a special study for new structures, which is a site-specific surface fault rupture investigation. The requirements of this special study are governed by the California Public Resources Code (CPRC), Division 2, Chapter 7.5. However, for the City of Inglewood Civic Center, due to both the age of the structures and type of construction, there are two exceptions that exempt this current project from meeting the requirements the special study requirements of the Alquist-Priolo Act (CPRC, Division 2, Section 2621.7):

Exception 1 [Section 2621.7, subdivision (b)]:

(b) Any development or structure in existence prior to May 4, 1975, except for an alteration or addition to a structure that exceeds the value limit specified in subdivision (c).
(c) An alteration or addition to any structure if the value of the alteration or addition does not exceed 50 percent of the value of the structure.

Exception 2 [Section 2621.7, subdivision (e)]:

(e) (1) Alterations that include seismic retrofitting, as defined in Section 8894.2 of the Government Code, to any of the following listed types of buildings in existence prior to May 4, 1975:
(A) Unreinforced masonry buildings, as described in subdivision (a) of Section 8875 of the Government Code.
(B) Concrete tilt-up buildings, as described in Section 8893 of the Government Code.
(C) Reinforced concrete moment resisting frame buildings as described in Applied Technology Council Report 21 (FEMA Report 154).

Nevertheless, since the Civic Center buildings have been in existence prior to May 4, 1975, and the new alterations only include seismic retrofitting of these structures, the proposed developments are exempt from site specific fault rupture hazard investigation (Section 2621.6 of Chapter 7.5 of the CPRC).

Since the buildings were constructed prior to 1975 and the planned seismic retrofit does not exceed 50 percent of the value of the structure, it meets the exception stated in Section 2621.7, subdivision (b). In addition, the buildings are reinforced concrete moment resisting frame buildings undergoing a voluntary seismic retrofit, so they also meet the exception stated in Section 2621.7, subdivision (e). Therefore, a special study for potential surface fault rupture is not required for this current project.

The proposed seismic retrofits will not increase the risk of damage to the structures if surface fault rupture occurs at the site (i.e., the retrofitted structures will not be more likely than unretrofitted structures to collapse due to fault displacement).

Note that future major renovation or modernization projects at the Civic Center site, or new planned structures or improvements will likely require a site-specific surface fault rupture investigation performed in accordance with California Geological Survey (CGS) Note 49, *Guidelines for Evaluating the Hazard of Surface Fault Rupture*.

4.2.2 Limited Earthquake Surface Fault-Rupture Hazard Evaluation

Although the project is exempted from the Alquist-Priolo Act, a limited assessment of the potential deformations associated with surface fault rupture has been performed to support the City of Inglewood's understanding of this risk. A simplified probabilistic evaluation of fault displacements was performed using the procedure developed by Caltrans for calculation of fault rupture (Shantz, 2013) at the BSE-1E and BSE-2E hazard levels. The resulting hazard curve is presented in Figure D-1 in Appendix D. Based on this methodology, the estimated fault displacements at the BSE-1E and BSE-2E hazard levels are negligible. Details of the evaluation are included in Appendix D.

Note that in a deterministic evaluation of fault displacement (i.e., assuming the fault ruptures to the surface in the characteristic earthquake, regardless of the likelihood of this occurring), the average displacement is estimated to be 0.95 meters (about 3.1 feet). However, the deterministic evaluation is provided only to better understand the risk of surface fault rupture at the site, since it does not take into account the likelihood that it will rupture within a given period of time, only the estimated fault slip that will occur when it does rupture.

This limited fault displacement hazard evaluation is for information only, and not intended for design purposes. If structural mitigation for the surface fault rupture hazard is desired, a more comprehensive study, such as additional field investigation and site-specific fault displacement evaluation, may be required.

4.3 Liquefaction and Seismic Settlement

Liquefaction is the sudden loss of soil shear strength within saturated, loose to medium dense, sands and non-plastic silts. Liquefaction is caused by the build-up of pore water pressure during strong ground shaking from an earthquake. Secondary effects of liquefaction are sand boils, settlement and instabilities within sloping ground that occur as lateral spreading, seismic deformation and flow sliding. Seismic shaking can also cause seismic compaction, which is the densification of loose to medium dense granular soils that are above groundwater. Loose unsaturated coarse-grained soils were generally not observed at the site.

Considering the dense condition of the soils underlying the site and the relatively deep groundwater, the potential for earthquake induced ground failure due to soil liquefaction and seismic compaction should be very low.

4.4 Landslides and Slope Stability

The site and site vicinity are relatively flat. Evidence of ancient landslides or slope instabilities were not observed during our literature review, site reconnaissance, or subsurface exploration. Based on our understanding of the current project, landslides and slope stability are not considered hazards for the site.

4.5 Tsunamis, Seiches, and Inundation

The site is located about 5.3 miles from the Pacific Ocean, with an approximate Elevation of 115 feet above Mean Sea Level (MSL). No bodies of water are located above or near the site. The site is not located within a Tsunami Inundation Area as mapped by the California Emergency

Management Agency (2009). Therefore, the potential for a tsunami, seiche, and inundation is considered very low for this site.

4.6 Flooding

The site is located within a densely populated and developed area. Storm water is largely controlled by engineered drainage. The site is outside of the 100-year flood plain and it is located in an area of minimal flood hazard (Federal Emergency Management Agency, 2020). Therefore, the risk of flooding is considered very low.

4.7 Compressible Soils

In general, the site is underlain by Holocene to Pleistocene-aged alluvial deposits that are not considered compressible and provide good support for any existing or new foundations. However, any locally encountered undocumented fill is considered compressible due to the variable physical characteristics and apparent densities that can stem from uncontrolled placement and compaction of the fill. These soils are typically removed entirely and recompacted where the new footings or other improvements are placed.

5.0 CONCLUSIONS

Based on the findings and interpretations from our investigation, the following is a summary of our primary findings.

- The existing City Hall, Police Station, and Main Library are supported within the underlying native Pleistocene-aged alluvial soils as recommended in the original geotechnical reports. The native alluvial soils are generally dense and firm, with low compressibility and high shear strength, and they are competent for support of foundations.

- These 3 structures are all supported by shallow spread and continuous footings, with the exception of four belled caissons supporting a portion of the Police Department building.

- Some existing fill underlies the site to a maximum depth of about 8 feet. Without review of as-built reports of the compaction of this fill, these materials are considered potentially compressible and therefore they are not suitable for the direct support of structures or slabs or walkways without remedial grading.

- The primary geologic hazards are the site are seismic shaking and surface fault rupture. There is high potential for strong ground shaking due to nearby or distant seismic events. This hazard is being managed through the seismic retrofit process following structural evaluation in accordance with ASCE 41-17. The subject site is located within the Alquist-Priolo Earthquake Fault Zone (A-P Zone) for the Newport-Inglewood fault, but it is exempt from a site-specific fault rupture hazard investigation for this voluntary seismic retrofit as discussed in the *Earthquake Surface Fault-Rupture Hazard* section of this report (Section 4.2).

6.0 RECOMMENDATIONS

The remainder of this report presents geotechnical parameters and recommendations for the analysis of existing foundations and retaining walls, which are supported by native alluvial deposits, as well as recommendations for the proposed new improvements. If these recommendations do not address a specific feature of the project, please contact Group Delta for additions or revisions.

6.1 Foundation Recommendations

6.1.1 Existing Shallow Foundations

Bearing Value

The net allowable bearing capacity for existing footings may be calculated through the following equations:

$$q'_{all}(\mathrm{ksf}) = 8.90 + 0.75 \times D_f + 0.27 \times B \qquad \text{(column footings)}$$

$$q'_{all}(\mathrm{ksf}) = 6.85 + 0.75 \times D_f + 0.34 \times B \qquad \text{(wall continuous footings)}$$

In the above equations, D_f, is the footing's embedment depth and B, is the footings width. It should be noted that for rectangular footings, $B = \sqrt{B_f \times L_f}$, where B_f and L_f are the actual width and length of the footing, respectively. The bearing capacities obtained from the above equations can be varied by 50 percent for upper and lower bounds.

Settlement

We don't expect significant increase in loads as a result of the planned retrofit. Therefore, the anticipated settlements resulting from the additional load of the retrofit are expected to be negligible for existing footings.

Lateral Resistance

Lateral loads against the structure may be resisted by friction between the bottoms of footings and the soil, and passive pressure from the portion of vertical foundation members embedded into the native alluvial deposits. A coefficient of friction of 0.45 and a passive pressure of 350 psf per foot of embedment may be used.

Ultimate Resistance

The recommended bearing and lateral resistance values provided above are allowable pressures used in Working Stress Design. For Ultimate Resistance, these allowable values may be multiplied by the following factors:

Parameter	Ultimate Factor
Bearing Value	3.0
Coefficient of Friction	1.5
Passive Pressure	1.5

Dynamic Modulus of Subgrade Reaction

For evaluation of transient loading, such as seismic loading, the load-deformation characteristics for existing foundations supported on native older alluvial deposits may be evaluated following Section 8.4.2 of ASCE 41-17. The initial (small-strain) shear modulus (G_0) may be calculated using Equation 8-4, and the effective shear modulus ratio (G/G_0) may be evaluated using Table 8-2 of ASCE 41-17. The parameters given in the following tables are recommended to be used for evaluation of the dynamic modulus of subgrade reaction for the existing footings:

Existing Foundations Location	Unit Weight (pcf)	Average Shear Wave Velocity, *Vs* (ft/s)	Poisson's Ratio	Average Initial Shear Modulus, G_0 (ksf)
City Hall (1st Basement Level)	120	1,150	0.35	4,930
City Hall (2nd Basement Level)	120	1,300	0.25	6,300
Police Department	120	1,150	0.35	4,930
Main Library	120	1,150	0.35	4,930

Based on site-specific shear wave velocity measurements, the variation in average shear wave velocity is about 15 percent (corresponding to a coefficient of variation of approximately 0.3). However, per Section 8.4.2, the coefficient of variation (C_V) may not be taken as less than 0.5 in the analyses for determining the upper and lower bounds of the dynamic stiffnesses.

Hazard Level	Effective Peak Acceleration $S_{XS}/2.5$	Effective Shear Modulus Ratio (G/G_0)
BSE-1E	0.346	0.786
BSE-2E	0.641	0.660

These values may be used with either Methods 1, 2, or 3 described in Sections 8.4.2.3, 8.4.2.4, or 8.4.2.5 of ASCE 41-17, depending on the rigidity of the foundation relative to the older alluvial deposits.

6.1.2 Existing Belled Caissons

Axial Capacity

The four existing belled caissons supporting a portion of the Police Department building may be evaluated use an ultimate axial capacity of 45 kips per square foot (ksf). If needed, allowable axial capacity can be estimated by applying a factor of safety of 3. The ultimate axial compressive capacity mobilization curve for the 10-foot and 12-foot diameter belled caissons are included in Figures 6A and 6B, along with a curve presenting the secant axial springs in kips per inch. The ultimate tensile capacity mobilization curves for the 10-foot and 12-foot diameter belled caissons are included in Figures 6C and 6D.

Lateral Capacity

Lateral capacity curves as a function of deformation for the two 10-foot belled caissons for both fixed head and pinned head conditions have been developed using the computer program LPILE, version 2019-11.005 (Ensoft, 2019). Nonlinear material properties were used for the upper portion of the drilled shaft with uniform cross-section and reinforcement, with concrete and reinforcing steel dimensions and properties obtained from the as-built plans. The belled portion of the drilled shaft was modeled as an elastic section with varied diameter from top to bottom of the bell. The elastic section was modeled assuming cracked concrete with 60% gross moment of inertia. Note that a sensitivity evaluation using a cracked section compared with an uncracked section in the bell resulted in very minor differences in overall behavior and capacity.

An axial load of 350 kips was used in the analyses (the static load on the drilled shafts provided by KPFF). The lateral capacity curves for two different pile cutoff elevations (at basement level and at first floor) are included in Figures 7A through 7D.

6.1.3 New Shallow Foundations

New shallow foundations planned as part of the retrofit should be embedded within the undisturbed native alluvial deposits for consistency with the other existing foundations supporting the structures.

New footings with a minimum depth of 24 inches below the lowest adjacent grade may use a net allowable dead-plus-live load capacity of 7,000 psf. This value may be increased by a factor of 3 for ultimate bearing capacity.

Settlement

We estimate that the total and differential settlement of the new foundations will be less than 1/2 inch and 1/4 inch in 40 feet, respectively.

Lateral Resistance

Lateral loads against the structure may be resisted by friction between the bottoms of footings and slabs and the soil, and passive pressure from the portion of vertical foundation members embedded into undisturbed native soils. A coefficient of friction of 0.45 and a passive pressure of 350 psf per foot of embedment may be used. The allowable lateral resistance may be increased by a factor of 1.5 for ultimate lateral resistance.

6.1.3.1 New Foundations Adjacent to Existing Foundations or Utilities

Numerous existing foundations are present supporting the existing buildings, walls, and other improvements at the site. New foundations planned adjacent to existing foundations or utilities, should be deepened such that they do not surcharge the existing foundations or utilities. For preliminary design, any new foundations should be placed such that a 1H:1V slope descending from the bottom of the new foundation projects below the adjacent existing footing or utility.

If foundations are planned in close proximity, it is recommended that the Geotechnical Engineer review the plans and provide recommendations as needed.

6.2 Seismic Design

6.2.1 BSE-2E and BSE-1E Site-Specific Response Spectra and Ground Motions

The seismic evaluation for the retrofit of Civic Center buildings will be performed in accordance with ASCE 41-17. Both site-specific response spectra and ground motion time histories for the BSE-2E and BSE-1E seismic hazard levels have been developed to support this evaluation. A detailed discussion of the analysis procedures, the resulting site-specific ARS and ground motion time histories are presented in Appendix C. The site-specific seismic design parameters for BSE-2E and BSE-1E are presented in the table below.

Site-Specific Seismic Design Acceleration Parameters

Design Parameters	Site-Specific Seismic Design Parameters (ASCE 7-16 Section 21.4)
Site Class	C
BSE-2E S_{XS} (g)	1.603
BSE-2E S_{X1} (g)	0.907
BSE-1E S_{XS} (g)	0.865
BSE-1E S_{X1} (g)	0.428

6.3 Lateral Earth Pressures for Existing Subterranean Walls

Existing permanent subterranean walls that are restrained from lateral movement (such as the structural basement walls) may be evaluated using an at-rest equivalent fluid pressure for static conditions, and an active-plus-seismic equivalent fluid pressure under seismic loading as shown in the table below.

Load Case	Equivalent Fluid Pressure (pcf)
Static At-Rest	55
BSE-2E	35 (active) + 12 (seismic increment)
BSE-1E	35 (active) + 8 (seismic increment)

Any surcharge loads, as a result of traffic, live loads, adjacent foundations, or others should be included in the evaluation as appropriate. Surcharge loading may be taken as a uniform lateral earth pressure of 0.35q, where 'q' is the surcharge in pounds per square foot (psf). Surcharges at least a distance of the wall height, H, away from the wall may be neglected. Other loading may be provided as needed upon request.

6.4 Grading and Excavation

Significant grading is not anticipated as part of the seismic retrofits but may be required locally for the enlargement of existing shallow foundations. The recommendations in the following sections are provided to support this anticipated scope of work.

Prior to the start of any earthwork or demolition, the project civil engineer should locate any existing utilities in the area. Existing utilities should be removed, relocated, or protected, as appropriate.

6.4.1 Site Restoration and Earthwork

During construction, any subgrade disturbed by demolition (not supporting structural foundations) should be restored to the satisfaction of the geotechnical engineering representative. All grading activities should conform to the requirements of the 2019 California Building Code and the general grading recommendations outlined below.

1. Any fill placed as part of this restoration should be placed in lifts that do not exceed an 8-inch loose lift thickness, and compacted to a relative compaction of 90 percent or more (or 95 percent, where specified) of the maximum dry density based on ASTM D1557.

2. The existing site soils may be reused for fill, as long as they are free of expansive clay, organics, debris, rocks greater than 3 inches in maximum dimension, and any other deleterious material. All soils to be used as fill should be approved by the project geotechnical engineer. Any import soils should be approved before being brought to the site. If expansive soils are found, they should be removed and replaced with non-expansive compacted fill.

3. All earthwork and grading should be performed under the observation of the project geotechnical engineer. Compaction testing of the fill soils shall be performed at the discretion of the project geotechnical engineer.

6.4.2 Excavation

Vertical cuts for the temporary excavations may be used provided that adjacent underground utilities and structures are adequately supported. The sides of the temporary excavations made in fine grained soils should stand with vertical cuts to a max depth of 4 feet. Temporary excavation slopes in sandier non-cohesive soils should be constructed at an angel of 1H:1V (horizontal to vertical ratio) or flatter or shoring should be used.

Surcharge loads from equipment or stockpiled material should be kept behind the top of the temporary excavations a horizontal distance of at least twice the depth of the excavation. Surface drainage should be controlled and prevented from running down the slope face. Ponding water should not be allowed within or near the excavation. Even with the implementation of the above recommendations, some sloughing of slopes and unstable soil zones may still occur within temporary excavations, and workmen should be adequately protected. Construction equipment and foot traffic should be kept off excavation slopes to minimize disturbance/ sloughing.

Where there is insufficient room to excavate slopes, or where an existing structure or other improvement requires protection, temporary shoring should be used.

The excavations should be readily accomplished using conventional heavy construction equipment. All excavation slopes and shoring systems should meet minimum requirements of the Occupational Safety and Health (OSHA) Standards. Maintaining safe and stable slopes on excavations is the responsibility of the contractor and will depend on the nature of the soils encountered and the contractor's method of excavation.

Excavations during construction should be carried out in such a manner that failure or ground movement will not occur. The short-term stability of excavation depends on many factors, including slope angle, engineering characteristics of the subsurface materials, height of the excavation, and length of time the excavation remains unsupported and exposed to equipment, vibrations, rainfall, and desiccation. The contractor should perform any additional studies deemed necessary to supplement the information contained in this report for planning and executing their excavation plan.

6.4.3 Subgrade Preparation

During excavation for any proposed new foundations, the exposed subgrade for the design foundation depth should be evaluated by the geotechnical engineering representative to confirm that the subgrade soils consist of undisturbed native soils, and that conditions are suitable for structural support. Should the exposed conditions encountered at the design foundation depth require over-excavation, any required over-excavation should be backfilled with three-sack sand and cement slurry with a 28-day compressive strength of 300 pounds per square inch (psi).

7.0 ADDITIONAL GEOTECHNICAL SERVICES

7.1 Response to Peer Review Comments and Plan Review

We understand this project will be peer-reviewed, and therefore we may need to respond to seismic peer review comments pertaining to the geotechnical recommendations as part of the peer review process.

We recommend that any demolition, grading, foundation retrofit, and new foundation plans be reviewed by Group Delta Consultants prior to construction. We anticipate that some changes may occur during the structural evaluation and retrofit design phase from the concepts used for this investigation. Such changes may require additional geotechnical evaluation, which may result in modifications or additions to the grading and foundation recommendations provided in this report.

7.2 Excavation and Grading Observations

Excavations for foundations and compaction of all fill should be observed by the project geotechnical consultant during construction. During grading, the geotechnical engineer's representative should provide observation and testing services continuously. Such observations are considered essential to identify field conditions that differ from those anticipated by this investigation, to adjust designs to the actual field conditions, and to determine that the remedial grading is accomplished in general accordance with the recommendations presented in this report. Geotechnical observations may include:

- Observe exposed subgrade for foundations, slabs, and areas to receive new fill to confirm appropriate subgrade conditions, probe as needed, observe any scarifying and recompaction, and determine if any overexcavation is required.

- Evaluate suitability of on-site soils and any import soils for new fill placement, obtain appropriate samples of selected soils for laboratory testing.

- Observe placement of fill and backfill and perform sufficient testing during grading operations for evaluation of the relative compaction and to support our professional opinion as to compliance with compaction recommendations.

The recommendations provided in this report are contingent upon Group Delta Consultants providing these services.

8.0 LIMITATIONS

The recommendations in this report are preliminary and subject to revision from changes that occur during design development or from the results of field testing or actual subsurface conditions encountered during construction. Group Delta needs to continue to be part of the project design and construction for these recommendations to remain valid. If another geotechnical consultant provides these services, they should prepare a letter indicating their intent to assume the responsibilities of the project Geotechnical Engineer-of-Record. This letter should also indicate their concurrence with the recommendations in the report or revise them as needed to assume the role of the project Geotechnical Engineer-of-Record.

The limited fault displacement hazard evaluation presented in Appendix D of this report is for information only, and not intended for design purposes. If structural mitigation for the surface fault rupture hazard is desired, a more comprehensive study, such as additional field investigation and site-specific fault displacement evaluation, may be required.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in similar localities. No warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

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

9.0 REFERENCES

Amec Foster Wheeler, (2017), "Report of Geotechnical Consultation Proposed Seismic Evaluation, Inglewood Civic Center Buildings, 1 West Manchester Boulevard, Inglewood, California", For Nabih Youssef and Associates.

American Public Works Association (2018). *Standard Specifications for Public Works Construction, Section 200-2.2*, Untreated Base Materials, Section 400-4, Asphalt Concrete: BNI, 761 p.

American Society for Testing and Materials (2019). *Annual Book of ASTM Standards, Section 4, Construction, Volume 04.08 Soil and Rock (I); Volume 04.09 Soil and Rock (II)*; Geosynthetics, ASTM, West Conshohocken, PA, Compact Disk.

American Society of Civil Engineers (ASCE). (2016). Minimum Design Loads for Buildings and Other Structures, ASCE 7-16.

American Society of Civil Engineers (ASCE). (2017). Seismic Evaluation and Retrofit of Existing Buildings, ASCE 41-17.

California Building Standards Commission (CBSC), (2019). *2019 California Building Code (CBC)*, California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, dated: July 1.

CDMG (1998), Seismic Hazard Zone Report 027, Seismic Hazard Zone Report for The Inglewood 7.5-Minute Quadrangle, Los Angeles County, California.

City of Inglewood, 1995, Safety Element of the General Plan, Adopted July 1995.

Federal Emergency Management Agency (FEMA) (2020), Flood Map Service Center, accessed on March 11, 2020, at https://msc.fema.gov/portal/home

Kennedy, M. P., and Tan, S. S. (2008). *Geologic Map of the San Diego 30'x60' Quadrangle, California*: California Geologic Survey, Scale 1:100,000.

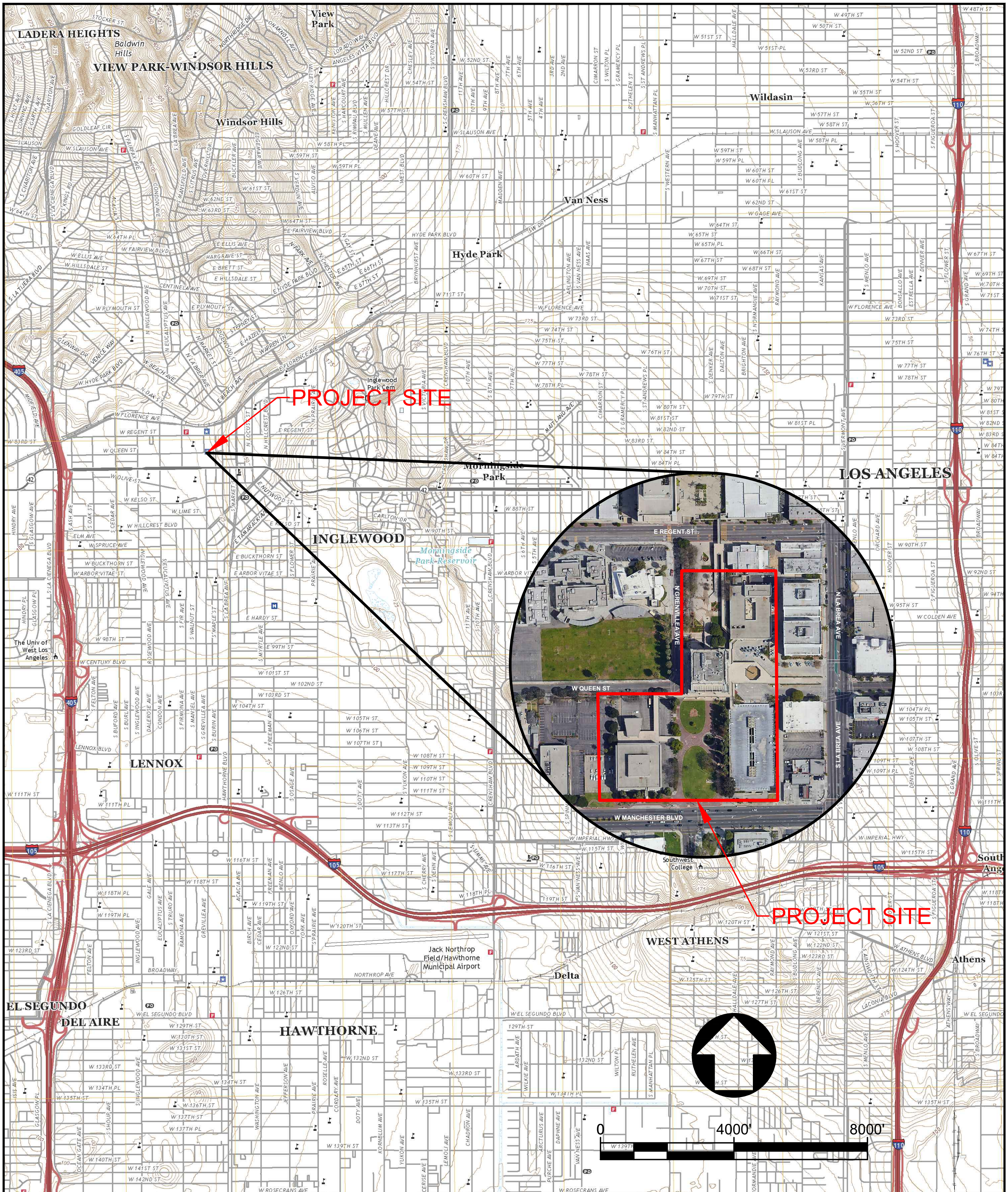
LeRoy Crandall and Associates, (1964), "Report of Foundation Investigation, Proposed Inglewood Civic Center Parking Structure, Grevillea Avenue between Queen Street and Manchester Boulevard, Inglewood, California", For the City of Inglewood.

LeRoy Crandall and Associates, (1970), "Report of Foundation Investigation, Proposed Civic Center, Grevillea Avenue Between Manchester Boulevard and Regent Street, Inglewood, California", For the City of Inglewood.

Shantz, Tom (2013). Caltrans Procedures for Calculation of Fault Rupture Hazard. Caltrans Division of Research and Innovation, February 2013.

Wells, D.L., Coppersmith, K.J. (1994). "New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement", *Bulletin of the Seismological Society of America*, v. 84, No. 4, p. 974-1002.

FIGURES



REFERENCE: USGS TOPO, INGLEWOOD QUADRANGLE 7.5-MINUTE SERIES

DATE: 12/17/2020	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	SITE VICINITY MAP	PROJECT NUMBER: LA1474
REVIEWED BY: LK	APPROVED BY: KC			1 W. MANCHESTER BOULEVARD	SCALE: AS SHOWN
PREPARED BY: -				INGLEWOOD, CALIFORNIA	FIGURE NUMBER: 1

N:\Projects\1400-1499\LA1474 KPFF City of Inglewood Retrofits\600 Drafting\LA1474.dwg, 12/17/2020 2:52:23 PM, DWG To PDF.pc3

REFERENCE: GOOGLE EARTH

LEGEND:

CPT-2	APPROXIMATE CPT LOCATION	20	APPROXIMATE PRIOR INVESTIGATION LOCATION (AMEC-1970)
SCPT-3	APPROXIMATE SCPT LOCATION	2	APPROXIMATE PRIOR INVESTIGATION LOCATION (AMEC-1964)

DATE: 12/17/2020	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	EXPLORATION LOCATION PLAN	PROJECT NUMBER: LA1474
REVIEWED BY: LK	APPROVED BY: KC		1 W. MANCHESTER BOULEVARD INGLEWOOD, CALIFORNIA	SCALE: AS SHOWN
REVISED: -				FIGURE NUMBER: 2

N:\Projects\1400-1499\LA1474 KPFF City of Inglewood Retrofits\600 Drafting\LA1474.dwg, 12/17/2020 2:52:40 PM, DWG To PDF.pc3

MAP UNITS

LATE HOLOCENE (SURFICIAL DEPOSITS

af **Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills

Qsu **Undifferentiated Surficial Deposits** - includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers

Qls **Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated

Qb **Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand

Qw **Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand

Qf **Alluvial Fan Deposits** - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment

Qa **Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment

Qt **Terrace Deposits** - includes marine and stream terrace deposits; marine deposits include slightly to moderately consolidated and bedded gravel and conglomerate, sand and sandstone, and silt and siltstone; river terrace deposits consist of unconsolidated thin- to thick-bedded gravel

Ql **Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites

Qe **Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand

Holocene to Late Pleistocene (Surficial Deposits)

Qyf **Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Qya **Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers

Late to Middle Pleistocene (Surficial Deposits)

Qof **Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Qoa **Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers

Qot **Old Terrace Deposits** - slightly to moderately consolidated, moderately dissected marine and stream terrace deposits

Qol **Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types

Middle to Early Pleistocene (Surficial Deposits)

Qvof **Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Qvoa **Very Old Alluvial Valley Deposits** - moderately to well-consolidated, highly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers; generally uplifted and deformed

Quaternary (Bedrock)

Qss **Coarse-grained formations of Pleistocene age and younger** - primarily sandstone and conglomerate

Qsh **Fine-grained formations of Pleistocene age and younger** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tertiary (Bedrock)

Tss **Coarse-grained Tertiary age formations** - primarily sandstone and conglomerate

Tsh **Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tv **Tertiary age formations of volcanic origin**

Mesozoic and Older (Bedrock)

Kss **Coarse-grained Cretaceous age formations of sedimentary origin**

Ksh **Fine-grained Cretaceous age formations of sedimentary origin**

pKm **Cretaceous and pre-Cretaceous metamorphic formations of sedimentary and volcanic origin**

sp **Serpentinite of all ages**

gr **Granitic and other intrusive crystalline rocks of all ages**

SYMBOL EXPLANATION

Contact

Gradational contact

Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map.

Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip

Lineament

Folds -- Showing direction of plunge where appropriate

Anticline

Overturned anticline

Syncline

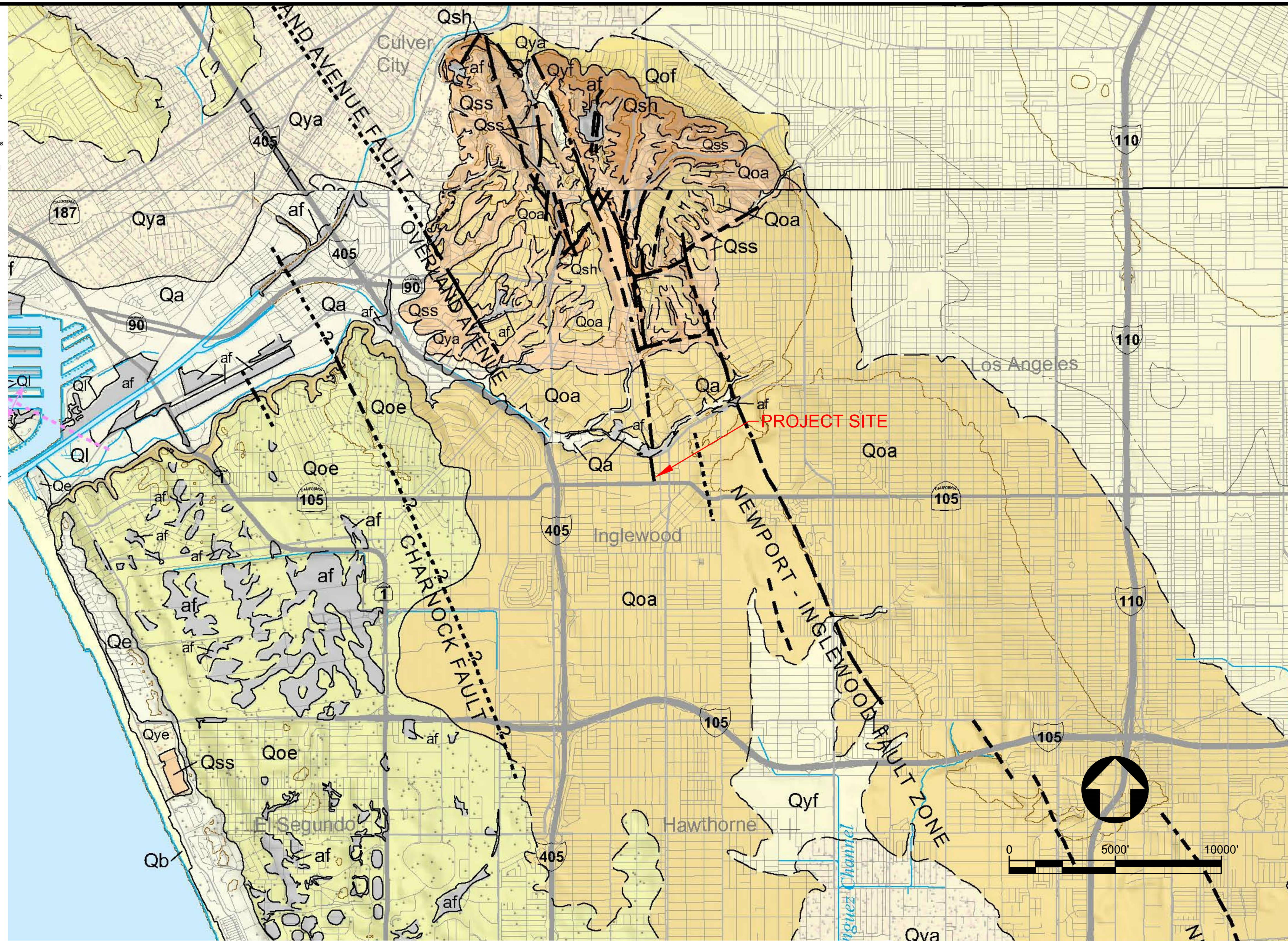
Dike

Stream

Spring

Road

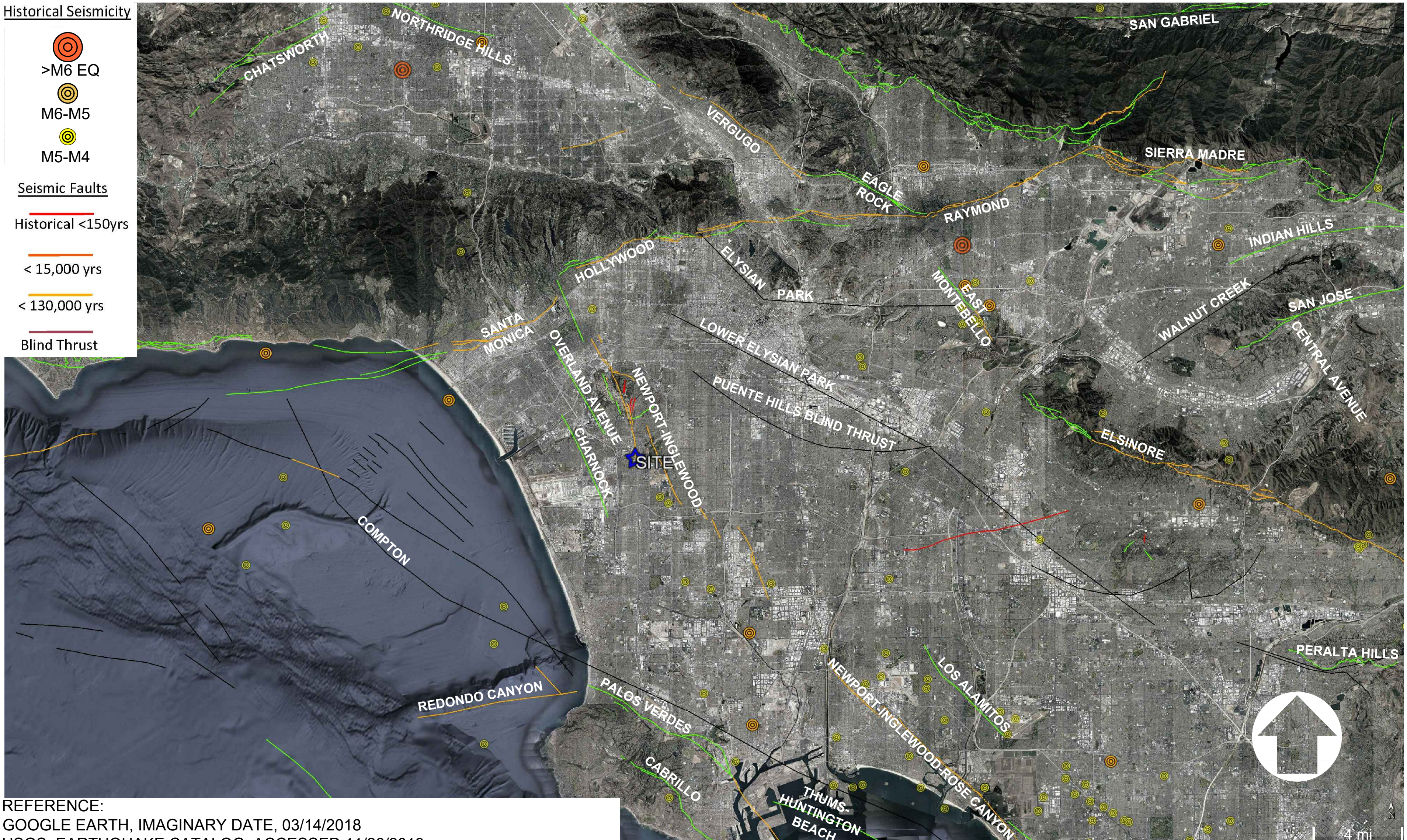
County boundary



REFERENCE: CGS, 2010, GEOLOGIC COMPILATION OF QUATERNARY SURFICIAL DEPOSITS IN SOUTHERN CALIFORNIA, LOS ANGELES AND LONG BEACH 30X60 QUADRANGLE

DATE: 12/17/2020	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	REGIONAL GEOLOGIC MAP	PROJECT NUMBER: LA1474
REVIEWED BY: LK	APPROVED BY: KC		1 W. MANCHESTER BOULEVARD	SCALE: AS SHOWN
REVISED: -			INGLEWOOD, CALIFORNIA	FIGURE NUMBER: 3

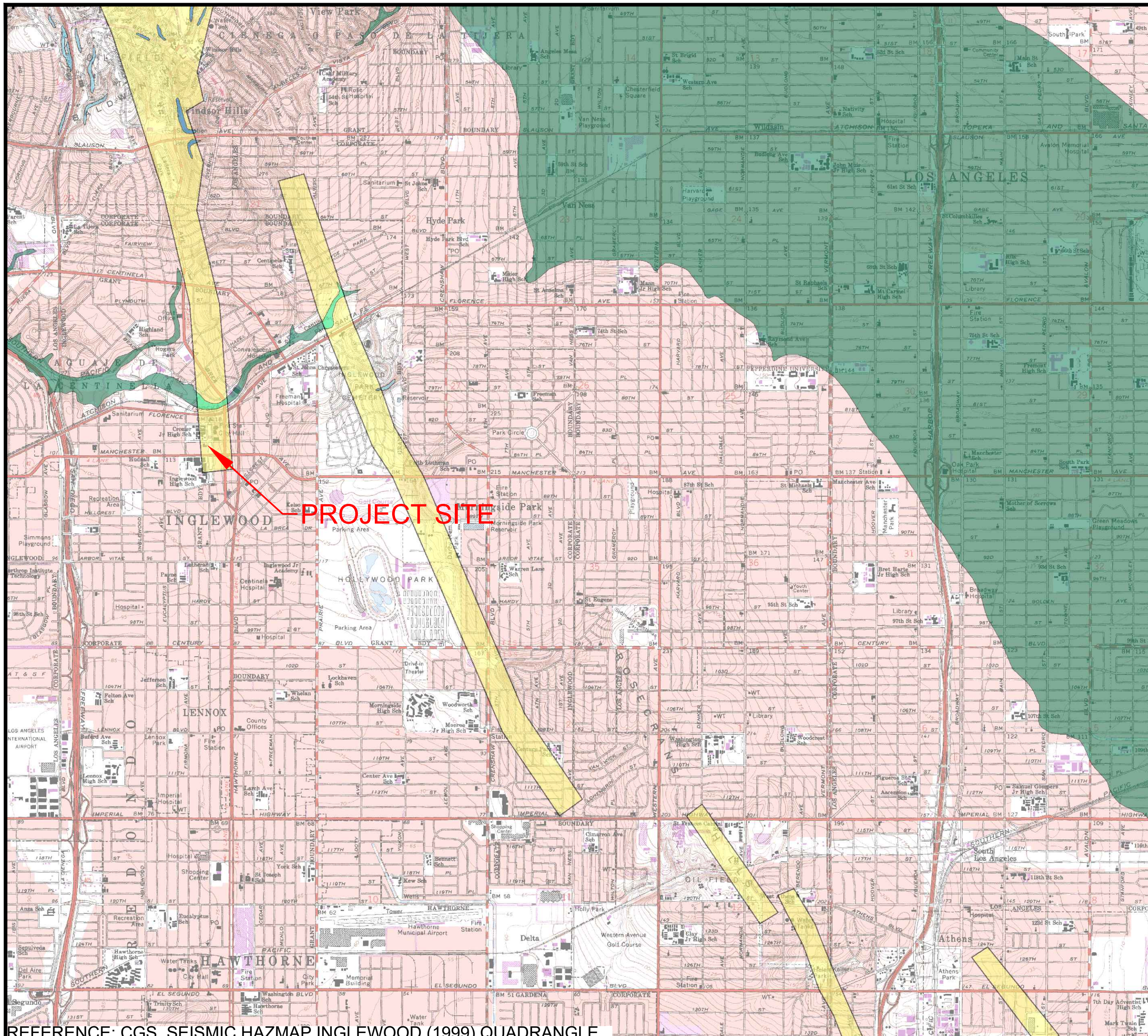
N:\Projects\1400-1499\LA1474 KPFF City of Inglewood Retrofits\600 Drafting\LA1474.dwg, 12/17/2020 2:52:56 PM, DWG To PDF.pc3



REFERENCE:
GOOGLE EARTH, IMAGINARY DATE, 03/14/2018
USGS, EARTHQUAKE CATALOG, ACCESSED 11/26/2019
USGS & CGS, QUATERNARY FAULT AND FOLD DATABASE, ACCESSED 3/4/2020

DATE: 12/17/2020	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	REGIONAL FAULT AND SEISMICITY MAP	PROJECT NUMBER: LA1474
REVIEWED BY: LK	APPROVED BY: KC			1 W. MANCHESTER BOULEVARD INGLEWOOD, CALIFORNIA	SCALE: AS SHOWN
PREPARED BY: -					FIGURE NUMBER: 4

N:\Projects\1400-1499\LA1474 KPFF City of Inglewood Retrofits\600 Drafting\LA1474.dwg, 12/17/2020 2:53:12 PM, DWG To PDF.pc3



REFERENCE: CGS, SEISMIC HAZMAP INGLEWOOD (1999) QUADRANGLE

MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

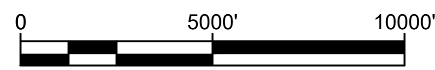
Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

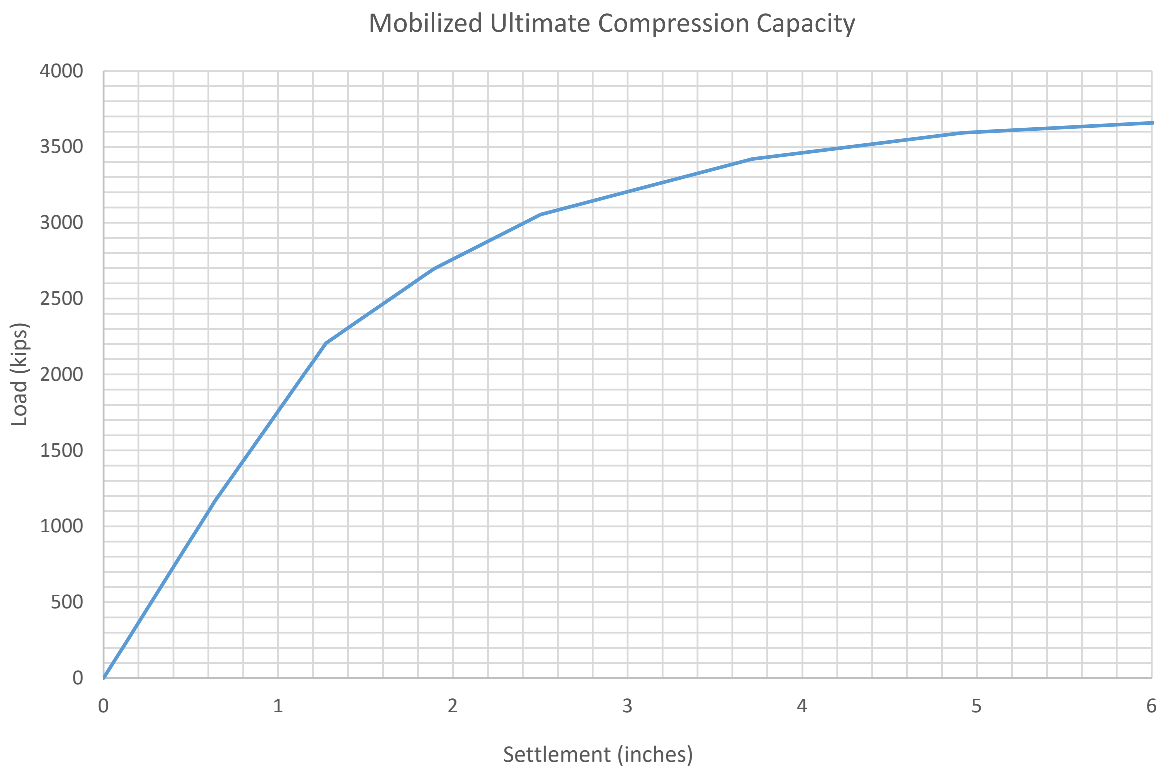
OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

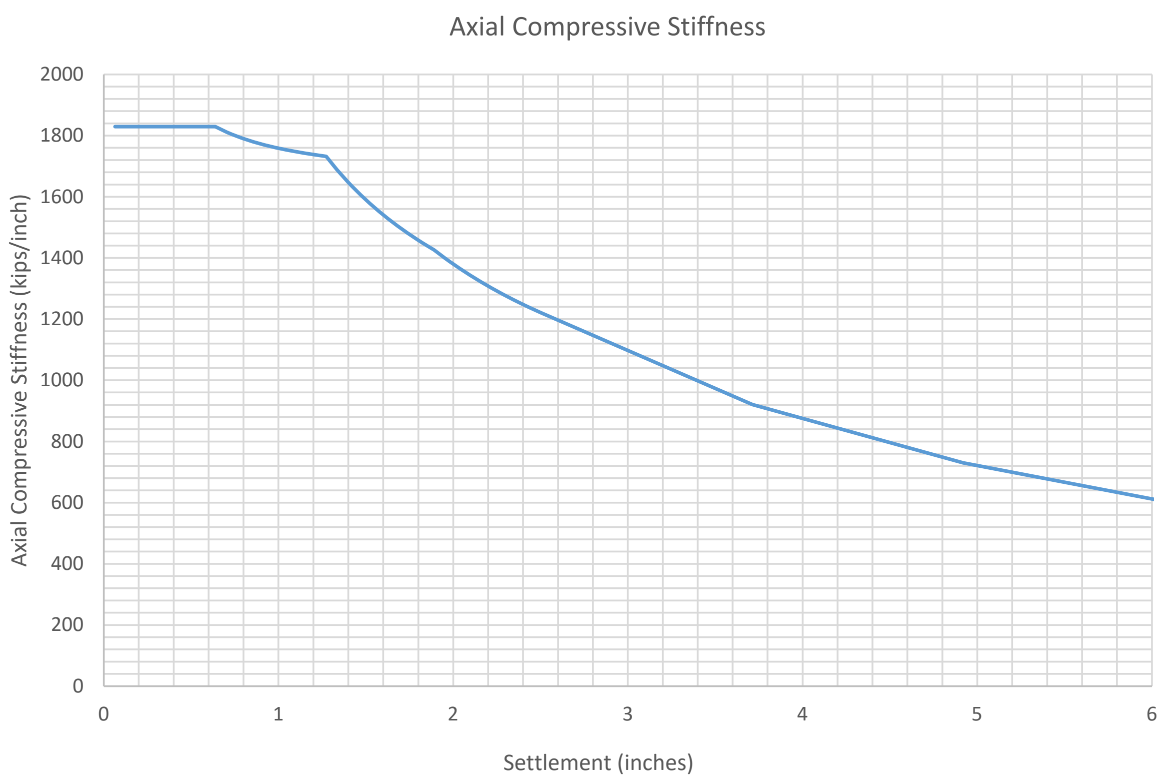
Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

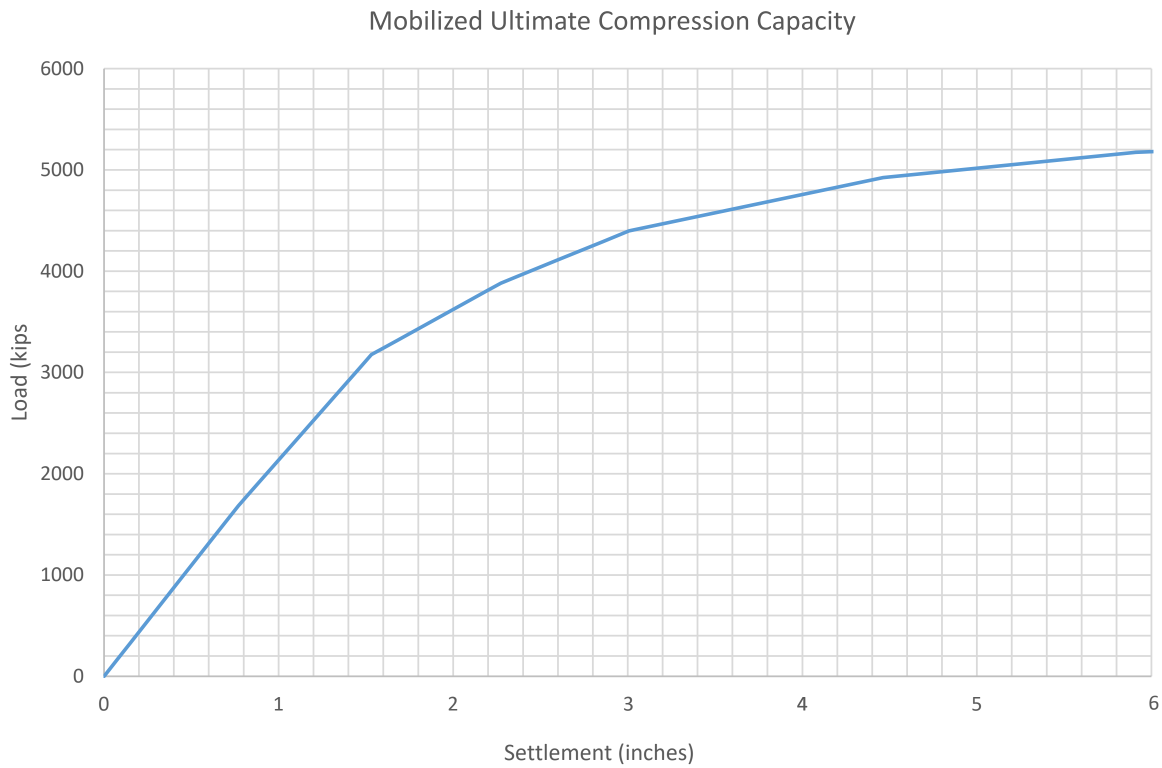


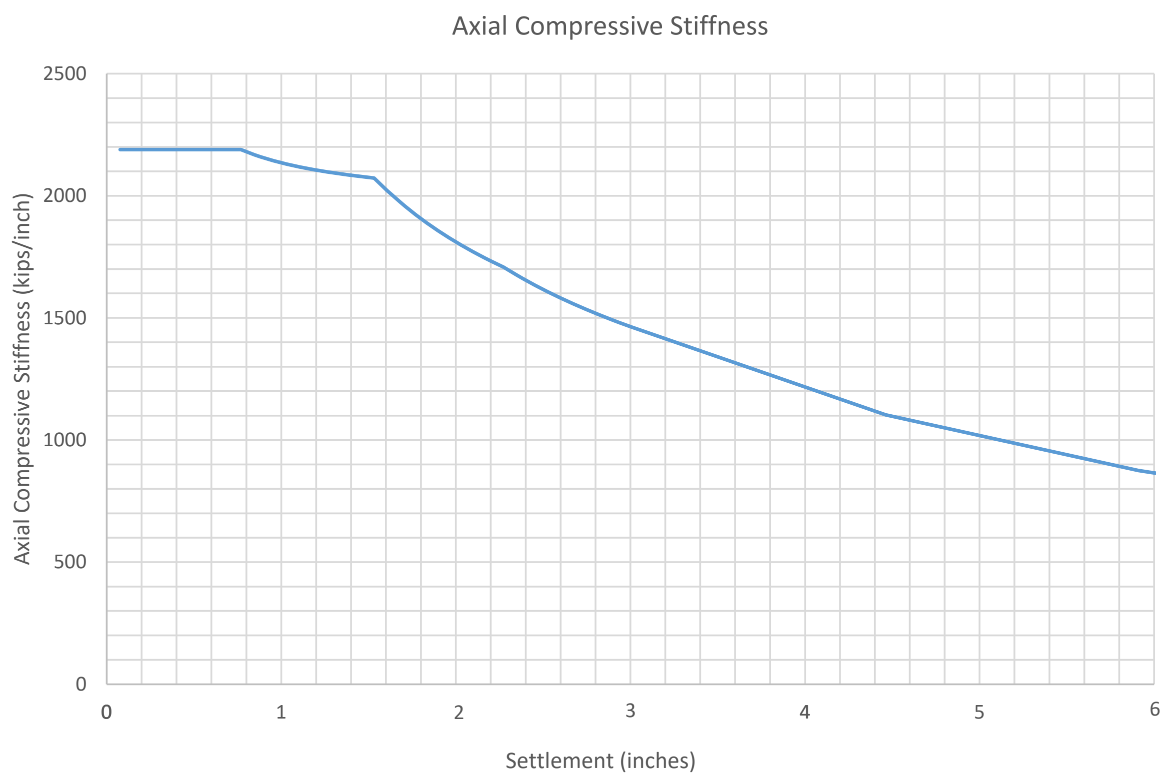
DATE: 12/17/2020	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP	PROJECT NUMBER: LA1474
REVIEWED BY: LK	APPROVED BY: KC		1 W. MANCHESTER BOULEVARD INGLEWOOD, CALIFORNIA	SCALE: AS SHOWN
PREPARED BY: -				FIGURE NUMBER: 5





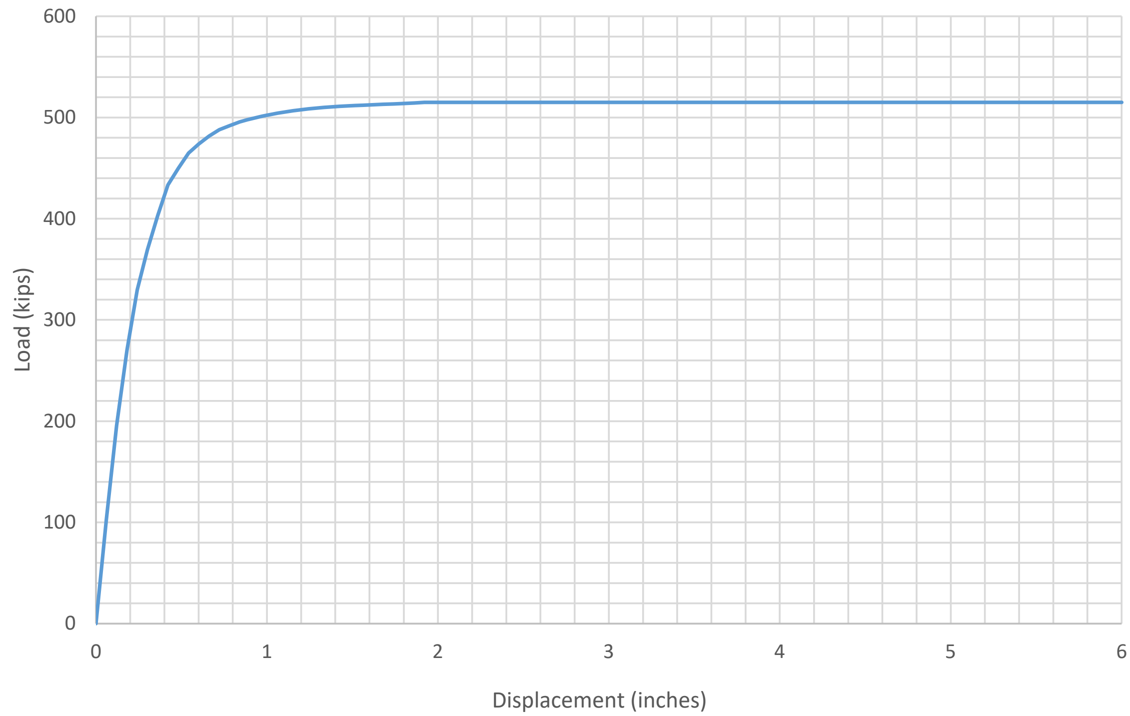
City of Inglewood Seismic Retrofits 1 W Manchester Blvd, Inglewood, CA	10 ft Diameter Belled Caisson Axial Capacity Mobilization	GROUP DELTA PROJECT NUMBER: LA-1474 FIGURE NUMBER: 6A



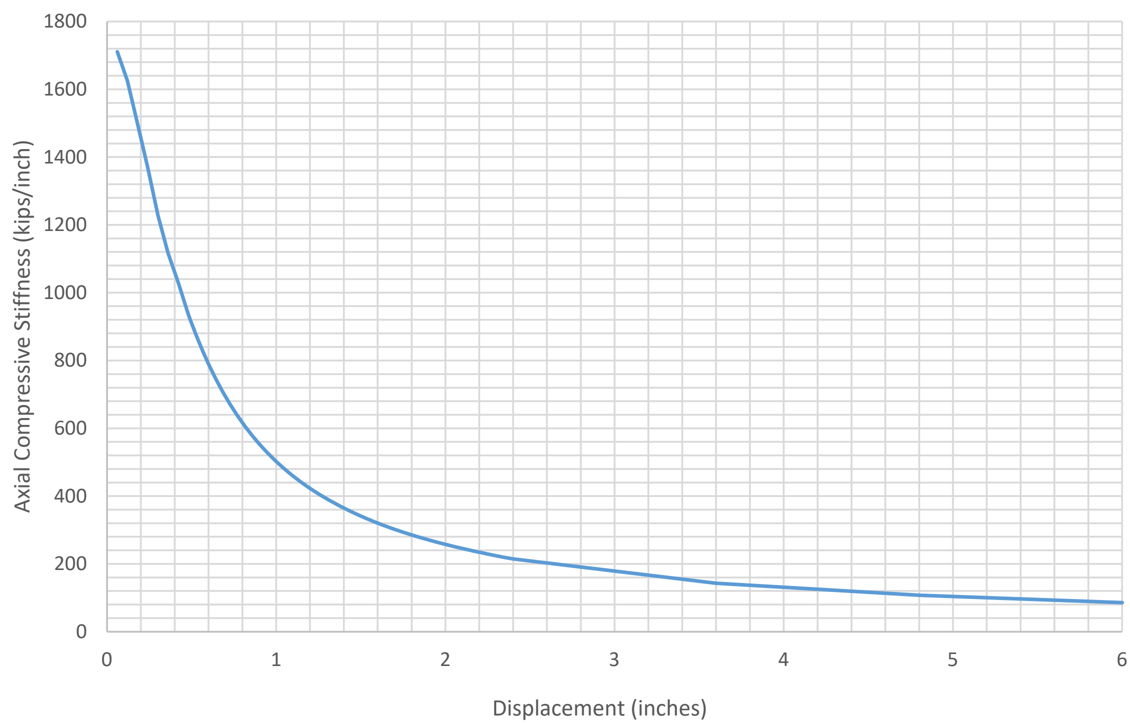


City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA**	**12 ft Diameter Belled Caisson Axial Capacity Mobilization**	GROUP DELTA PROJECT NUMBER LA-1474	FIGURE NUMBER **6B**

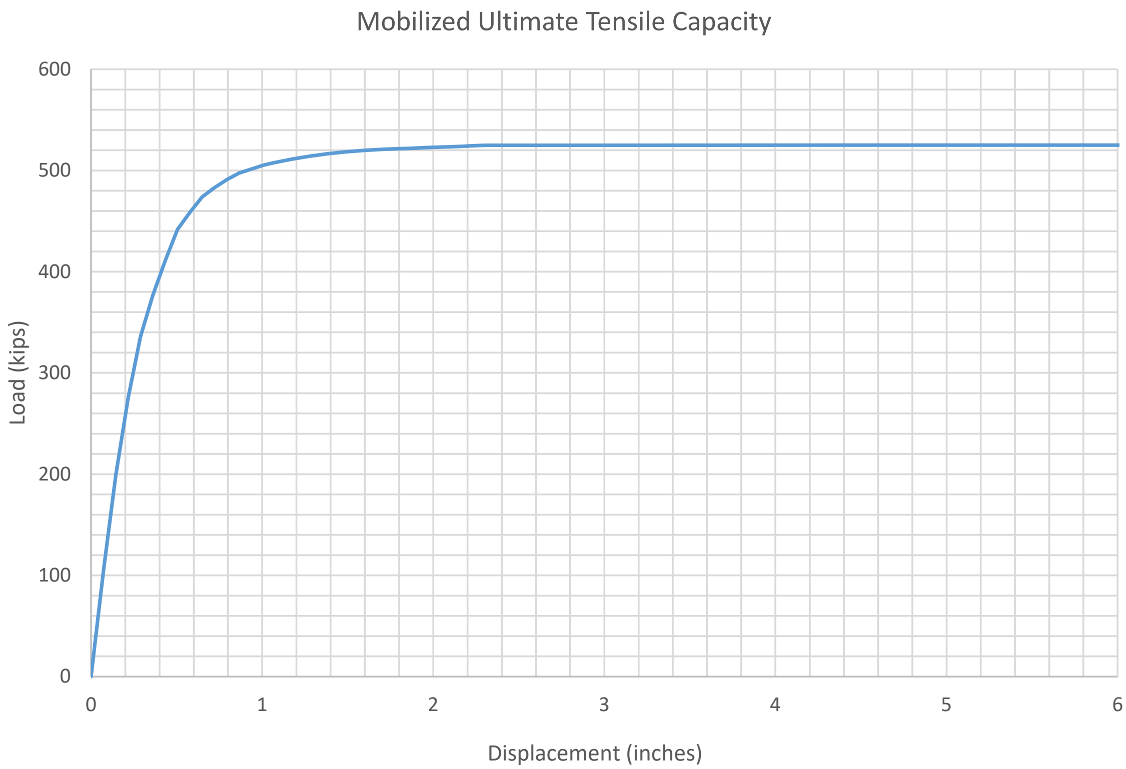
Mobilized Ultimate Tensile Capacity

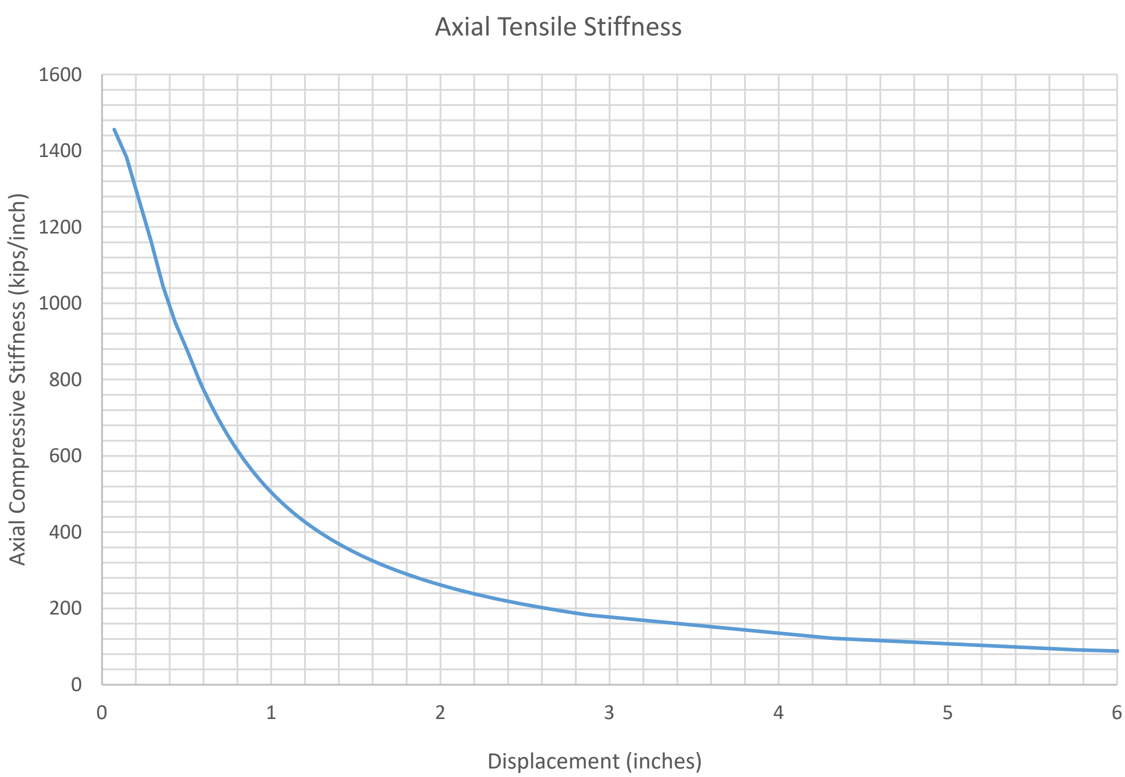


Axial Tensile Stiffness

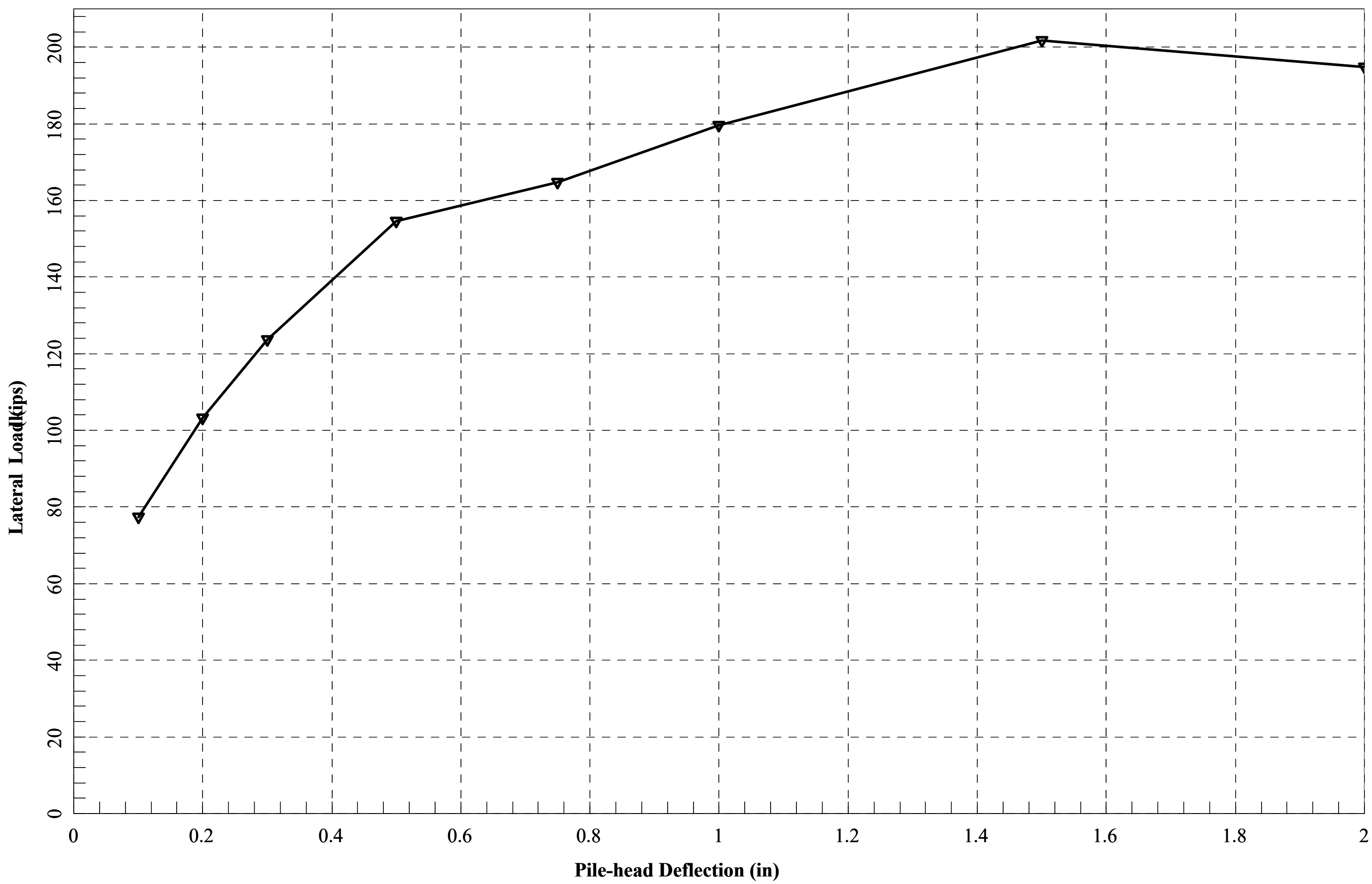


City of Inglewood Seismic Retrofits **1 W. Manchester Blvd., Inglewood, CA**	**10 ft Diameter Belled Caisson Tensile Capacity Mobilization**	GROUP DELTA PROJECT NUMBER LA1474	FIGURE NUMBER **6C**

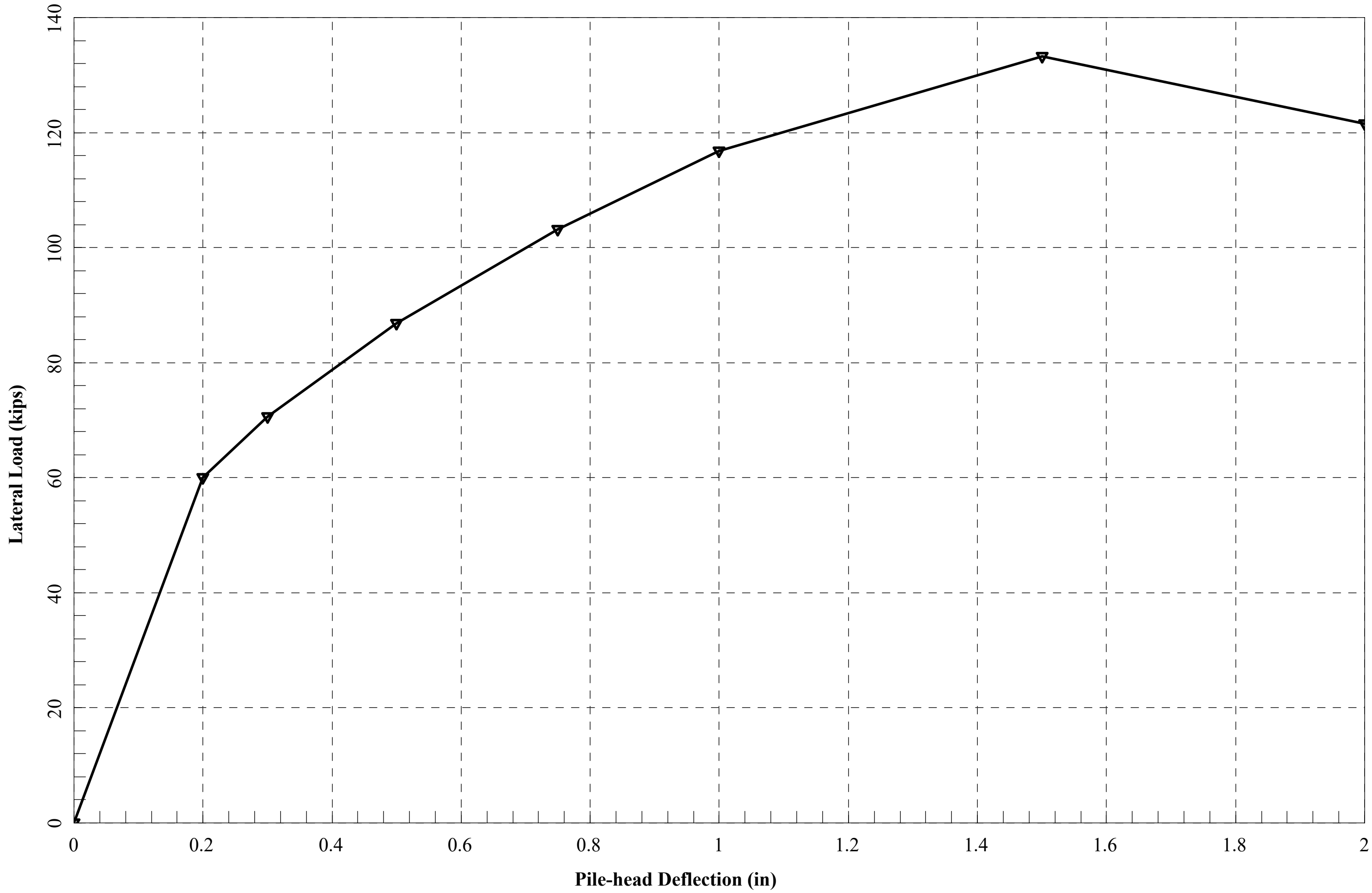




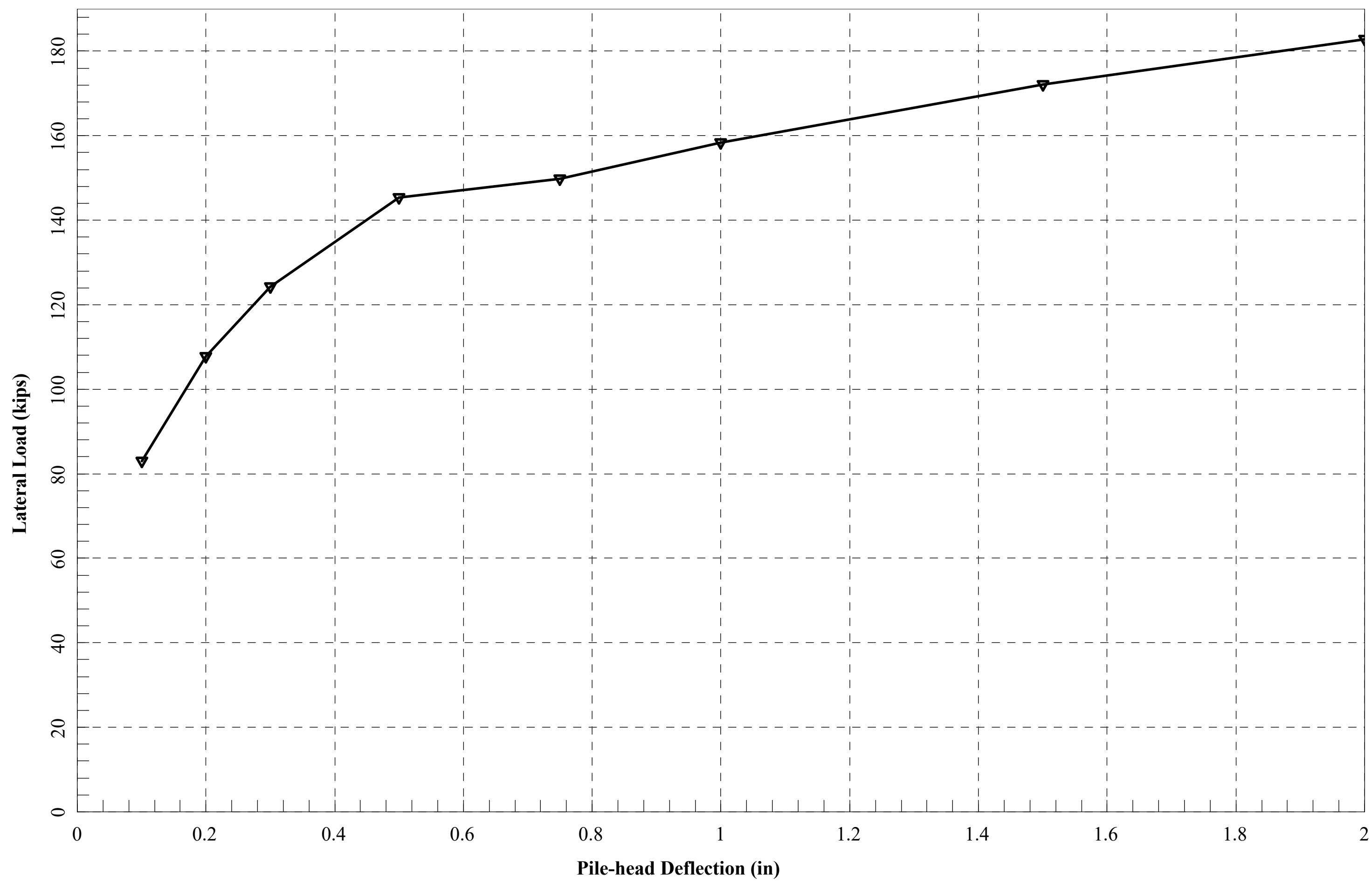
City of Inglewood Seismic Retrofits **1 W. Manchester Blvd., Inglewood, CA**	**12 ft Diameter Belled Caisson Tensile Capacity Mobilization**	GROUP DELTA PROJECT NUMBER LA1474	FIGURE NUMBER **6D**



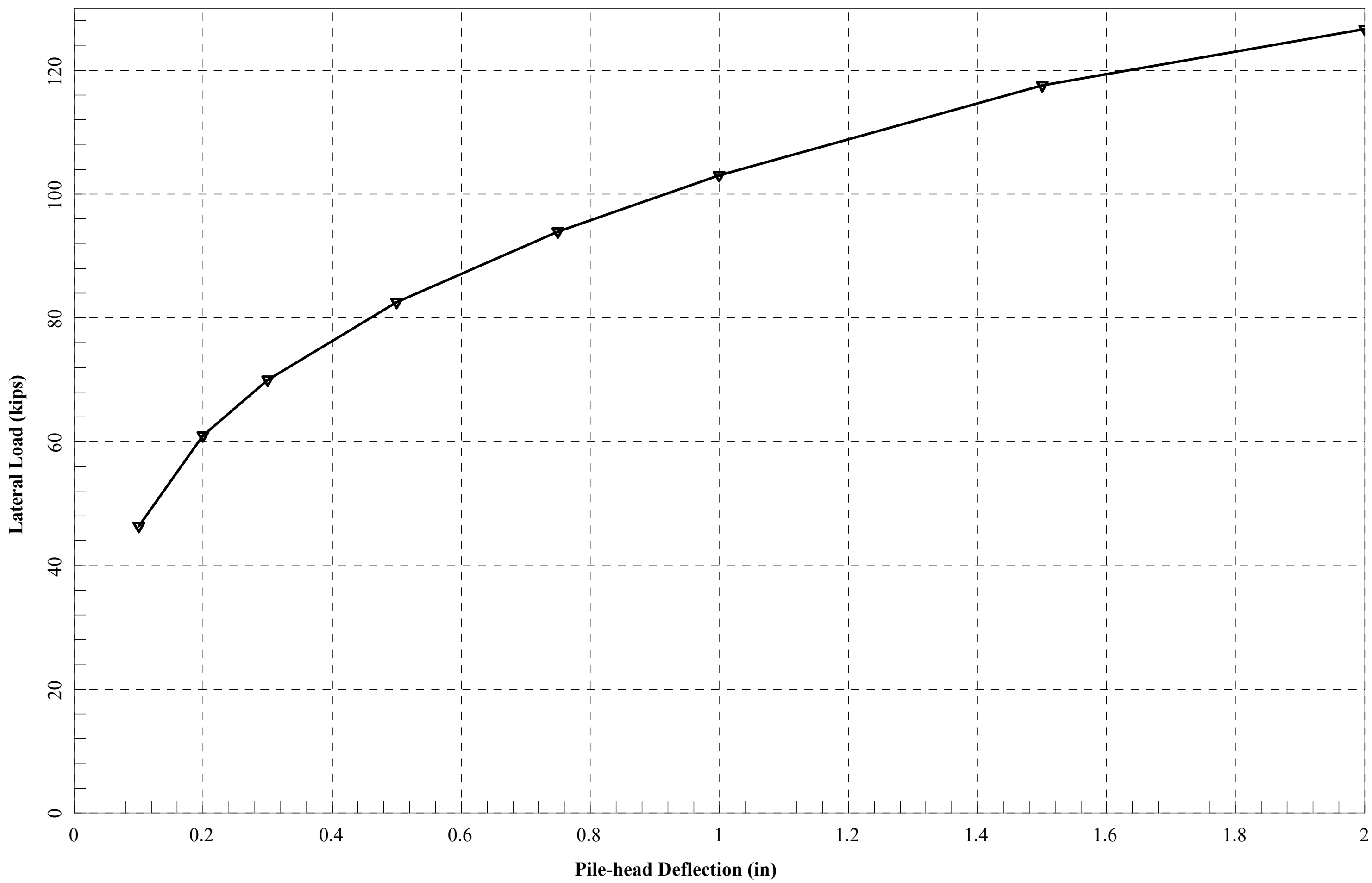
PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Pile Head Shear Vs. Displacement** **Fixed Head Condition** **Pile Head at Basement Level**	PROJECT NUMBER LA-1474	FIGURE NUMBER 7A



PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Pile Head Shear Vs. Displacement** **Pinned Head Condition** **Pile Head at Basement Level**	PROJECT NUMBER LA-1474	FIGURE NUMBER 7B



PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Pile Head Shear Vs. Displacement** **Fixed Head Condition** **Pile Head at First Floor Level**	PROJECT NUMBER LA-1474	FIGURE NUMBER 7C



PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Pile Head Shear Vs. Displacement** **Pinned Head Condition** **Pile Head at First Floor Level**	PROJECT NUMBER LA-1474	FIGURE NUMBER 7D

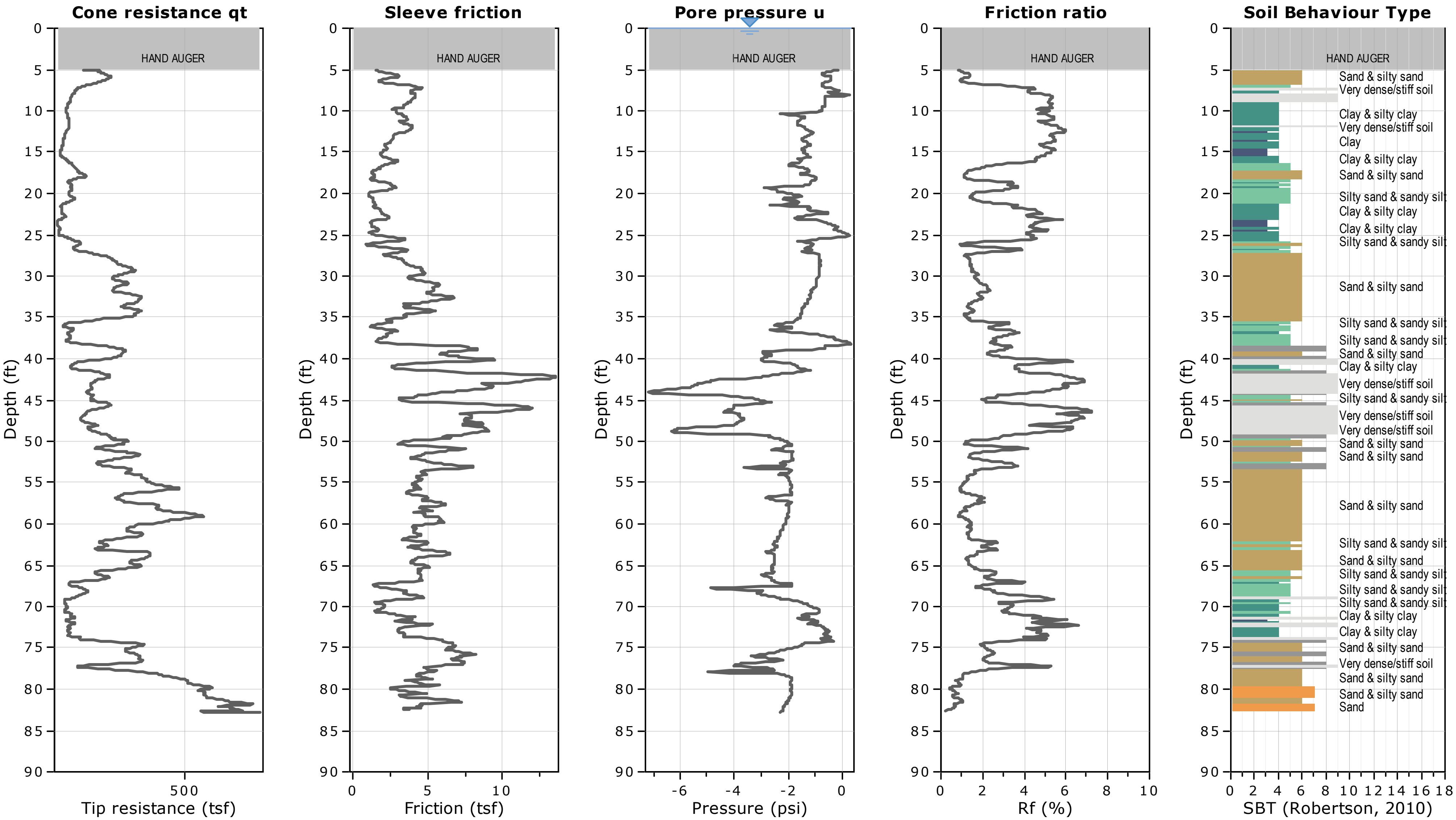
APPENDIX A

FIELD EXPLORATION

Group Delta Consultants
370 Amapola Avenue, Suite 212
Torrance, CA 90501
http://www.groupdelta.com

Project: **City of Inglewood Seismic Retrofits**
Location: **1 W Manchester Blvd., Inglewood, CA 90301**

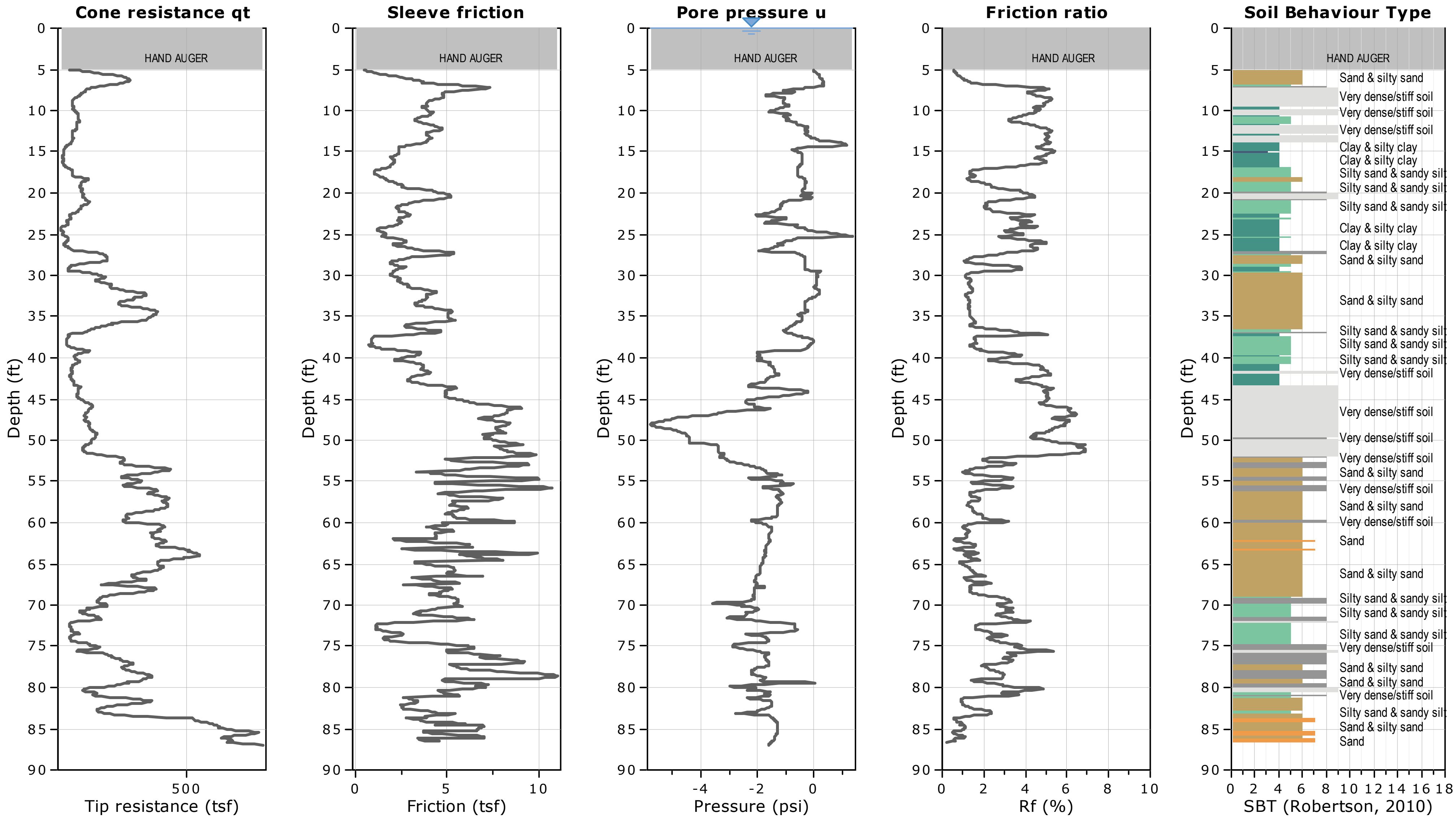
CPT: SCPT-1
Total depth: 82.88 ft, Date: 10/23/2020



Group Delta Consultants
370 Amapola Avenue, Suite 212
Torrance, CA 90501
http://www.groupdelta.com

Project: City of Inglewood Seismic Retrofits **CPT: CPT-2**

Location: 1 W Manchester Blvd., Inglewood, CA 90301 Total depth: 86.95 ft, Date: 10/23/2020



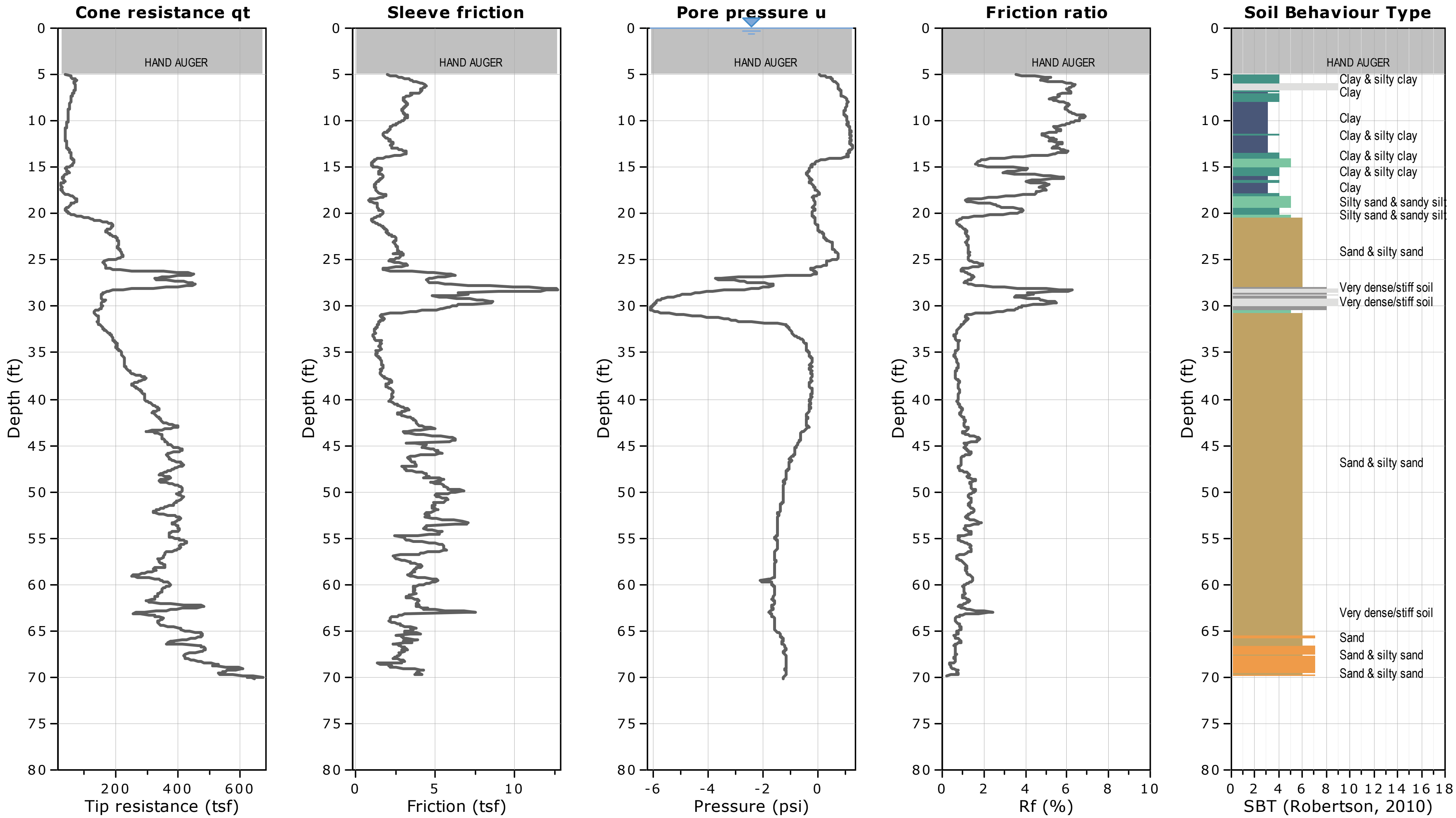
Group Delta Consultants
370 Amapola Avenue, Suite 212
Torrance, CA 90501
http://www.groupdelta.com

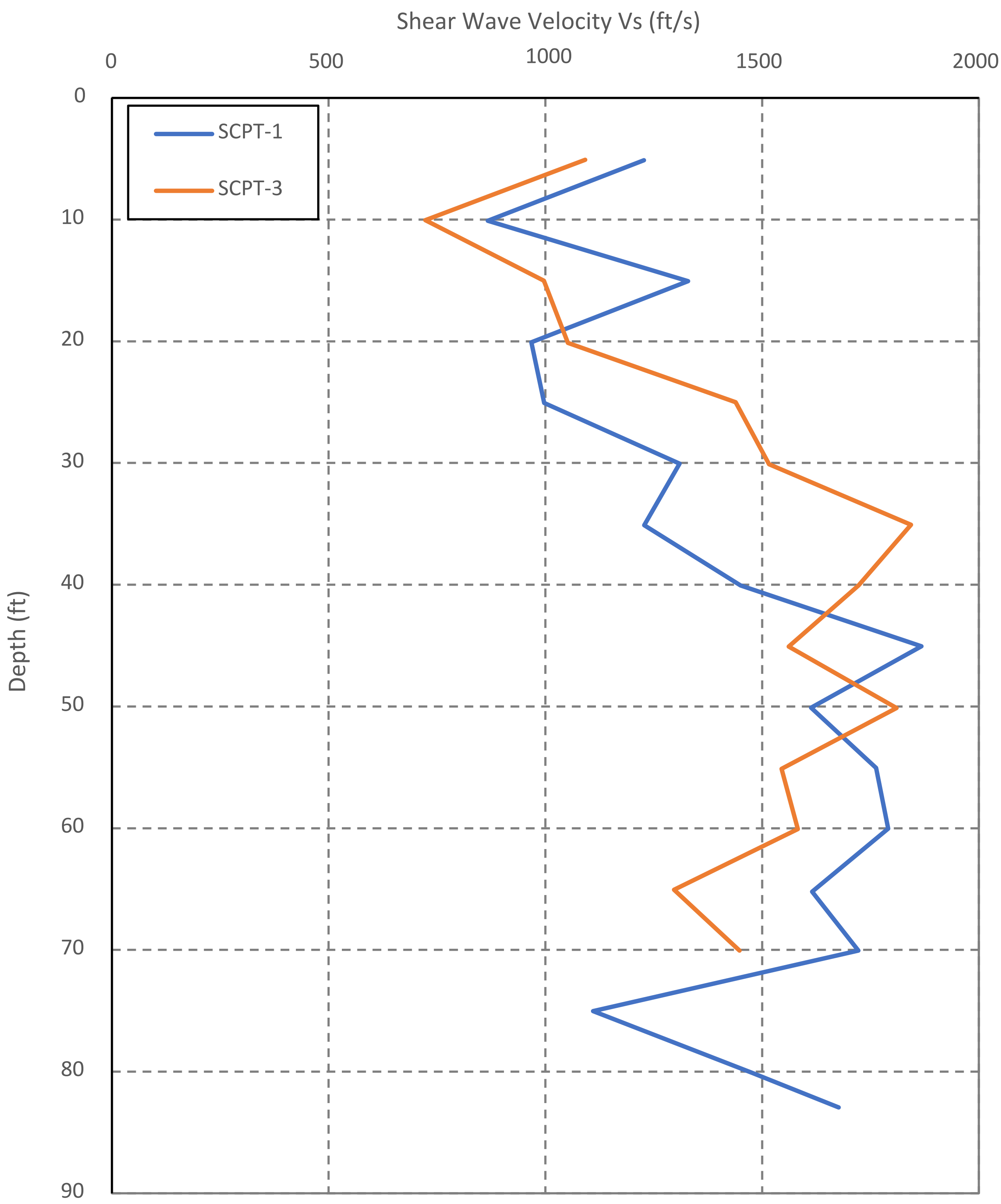
Project: **City of Inglewood Seismic Retrofits**

Location: **1 W Manchester Blvd., Inglewood, CA 90301**

CPT: SCPT-3

Total depth: 70.17 ft, Date: 10/23/2020

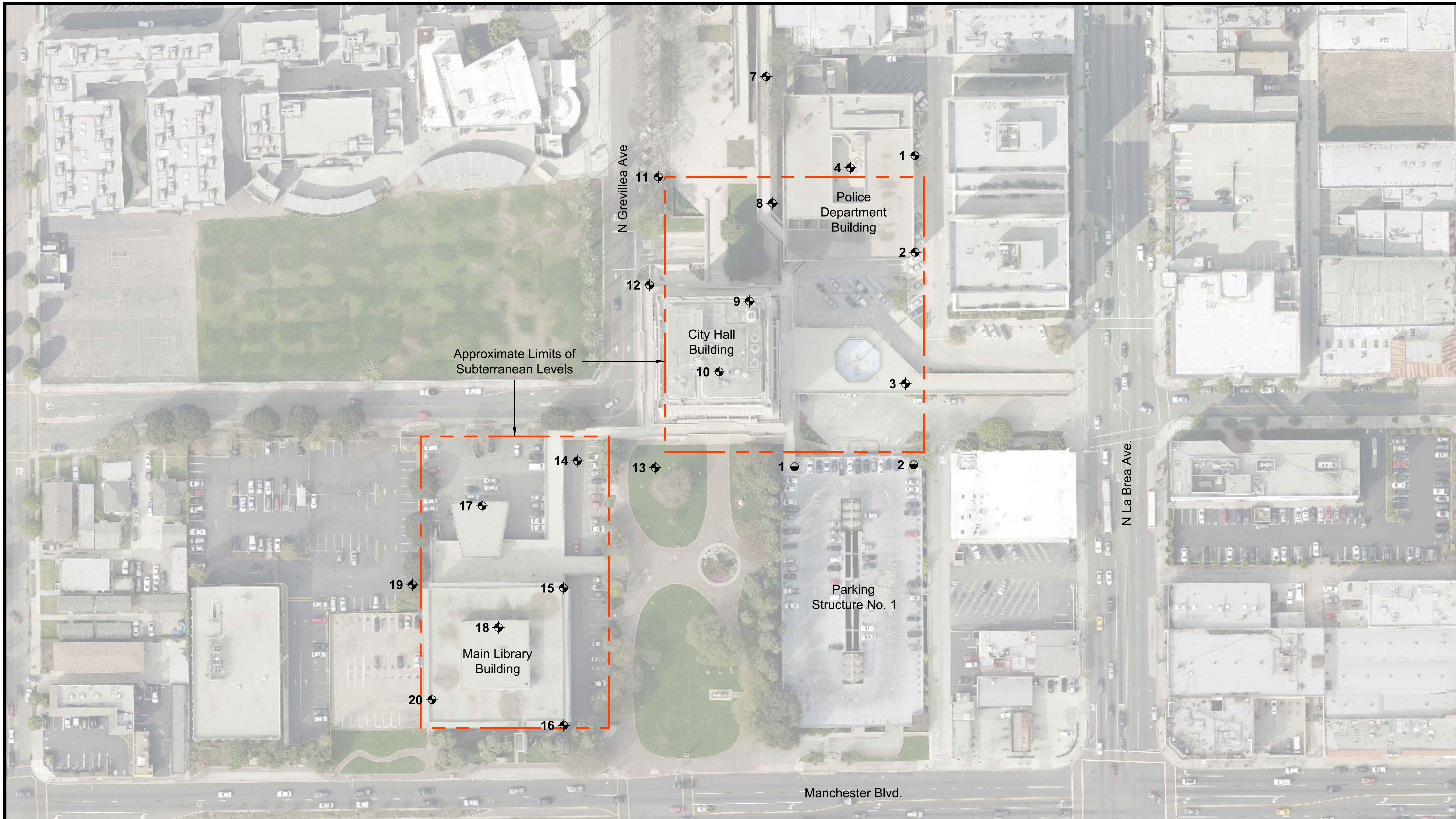




DATE: 11/12/2020	DRAWN BY: LK	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	Shear Wave Velocity Measurements	PROJECT NUMBER: LA-1474
REVIEWED BY: -	APPROVED BY: -			CITY OF INGLEWOOD SEISMIC RETROFITS INGLEWOOD, CALIFORNIA	SCALE: AS SHOWN
PREPARED BY: -					FIGURE NUMBER: A-4

APPENDIX B

PRIOR FIELD INVESTIGATIONS AND LABORATORY TESTING



LEGEND

20 ⌖ PRIOR INVESTIGATION (AMEC-1970)

2 ◒ PRIOR INVESTIGATION (AMEC-1964)

Reference:
Los Angeles County Orthoimagery, 2013

0 50' 100' 200'

SCALE: 1" = 100'

amec foster wheeler

Amec Foster Wheeler
Environment & Infrastructure, Inc.
6001 Rickenbacker Road
Los Angeles, CA 90040
Phone (323) 889-5300
Fax (323) 721-6700

PROPOSED SEISMIC EVALUATION
INGLEWOOD CIVIC CENTER
1 WEST MANCHESTER BOULEVARD, INGLEWOOD, CA

LT,LNG:	
PREPARED BY:	VMN
SCALE:	1" = 100'
DRAWN:	VMN
CHKD:	MM
DATE:	11/15/2017

PLOT PLAN

FIGURE NO.
2

PROJECT NO.
4953-17-1031

Path: G:\4953_Geotech\2017\171031 Inglewood Civic Ctr Seismic Eval\CAD\DWG\4953-17-1031_Fig-2_PlotPlan.dwg [B-17x11]
Date: November 15, 2017 - 3:00pm By: vo.nguyen

PRIOR FIELD INVESTIGATIONS AND LABORATORY TESTING (AMEC, 1970)

GROUP DELTA

JOB A-70230 DATE 10-26-70 DR T.F. O.E. JM CHKD [illegible]

BORING 1

DATE DRILLED October 6, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 125.7 *

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		DESCRIPTION
125						3" Asphaltic Paving - 6" Sand Base
					ML	CLAYEY SILT - brown
		12.5	120	■		
	5					
120		13.1	118	■	SM	SILTY SAND - fine, brown
		14.8	122	■	CL	SILTY CLAY - brown
115	10					
						Sandy
		18.0	113	■		
110	15				ML	SANDY SILT - brown
		15.2	111	■		
					ML	CLAYEY SILT - brown
105	20	33.0	87	■		
					SM	SILTY SAND - fine, greyish-brown
		14.2	114	■		
	25	8.2	106	■		
100					SP	SAND - fine, greyish-brown
		6.3	109	■		
					SM	SILTY SAND - fine, brown
	30	8.6	109	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 1 (CONTINUED)

DATE DRILLED: October 6, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE	Classification	Description
95						
		8.9	119	■		Clayey
90	35					
		7.8	117	■		
85	40					
					SP	SAND - fine, light brown
		4.7	108	■		
80	45					
		6.2	105	■		
	50					

NOTE: Water not encountered. No caving.

* Elevations refer to datum of reference drawing; see Plate 1.

Soils classified in accordance with the Unified Soil Classification System.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A70230 DATE 10-26-70 DR JZ O.E. JK CHKD. ML

JOB A-70230 DATE 10-26-70 DR T.F. O.E. JM CHKD. [illegible]

BORING 2

DATE DRILLED October 7, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 125.1

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
125						4" Asphaltic Paving - 4" Silty Sand Base
		11.1	109		ML	SANDY SILT - brown
		11.4	122			
120	5				SM	SILTY SAND - fine, brown
		10.1	117			
					CL	SILTY CLAY - brown
115	10	14.8	121			
		14.4	115			
110	15				ML	SANDY SILT - light greyish-brown
		16.1	110			
105	20					
		12.0	117			
					SM	SILTY SAND - fine, light greyish-brown
		11.2	121			
	25					

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 2 (CONTINUED)
DATE DRILLED: October 7, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
		9.3	107	■		
					SP	SAND - fine, light brown
		2.8	104	■		
100	30	17.0	95	■	SM	SILTY SAND - fine, greyish-brown
					SC	CLAYEY SAND - fine, greyish-brown
95	35	10.1	127	■		
					SM	SILTY SAND - fine, clayey, brown
90	40	8.4	123	■		
85	45	10.1	105	■	SP	SAND - fine, light brown
	50	6.9	95	■		

NOTE: Water not encountered. No caving.

JOB A-70230 DATE 10-26-70 DR T.F. O.E. JM CHKD.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-26-70 DR 7.7. O.E. JM CHKD

BORING 3

DATE DRILLED: October 7, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 123.5

Elevation (ft.)	Depth (ft.)	Moisture (% of dry wt.)	Dry Density (lbs./cu.ft.)	Sample	Soil	Description
						9" Concrete Slab
					ML	SANDY SILT - brown
120		14.2	101	■		
	5	13.5	121	■	SC	CLAYEY SAND - fine, brown
115		15.1	113	■	CL	SILTY CLAY - light brown
	10	18.4	106	■		
110					ML	SANDY SILT - light greyish-brown
	15	16.1	106	■		
105		19.7	99	■		
	20	13.9	113	■	SM	SILTY SAND - fine, light brown
100		10.3	112	■		
	25				SP	SAND - fine, light brown
		5.9	101	■	SM	SILTY SAND - fine, light brown
95						
	30	7.7	94	■		
90					SC	CLAYEY SAND - fine, brown
	35	11.1	125	■		
85						
	40	7.3	111	■		
80						
	45					

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-26-70 DR T.T. OE JM MU CHKD. [illegible]

BORING 4

DATE DRILLED: October 6, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 122.8

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
					CL ML	FILL - CLAY and SILT - brown 4" clay pipe Piece of concrete Piece of brick
120		15.4	112			
	5					
					CL	FILL - SILTY CLAY - mottled brown Piece of concrete
115					CL	SILTY CLAY - brown
		16.8	116			
	10					
					ML	SANDY SILT - greyish-brown
110		15.3	113			
	15				SM	SILTY SAND - fine, light brown
					ML	CLAYEY SILT - brown
105		15.3	111			
	20				ML	SANDY SILT - greyish-brown
					SM	SILTY SAND - fine, greyish-brown
100		13.5	113			
					SP	SAND - fine, light grey
	25	1.7	103			

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-26-70 DR T.F. O.E. ____ CHKD. ____

BORING 4 (CONTINUED)

DATE DRILLED: October 6, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
95		2.2	103	■		
	30				SM	SILTY SAND - fine, brown
		7.1	117	■		
90						Few gravel
	35					
85						
	40				SP	SAND - fine, brown
		5.7	110	■		
80						
	45					Light brown
		5.1	101	■		
75						
	50	5.2	105	■		
70						
	55					

NOTE: 1) Water not encountered. Caving from 23½' to 30' (to 2' in diameter).

2) Encountered 4" clay pipe at 18". Moved drilling rig 3' east and completed boring.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-26-70 DR T.K. O.E. JK CHKD. __

BORING 7

DATE DRILLED: October 7, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 119.8

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
					ML	SANDY SILT - brown
		13.5	119	■		
115	5	17.3	113	■	CL	SANDY CLAY - brown
					CL	SILTY CLAY - light brown
		16.1	115	■		
110	10	19.9	105	■		
					ML	SANDY SILT - light greyish-brown
		17.8	109	■		
105	15					
		31.0	90	■		
					SM	SILTY SAND - fine, light greyish-brown
	20	15.3	113	■		

NOTE: Water not encountered. No caving.

LOG OF BORING

A2-37 of 84

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-26-70 DR T.L. O.E. VM CHKD. [illegible]

BORING 8

DATE DRILLED: October 6, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 119.3

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		Description
					ML	SANDY SILT - brown
		13.6	120	■		
					CL	SANDY CLAY - light brown
115	5					
					CL	SILTY CLAY - brown
		14.2	116	■		
						Light brown
110	10					
		26.1	98	■		
					ML	SANDY SILT - greyish-brown
105	15	20.4	103	■		
		28.6	90	■		
		18.0	108	■		
100	20					
		15.6	111	■		
					SM	SILTY SAND - fine, light brown
95	25	19.5	94	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 8 (CONTINUED)
DATE DRILLED October 6, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE
90	30	3.8	102	■
85	35	8.6	120	■
80	40	8.1	111	■
75	45	7.4	108	■
70	50	5.9	99	■

SP SAND - fine, few gravel, light grey

SC CLAYEY SAND - fine, brown

SP SAND - fine, light brown
Few gravel

NOTE: Water not encountered. No caving.

JOB A-70230 DATE 10-26-70 DR T.F. O.E. VM ECHKD. [illegible]

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 9

DATE DRILLED: October 6, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 118.3

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE
		10.1	109	■
115	5	12.7	122	■
110		18.7	110	■
	10	17.4	111	■
105	15	18.4	109	■
100		18.4	99	■
	20	19.1	105	■
95	25	11.0	97	■
90	30	1.9	101	■

	3" Asphaltic Paving
ML	SANDY SILT - brown
CL	SANDY CLAY - brown
CL	SILTY CLAY - brown
ML	SANDY SILT - light greyish-brown
SM	SILTY SAND - fine, light brown
SP	SAND - fine, few gravel, light grey

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

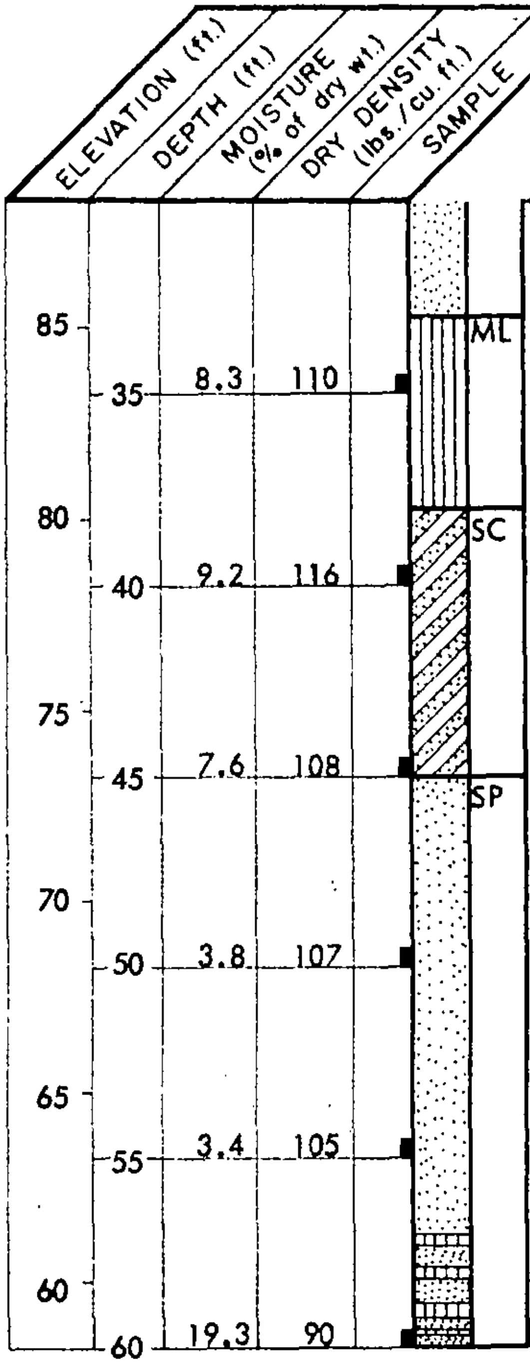
JOB A-70230 DATE 10-26-70 DR T.F. O.E. Jh CHKD. MC 28

LEROY CRANDALL AND ASSOCIATES

BORING 9 (CONTINUED)

DATE DRILLED: October 6, 1970

EQUIPMENT USED: 18"-Diameter Bucket



SANDY SILT - clayey, light brown

CLAYEY SAND - fine, brown

SAND - fine, light brown

Layers of Sandy Silt

NOTE: Water not encountered. Caving from 28' to 33' (to 3' in diameter).

LOG OF BORING

A2-41 of 84

LEROY CRANDALL AND ASSOCIATES

PLATE A-14

JOB A-70230 DATE 10-26-70 DR T.T. O.E. VA CHKD. [illegible]

JOB A-70230 DATE 10-26-70 DR J.F. O.E. J/h CHKD mcB

BORING 10

DATE DRILLED: October 8, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 118.9

MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)		
		ML	SANDY SILT - brown
13.2	118		
		SM	SILTY SAND - fine, light brown
13.0	115		
		CL	SILTY CLAY - brown
15.7	114		
19.4	107		
		ML	CLAYEY SILT - light brown
24.5	99		
31.6	84		Layer of Clay
		ML	SANDY SILT - light greyish-brown
17.6	112		
28.0	91		
14.4	104		
		SM	SILTY SAND - fine, light grey
8.8	109		
		SP	SAND - fine, light grey
22.9	106		Layer of Silty Sand

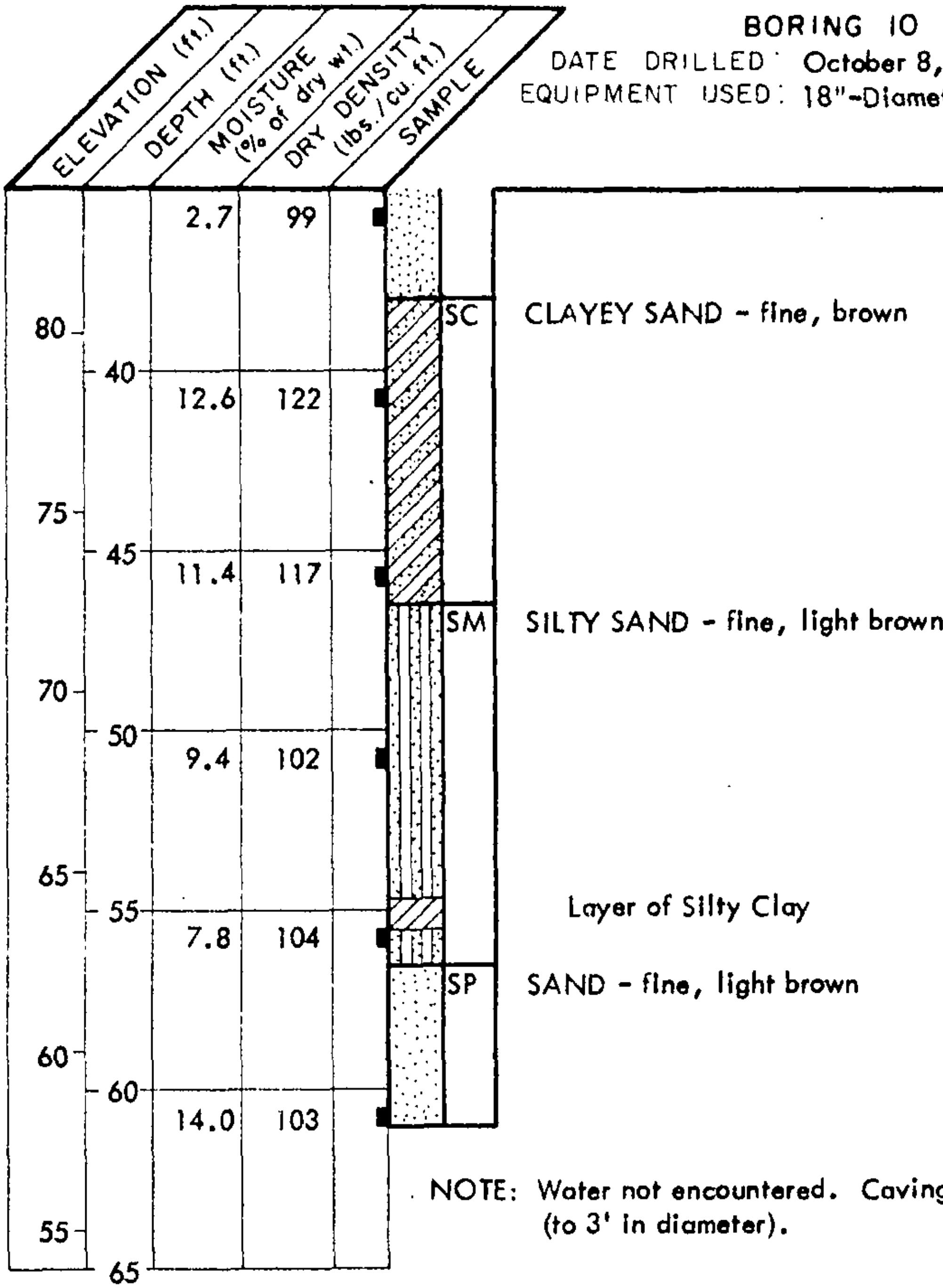
ELEVATION (ft): 115, 110, 105, 100, 95, 90, 85
DEPTH (ft): 5, 10, 15, 20, 25, 30, 35

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 10 (CONTINUED)
DATE DRILLED: October 8, 1970
EQUIPMENT USED: 18"-Diameter Bucket



NOTE: Water not encountered. Caving from 32' to 38' (to 3' in diameter).

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-26-70 DR T.F. O.E. JM CHKD.

BORING 11

DATE DRILLED: October 5, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 118.8

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
					ML	SANDY SILT - brown
					SM	SILTY SAND - fine, brown
		12.8	116	■		
115						
	5	15.0	115	■		
					SC	CLAYEY SAND - fine, brown
110		17.3	114	■		
	10				CL	SILTY CLAY - brown
		16.2	116	■		
105						
	15	17.9	111	■		
					SM	SILTY SAND - fine, greyish-brown
		21.6	105	■		Layer of Clayey Silt
100					ML	SANDY SILT - brown
	20	20.9	103	■		Layers of Clayey Silt
95						
	25	23.8	101	■		
					SM	SILTY SAND - fine, light greyish-brown
90		7.1	100	■		
	30				SP	SAND - fine, light greyish-brown
		8.8	103	■		
85	35					

(CONTINUED ON FOLLOWING PLATE)

JOB A-70230 DATE 10-27-70 DR T.F. O.E. VH CHKD. RK

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 11 (CONTINUED)
DATE DRILLED: October 5, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
		4.6	108	■		Layer of Sandy Silt
80		17.9	112	■	ML	SANDY SILT - grey
	40				SM	SILTY SAND - fine, brown
		9.3	131	■		
75	45	9.0	118	■		
					SP	SAND - fine, light brown
70	50	6.6	109	■		
65	55	5.1	114	■		
60	60	4.3	109	■		

NOTE: Water not encountered. No caving.

JOB A-70230 DATE 10-27-70 DR T.K. O.E. JM CHKD RC

LOG OF BORING

A2-45 of 84

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR T.K. O.E. JM CHKD

BORING 12

DATE DRILLED: October 8, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 116.1

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE
115				
		11.5	118	■
110	5	11.6	120	■
		15.7	116	■
105	10	16.3	114	■
100	15	29.2	90	■
		17.2	107	■
95	20			
		17.7	108	■
90	25	10.5	110	■
		7.7	109	■
85	30	19.0	101	■
		15.3	108	■
80	35	13.6	121	■
	40			

4" Asphaltic Paving - 8" Base Course

SM SILTY SAND - fine, brown

CL SILTY CLAY - brown

ML CLAYEY SILT - brown

ML SANDY SILT - brown

Layer of Clayey Silt

SM SILTY SAND - fine, brown

SP SAND - fine, light grey

ML SANDY SILT - greyish-brown

SM SILTY SAND - fine, brown

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 12 (CONTINUED)
DATE DRILLED: October 8, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
75						
		10.8	120	■		
	45					
70						
		10.6	114	■		
	50				SP	SAND - fine, light brown
65						
		4.7	108	■		
	55					
60						
		3.4	103	■		
	60					Layer of Silty Sand
55						
		12.8	94	■		Layer of Sandy Silt
					ML	CLAYEY SILT - brown
	65					
50						
		33.9	87	■		
					ML	SANDY SILT - light brown
	70					
					SP	SAND - fine, light brown
45		9.7	106	■		
						Layer of Sandy Silt
	75					
40						
		3.3	112	■		
	80					

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR T.F. O.E. JM [handwritten initials] CHKD [handwritten initials]

JOB A-70230 DATE 10-27-70 DR T.F. O.E. VM CHKD. RCB

BORING 13

DATE DRILLED October 7, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 115.8

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
115						4½" Asphaltic Paving - 6" Base Course
					SM	SILTY SAND - fine, brown
		9.7	117	■		
110	5					
		13.1	121	■		
					CL	SILTY CLAY - brown
		16.3	113	■		
105	10					
		16.3	112	■		
		18.4	112	■		
100	15					
		28.8	95	■		
					SM	SILTY SAND - fine, brown
		12.4	115	■		
95	20					
					ML	SANDY SILT - greyish-brown
		16.2	110	■		
					ML	CLAYEY SILT - greyish-brown
		34.3	86	■		
	25					

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 13 (CONTINUED)

DATE DRILLED: October 7, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
90						
		14.7	104	■	SM	SILTY SAND - fine, brown
	30	5.7	111	■	SP	SAND - fine, greyish-brown
85						Layer of Silty Sand
						Light grey
	35	4.7	95	■		Layer of Clayey Silt
80					SM	SILTY SAND - fine, brown
	40	11.7	125	■		
75						
	45	11.7	123	■		
70						
	50	9.1	118	■		
65					SP	SAND - fine, light grey
	55	5.3	110	■		Layer of Silty Sand
60						
	60	4.4	104	■		
55						
	65					

NOTE: Water not encountered. No caving.

JOB A-70230 DATE 10-27-70 DR T.T. O.E. JA CHKD RC

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR T.K. O.E. JM CHKD RCB

BORING 14

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 115.7

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
115					ML	SANDY SILT - brown
		6.6	108	■		
		4.6	108	■		
110	5	7.2	113	■		
					SC	CLAYEY SAND - fine, brown
		11.9	116	■		
					CL	SANDY CLAY - light brown
105	10	16.5	111	■		
					CL	SILTY CLAY - light brown
	15	18.5	109	■		
100						
					ML	SANDY SILT - light brown
	20	16.5	97	■		

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 15

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 116.4

Moisture (% of dry wt.)	Dry Density (lbs./cu. ft.)	Classification	Description
			3" Asphaltic Paving - 4" Base Course
		SM	FILL - SILTY SAND - fine, few pieces of concrete, brown
8.2	128	SM	SILTY SAND - fine, brown
8.6	113		
7.7	118		
8.1	117	SC	CLAYEY SAND - fine, brown
15.1	116	CL	SILTY CLAY - brown
18.9	110		
28.3	94	ML	CLAYEY SILT - brown
		ML	SANDY SILT - brown
16.2	113		Layer of Sand
		SM	SILTY SAND - fine, brown
18.5	105	ML	CLAYEY SILT - brown
		SM	SILTY SAND - fine, brown
8.1	110	SP	SAND - fine, brown

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR T.F. O.E JM CHKD.

BORING 15 (CONTINUED)

DATE DRILLED: October 9, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
80						Layer of Silty Sand
		14.7	114	■	ML	SANDY SILT - brown
	40					
75					SM	SILTY SAND - fine, brown
		18.5	109	■	ML	CLAYEY SILT - greyish-brown
	45					
70						
		12.0	122	■	SM	SILTY SAND - fine, greyish-brown
	50					

NOTE: Water not encountered. Caving from 36½' to 38½' (to 2½' in diameter).

LOG OF BORING

A2-52 of 84

LEROY CRANDALL AND ASSOCIATES

PLATE A-25

JOB A-70230 DATE 10-27-70 DR. J.F. O.E. JH CHKD. [illegible]

BORING 16

DATE DRILLED: October 12, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 116.1

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE
115				
		10.5	114	■
	5	8.7	113	■
110		8.9	107	■
		6.4	114	■
105	10	13.1	114	■
100	15	14.7	111	■
95	20	4.0	105	■
	25			

3" Asphaltic Paving - 6" Base Course

SM SILTY SAND - fine, brown

SC CLAYEY SAND - fine, brown

CL SILTY CLAY - light brown

SM SILTY SAND - fine, light brown

SP SAND - fine, light brown

SM SILTY SAND - fine, light brown

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

A2-53 of 84

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR JK O.E. JH CHKD RL

JOB A-70230 DATE 10-27-70 DR TL O.E. JH CHKD

BORING 16 (CONTINUED)

DATE DRILLED: October 12, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
90		17.0	114	■	ML	CLAYEY SILT - greyish-brown
					ML	SANDY SILT - light brown
85	30	28.6	88	■		Clayey
					SM	SILTY SAND - fine, light brown
80	35	7.5	111	■		
					SP	SAND - fine, light brown
75	40	20.2	102	■	ML	SANDY SILT - light greyish-brown
70	45	16.5	112	■		
	50	16.4	108	■		

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR T.F. O.E. JH CHKD. ALB

BORING 17

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 115.5

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
115						3" Asphaltic Paving
					ML	SANDY SILT - brown
		5.3	105	■		
		7.2	119	■		
110	5					
					SM	SILTY SAND - fine, light brown
		6.1	106	■		
		7.8	120	■		
					SC	CLAYEY SAND - fine, brown
105	10					
					CL	SILTY CLAY - brown
		14.5	110	■		
100	15					
		16.6	111	■		
					ML	SANDY SILT - light brown
95	20					
		11.1	103	■		
	25					

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-27-70 DR TZK O.E. JM CHKD REL

BORING 17 (CONTINUED)

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		Description
90		20.0	98	■	SM	SILTY SAND - fine, brown
						Layer of Clay
85	30	10.3	105	■		
					SP	SAND - fine, light brown
80	35	5.1	102	■		
					ML	SANDY SILT - grey
75	40					
70	45	18.0	108	■		
					SC	CLAYEY SAND - fine, brown
65	50	10.4	120	■		
	55					

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 18

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 116.6

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
						2" Asphaltic Paving - 6" Base Course
115					ML	SANDY SILT - brown
		6.0	117	■		
	5				SM	SILTY SAND - fine, brown
		6.6	102	■		
110						
		8.6	116	■		
	10	7.3	109	■		
105		14.1	116	■	CL	SANDY CLAY - brown
	15	15.4	115	■		
100						
					CL	SILTY CLAY - brown
	20	20.6	102	■		
95					ML	SANDY SILT - light brown
					SM	SILTY SAND - fine, brown
	25	13.3	102	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

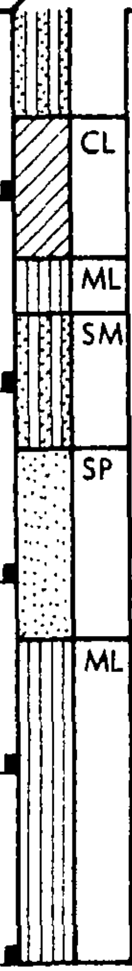
LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-23-70 DR T.F. O.E. VM CHKD lkb

JOB A-70230 DATE 10-23-70 DR T.F. O.E. JM CHKD. KD

BORING 18 (CONTINUED)

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket



ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE	
90					
				CL	SILTY CLAY - brown
	30	36.2	82	■	
85				ML	CLAYEY SILT - light brown
				SM	SILTY SAND - fine, brown
	35	8.4	105	■	
80				SP	SAND - fine, light brown
	40	5.1	97	■	
75				ML	SANDY SILT - light brown
	45	14.7	110	■	
70					
	50	13.5	105	■	

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 19

DATE DRILLED: October 9, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 115.2

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
115						2" Asphaltic Paving - 3" Silty Sand Base
		8.2	106	■	SM	SILTY SAND - fine, brown
		8.1	123	■		
110	5					
		7.7	117	■		
		6.8	113	■		
105	10					
		9.6	116	■		
		15.6	115	■	CL	SILTY CLAY - brown
100	15					
					ML	CLAYEY SILT - greyish-brown
		17.4	114	■		
95	20				SM	SILTY SAND - fine, brown
		9.1	108	■		
	25					

(CONTINUED ON FOLLOWING PLATE)

JOB A-70230 DATE 10-23-70 DR T.F. O.E. JM CHKD [illegible]

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 19 (CONTINUED)

DATE DRILLED: October 9, 1970
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu ft.)	SAMPLE		
90						
		32.5	87		ML	CLAYEY SILT - brown
85	30					
		6.4	112		SP	SAND - fine, light brown
80	35					
		3.1	101			
75	40				ML	SANDY SILT - greyish-brown
		16.0	117			
70	45					
		16.2	113			
	50					

NOTE: Water not encountered. No caving.

JOB A-70230 DATE 10-23-70 DR T.F. O.E. JM CHKD. AH

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

JOB A-70230 DATE 10-23-70 DR ____ O.E. JM ____ CHKD. MWB

BORING 20

DATE DRILLED October 12, 1970

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 115.1

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		DESCRIPTION
115						2" Asphaltic Paving
					SM	SILTY SAND - fine, brown
		8.0	97	■		
		8.1	114	■		
110	5	8.0	116	■		
		9.5	113	■		
105	10				SC	CLAYEY SAND - fine, brown
		9.3	107	■		
					ML	SANDY SILT - light brown
100	15	17.8	106	■		
					ML	CLAYEY SILT - light brown
95	20	17.9	104	■		
					SM	SILTY SAND - fine, light brown
	25	12.9	95	■		

(CONTINUED ON FOLLOWING PLATE)

LOG OF BORING

LEROY CRANDALL AND ASSOCIATES

BORING 20 (CONTINUED)

DATE DRILLED October 12, 1970

EQUIPMENT USED 18"-Diameter Bucket

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE
90				
85	30	30.5	92	■
80	35	26.4	95	■
75	40	4.9	104	■
70	45	13.9	113	■
	50	12.8	114	■

ML CLAYEY SILT - light brown

SP SAND - fine, light brown

ML SANDY SILT - greyish-brown

NOTE: Water not encountered. No caving.

LOG OF BORING

A2-62 of 84

LEROY CRANDALL AND ASSOCIATES

PLATE A-35

JOB A-70230 DATE 10-23-70 DR J.F. O.E. JM CHKD. [illegible]

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 NO. 40 NO. 10 NO. 4 ¾ in. 3 in. (12 in.)

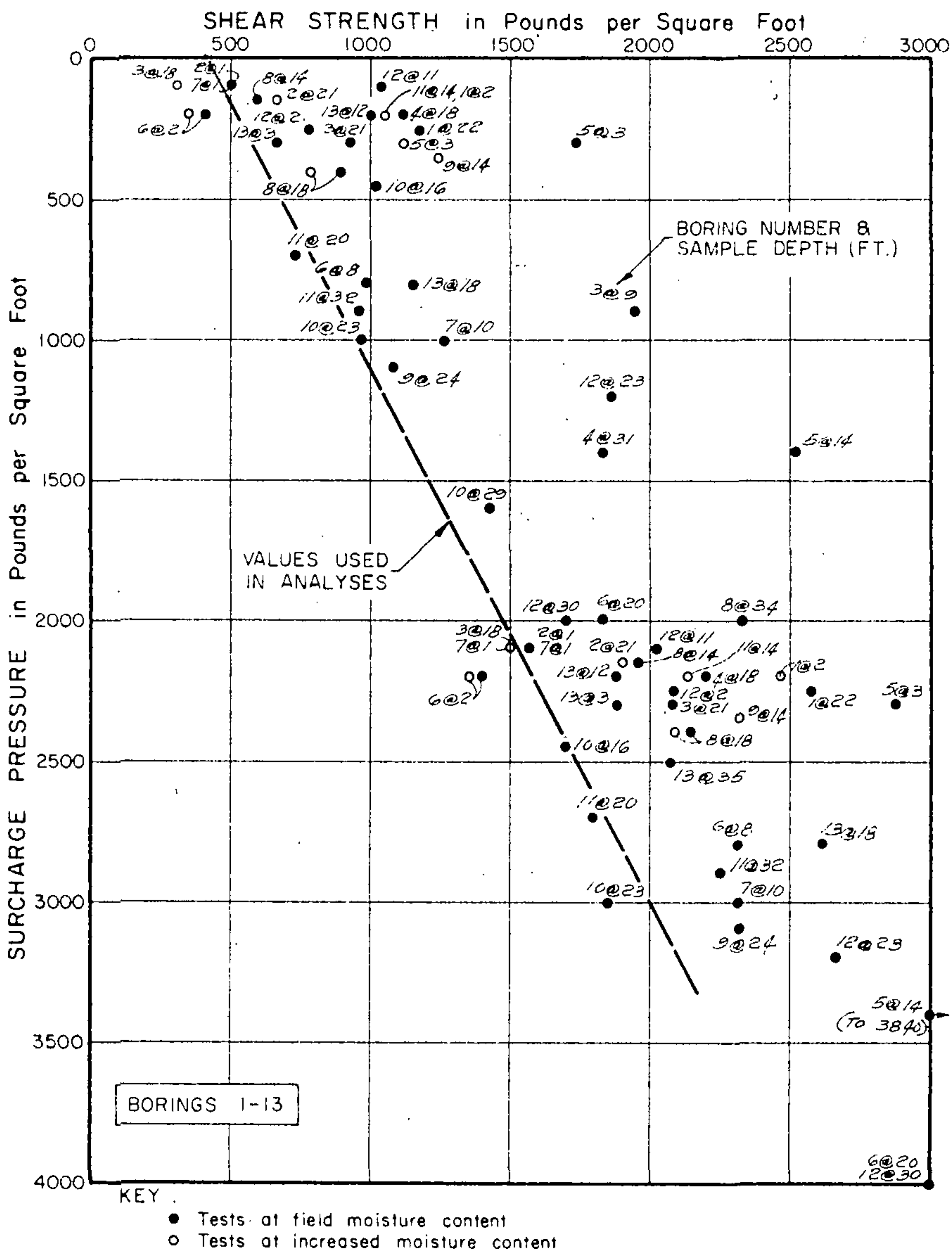
U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

Reference :
The Unified Soil Classification System, Corps of Engineers, U. S. Army Technical Memorandum No. 3-357, Vol. I, March, 1953. (Revised April, 1960)

LEROY CRANDALL & ASSOCIATES

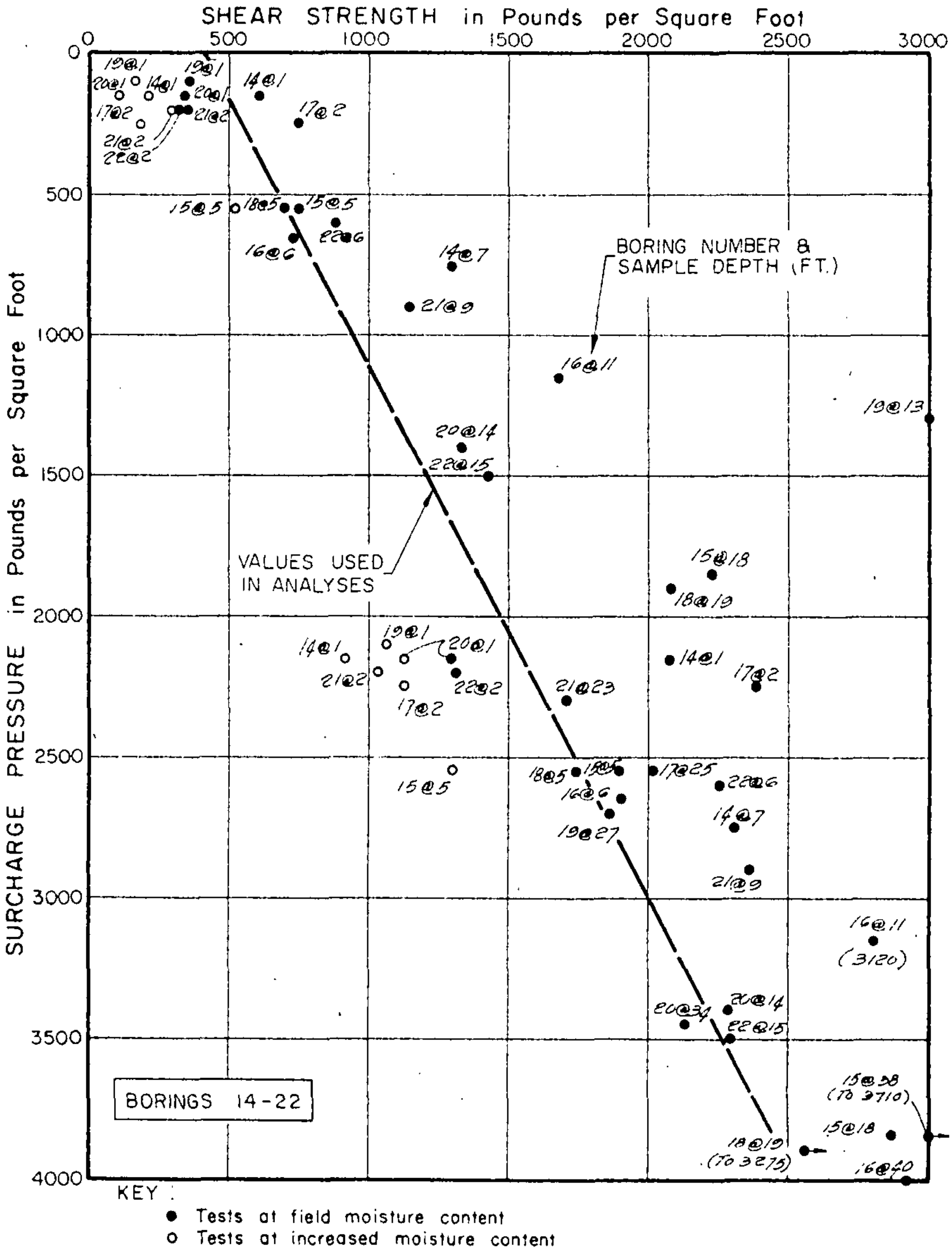
PLATE B



DIRECT SHEAR TEST DATA

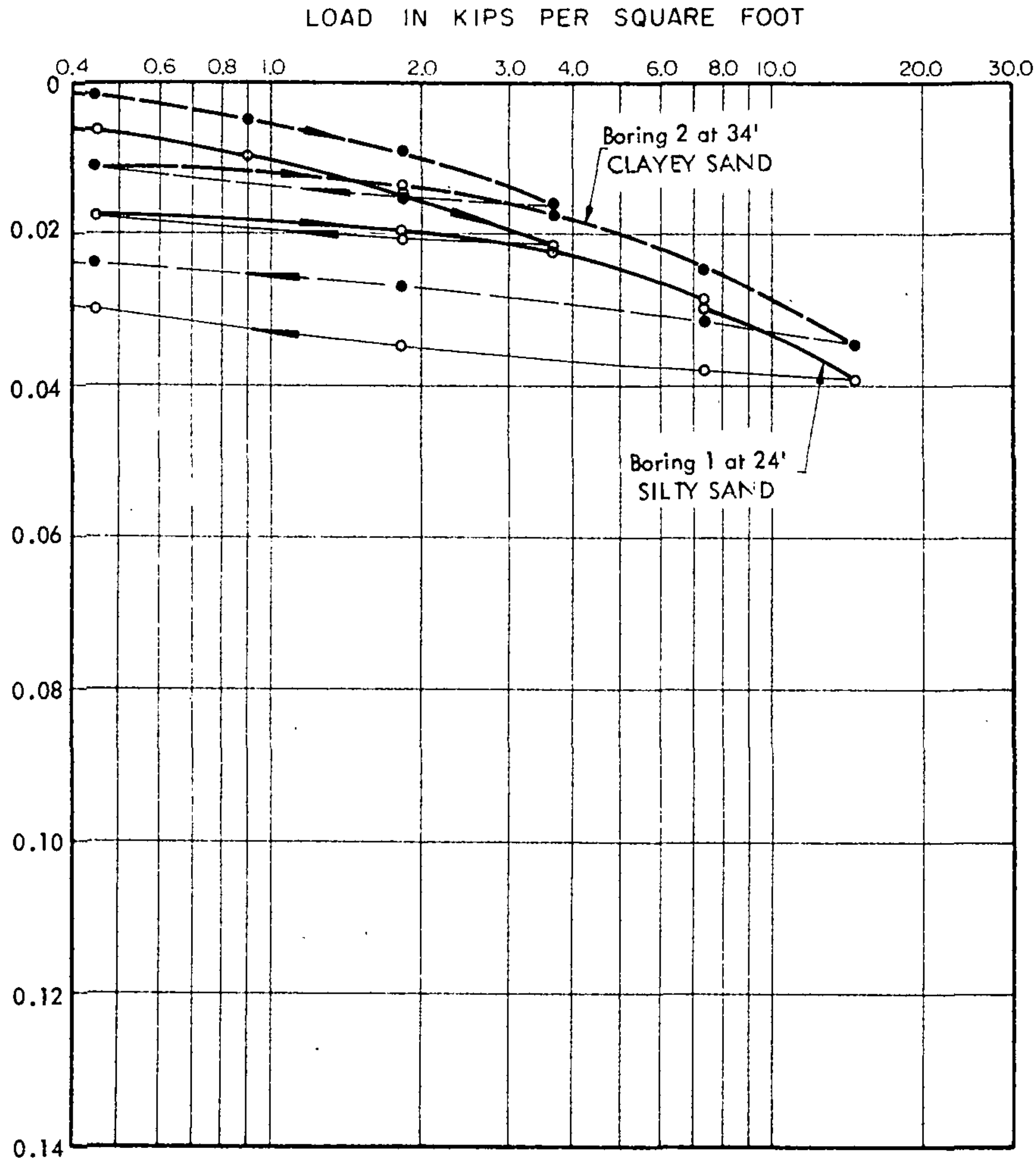
JOB A-70230 DATE 10-22-70 DR. T.F. O.E. JM CHKD.

LEROY CRANDALL & ASSOCIATES



DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES



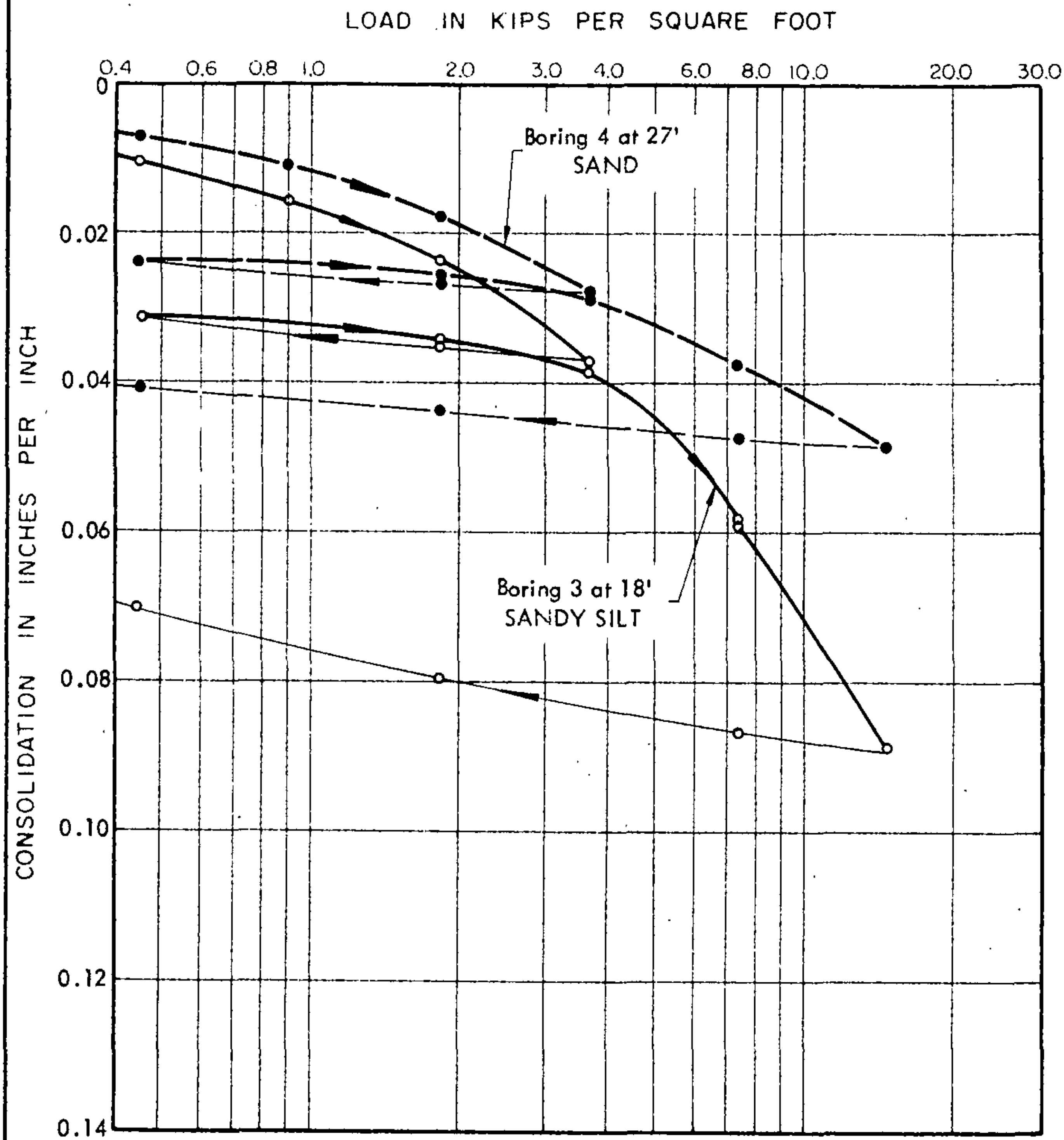
NOTE: Water added to sample from Boring 1 after consolidation under a load of 7.2 kips per square foot. The other sample tested at field moisture content.

JOB A-70230 DATE 10-22-70 DR. T.T. O.E. JH 1b CHKD. [illegible]

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-1

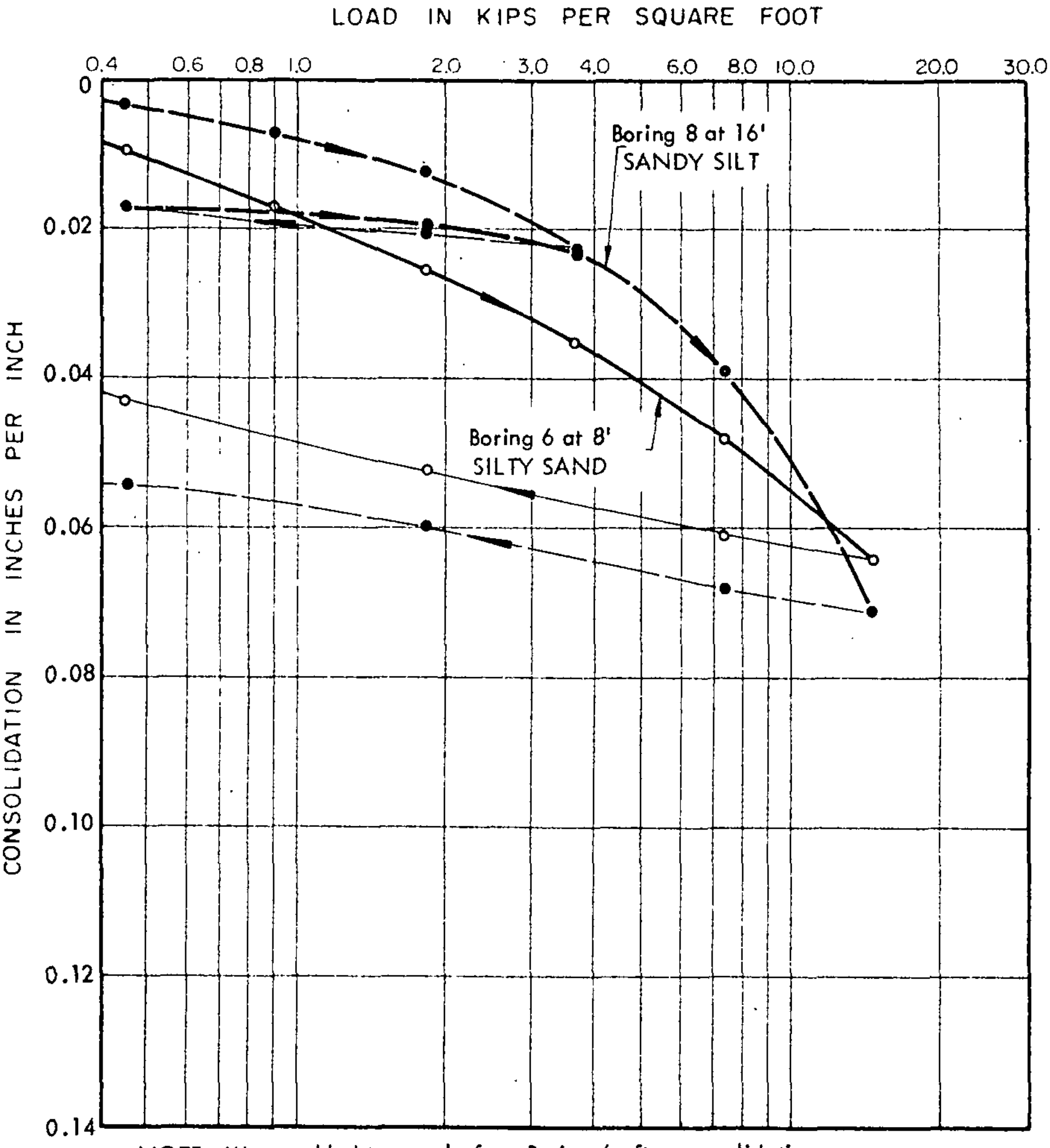


NOTE: Water added to sample from Boring 3 after consolidation under a load of 7.2 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-2



NOTE: Water added to sample from Boring 6 after consolidation under a load of 7.2 kips per square foot. The other sample tested at field moisture content.

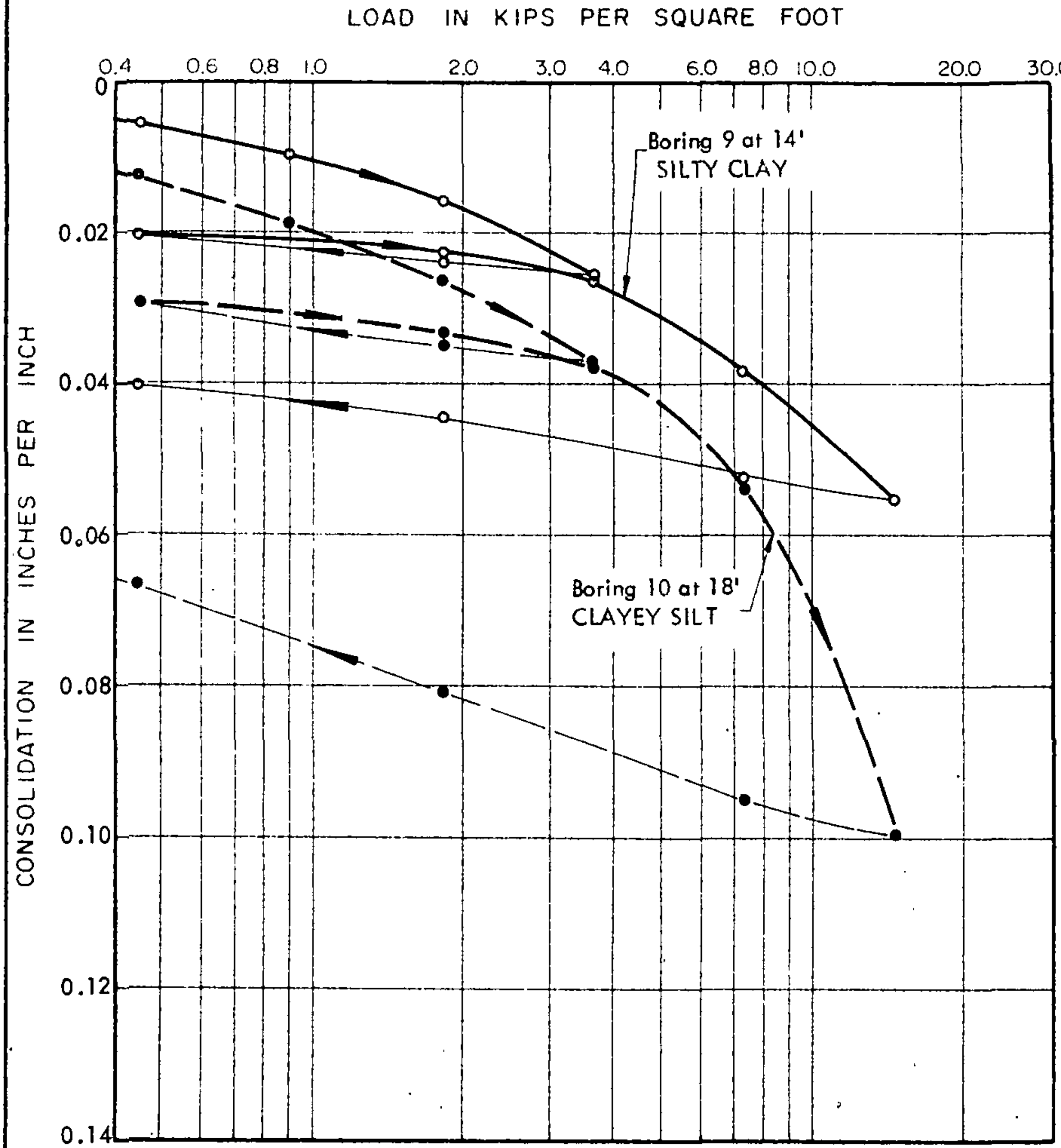
CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-3

JOB A-70230 DATE 10-22-70 DR T.K. CHKD. [illegible] lb O.E. JM

JOB A-70230 DATE 10-22-70 DR. T.F. O.E. JM ib CHKD. [illegible]

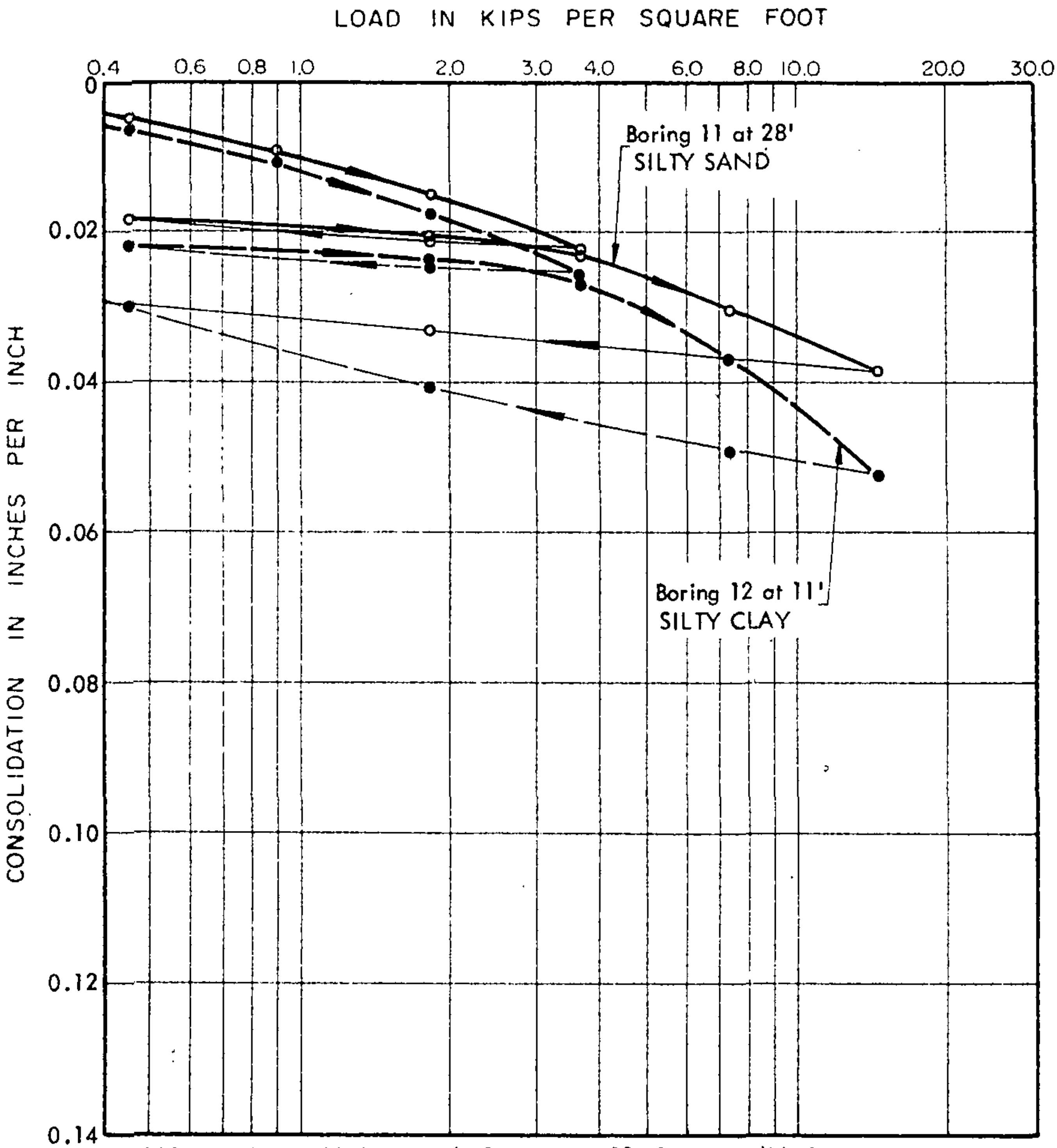


NOTE: Water added to sample from Boring 10 after consolidation under a load of 7.2 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

JOB A-70230 DATE 10-22-70 DR. T.F. O.E. JM CHKD. BB

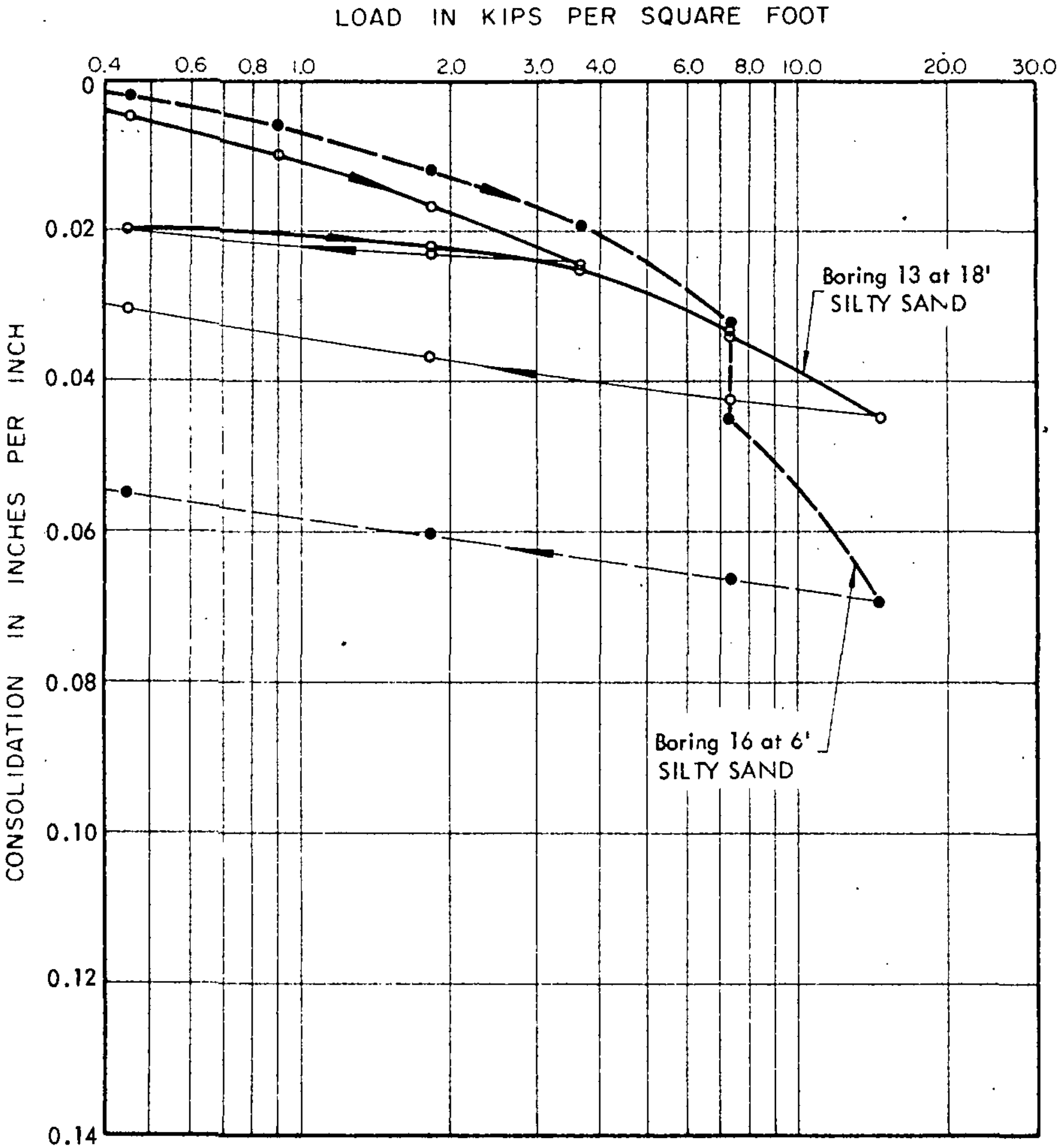


NOTE: Water added to sample from Boring 12 after consolidation under a load of 7.2 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

JOB A-70230 DATE 10-22-70 DR T.K. O.E. JM Ib CHKD. [illegible]



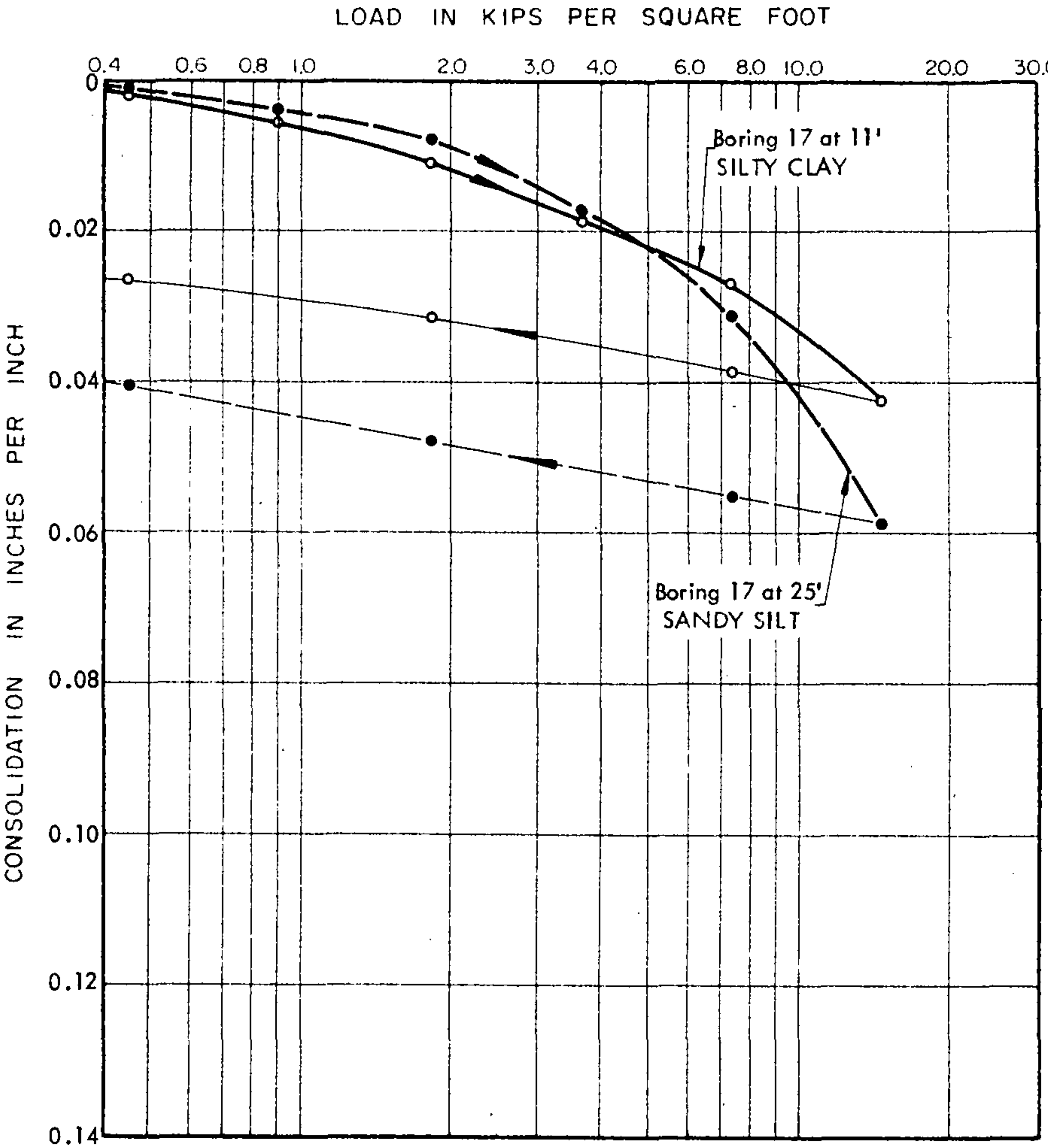
NOTE: Water added to samples after consolidation under a load of 7.2 kips per square foot.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-6

JOB A-70230 DATE 10-22-70 DR T.F. O.E. JM Ib CHKD.

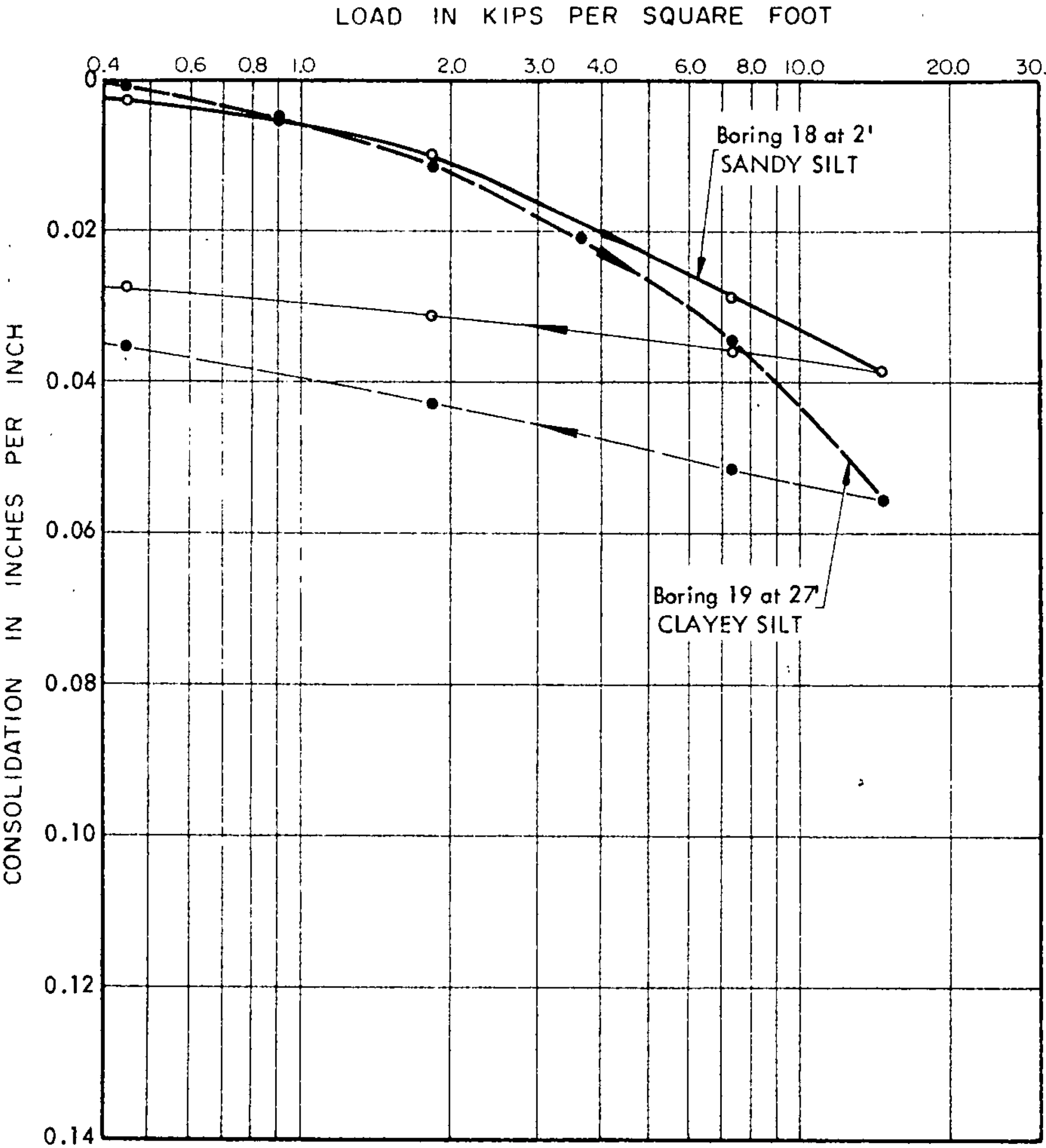


NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LeROY CRANDALL & ASSOCIATES

JOB 4-70230 DATE 10-22-70 DR. ___ CHKD. ___

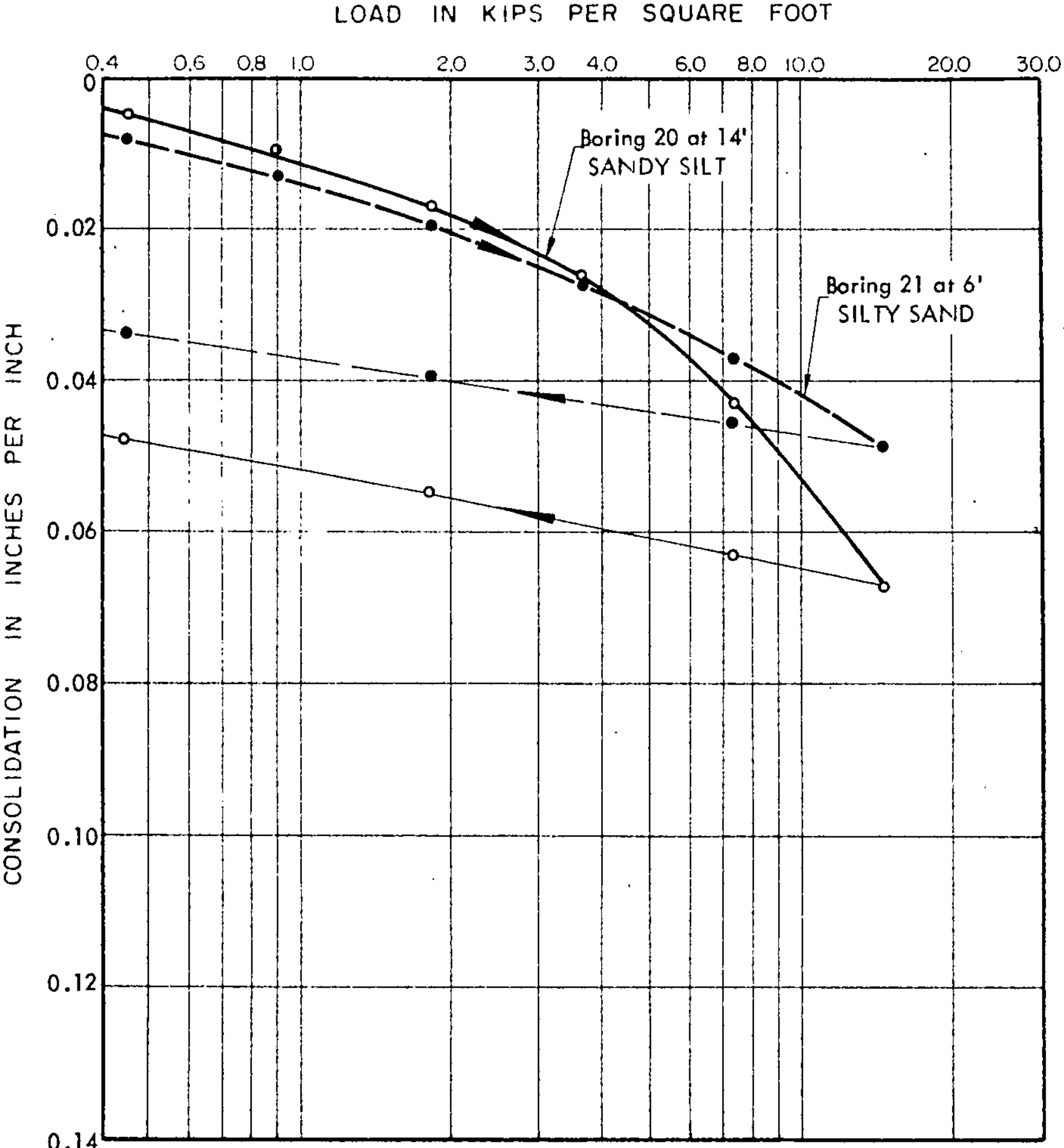


NOTE: Water added to sample from Boring 18 after consolidation under a load of 3.6 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-8



NOTE: Water added to sample from Boring 21 after consolidation under a load of 7.2 kips per square foot. The other sample tested at field moisture content.

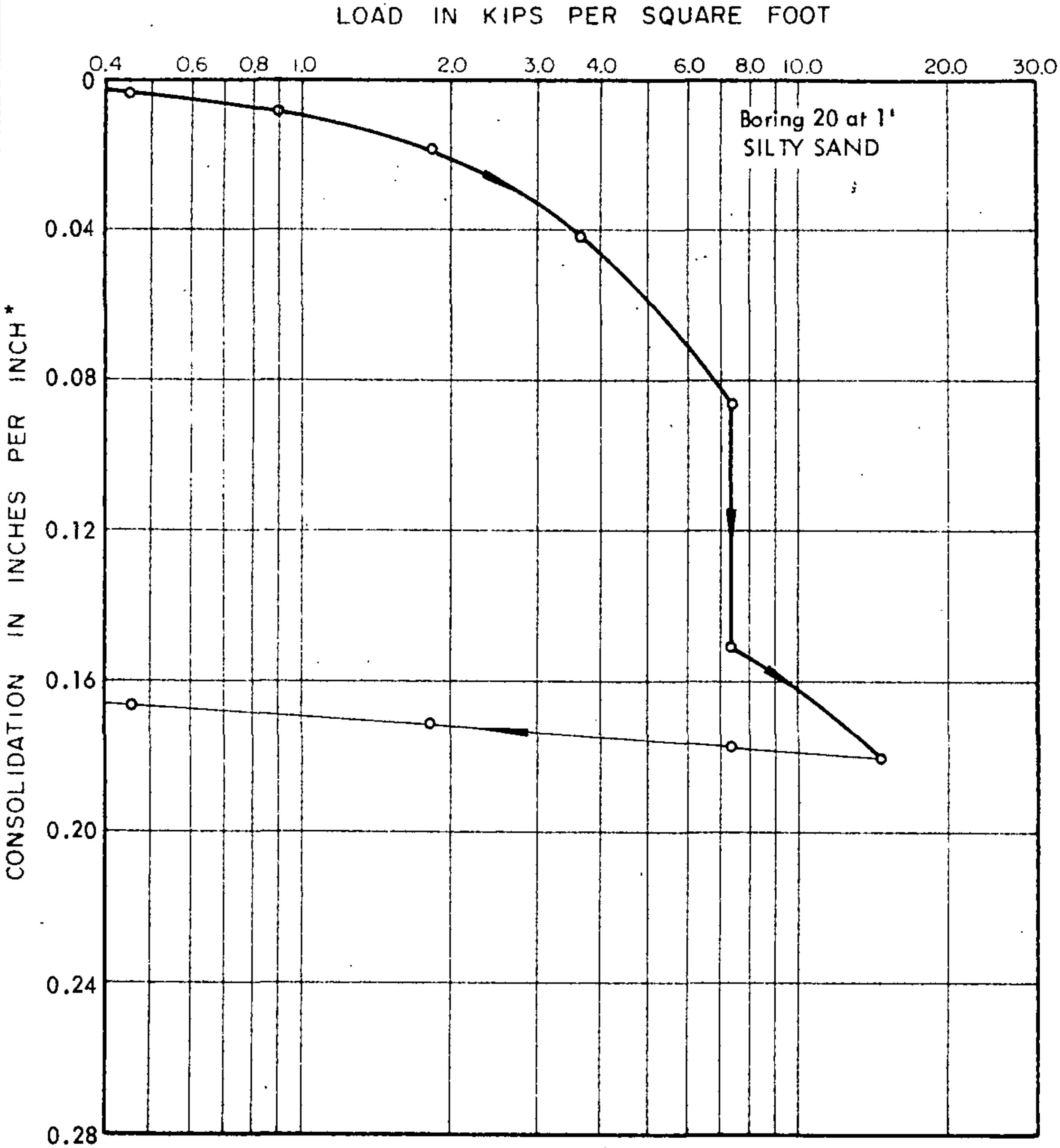
CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-9

JOB A-70230 DATE 10-22-70 DR T.F. O.E. JM lb CHKD. ec 28

JOB A-70230 DATE 10-22-70 DR. T.F. O.E. JM CHKD



NOTE: Water added to sample after consolidation under a load of 7.2 kips per square foot.

* NOTE CHANGE IN SCALE.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

BORING NUMBER AND SAMPLE DEPTH:	2 at 8½' to 14'	10 at 19' to 26'	13 at 1' to 7½'
SOIL TYPE:	SILTY CLAY	SANDY SILT	SILTY SAND
MAXIMUM DRY DENSITY : (Lbs./Cu.Ft.)	118	114	128
OPTIMUM MOISTURE CONTENT : (% of Dry Wt.)	12	14	10

TEST METHOD: ASTM Designation D1557-70 modified to use three layers instead of five.

COMPACTION TEST DATA

LEROY CRANDALL AND ASSOCIATES

BORING NUMBER AND SAMPLE DEPTH:	1 at 2'	7 at 4'	12 at 15'
SOIL TYPE:	CLAYEY SILT	SANDY CLAY	CLAYEY SILT
CONFINING PRESSURE: (Lbs./Sq.Ft.)	200	200	200
FIELD MOISTURE CONTENT: (%)	12.5	17.3	29.2
EXPANSION FROM FIELD TO SOAKED MOISTURE CONTENT: (%)	0	0.1	1.5
SOAKED MOISTURE CONTENT: (%)	13.5	18.0	31.9
SHRINKAGE FROM FIELD TO AIR-DRIED MOISTURE CONTENT: (%)	5.1	5.8	13.0
AIR-DRIED MOISTURE CONTENT: (%)	2.1	3.6	8.6
TOTAL VOLUME CHANGE: (%)	5.1	5.9	14.5

EXPANSION TEST DATA

LEROY CRANDALL AND ASSOCIATES

BORING NUMBER AND SAMPLE DEPTH.	13 at 9'	18 at 2'
SOIL TYPE:	SILTY CLAY	SANDY SILT
CONFINING PRESSURE: (Lbs./Sq.Ft.)	200	200
FIELD MOISTURE CONTENT: (%)	16.3	6.0
EXPANSION FROM FIELD TO SOAKED MOISTURE CONTENT: (%)	0.2	0
SOAKED MOISTURE CONTENT: (%)	17.4	13.0
SHRINKAGE FROM FIELD TO AIR-DRIED MOISTURE CONTENT: (%)	7.4	2.4
AIR-DRIED MOISTURE CONTENT: (%)	3.6	0.7
TOTAL VOLUME CHANGE: (%)	7.6	2.4

EXPANSION TEST DATA

LEROY CRANDALL AND ASSOCIATES

PRIOR FIELD INVESTIGATIONS AND LABORATORY TESTING
(AMEC, 1964)

JOB 64536 DATE 8-27-64 DR MC O.E. CHKD. FKL

BORING 1

DATE DRILLED: August 10, 1964
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 120.3*

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
120					SM	2" ASPHALTIC PAVING FILL - SILTY SAND - fine, pieces of asphalt, brown
		8.8	121	■		
		7.5	114	■	SM	SILTY SAND - fine, brown
		15.1	117	■	CL	SANDY CLAY - dark brown
110	10					
		16.5	114	■		
		27.0	97	■	CL	SILTY CLAY - brown
		5.2	103	■	SM	SILTY SAND - fine, brown
					ML	SANDY SILT - greyish-brown
100	20	16.1	105	■		
		31.1	89	■	CL	SILTY CLAY - brown
		10.3	107	■	SM	SILTY SAND - fine, brown
		6.9	111	■		
		4.6	106	■	SP	SAND - fine, greyish-brown
90	30	7.0	103	■		
						Grading light greyish-brown
		4.3	108	■	SM	SILTY SAND - fine, brown
80	40	10.0	115	■		
		9.8	116	■		
					SP	SAND - fine, some silt, light brown
70	50	4.9	100	■		
	60					

NOTE: Water not encountered; caving from 27' to 35' (to 24" in diameter).

Soils classified in accordance with the Unified Soil Classification System.

* Elevations refer to benchmark shown on Plate 1.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

JOB 64536 DATE 8-27-64 DR. MC O.E. CHKD.

BORING 2

DATE DRILLED: August 10, 1964
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 124.5

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
		10.4	120		SM	2" ASPHALTIC PAVING FILL - SILTY SAND - fine, pieces of brick and asphalt, brown
120		9.9	110		SM	SILTY SAND - fine, very silty, dark brown Grading light brown
		16.8	114		CL	SANDY CLAY - light brown
	10	14.7	117			
110		23.6	101		CL	SILTY CLAY - some sand, greyish-brown
		20.2	106			Layer of SILTY SAND - fine, brown
	20	16.4	98		ML	SANDY SILT - greyish-brown Grading very sandy
		13.2	111			
100		14.0	109		SM	SILTY SAND - fine, brown
		9.3	105			
		7.1	106			Layers of SAND - fine, light greyish-brown Some clay
	30	15.2	109			
90		8.3	115			Some clay, reddish-brown
	40	7.2	114			
80		5.7	105		SP	SAND - fine, light brown
	50	4.0	105			

NOTE: Water not encountered; no caving or raveling.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 NO. 40 NO. 10 NO. 4 ¾ in. 3 in. (12 in.)

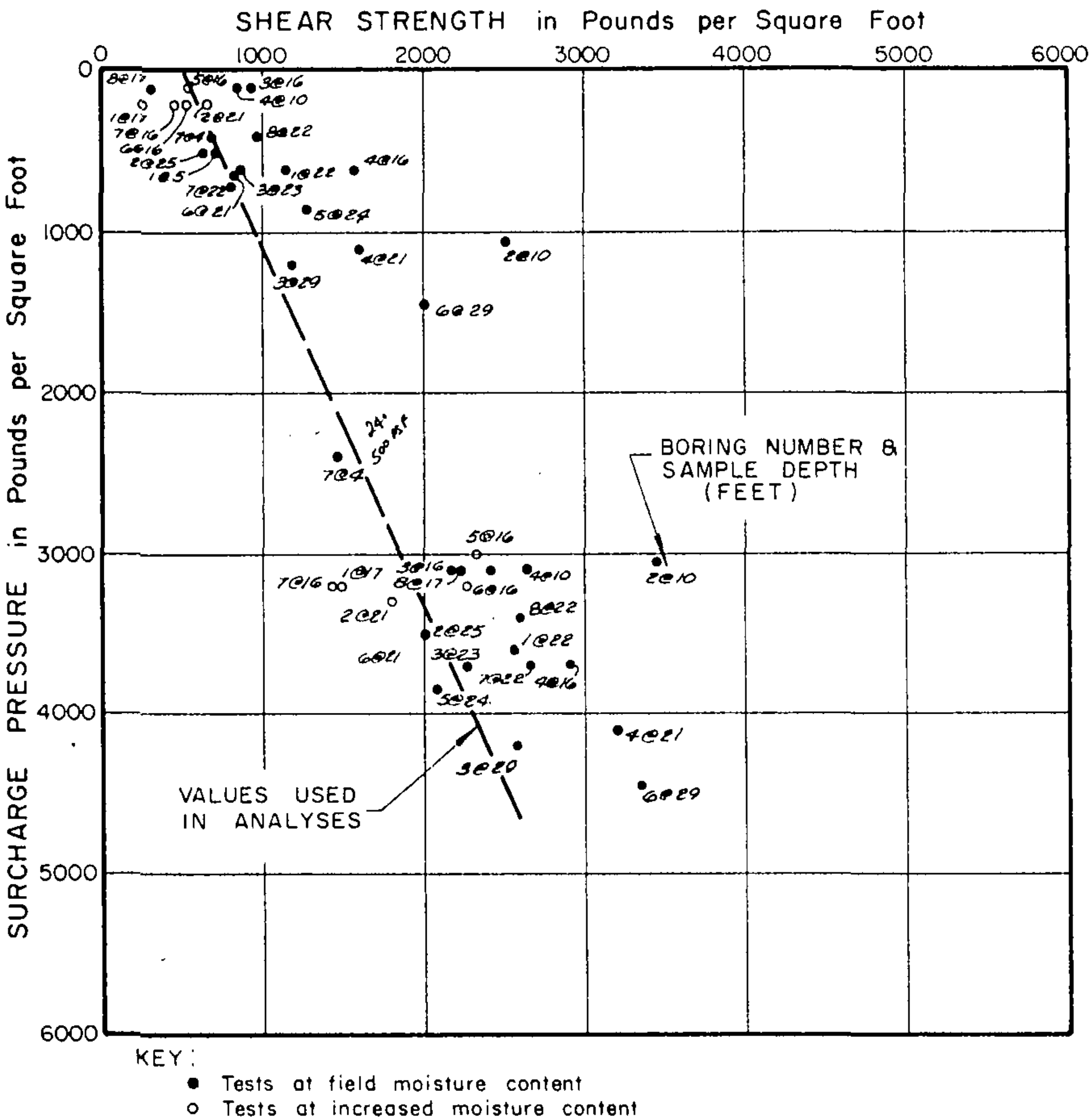
U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

Reference:
The Unified Soil Classification System, Corps of Engineers, U. S. Army Technical Memorandum No. 3-357, Vol. I, March, 1953. (Revised April, 1960)

LEROY CRANDALL & ASSOCIATES

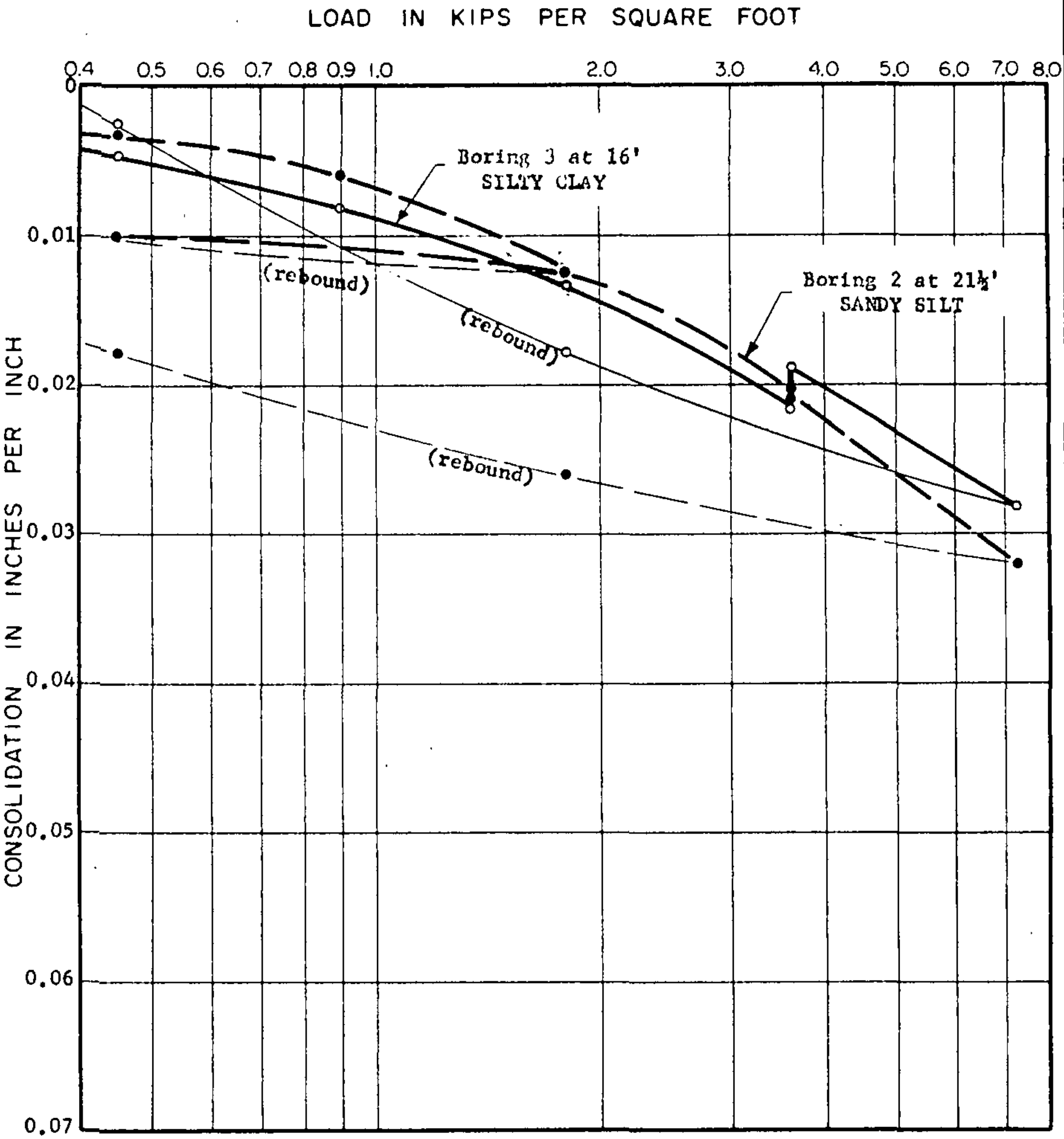
JOB 64536 DATE 8-31-64 DR O.E. CHKD.



DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES

JOB 64530 DATE 8-26-64 DR. O.E. CHKD.



NOTE: Water added to samples after consolidation under a load of 3.6 kips per square foot.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

APPENDIX C

SITE SPECIFIC SEISMIC ANALYSES AND GROUND MOTIONS STUDIES

C.1 Introduction

C.1.1 Background

The City of Inglewood Civic Center buildings, including City Hall, the Police Department and the Main Library, were constructed in 1971. An evaluation for seismic retrofitting of these buildings is currently being undertaken.

An ASCE 41-17 Tier 3 Seismic Evaluation is being performed as part of the seismic retrofit. The Tier 3 Seismic Evaluation will evaluate the performance of the existing structures for two Basic Safety Earthquake (BSE) hazard levels, BSE-2E and BSE-1E. In order to minimize retrofit work, nonlinear dynamic analyses are also planned for the BSE-1E and BSE-2E hazard levels, which require earthquake ground motion records as input.

We understand that the preliminary estimated fundamental periods for the Civic Center structures vary between 1.1 seconds to 1.2 seconds.

This appendix documents the development of the site-specific Acceleration Response Spectra (ARS) curves, as well as the BSE-1E and BSE-2E earthquake ground motions.

C.1.2 Purpose and Scope of Services

The purpose of our services was to develop earthquake ground motion recommendations to support the ASCE 41-17 Tier 3 Seismic Evaluation. We provided the following services:

- Performing document review and site characterization for seismic design, including the shear wave velocity in the upper 30 meters ($V_{S,30}$) for the underlying soil and rock at the project site;
- Performing site-specific seismic hazard evaluation for the BSE-2E and BSE-1E hazard levels based on the provisions of ASCE 41-17;
- Development of site-specific ARS for the BSE-2E and BSE-1E hazard levels, following the ground motion requirements of ASCE 41-17 and ASCE 7-16;
- Evaluation of deaggregation of the seismic hazard at the site for the BSE-2E and BSE-1E hazard levels;
- Development of criteria and selection of eleven sets of ground motion time histories for the BSE-1E, and BSE-2E hazard levels based on the provisions of Section 16.2 of ASCE 7-16, as required by ASCE 41-17;
- Development of amplitude-scaled time histories of the eleven sets of ground motion time histories selected for the BSE-1E, and BSE-2E hazard levels; and
- Preparation of this report documenting our analyses and recommendations.

C.2 Seismic Setting

The City of Inglewood Civic Center Buildings are located in a region with high seismic activity. The adopted coordinates of the site for use in the analyses are latitude of 33.9631º north and longitude of 118.3549º west.

Figure 4 of the main report presents a Regional Fault Map, and Figure C-1 presents the Seismic Source Fault Map to be read with Table C-1, which lists the active faults closest to the Site, along with their Fault Type, Maximum Magnitude (Mw), and Site-To-Source Rupture Distance (R_{rup}), which is the closest distance of the fault to the Site. These faults are from the Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) (Field et al., 2013) seismic source model developed by the Working Group on California Earthquake Probabilities (WGCEP) in 2013. The UCERF3 model was subsequently adopted by the 2014 U.S. National Seismic Hazard Mapping Program (NSHM) (Petersen et al., 2014) to develop probabilistic seismic hazard maps.

Table C-1: Significant Active Faults Near the Site

Fault	Fault Type	Maximum Magnitude, M_W	Site-to-Source Distance, R_{rup} (km)
Newport-Inglewood	Strike-slip	7.2	0.04
Newport-Inglewood Connected	Strike-slip	7.5	0.04
Puente Hills (LA)	Reverse	7.0	7.4
Puente Hills	Reverse	7.1	10.6
Santa Monica	Reverse	6.8	12.4
Palos Verdes	Strike-slip	7.4	13.5
Hollywood	Strike-slip	6.7	14.0
Compton	Reverse	7.5	14.6
Elysian Park (Upper)	Reverse	6.7	14.8
Redondo Canyon	Reverse	6.7	16.2
Anacapa-Dume	Reverse	7.2	19.5
Puente Hills (Santa Fe Springs)	Reverse	6.6	20.3
Whittier	Strike-slip	7.0	28.0
Anaheim	Reverse	6.4	28.1
Elsinore (Coyote Mountain + Julian + Temecula + stepovers combined + Glen Ivy + Whittier)	Strike-slip	7.8	28.3
Malibu Coast	Strike-slip	7.0	28.9

Fault	Fault Type	Maximum Magnitude, M_W	Site-to-Source Distance, R_{rup} (km)
Sierra Madre	Reverse	7.2	34.4
Puente Hills (Coyote Hills)	Reverse	6.8	40.6
Southern San Andreas (Parkfield + Cholame + Carrizo + Big Bend + Northern Mojave + Southern Mojave + Northern San Bernardino + Southern San Bernardino + San Gorgonio Pass-Garnet Hill + Coachella)	Strike-slip	8.2	68.0
San Jacinto (San Bernardino + San Jacinto Valley +Anza + Coyote Creek + Borrego Mountain + Superstition Mountain)	Strike-slip	7.8	84.8

The Newport-Inglewood fault traverses the project site through the Police Department and City Hall buildings (Figure 4 in main report). The Newport-Inglewood Fault Zone (NIFZ), is a zone of discontinuities, folds and faults which stretches across the Los Angeles basin in a Northwest-Southeast direction from Beverly Hills to Newport Beach. The Newport-Inglewood fault continues offshore to the south to connect with the Rose Canyon fault, parallel to the coastline towards San Diego and Baja California. The scenario of the Newport-Inglewood rupturing together (Newport-Inglewood connected), has been also considered as a separate system of active faults near the site.

The maximum magnitudes and scenarios adopted are consistent with the published Building Seismic Safety Council 2014 Event Set (the adopted deterministic ruptures used for the 2014 USGS NSHM) (BSSC, 2015). For very active, multi-segment faults (such as Newport-Inglewood or Santa Monica), where different earthquake scenarios are considered, the one producing the largest magnitude was reported in the table along with its combined segments.

C.3 Site Characterization

The geotechnical investigation for the original construction of the Civic Center development were performed by Leroy Crandall and Associates in 1964 and 1970. The geotechnical information (boring logs and laboratory test data) from these investigations, along with the recent geotechnical report by AMEC (2017), for the support of the previous Tier 2 seismic evaluations of the Civic Center buildings, were reviewed in support the current investigation.

In addition to reviewing of the previous geotechnical investigations, and for better understanding of the site geotechnical conditions, we performed a field investigation consisting of two (2) seismic cone penetration tests (SCPT) and one (1) cone penetration test (CPT). The locations of these explorations are shown in Figure 2 in the main report.

The entire site is underlain by dense Pleistocene-age older alluvial deposits (Figure 3), with localized shallow fill up to a maximum depth of 8 feet (LeRoy Crandall and Associates, 1970).

These alluvial deposits supporting the existing Civic Center Buildings have high shear strength and low compressibility and are very competent for structural support.

Historical topographic maps were reviewed for the site, prior to construction of the Civic Center development. Figure C-2 shows an excerpt from the 1924 USGS topographic maps of the Inglewood quadrangle, with the locations of project site marked. This topographic map indicates that the site was generally level with a slight slope from North-East to South-West, and therefore, we expect the subsurface conditions to be consistent across the project site.

Shear wave velocity measurements were obtained within the SCPTs at 5-foot intervals to the depths explored (Appendix A) and were used to obtain shear wave velocity in the upper 30 meters (100 feet) ($V_{s,30}$) for the underlaying soil and rock, which are required for input to the seismic hazard evaluation. Table C-2 presents the $V_{s,30}$ values obtained from SCPT-1, and SCPT-3. The $V_{s,30}$ values at both locations were found to be highly comparable. Therefore, an average $V_{s,30}$ value of 446.5 meters per second (m/s) was selected for use in developing the ground motions for this seismic evaluation.

Table C-2: Shear Wave Velocity Measurements

Exploration	$V_{S,30}$ (m/s)
SCPT-1	438.0
SCPT-3	455.0
Average	**446.5**

C.4 Site-specific Seismic Hazard Analysis

Probabilistic Seismic Hazard Analyses (PSHA) were performed to develop the horizontal 5-percent damped ARS for the two hazard levels, BSE-2E and BSE-1E, as defined by ASCE 41-17. In accordance with ASCE 41-17, the BSE-2E hazard level is associated with a 5-percent probability of exceedance in 50 years, in the direction of maximum horizontal response, and the BSE-1E to a 20-percent probability of exceedance in 50 years, in the direction of maximum horizontal response.

In accordance with ASCE 41-17, Section 2.4.2.1, site-specific ARS are developed following the procedures of Chapter 21 of ASCE 7-16. Details are discussed in the following sections.

C.4.1 Ground Motion Models

Site-specific ground motions are influenced by type of faulting, magnitude of characteristic earthquakes, and local soil conditions. Numerous ground motion models, also referred to as Ground Motion Prediction Equations (GMPEs) have been developed to estimate the variation of spectral acceleration with earthquake magnitude and source-to-site distance, among other parameters. The Pacific Earthquake Engineering Research (PEER) coordinated a large multidisciplinary project entitled "NGA (Next Generation Attenuation)-West 2 Research Project" (Bozorgnia et al., 2014), referred to as NGA-West2. In NGA-West2, five teams have developed and presented horizontal ground motion models for shallow crustal earthquakes in active tectonic regions including Western North America. These teams are Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), Chiou and Youngs (2014), and Idriss (2014).

We used four of these five models in our probabilistic seismic hazard analysis (PSHA). The Idriss (2014) model was not used as this ground motion model is only applicable to $V_{S,30} > 450$ m/s. The four models were each assigned a weight of 0.25. The Intensity Measure (IM) provided by these horizontal ground motion models corresponds to the 50th percentile of the rotated orientation-independent horizontal component, RotD50, defined by Boore (2010). The RotD50 is generally consistent with a geometric mean ground motion.

The NGA-West2 relationships use measured values of $V_{S,30}$ as input. As previously discussed, we have adopted an average $V_{S,30}$ of 446.5 m/s to represent the underlying soil and rock conditions at the site. In addition, some of the ground motion models require input for $Z_{1.0}$ (defined as the depth in meters to a shear wave velocity of Vs = 1 km/s) and $Z_{2.5}$ (defined as the depth in km to a shear wave velocity of Vs = 2.5 km/s). These two parameters are used to capture the basin effect on site response.

The Caltrans basin maps of $Z_{1.0}$, and $Z_{2.5}$ for the Los Angeles basin, SCEC Community Velocity Model (CVM) Version 4, correlations included in the Campbell and Bozorgnia (2013) ground motion model, and extrapolation from shear wave velocity trends in the seismic CPTs were reviewed for the selection of $Z_{1.0}$, and $Z_{2.5}$ values. The values from SCEC CVM and Caltrans basin maps were comparable, and the $Z_{1.0}$ value of 575 meters, and $Z_{2.5}$ of 3.4 km were adopted for the analyses based on the Caltrans basin map.

C.4.2 Probabilistic Seismic Hazard Analyses

Site-specific Probabilistic Seismic Hazard Analyses (PSHA) were performed using the computer tool OpenSHA (Fields, 2003), using the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) seismic source model and the updated NGA-West2 ground motion models. Uniform hazard horizontal ARS were developed up to a period of 10 seconds. All spectra were developed for 5-percent damping. Figure C-3 presents the results of probabilistic seismic hazard analyses for the two seismic hazard levels, BSE-2E and BSE-1E.

In addition, supplementary PSHA were performed using the Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/), using the dynamic version of the Conterminous U.S. 2014 (v4.2.0) at available spectral periods, for the Site Class C ($V_{S,30}$ = 537 m/s) and Site Class C/D boundary ($V_{S,30}$ = 360 m/s). Results of these supplementary analyses showed good agreement with results obtained from the OpenSHA analyses.

The median (RotD50) ground motion was adjusted to the maximum rotated component of ground motion (RotD100) using the factors proposed in Section 21.2 of ASCE 7-16. Figure C-3 presents the comparison of the median ground motion to maximum horizontal component at each hazard level.

C.4.3 Site-Specific BSE-1E and BSE-2E Acceleration Response Spectra

Development of the site-specific ARS for ASCE 41-17 is performed following the procedures recommended in Chapter 21 of ASCE 7-16. For a probabilistically defined spectrum (BSE-1E and BSE-2E), the procedure is followed, but neglects the deterministic ground motion development, and instead uses a cap of the BSE-1N (Design Earthquake per ASCE 7) or BSE-2N (MCE_R per ASCE 7) spectra, respectively. Per Commentary C2.4.2.1 of ASCE 41-17, the site-specific ARS is anchored to the general response spectrum as described in Chapter 21 of ASCE 7-16. Therefore, the site-specific BSE-1E or BSE-2E spectra may not be less than 80-percent of the general spectrum developed using mapped seismic parameters at each respective hazard level.

The site-specific BSE-1E and BSE-2E spectra developed in the previous section are compared to the BSE-1N and BSE-2N spectra, respectively, as shown in Figure C-4. Both BSE-1E and BSE-2E spectra are less than their respective caps of BSE-1N and BSE-2N.

Figure C-5 presents the 5-percent damped horizontal ARS along with tabulated values for the BSE-1E and BSE-2E hazard levels. The two site-specific ARS are also compared with the generic spectra produced with mapped values, as well as the 80-percent lower limit.

C.4.4 Site-Specific BSE-1E and BSE-2E Design Acceleration Parameters

The site-specific short period design spectral acceleration (S_{DS}) and 1-second period design spectral acceleration (S_{D1}) parameters were determined in accordance with ASCE 7-16 Section 21.4. The parameter S_{DS} is taken as 90-percent of the maximum spectral acceleration from the site-specific design spectrum at periods between 0.2 and 5 seconds. The parameter S_{D1} is taken as the maximum of the product between period and spectral acceleration for periods from 1 to 2 seconds for sites with $V_{S,30}$ > 1,200 ft/s (365 m/s). The values obtained shall not be less than 80-percent of the mapped values for each hazard level, respectively. Mapped design acceleration parameters were obtained from the online SEAOC/OSHPD Seismic Design Maps tool (SEAOC/OSPHD, 2020).

Table C-3 presents the mapped seismic design acceleration parameters and the site-specific seismic design acceleration parameters.

Table C-3: Mapped and Site-Specific Seismic Design Acceleration Parameters

Hazard Level	Parameter	Mapped Value	Site-Specific Value
BSE-1	S_{XS}	0.840	0.865
	S_{X1}	0.337	0.428
BSE-2	S_{XS}	1.738	1.603
	S_{X1}	0.744	0.907

C.5 Seismic Deaggregation

Deaggregation analyses of the seismic hazard were performed for the BSE-1E and BSE-2E seismic hazard levels to identify the key contributors to the hazard in terms of earthquake magnitude, distance to the seismogenic source, and types of seismogenic sources. Deaggregation analyses were performed using the Unified Hazard Tool (USGS, 2019). The edition used is the Dynamic Conterminous U.S. NSHM 2014 edition (Version 4.2.0). Deaggregation analyses were performed at selected spectral periods of interest between PGA and 3 seconds (PGA, 0.3 second, 0.5 second, 0.75 second, 1 second, 2 second, and 3 second) for the Site, for Site Class C ($V_{S,30}$ = 537 m/s) as well as Site Class C/D ($V_{S,30}$ = 360 m/s). In general, results between the two analyses were comparable. Results of the deaggregation in terms of modal and mean values for magnitude, distance, and epsilon and for the BSE-1E and BSE-2E hazard levels for Site Class C, are summarized in Tables C-4 and C-5, respectively. Results of the deaggregation displayed as three-dimensional bar charts showing the relationship of magnitude and distance with hazard contribution for the BSE-1E and BSE-2E hazard levels for Site Class C, are presented in Figures C-6 and C-7, respectively.

The deaggregation of the hazard by magnitude and distance reveals that majority of the hazard for PGA and selected spectral periods for the BSE-1E hazard level is generally controlled by medium to large magnitude earthquakes (M6.3 to M7.2) at distances between 1.7 km and 30 km. The controlling magnitude earthquakes and distances for the BSE-2E hazard level are (M6.4 to M7.3) and 1.7 to 18 km, respectively.

Table C-4: Seismic Deaggregation for the BSE-1E Hazard Level (Site Class C)

Spectral Period (sec)	Magnitude		Distance (km)		Epsilon	
	Mean	Modal	Mean	Modal	Mean	Modal
PGA	6.6	6.3	13.9	1.8	0.43	-0.68
0.3	6.7	6.4	15.3	1.7	0.45	-0.64
0.5	6.8	6.4	17.8	1.8	0.44	-0.66
0.75	6.9	6.4	20.0	1.8	0.44	-0.66
1	7.0	6.4	21.0	1.8	0.42	-0.65
2	7.1	6.4	26.2	1.9	0.42	-0.21
3	7.2	6.4	30.0	1.8	0.41	-0.27

Table C-5: Seismic Deaggregation for the BSE-2E Hazard Level (Site Class C)

Spectral Period (sec)	Magnitude		Distance (km)		Epsilon	
	Mean	Modal	Mean	Modal	Mean	Modal
PGA	6.7	6.4	8.0	1.7	0.90	0.43
0.3	6.8	6.4	9.0	2.3	0.91	0.60
0.5	6.9	6.4	10.0	2.1	0.88	0.64
0.75	7.0	6.4	10.9	1.8	0.87	0.38
1	7.0	6.4	11.5	1.8	0.86	0.40
2	7.2	6.4	15.2	1.9	0.92	0.82
3	7.3	6.4	18.0	1.8	0.94	0.78

In addition to the controlling modal and mean magnitude and distances, the percentage of source contribution for at 1-second, 2-second, and 3-second periods are presented in Tables C-6 and C-7 for BSE-1E and BSE-2E hazard levels, respectively.

Table C-6: Source Contribution (Site Class C) for the BSE-1E Hazard Level

Fault	% Contribution at 1-second Period	% Contribution at 2-second Period	% Contribution at 3-second Period
Newport-Inglewood	19.5	18.2	17.2
San Andreas	8.3	13.0	16.8
Palos Verdes	9.4	9.9	10.0
Puente Hills	4.4	4.5	4.3
Santa Monica	4.1	4.2	4.2
Compton	3.0	2.9	2.6
Sierra Madre	<1%	2.4	2.6
Hollywood	2.1	2.1	2.0
Elysian Park (Upper)	1.2	1.0	<1%
Background Seismicity	20.0	12.8	9.4
All Other Sources	<1%	<1%	<1%

Table C-9: Source Contribution (Site Class C) at the BSE-2E Hazard Level

Fault	% Contribution at 1-second Period	% Contribution at 2-second Period	% Contribution at 3-second Period
Newport-Inglewood	39.8	35.4	32.9
Palos Verdes	8.8	10.4	11.3
San Andreas	3.2	6.8	10.1
Compton	9.8	8.9	6.6
Santa Monica	4.5	5.2	5.5
Puente Hills	4.0	4.3	4.3
Sierra Madre	<1%	<1%	2.2
Hollywood	1.74	1.87	1.8
San Pedro Escarpment	1.04	1.03	<1%
Background Seismicity	12.22	7.84	5.8
All Other Sources	<1%	<1%	<1%

C.6 Ground Motion Selection

According to ASCE 41-17, Section 2.4.3, ground motion acceleration time histories should be developed in accordance with Section 16.2 of ASCE 7-16. Time histories were selected following criteria recommended in the publications "Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses" (NIST, 2012). In accordance with Section 16.2 of ASCE 7-16, a minimum of eleven sets of time histories should be used nonlinear dynamic analyses.

The selection process was generally consistent with the criteria based on the deaggregation of the seismic hazard for the BSE-1E and BSE-2E hazard levels, station site conditions, scale factor of the time history, and significant duration of the record. The criteria used for the selection process is discussed further in the following sections.

C.6.1 Selection Criteria

C.6.1.1 Summary of the Seismic Deaggregation

Deaggregation results summarized in Tables C-4 through C-7 reveal that majority of the hazard for the selected spectral periods for the BSE-1E hazard level is controlled by medium to large magnitude earthquakes (M6.3 to M7.6) at distances between about 2 to 20 km caused by nearby active faults with strike-slip and reverse (thrust) fault mechanisms. Contribution of the large magnitude earthquakes (M7.7 to M8.0) at distances of 32 to 68 km increases from less than 10-percent of the hazard for shorter periods (less than 1 second) to 20-percent for larger periods (2 and 3 seconds). Distant active faults (e.g., San Andreas, Sierra Madre) capable of generating these earthquakes, are characterized as strike-slip and reverse (thrust) fault mechanisms.

For the BSE-2E level, the hazard is generally controlled by medium to large magnitude earthquakes (M6.4 to M7.6) at distances between about 2 to 15 km caused by nearby active faults with strike-slip and reverse (thrust) fault mechanisms. Large magnitude earthquakes (M7.7 to M8.0) at distances of 32 to 68 km contribute to less than 5-percent of the hazard at shorter periods (less than 1 second) to 12-percent for larger periods (2 and 3 seconds).

Considering the similarity of the deaggregation results obtained for the BSE-1E and BSE-2E levels, the same selection criteria was adopted for both levels.

Based on the mean magnitude, site-to-source distance, and source mechanism, the following general criteria was used for the selection process:

- Medium to large magnitude earthquakes at short distances
    - Magnitude range: M6.0 – M7.5
    - Site-to-source rupture distance, Rrup: 0 – 20 km
    - Fault type: strike-slip or reverse
- Large magnitude and distant earthquakes:
    - Magnitude range: M6.5 to M8.0
    - Site-to-source rupture distance, Rrup: 45-80
    - Fault type: strike slip or reverse

For the range of useable frequencies, the selected earthquake records need to satisfy the lowest usable frequency (corresponding to the highest usable period, T) that exceeds the range of interest in the response spectrum. The fundamental period of the Civic Center buildings ranges from 1.1 seconds to 1.2 seconds and based on conversations with the project Structural Engineer, the range of interest would extend to 1.8 seconds. Therefore, the lowest useable frequency must be 0.55 Hz or lower.

C.6.1.2 Site Condition

The average $V_{S,30}$ at the site is estimated at 446.5 meters per second (m/s) as discussed in Section C.3 above. Shear wave velocity at the recording station site is typically important because of its impact on overall spectral shape. A general range of shear wave velocity of 350 m/s to 600 m/s was considered for this site.

C.6.1.3 Proportion of Pulse Motions

Near-fault sites have a probability of experiencing pulse-like ground motions. That probability is not unity, and therefore, while some of the selected motions should exhibit pulse-like characteristics, the remainder should be motions not containing pulses. The empirical relationships by NIST (2012), shown in Equation (1) below, and Hayden et al. (2014) shown in Equation (2), were used to estimate the proportion of pulse-like motions, based on the distance and epsilon values obtained from deaggregation.

$$Proportion\ of\ Pulse\ Motions = \frac{exp(0.905-0.188R+1.337\varepsilon)}{1+exp(0.905-0.188R+1.337\varepsilon)} \qquad (1)$$

$$Proportion\ of\ Pulse\ Motions = \frac{1}{1+exp[-3.87+1.04\times R^{0.5}+15.99\times(\varepsilon+3)^{-2}]} \qquad (2)$$

The proportion of pulse motions based on deaggregation results at the site for the BSE-1E and BSE-2E levels are summarized in Table C-9 and C-10, respectively.

Table C-9: Proportion of Pulse Motions for the BSE-1E Hazard Level

Period (sec)	Modal Distance (km)	Modal Epsilon	Proportion of Pulses		Mean Distance (km)	Mean Epsilon	Proportion of Pulses	
			NIST (2012)	Hayden et al. (2014)			NIST (2012)	Hayden et al. (2014)
PGA	1.8	-0.68	0.41	0.38	13.9	0.43	0.24	0.20
0.3	1.7	-0.64	0.43	0.41	15.3	0.45	0.20	0.18
0.5	1.8	-0.66	0.42	0.39	17.8	0.44	0.14	0.13
0.75	1.8	-0.66	0.42	0.39	20.0	0.44	0.09	0.11
1	1.8	-0.65	0.42	0.39	21.0	0.42	0.08	0.09
2	1.9	-0.21	0.57	0.60	26.2	0.42	0.03	0.06
3	1.8	-0.27	0.55	0.58	30.0	0.41	0.02	0.04

Table C-10: Proportion of Pulse Motions for BSE-2E Hazard Level

Period (sec)	Modal Distance (km)	Modal Epsilon	Proportion of Pulses		Mean Distance (km)	Mean Epsilon	Proportion of Pulses	
			NIST (2012)	Hayden et al. (2014)			NIST (2012)	Hayden et al. (2014)
PGA	1.7	0.43	0.76	0.76	8.0	0.90	0.65	0.47
0.3	2.3	0.60	0.78	0.74	9.0	0.91	0.61	0.43
0.5	2.1	0.64	0.80	0.76	10.0	0.88	0.55	0.38
0.75	1.8	0.38	0.75	0.74	10.9	0.87	0.50	0.35
1	1.8	0.40	0.75	0.75	11.5	0.86	0.47	0.33
2	1.9	0.82	0.84	0.79	15.2	0.92	0.33	0.23
3	1.8	0.78	0.83	0.79	18.0	0.94	0.23	0.17

The summary of proportion of pulse motions for modal values indicates in the period range of interest ranges from 0.39 to 0.60 (corresponding to about 4 to 7 motions containing pulses) for the BSE-1E level and from 0.74 to 0.84 (corresponding to about 8 to 9 motion containing pulses) for the BSE-2E level. The selection included 7 motions containing pulses for the BSE-1E level and 8 motions for the BSE-2E level.

C.6.1.4 Significant Duration and Arias Intensity

Intensity measures such as Significant Duration and Arias Intensity (I_A) are useful to characterize the energy realized during earthquake.

The Arias Intensity (Arias, 1970) is a cumulative ground motion intensity measure calculated as the time integral of the squared acceleration. The Arias Intensity was not used as a rigid criterion but was used to aid in the selection process.

The Significant Duration is often represented by the interval of time between of 5- and 95-percent of the Arias Intensity ($D_{a5\text{-}95}$). Kempton & Stewart (2006) and Bommer et al. (2009) have proposed equations to estimate the $D_{a5\text{-}95}$ based on earthquake magnitude, site-to-source distance, and site effects. Using these methods, the $D_{a5\text{-}95}$ estimated values range from about 9 to 20 seconds for BSE-1E and from about 20 to 45 seconds for the BSE-2E hazard level These values were considered in the selection criteria.

Scaling ground motions can often have a large impact on the intensity measures of a ground motion; for example, since Arias Intensity is related to the square of acceleration, if a seed ground motion is scaled by a factor of 3, the Arias Intensity may be 9 times higher. Therefore, as part of the selection criteria, a maximum scale factor of 4 for was adopted to avoid causing excessive modification of the seed ground motions.

C.6.2 Seed Time Histories

The PEER NGA-West2 database (Ancheta et al. 2013, 2014) was used as the primary source for selecting ground motion records. Generally following the selection criteria above and rev, candidate ground motion time histories were pre-selected to represent the shaking corresponding to BSE-1E and BSE-2E seismic hazard levels after a thorough search in the database. Of the pre-selected time histories, the final set was selected based on review of the mean squared error of the candidate motions compared to the target spectrum within the period range of interest ($0.2T_{MIN}$ to $1.5T_{MAX}$) and consideration of the overall balance of the motions meeting the criteria. Tables C-11 and C-12 present the characteristics of the final selected earthquakes and recorded seed ground motions for the BSE-1E and BSE-2E levels, respectively

Table C-11: Summary of Selected Seed Ground Motions Characteristics for the BSE-1E Hazard Level

PEER NGA Database RSN	Earthquake	Year	Mechanism	Recording Station	Magnitude, M_w	Distance, R_{rup} (km)	Pulse Period, T_p (sec)	$V_{S,30}$ (m/s)	Lowest Usable Freq. (Hz)	Significant Duration, $D_{5\text{-}95}$ (sec)
184	Imperial Valley-06	1979	Strike Slip	El Centro Differential Array	6.53	5.09	6.265	202.26	0.02875	7.0
802	Loma Prieta	1989	Reverse Oblique	Saratoga, - Aloha Ave	6.93	8.5	4.571	380.89	0.125	9.4
836	Landers	1992	Strike Slip	Baker Fire Station	7.28	87.94	-	324.62	0.1	25.7
1004	Northridge-01	1994	Reverse	LA – Sepulveda VA Hospital	6.69	8.44	0.931	380.06	0.182	8.5
1489	Chi Chi, Taiwan	1999	Reverse Oblique	TCU049	7.62	3.76	10.22	487.27	0.025	22.7
1602	Duzce, Turkey	1999	Strike Slip	Bolu	7.14	12.04	0.882	293.57	0.0625	9.0
2107	Denali, Alaska	2002	Strike Slip	Carlo	7.9	50.94	-	399.35	0.078	24.3
4228	Niigata, Japan	2004	Reverse	NGH11	6.63	8.93	1.799	375.0	0.05	12.2
4886	Chuetsu-oki, Japan	2007	Reverse	Kawanishi Izumozaki	6.8	11.75	-	338.32	0.1125	13.9
5779	Iwate, Japan	2008	Reverse	Sanbongi Osaki City	6.9	36.34	-	539.87	0.0875	29.1
6897	Darfield, New Zeeland	2010	Strike Slip	DSLC	7.0	8.46	7.826	295.74	0.075	19.6

Table C-12: Summary of Selected Seed Ground Motions Characteristics for the BSE-2E Hazard Level

PEER NGA Database RSN	Earthquake	Year	Mechanism	Recording Station	Magnitude, M_w	Distance, R_{rup} (km)	Pulse Period, T_p (sec)	$V_{S,30}$ (m/s)	Lowest Usable Freq. (Hz)	Significant Duration, D_{5-95} (sec)
15	Kern County	1952	Reverse	Taft Lincoln School	7.28	38.89	-	385.43	0.125	30.3
161	Imperial Valley-06	1979	Strike Slip	Brawley Airport	6.53	10.42	4.396	208.71	0.05375	14.9
184	Imperial Valley-06	1979	Strike Slip	El Centro Differential Array	6.53	5.09	6.265	202.26	0.02875	7.0
802	Loma Prieta	1989	Reverse Oblique	Saratoga, - Aloha Ave	6.93	8.5	4.571	380.89	0.125	9.4
832	Landers	1992	Strike Slip	Amboy	7.28	69.21	-	382.93	0.1	28.5
1004	Northridge-01	1994	Reverse	LA – Sepulveda VA Hospital	6.69	8.44	0.931	380.06	0.182	8.5
1489	Chi Chi, Taiwan	1999	Reverse Oblique	TCU049	7.62	3.76	10.22	487.27	0.025	22.7
1605	Duzce, Turkey	1999	Strike Slip	Duzce	7.14	6.58	5.9	281.86	11.1	11.1
4228	Niigata, Japan	2004	Reverse	NGH11	6.63	8.93	1.799	375.0	0.05	12.2
4866	Chuetsu-oki, Japan	2007	Reverse	Kawanishi Izumozaki	6.8	11.75	-	338.32	0.1125	13.9
6911	Darfield, New Zeeland	2010	Strike Slip	HORC	7.0	7.29	9.819	326.01	0.125	9.5

C.7 Ground Motion Modification

The final eleven sets of selected seed ground motions require modification to become compatible with the target horizontal acceleration response spectra. For the City Center buildings, ground motion modification was performed using linear scaling in accordance with Section 16.2.3 of ASCE 7-16, with modifications as appropriate based on recommendations of Section 2.4.3 of ASCE 41-17.

Since the site is considered "near-fault" as defined by Section 11.4.1 of ASCE 7-16, the ground motion time histories must be rotated from the recorded horizontal components (H1/H2) to their respective fault normal (FN) and fault parallel (FP) components and then applied to the buildings in the respective FN and FP orientations based on the causative fault (which is Newport-Inglewood at this site).

The selected ground motion time histories are first rotated, then amplitude-scaled such that they meet the criteria defined in Section 16.2.3.2 of ASCE 7-16. Details are presented in the following sections.

C.7.1 Rotation

The procedure recommended in Reyes and Kalkan (2012) was used for the ground motion rotation, as shown in Equations (3) and (4) below.

$$u\ddot{}_{FP} = \ddot{u}_1 \cos(\beta_1) + \ddot{u}_2 \cos(\beta_2) \tag{3}$$

$$u\ddot{}_{FN} = \ddot{u}_1 \sin(\beta_1) + \ddot{u}_2 \sin(\beta_2) \tag{4}$$

where,

$$\beta_1 = \alpha_{strike} - \alpha_1$$

$$\beta_2 = \alpha_{strike} - \alpha_2$$

and where, α_{strike} is the strike of the fault, and α_1 and α_2 are the original azimuths of the horizontal components of ground motion (as illustrated in Figure C-8. The orientation of the strikes of the causative faults for each set of recorded time histories was obtained from the NGA-West2 flatfile (PEER, 2015), and are listed in Table C-13 along with the original azimuths of the pairs of horizontal components.

Table C-13: Azimuths of the Horizontal Components and Strike of the Causative Fault for Seed Motions for the BSE-1E and BSE-2E Levels

BSE-1E Hazard Level				BSE-2E Hazard Level			
PEER NGA Database RSN	H1 Azimuth (deg.)	H2 Azimuth (deg.)	Causative Fault Strike Azimuth (deg.)	PEER NGA Database RSN	H1 Azimuth (deg.)	H2 Azimuth (deg.)	Causative Fault Strike Azimuth (deg.)
184	270	0	323	15	21	111	51
802	0	90	128	161	225	315	323
836	50	140	336	184	270	0	323
1004	270	0	122	802	0	90	128
1489	90	0	20	832	0	90	336
1602	0	90	270	1004	270	0	122
2107	90	0	298	1489	90	0	20
4228	0	90	212	1605	180	270	270
4886	0	90	34	4228	0	90	212
5779	0	90	209	4866	0	90	34
6897	333	63	85	6911	18	108	85

C.7.2 Amplitude Scaling

Per ASCE 7-16 section 16.2.3.2, For each horizontal ground motion pair, a maximum-direction spectrum (RotD100) shall be constructed for the two horizontal components. Each ground motion shall be scaled, with an identical scale factor applied to both horizontal components, such that the average of the maximum-direction spectra from all ground motions (suite of 11 ground motions) generally matches or exceeds the target spectrum, over the period range from $0.2T_{MIN}$ to $2.0T_{MAX}$, where T_{MIN} and T_{MAX} are the lowest and highest fundamental period in either the longitudinal or transverse direction of the structure, respectively. Note that ASCE 41-17 modifies the upper bound of the range to 1.5T, however, the upper bound of the range cannot be less than 1-second.

For the given range of horizontal fundamental periods for Civic Center Buildings (~1.1 seconds to 1.2 seconds), amplitude scaling was generally aimed to match the target spectra between 0.22 seconds and 1.8 seconds, as discussed with project Structural Engineer.

The maximum direction spectra (RotD100 component) for each pair of time histories was developed following the procedure documented by Boore (2010), and the amplitude scaling of the ground motions was performed in accordance with section 16.2.3.2 of ASCE 7-16.

Maximum direction spectra for each of the 11 modified ground motions, as well as the resulting average maximum direction spectra compared with the target spectrum, for BSE-1E and BSE-2E hazard levels are presented in Figures C-9 and C-10. Scaling factors used for the selected motions are presented in Table C-14.

Table C-14: Scaling Factors for Seed Motions for the BSE-1E and BSE-2E Levels

BSE-1E Hazard Level		BSE-2E Hazard Level	
PEER NGA Database RSN	**Scaling Factor**	**PEER NGA Database RSN**	**Scaling Factor**
184	0.65	15	3.48
802	0.90	161	2.50
836	2.81	184	1.34
1004	0.40	802	1.79
1489	1.15	832	4.0
1602	0.45	1004	0.83
2107	3.80	1489	2.30
4228	0.75	1605	1.08
4866	0.90	4228	1.56
5779	2.25	4866	1.82
6897	1.1	6911	0.90

C.7.3 Rotation to Structure Axes

The 11 sets of modified time histories have a final rotation performed to maintain the FN/FP components from the causative fault to the site, which is the Newport-Inglewood fault. The azimuth of the strike of the Newport-Inglewood fault is generally reported as 333 degrees (N27°W), but at the location of the City of Inglewood, the azimuth varies to 350 degrees. Therefore, the azimuth of the fault normal component is 260 degrees, and the fault parallel component is 350 degrees. The Civic Center buildings have their longitudinal and transverse axes oriented generally in the North-South and East-West directions. Therefore, the final rotation included rotating the FN and FP components to 90 and 180 degrees, respectively, for application to the structure. The method previously documented by Reyes and Kalkan (2012) was used for this rotation.

C.8 REFERENCES

Abrahamson, N. (1992). Non-stationary spectral matching. Seismological Research Letters. Vol. 63, No. 1, pg. 30.

Abrahamson, N., Silva, W. and Kamai, R. (2014), "Summary of the ASK14 Ground Motion Relation for Active Crustal Regions." *Earthquake Spectra*: August 2014, Vol. 30, No. 3, pp. 1025-1055.

Al Atik, L. and Abrahamson, N. (2010). An Improved Method for Nonstationary Spectral Matching, *Earthquake Spectra*, August 2010, Vol. 26, No. 3, pp. 601-617.

Ancheta, T., Darragh, R., Stewart, J.P., Seyhan, E., Silva, W., Chiou, B., Wooddell, K., Graves, R., Kottke, A., Boore, D., Kishida, T., and Donahue, J. (2013), "PEER NGA-West 2 Database," PEER Report 2013/03.

Arias, A. (1970). "A measure of earthquake intensity" in *Seismic design for nuclear power plants*, edited by R. Hansen, MIT Press, Cambridge, Massachusetts, p. 438-483.

(ASCE) American Society of Civil Engineers (2017). ASCE standard, ASCE 41-17, "Seismic Evaluation and Retrofit of Existing Buildings," ASCE, Reston, Virginia.

(ASCE) American Society of Civil Engineers (2017). ASCE standard, ASCE/SEI 7-16, "Minimum design loads for buildings and other structures," ASCE, Reston, Virginia.

(ASCE) American Society of Civil Engineers (2018). ASCE standard, ASCE/SEI 7-16, "Minimum design loads for buildings and other structures," Supplement 1, ASCE, Reston, Virginia.

Bommer, J.J., Stafford, P.J. and Alarcon, J.E. (2009). Empirical Equations for the Prediction of the Significant, Bracketed, and Uniform Duration of Earthquake Ground Motion. *Bulletin of the Seismological Society of America*, Vol. 99, No. 6, pp. 3217-3233, December 2009.

Boore, D.M. (2010), "Orientation-independent, Nongeometric-mean Measures of Seismic Intensity from two Horizontal Components of Motion," *Bulletin of the Seismological Society of America*, Vol. 99, pp. 2393-2409.

Boore, D., Stewart, J.P., Seyhan, E. and Atkinson, G.M. (2014), "NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes." *Earthquake Spectra*: August 2014, Vol. 30, No. 3, pp. 1057-1085.

California Building Standards Commission (CBSC), (2019). *2019 California Building Code (CBC)*, California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, dated: July 1.

Campbell, K.W., and Bozorgnia, Y. (2014), "NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra." *Earthquake Spectra*: August 2014, Vol. 30, No. 3, pp. 1087-1115.

Chiou, B. and Youngs, R. (2014), "Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra." *Earthquake Spectra*: August 2014, Vol. 30, No. 3, pp. 1117-1153.

Field, E.H., T.H. Jordan, and C.A. Cornell (2003), OpenSHA: A Developing Community-Modeling Environment for Seismic Hazard Analysis, *Seismological Research Letters*, **74**, no. 4, p. 406-419.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., and Zeng, Y. (2013). *Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) – The Time-Independent Model*, U.S. Geological Survey Open-File Report 2013-1165, U.S. Geological Survey, Reston, Virginia, 2013.

Hayden, C.P., Bray, J.D., and Abrahmson, N.A. (2014), "Selection of Near-Fault Pulse Motions," *Journal of Geotechnical and Geoenvironmental Engineering*: July 2014, Vol. 140, No. 7.

Idriss, I.M. (2014), "An NGA-West2 Empirical Model for Estimating the Horizontal Spectral Values Generated by Shallow Crustal Earthquakes. *Earthquake Spectra*: August 2014, Vol. 30, No. 3, pp. 1155-1177.

Kempton, J.J. and Stewart, J.P. (2006). Prediction Equations for Significant Duration of Earthquake Ground Motions Considering Site and Near-Source Effects. *Earthquake Spectra*, Volume 22, No. 4, pages 985-1013, November 2006.

LeRoy Crandall and Associates, (1964), "Report of Foundation Investigation, Proposed Inglewood Civic Center Parking Structure, Grevillea Avenue between Queen Street and Manchester Boulevard, Inglewood, California", For the City of Inglewood.

LeRoy Crandall and Associates, (1970), "Report of Foundation Investigation, Proposed Civic Center, Grevillea Avenue Between Manchester Boulevard and Regent Street, Inglewood, California", For the City of Inglewood.

Luco, N., Bachman, R. E., Crouse, C. B., Harris, J. R., Hooper, J. D., Kircher, C. A., Caldwell, P. J., and Rukstales, K. S. (2015). Updates to Building-Code Maps for the 2015 NEHRP Recommended Seismic Provisions, *Earthquake Spectra*, vol. 31, no. s1, pp. S245-S271, http://dx.doi.org/10.1193/042015EQS058M .

NIST (2012), "Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses," NIST GCR 11-917-15, NEHRP Consultants Joint Venture.

(PEER) Pacific Earthquake Engineering Research (2015). Updated NGA-West2 database "flatfiles", January 17, 2015.

Petersen, M.D., M. P. Moschetti, P. M. Powers, C. S. Mueller, K. M. Haller, A. D. Frankel, Y. Zeng, S. Rezaeian, S. C. Harmsen, O. S. Boyd, N. Field, R. Chen, K. S. Rukstales, N. Luco, R. L. Wheeler, R. A. Williams, and A. H. Olsen (2014). Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014–1091, 243 p., http://dx.doi.org/10.3133/ofr20141091 .

Reyes, J.C., and Kalkan, E. (2012). Should Ground-Motion Records be Rotated to Fault-Normal/Parallel or Maximum Direction for Response History Analysis of Buildings?: USGS Open-File Report 2012-1261, 89 p.

Structural Engineers Association of California and Office of Statewide Health Planning and Development (SEAOC/OSHPD, 2019). *Seismic Design Maps* Online Tool, https://seismicmaps.org/, accessed April 8.

United States Geological Survey (USGS) (2019), USGS National Seismic Hazard Mapping Program Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/.

2014 NSHM Fault Sources

Normal
Strike Slip
Thrust
Unassigned

N

Reference: USGS Quaternary Faults, NSHM 2014 Fault Sources https://usgs.maps.arcgis.com/apps/webappviewer/index.html

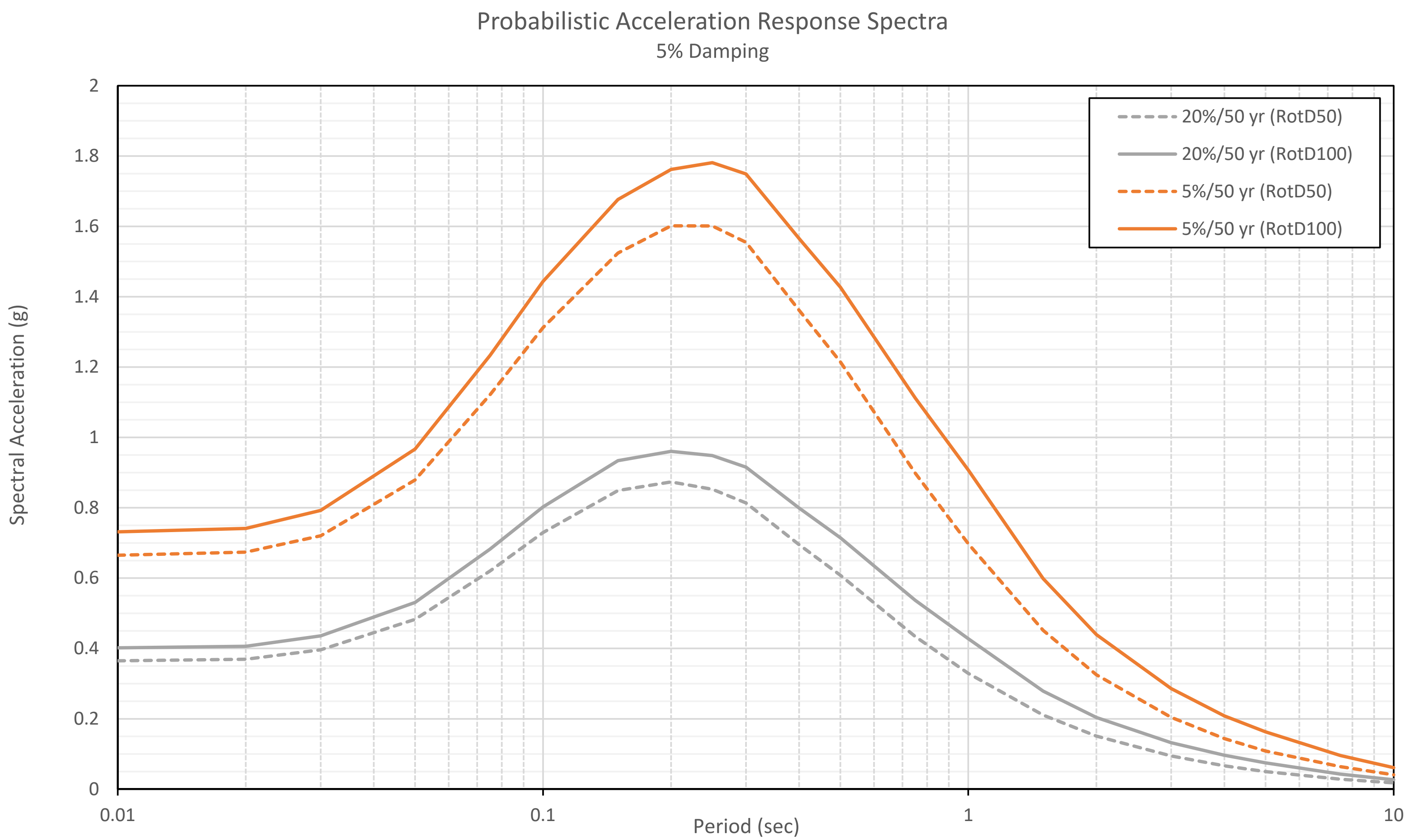
PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Fault Map** **Seismic Source Fault Map (2014 NSHM)**	PROJECT NUMBER LA-1474	FIGURE NUMBER C-1

Reference: USGS Historical Topographic Maps, Inglewood, CA, 1:24,000 Quadrangle

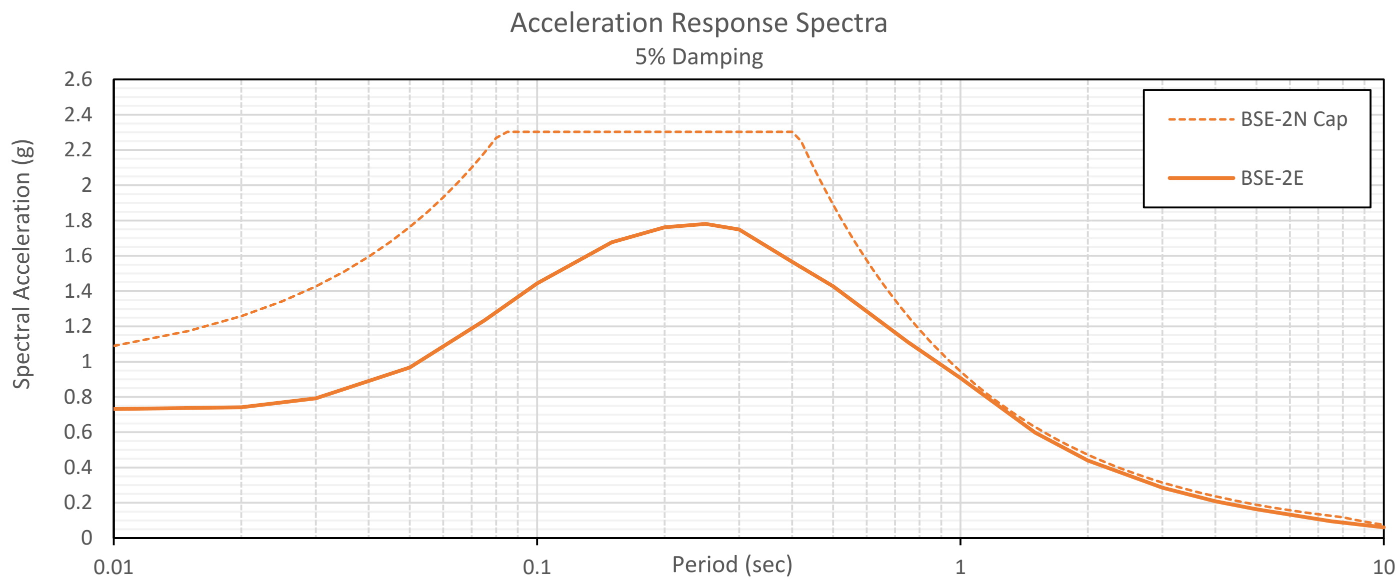
N

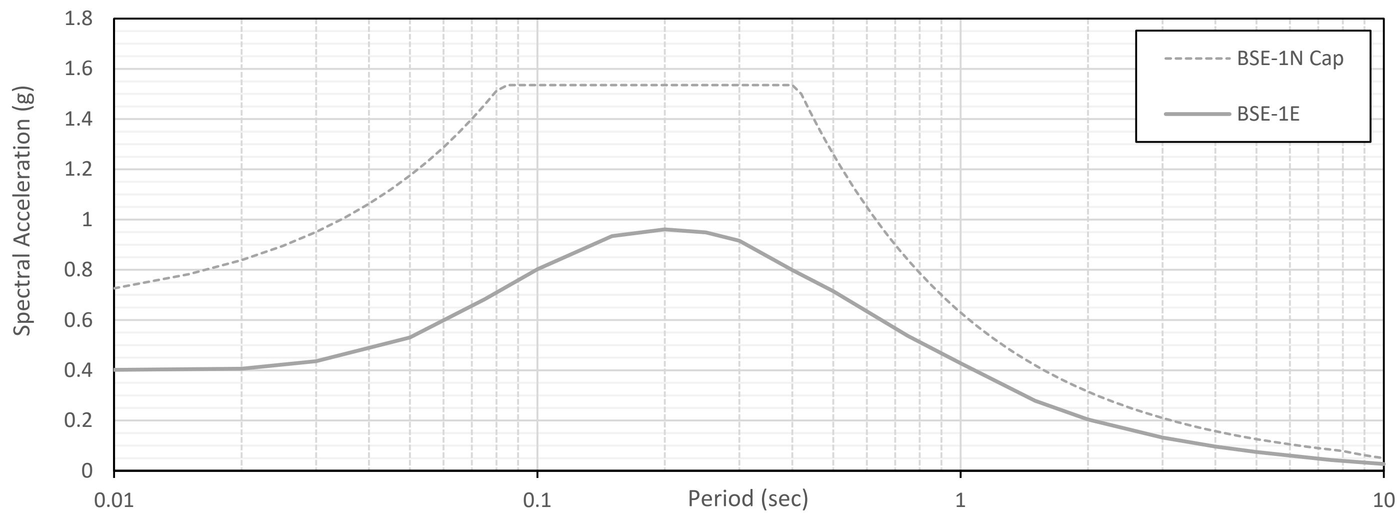
NO SCALE

PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Historical Topographic Map (1924)**	PROJECT NUMBER LA-1474	FIGURE NUMBER C-2



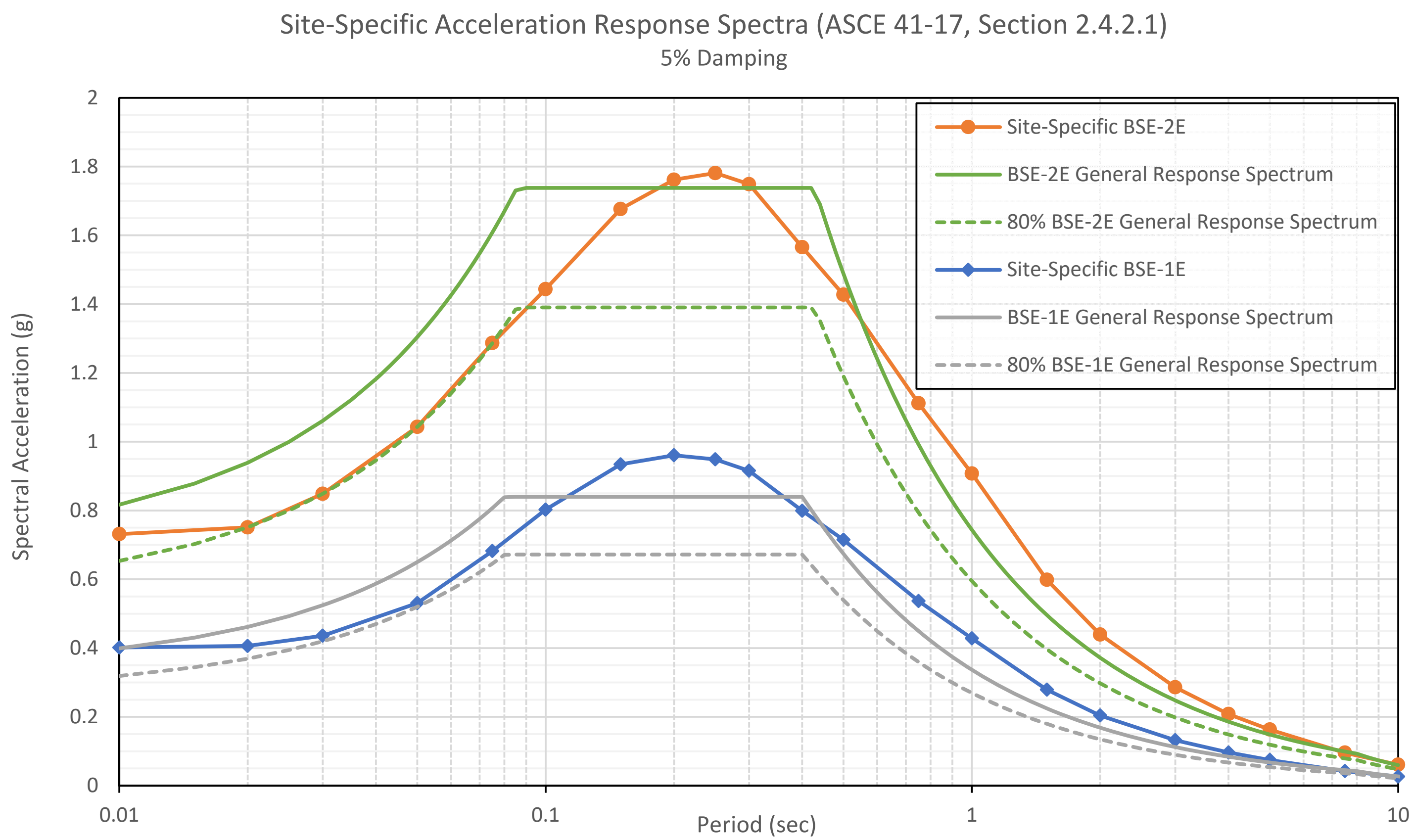
PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Probabilistic Seismic Hazard Analyses** **BSE-2E and BSE-1E Hazard Levels**	PROJECT NUMBER LA-1474	FIGURE NUMBER C-3





PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Site-Specific BSE-1E and BSE-2E** **Comparison with BSE-1N and BSE-2N**	PROJECT NUMBER LA-1474	FIGURE NUMBER C-4

ASCE 41-17 Site-Specific Acceleration Response Spectra		
Period (sec)	BSE-1E Sa (g)	BSE-2E Sa (g)
0.01	0.402	0.731
0.02	0.406	0.751
0.03	0.436	0.848
0.05	0.531	1.043
0.075	0.682	1.287
0.1	0.802	1.444
0.15	0.934	1.677
0.2	0.961	1.762
0.25	0.949	1.781
0.3	0.916	1.749
0.4	0.799	1.566
0.5	0.715	1.427
0.75	0.537	1.112
1	0.428	0.907
1.5	0.279	0.598
2	0.204	0.439
3	0.132	0.286
4	0.096	0.208
5	0.075	0.163
7.5	0.043	0.096
10	0.027	0.061



Site-Specific Design Acceleration Parameters

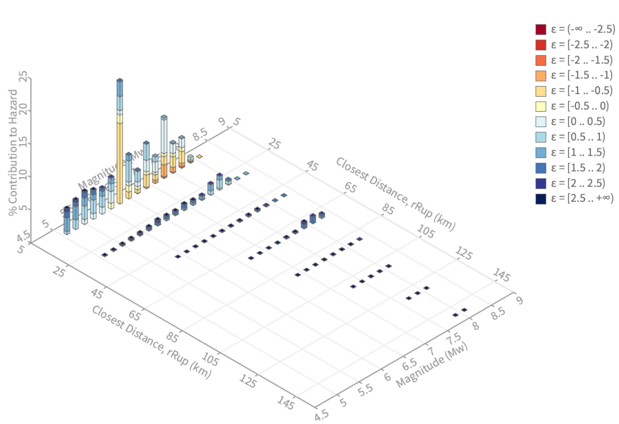
S_{XS} = 90% of the peak S_a from T =0.2 to 5 s (not less than 80% of mapped S_{XS})

S_{X1} = Peak $T*S_a$ between periods of 1 second and 2 seconds (not less than 80% of mapped S_{X1})

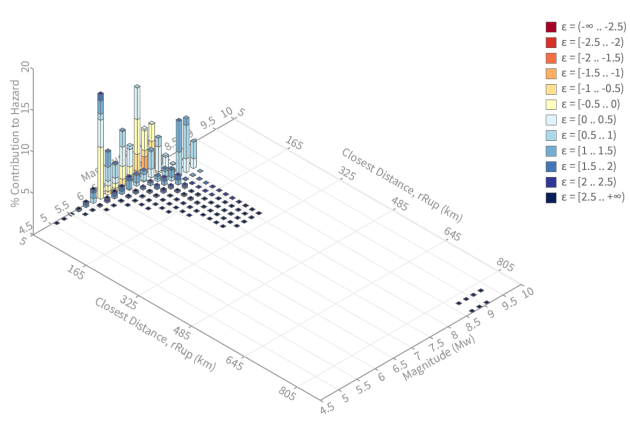
$S_{XS,BSE-1E}$ =	0.865	$S_{XS,BSE-2E}$ =	1.603
$S_{X1,BSE-1E}$ =	0.428	$S_{X1,BSE-2E}$ =	0.907

PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**ASCE 41-17 Site-Specific BSE-1E and BSE-2E ARS and Design Acceleration Parameters**	PROJECT NUMBER LA-1474	FIGURE NUMBER C-5

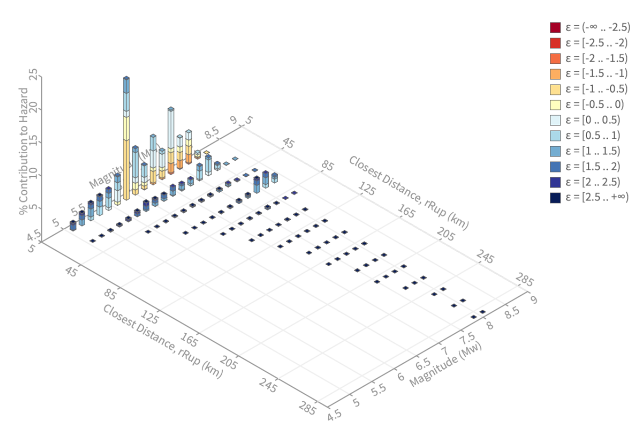
Spectral Period = PGA



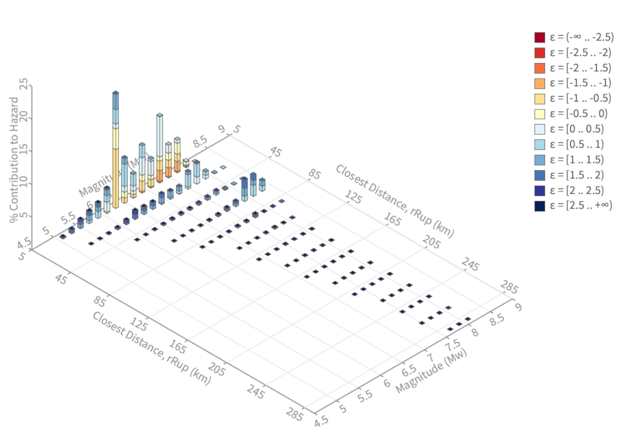
Spectral Period = 0.5 sec



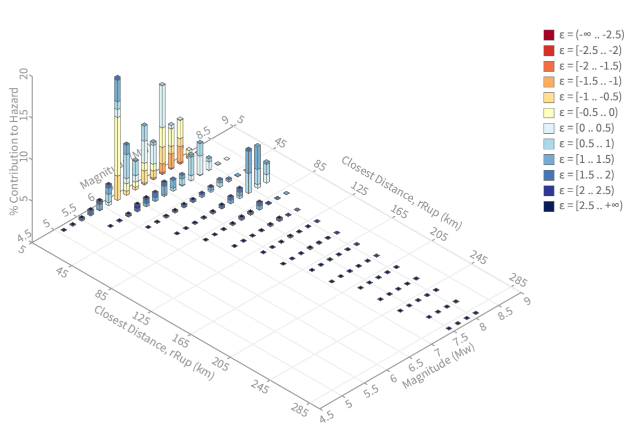
Spectral Period = 0.75 sec



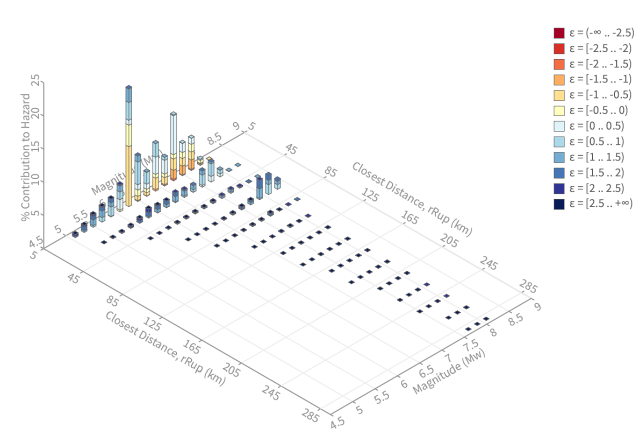
Spectral Period = 1 sec



Spectral Period = 2 sec



Spectral Period = 3 sec



PROJECT NAME

City of Inglewood Seismic Retrofits
1 W Manchester Blvd, Inglewood, CA 90301

FIGURE NAME

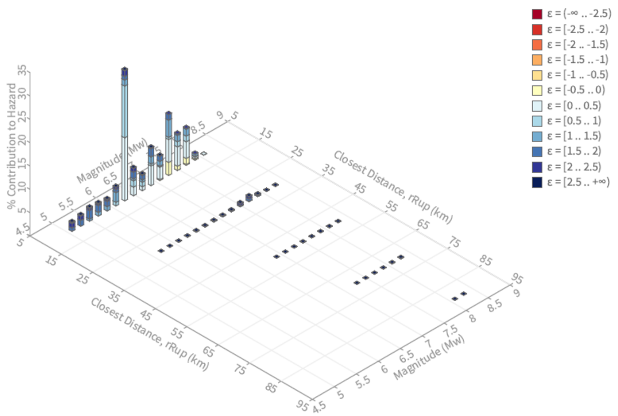
DEAGGREGATION OF THE BSE-1E SEISMIC HAZARD

GROUP DELTA

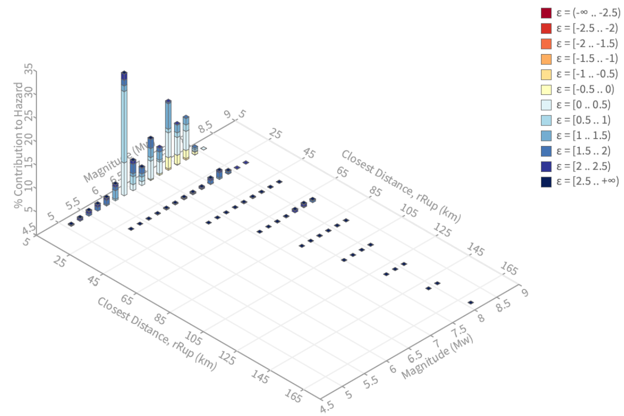
PROJECT NUMBER: LA-1474

FIGURE NUMBER: **C-6**

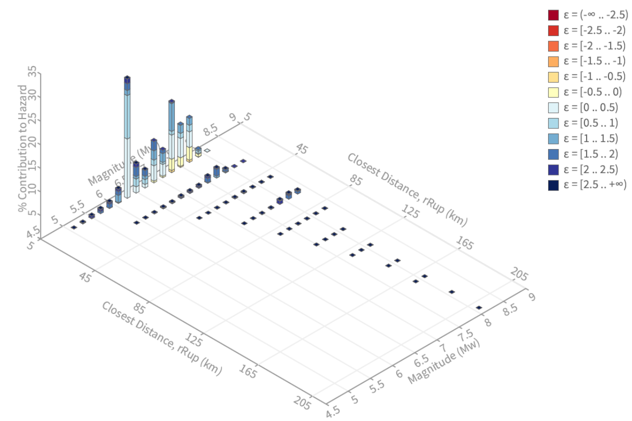
Spectral Period = PGA



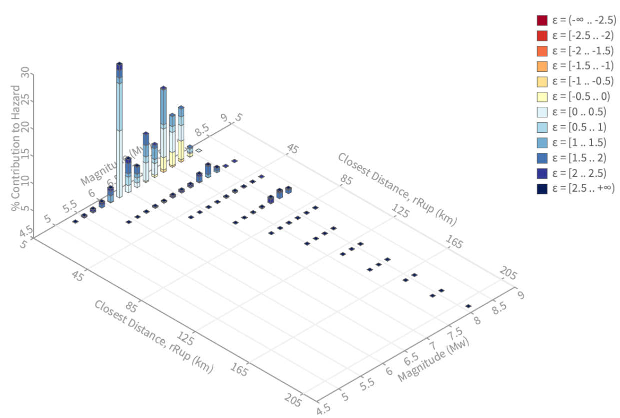
Spectral Period = 0.5 sec



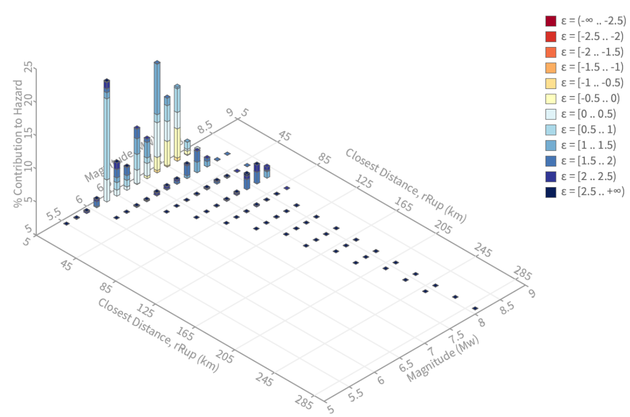
Spectral Period = 0.75 sec



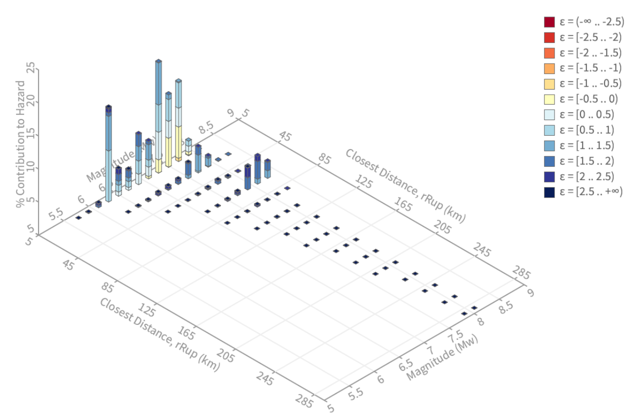
Spectral Period = 1 sec



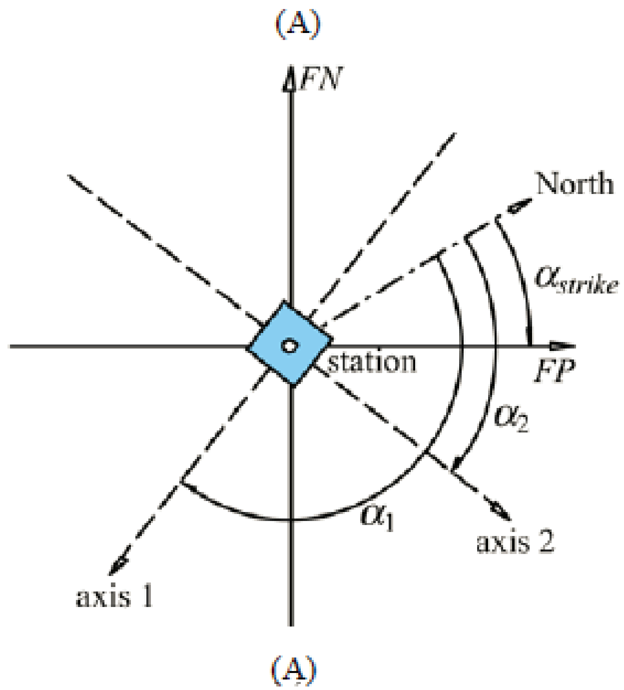
Spectral Period = 2 sec



Spectral Period = 3 sec

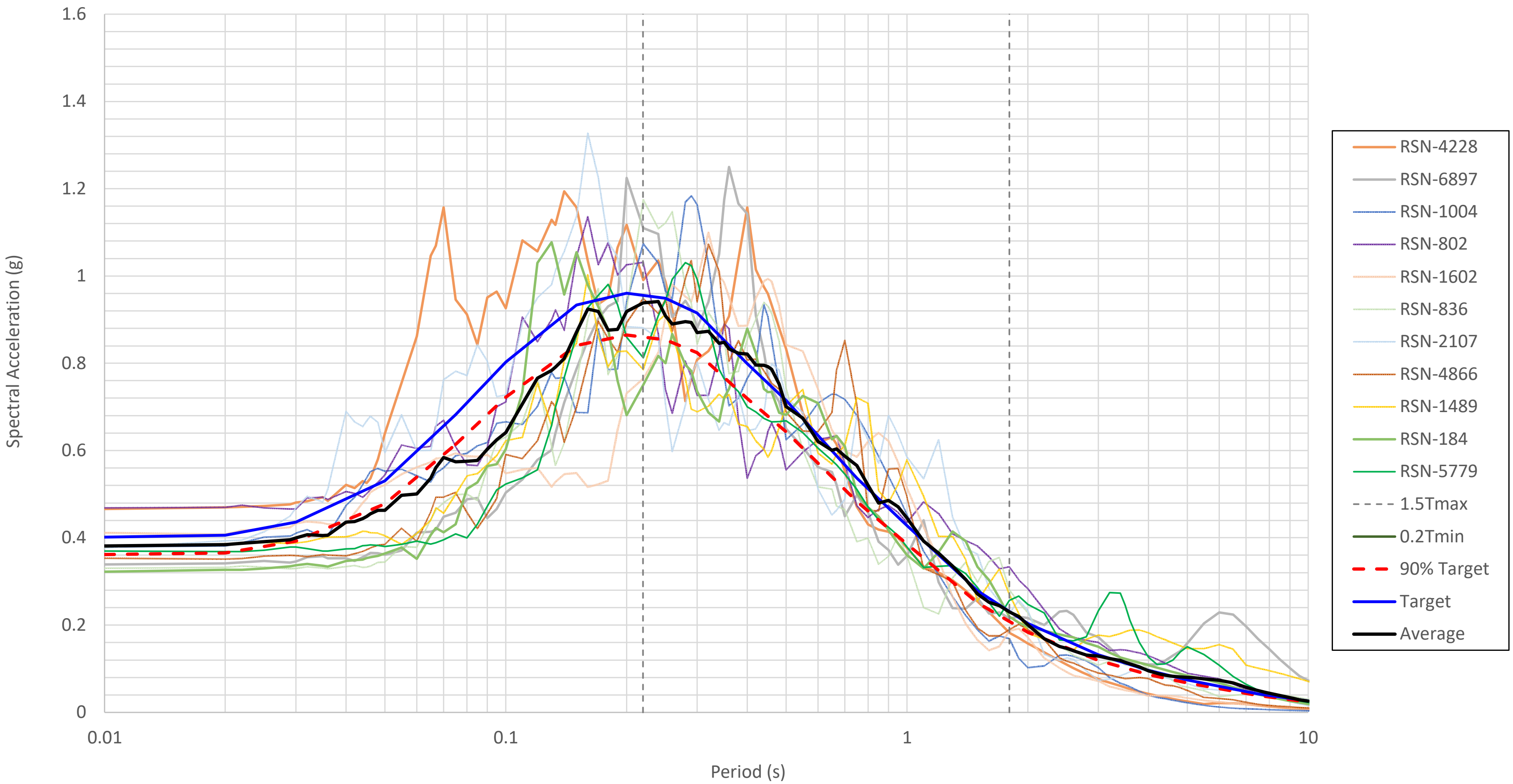


PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**DEAGGREGATION OF THE BSE-2E SEISMIC HAZARD**	PROJECT NUMBER LA-1474	FIGURE NUMBER **C-7**

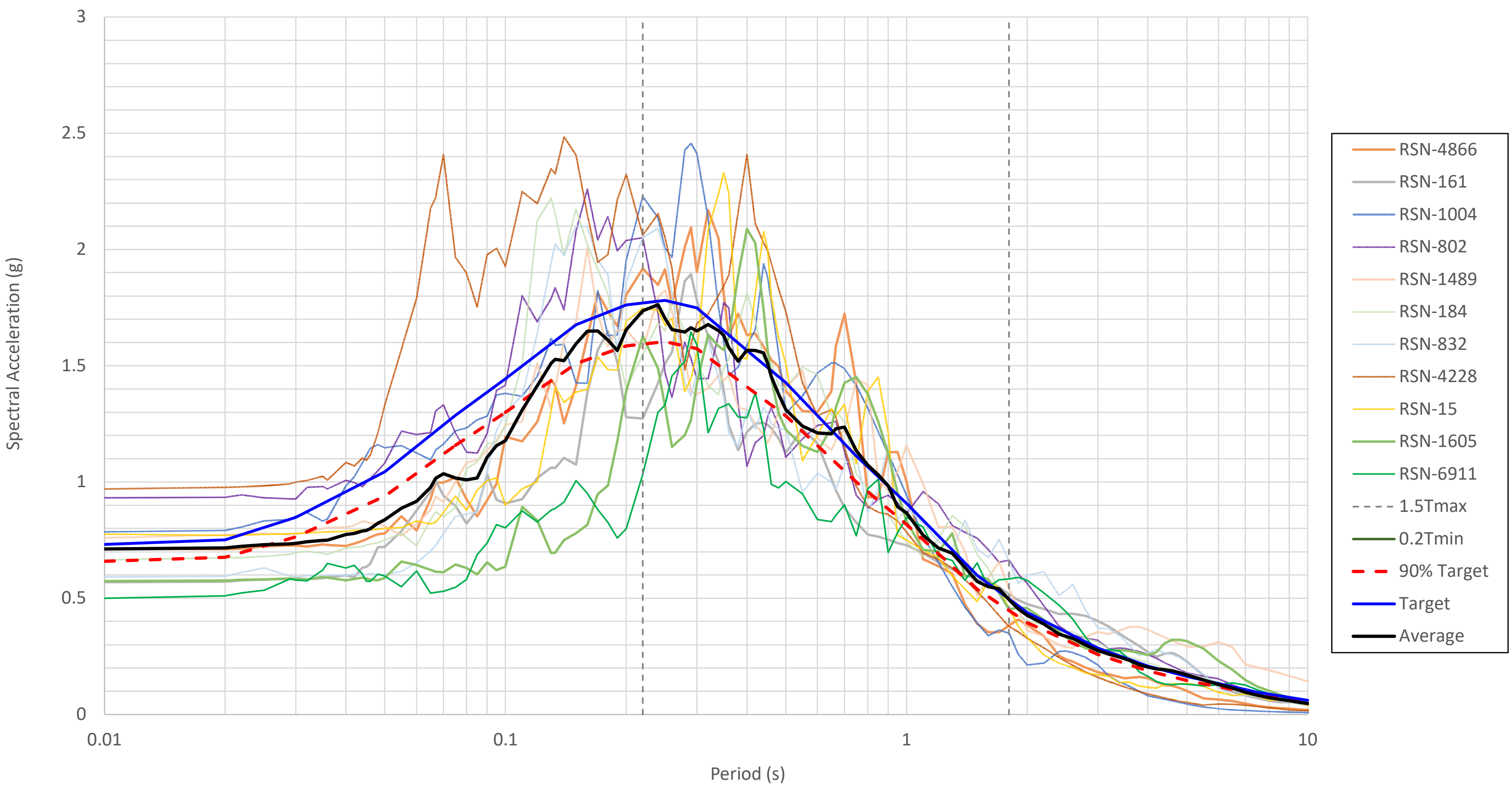


PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	REFERENCE AXES FOR GROUND MOTION ROTATION	PROJECT NUMBER LA-1474	FIGURE NUMBER **C-8**

REFERENCE: Figure 2 from Reyes and Kalkan (2012)



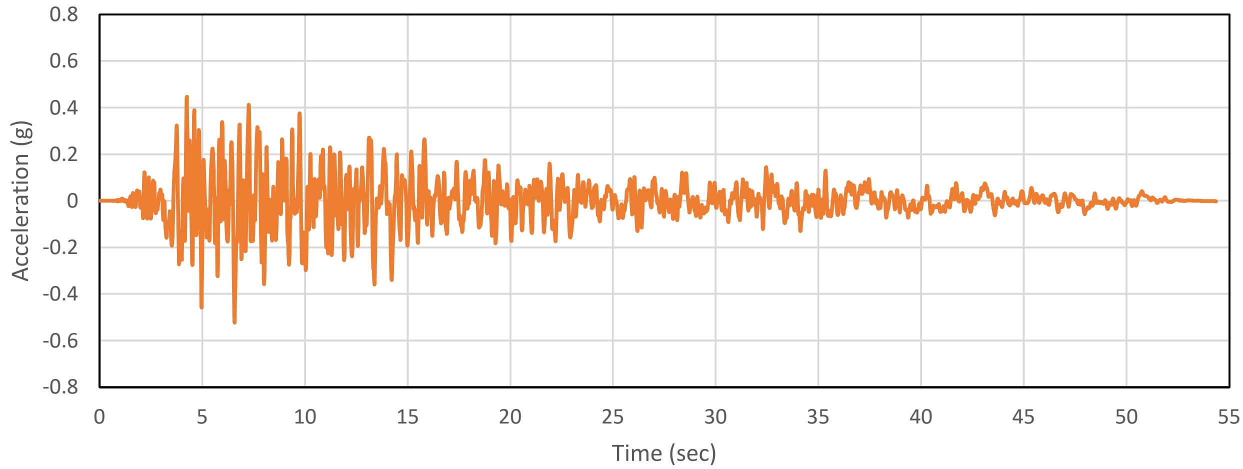
PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Comparison of Maximum Direction Spectra with BSE-1E Target**	PROJECT NUMBER LA-1474	FIGURE NUMBER C-9

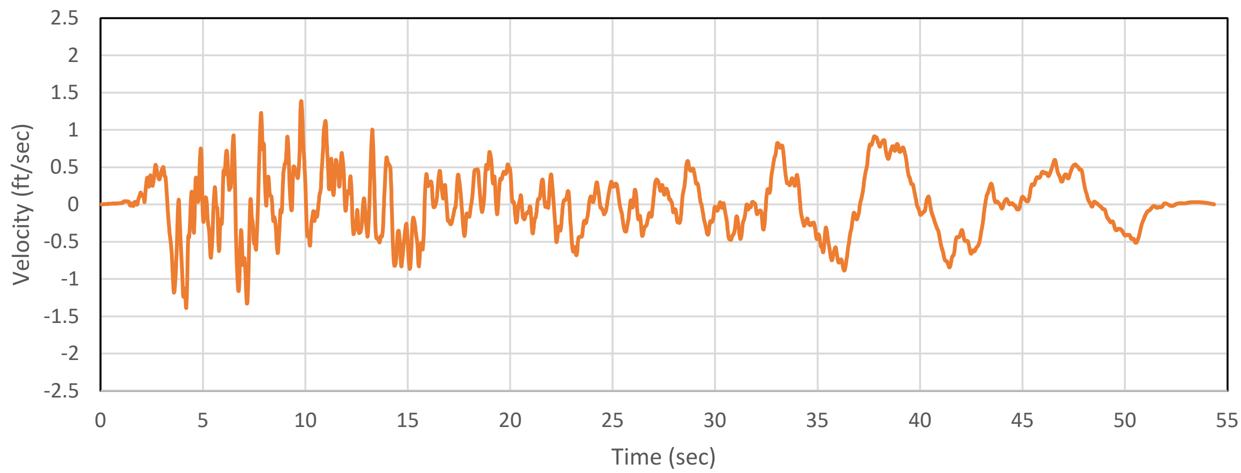


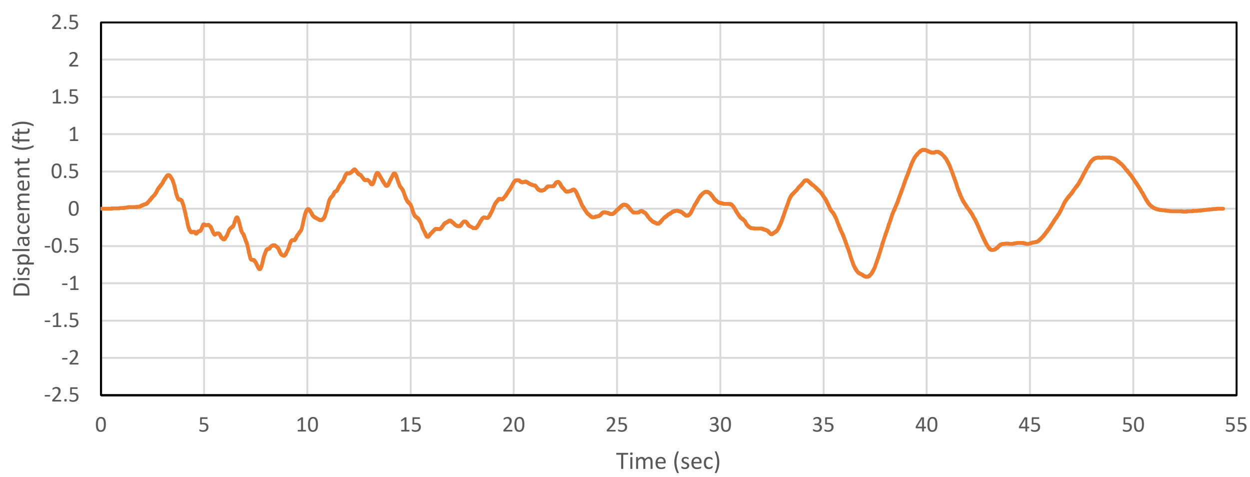
PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Comparison of Maximum Direction Spectra with BSE-2E Target**	PROJECT NUMBER LA-1474	FIGURE NUMBER C-10

RSN 15

Station: Taft Lincoln School, Earthquake: Kern County



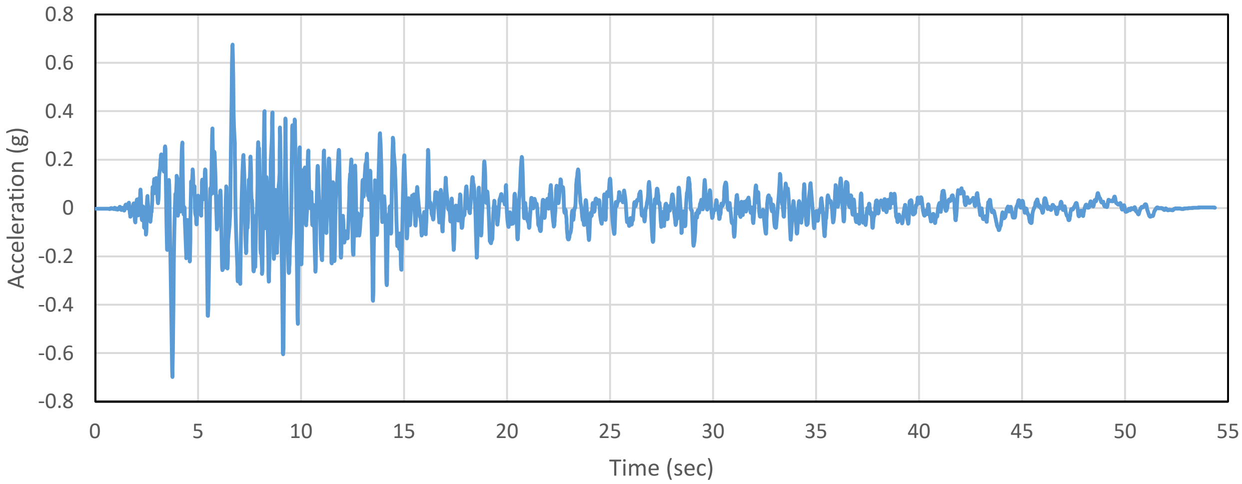


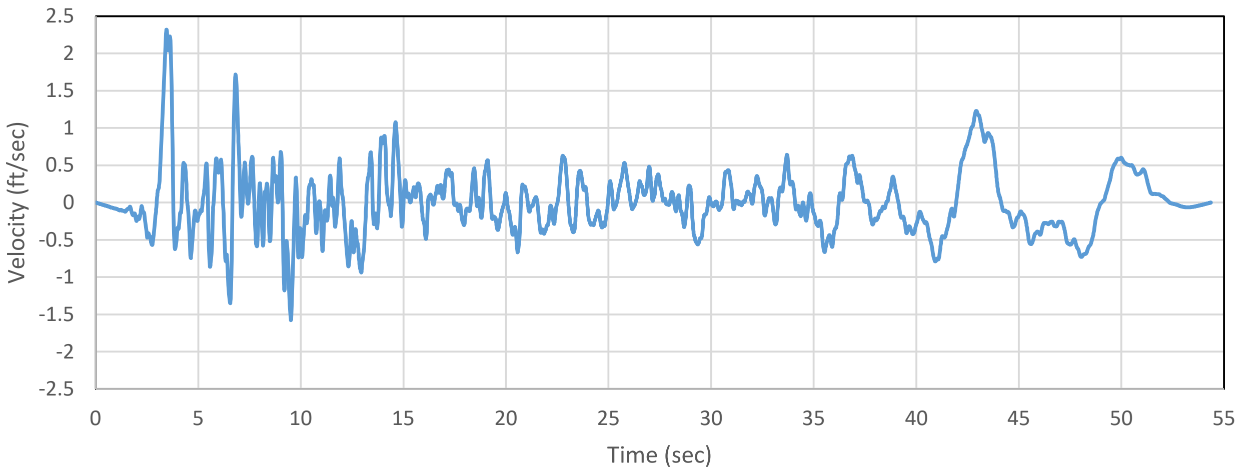


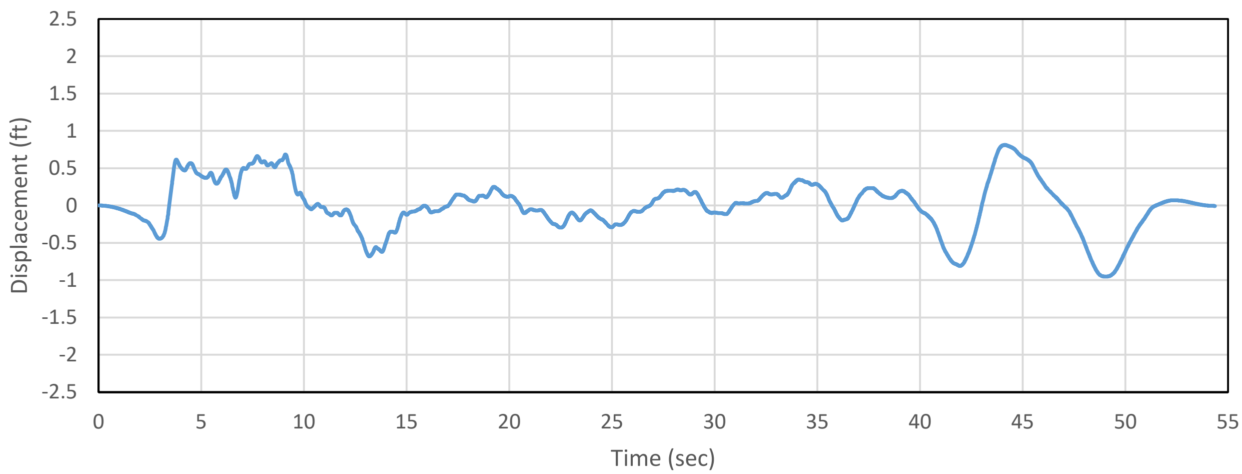
GROUP DELTA	**Scaled Time Histories - RSN 15 (FN)**		Project Number: LA1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-11a**

RSN 15

Station: Taft Lincoln School, Earthquake: Kern County

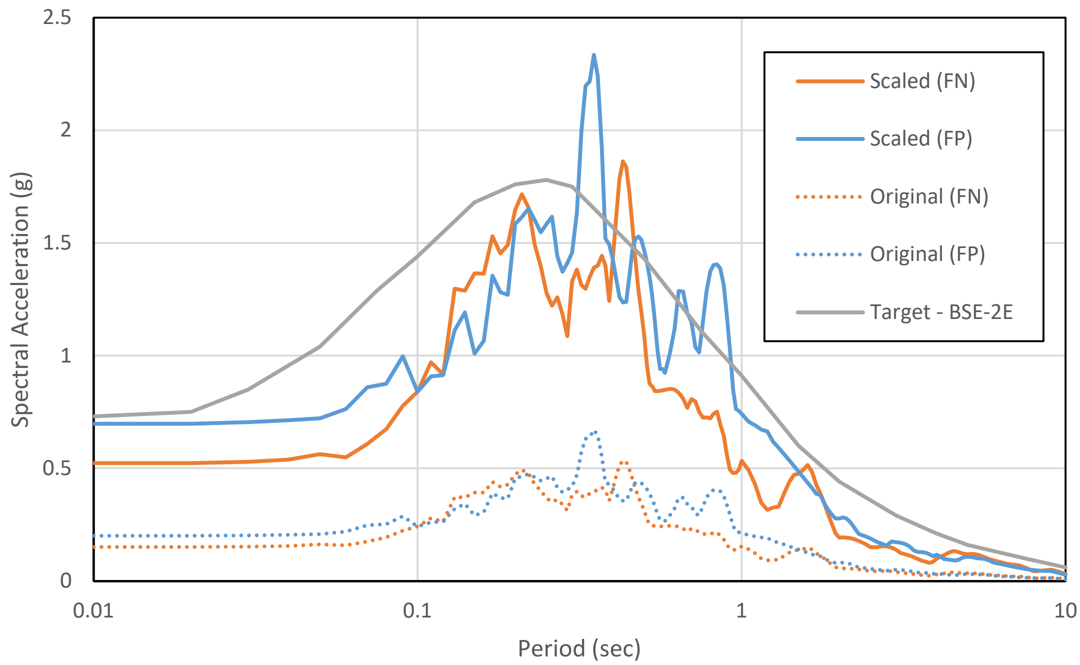


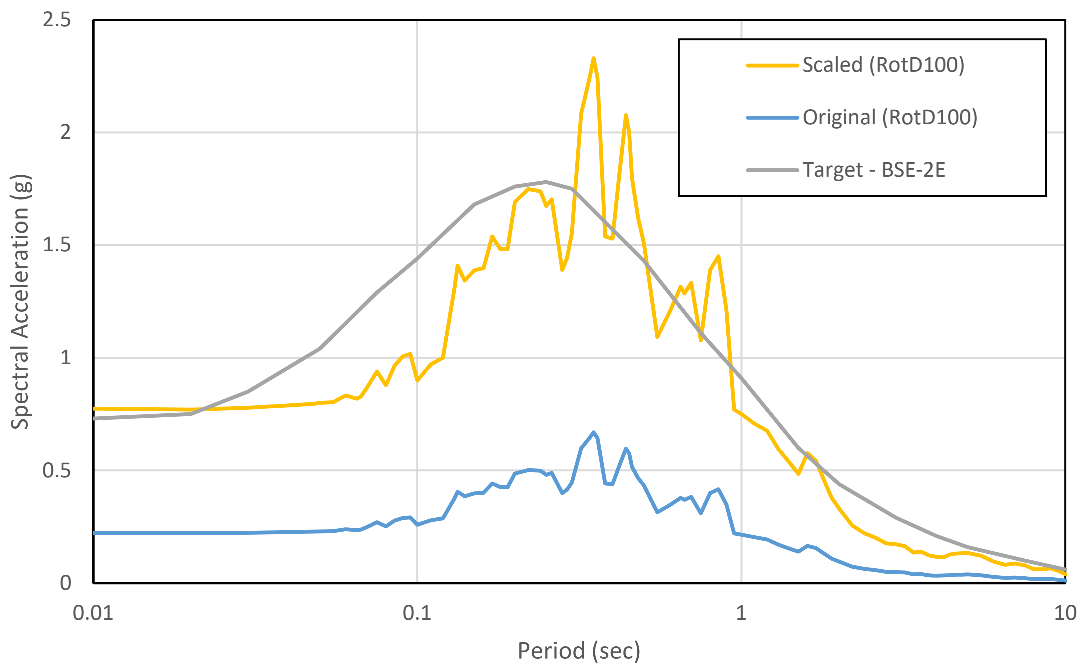




GROUP DELTA	**Scaled Time Histories - RSN 15 (FP)**		Project Number: LA1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-11b**

Response Spectra (5% damping)
Station: Taft Lincoln School, Earthquake: Kern County

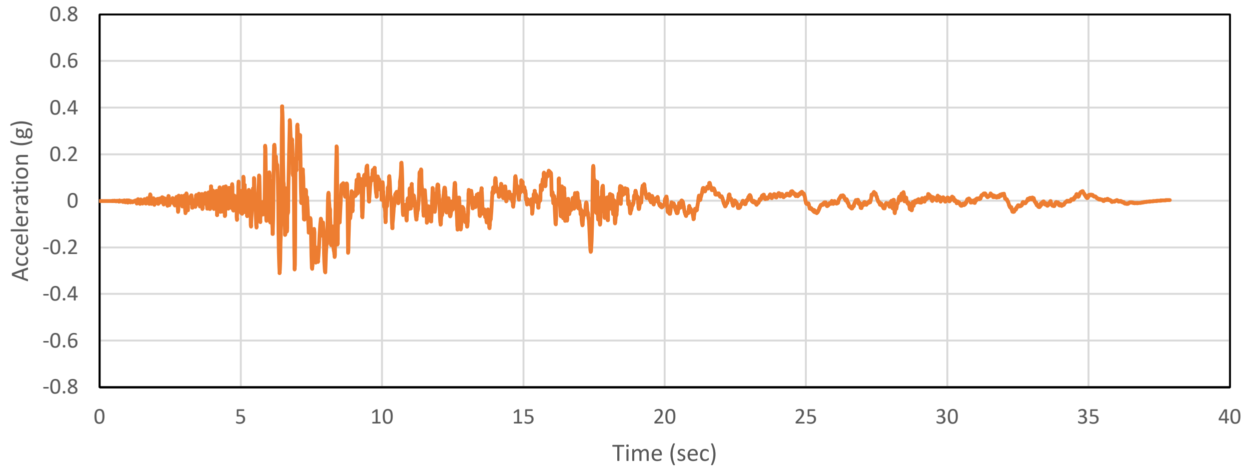


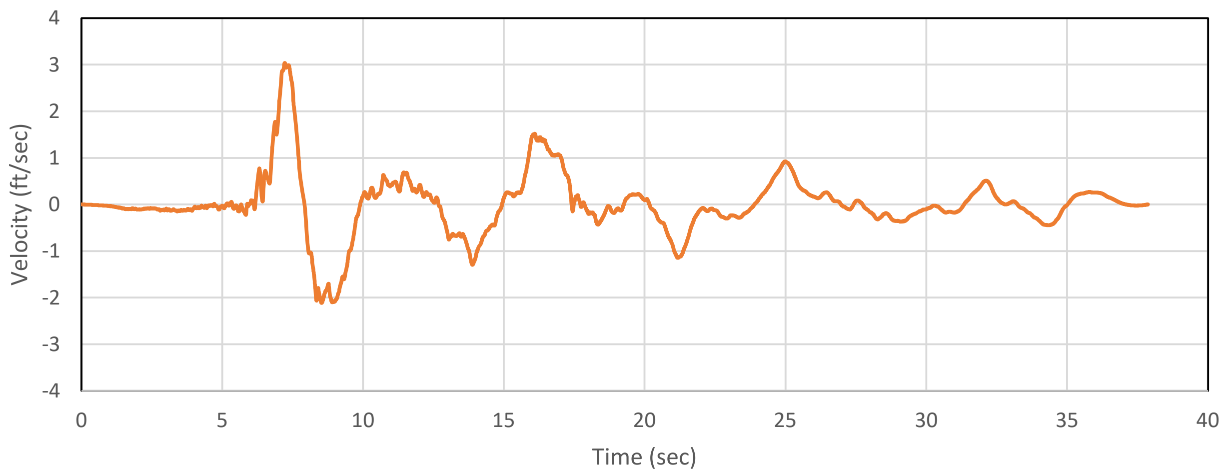


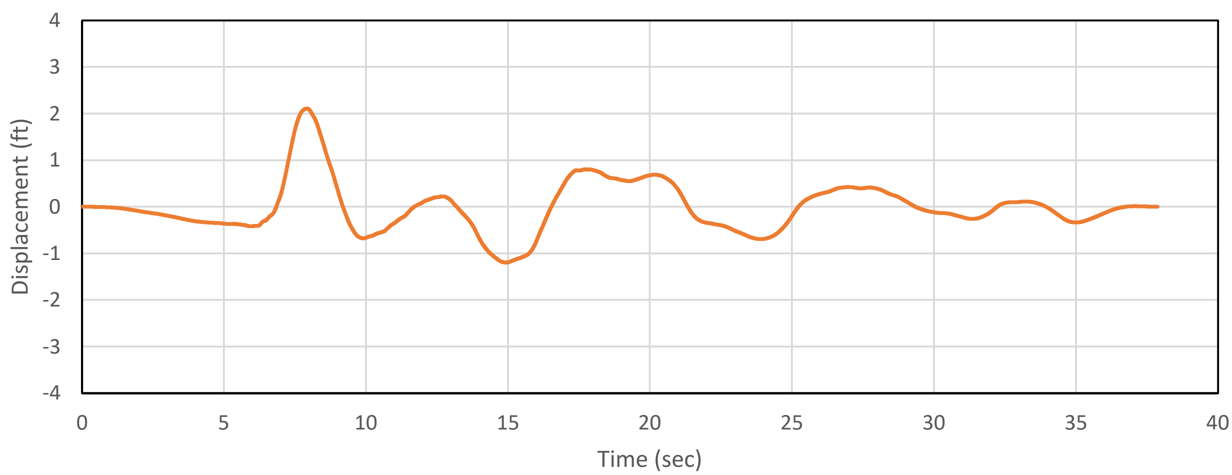
GROUP DELTA	**Response Spectra - RSN 15**	Project Number: LA1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-11c**

RSN 161

Station: Brawley Airport, Earthquake: Imperial Valley-06



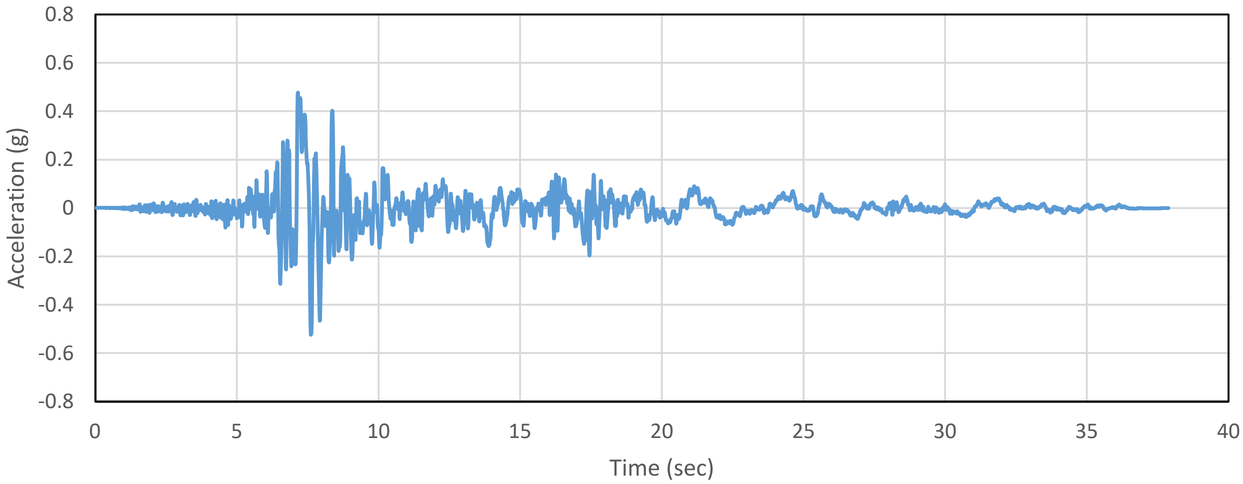


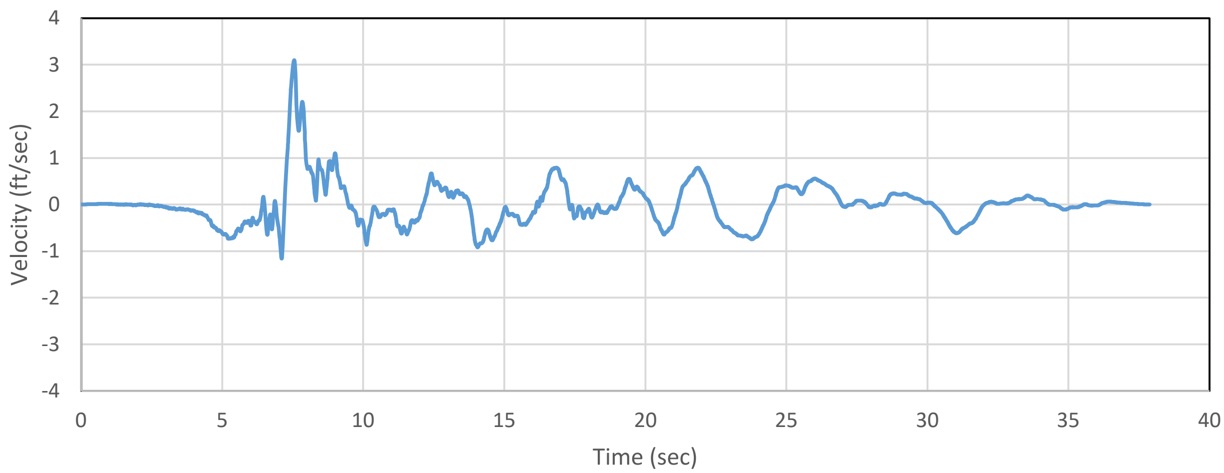


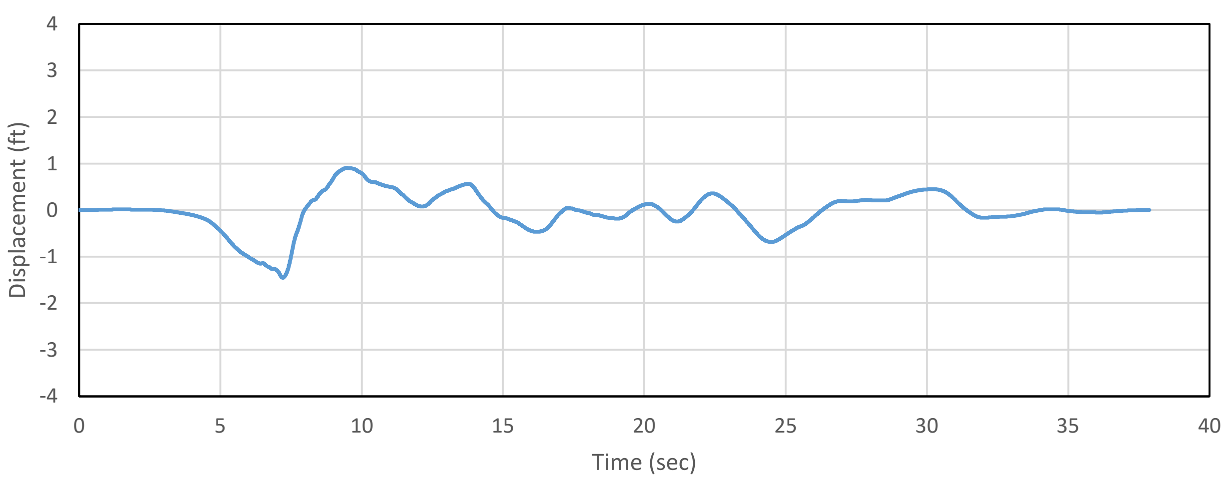
GROUP DELTA	**Scaled Time Histories - RSN 161 (FN)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-12a**

RSN 161

Station: Brawley Airport, Earthquake: Imperial Valley-06



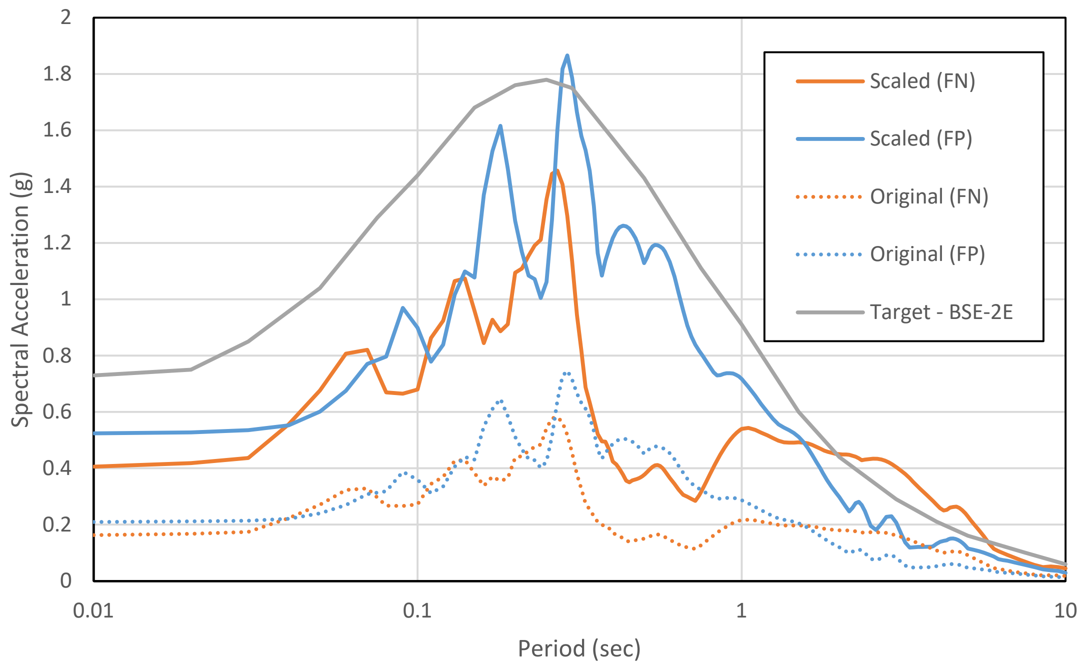


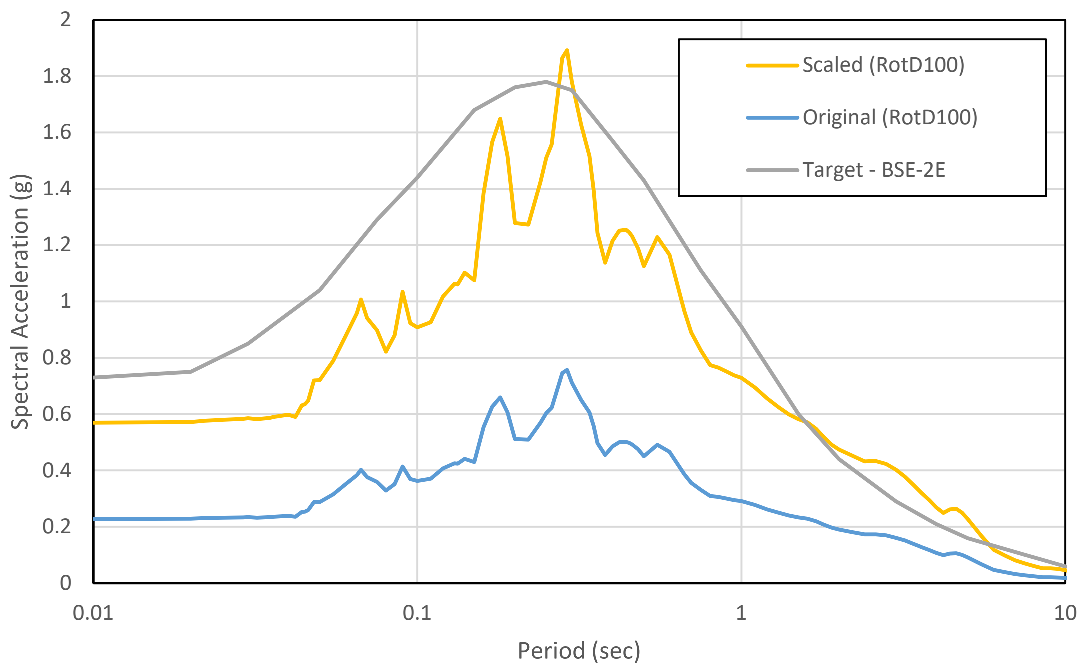


GROUP DELTA	**Scaled Time Histories - RSN 161 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-12b**

Response Spectra (5% damping)

Station: Brawley Airport, Earthquake: Imperial Valley-06

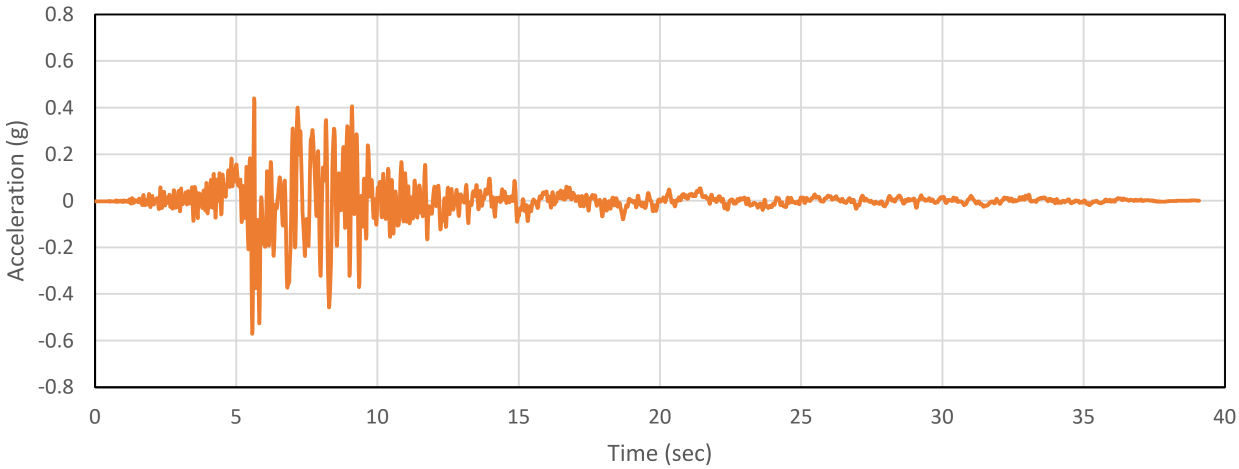


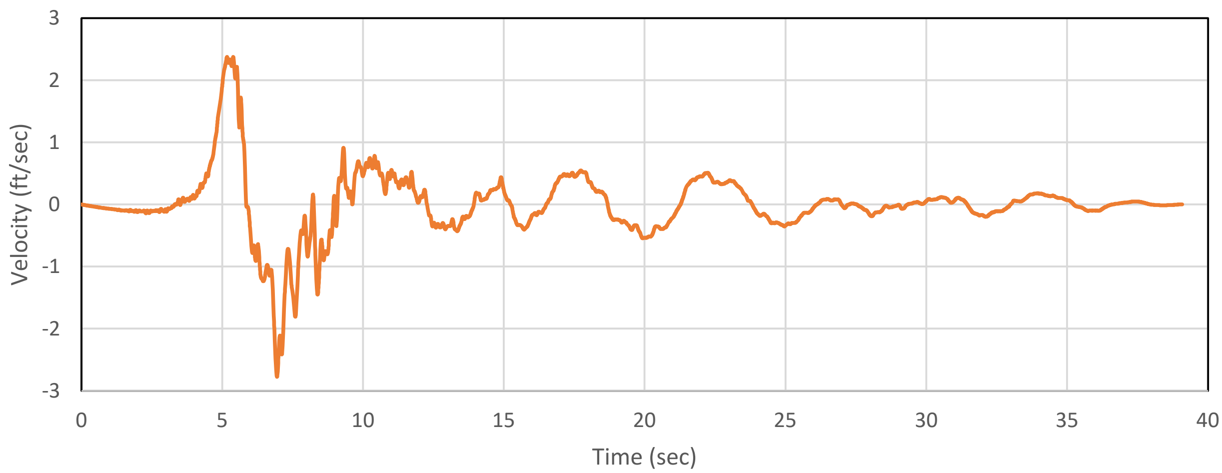


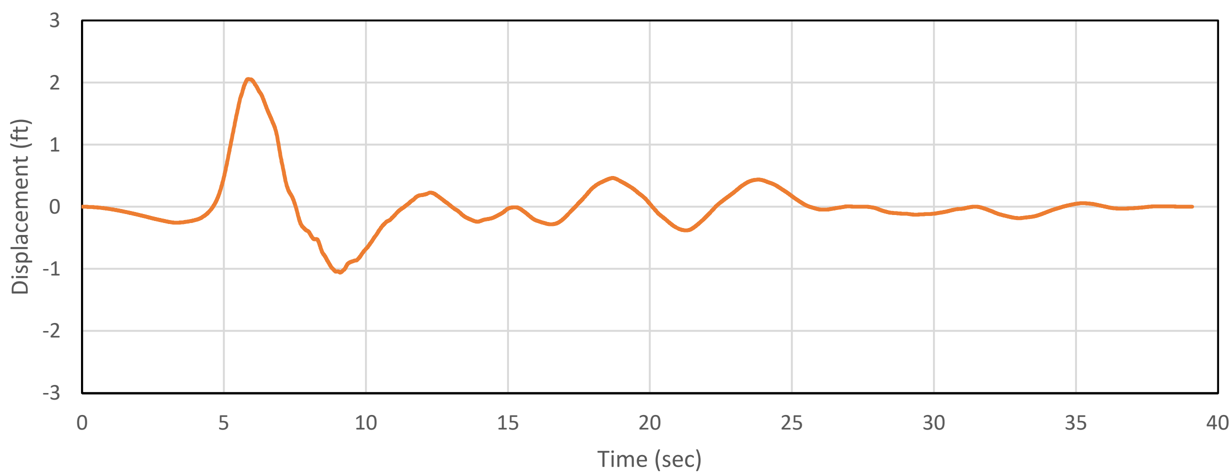
GROUP DELTA	**Response Spectra - RSN 161**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-12c**

RSN 184

Station: El Centro Differential Array, Earthquake: Imperial Valley-06



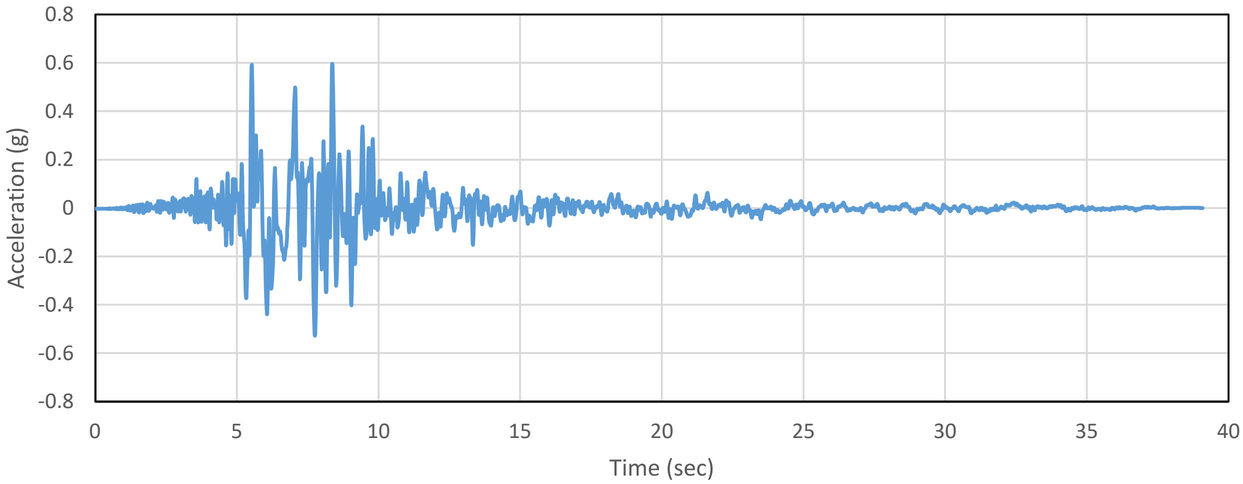


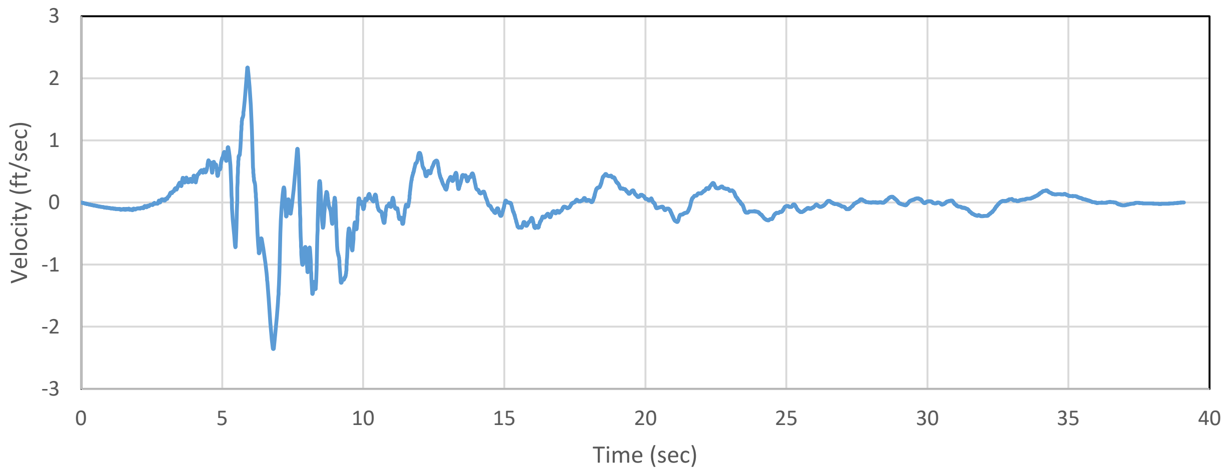


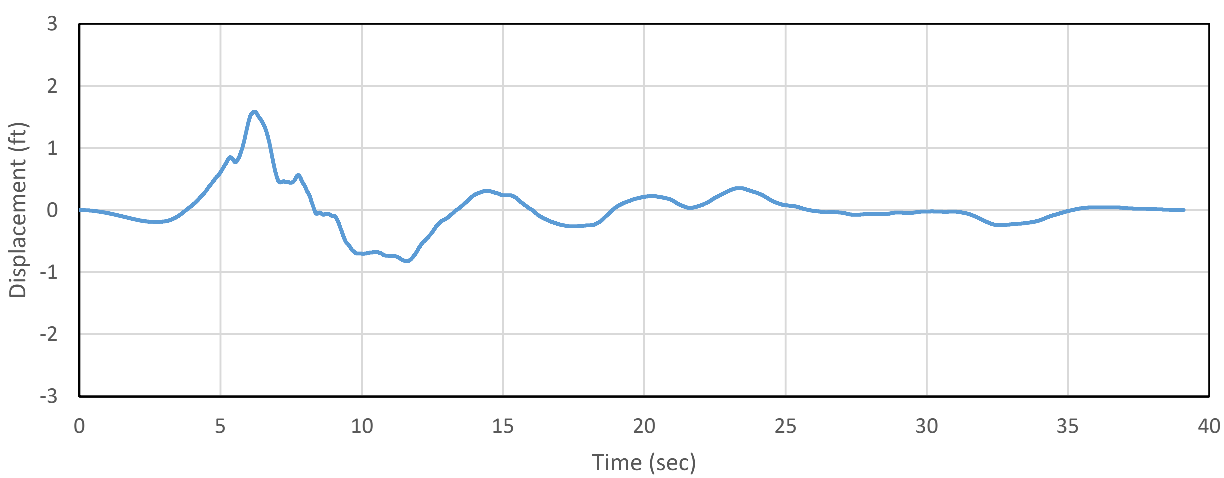
GROUP DELTA	**Scaled Time Histories - RSN 184 (FN)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-13a**

RSN 184

Station: El Centro Differential Array, Earthquake: Imperial Valley-06



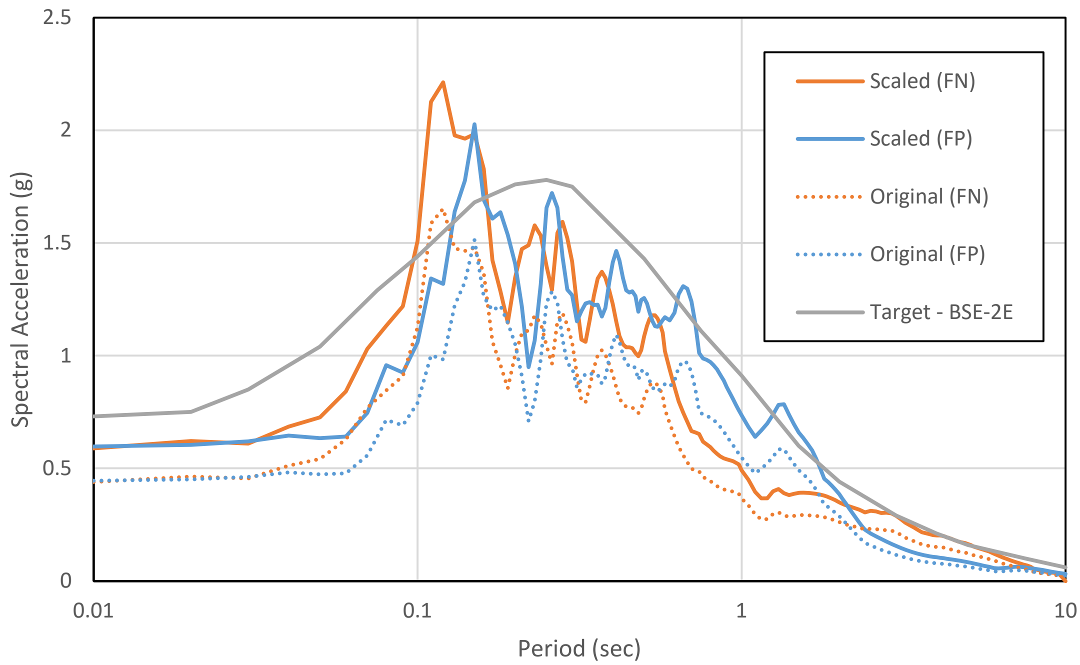


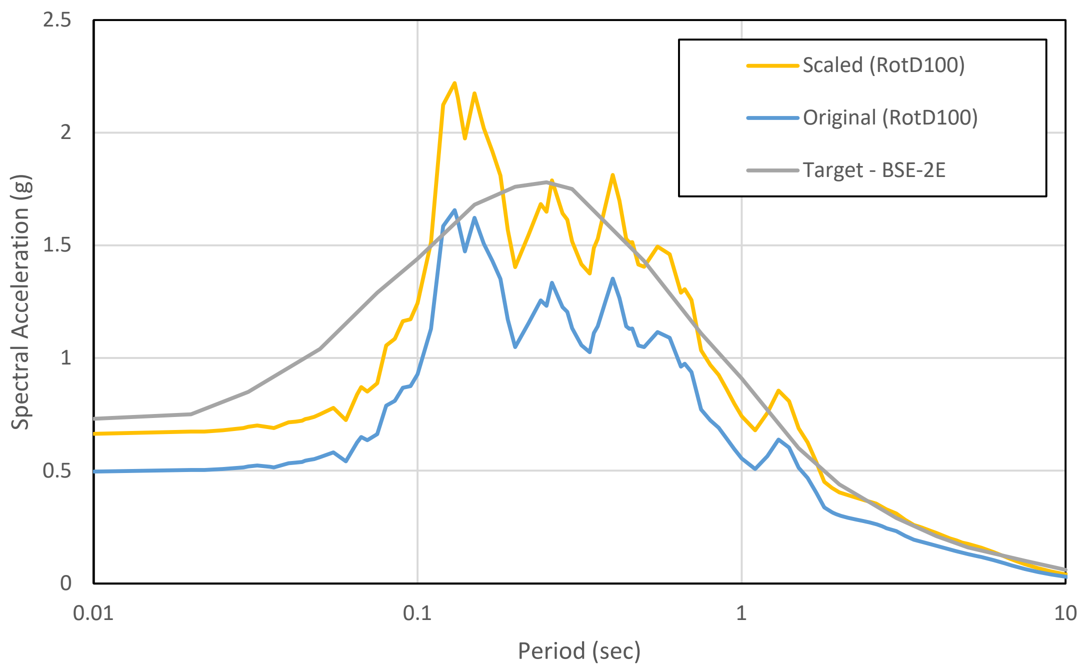


GROUP DELTA	**Scaled Time Histories - RSN 184 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-13b**

Response Spectra (5% damping)

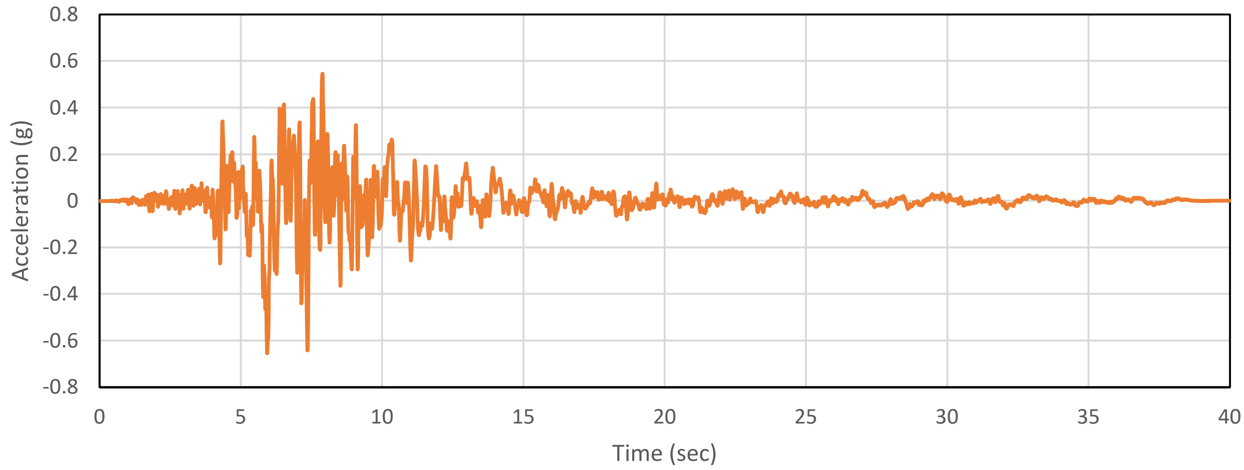
Station: El Centro Differential Array, Earthquake: Imperial Valley-06

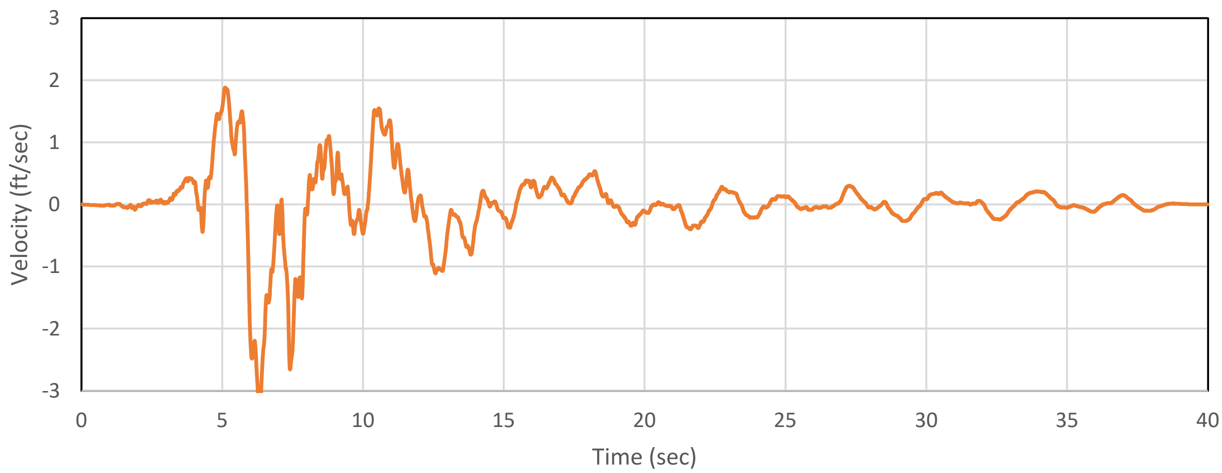


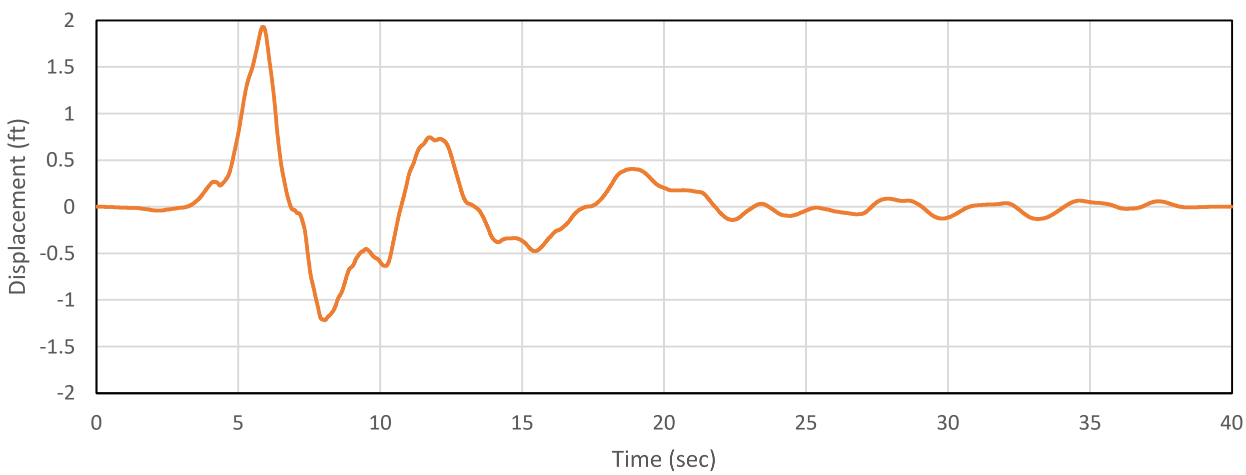


GROUP DELTA	**Response Spectra - RSN 184**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-13c**

RSN 802
Station: Saratoga - Aloha Ave Earthquake: Loma Prieta







GROUP DELTA

Scaled Time Histories - RSN 802 (FN)

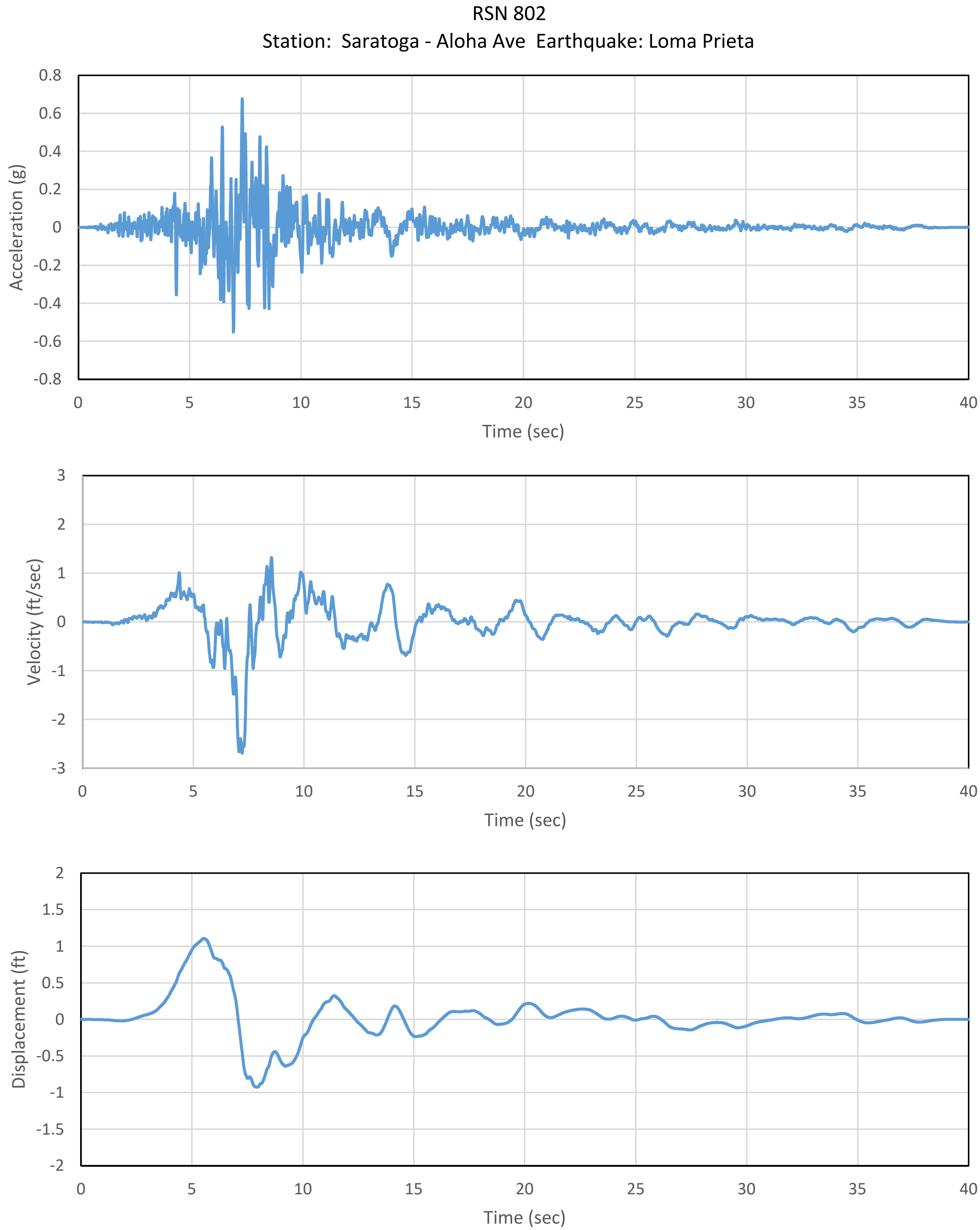
Project Number: LA-1417

Date: 12/18/2020

GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-14a



GROUP DELTA

Scaled Time Histories - RSN 802 (FP)

Project Number: LA-1417

Date: 12/18/2020

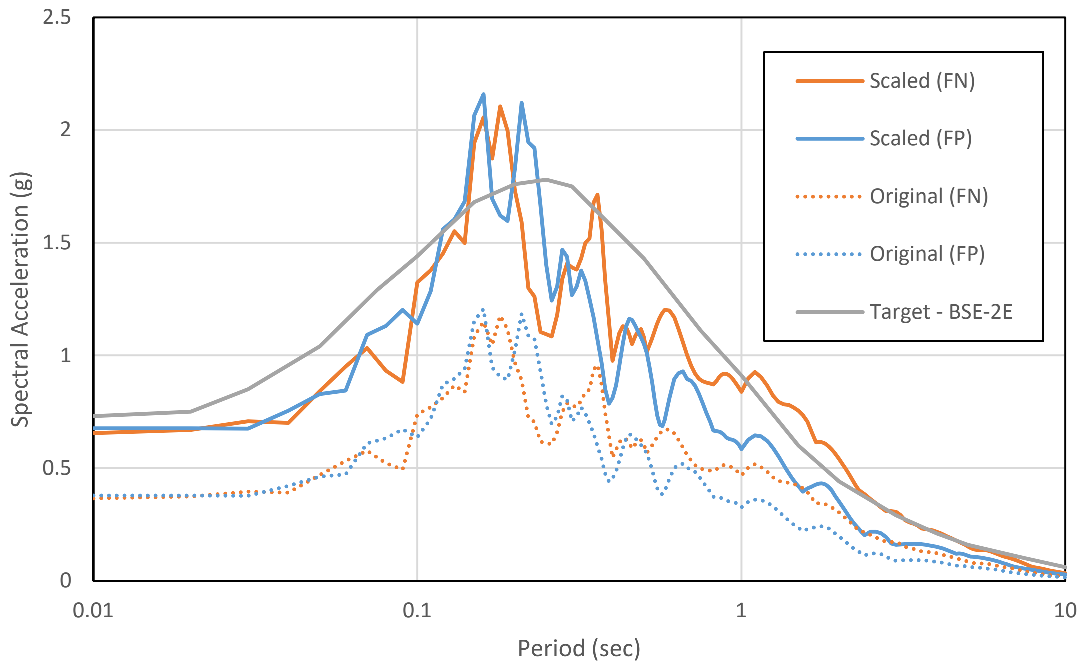
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

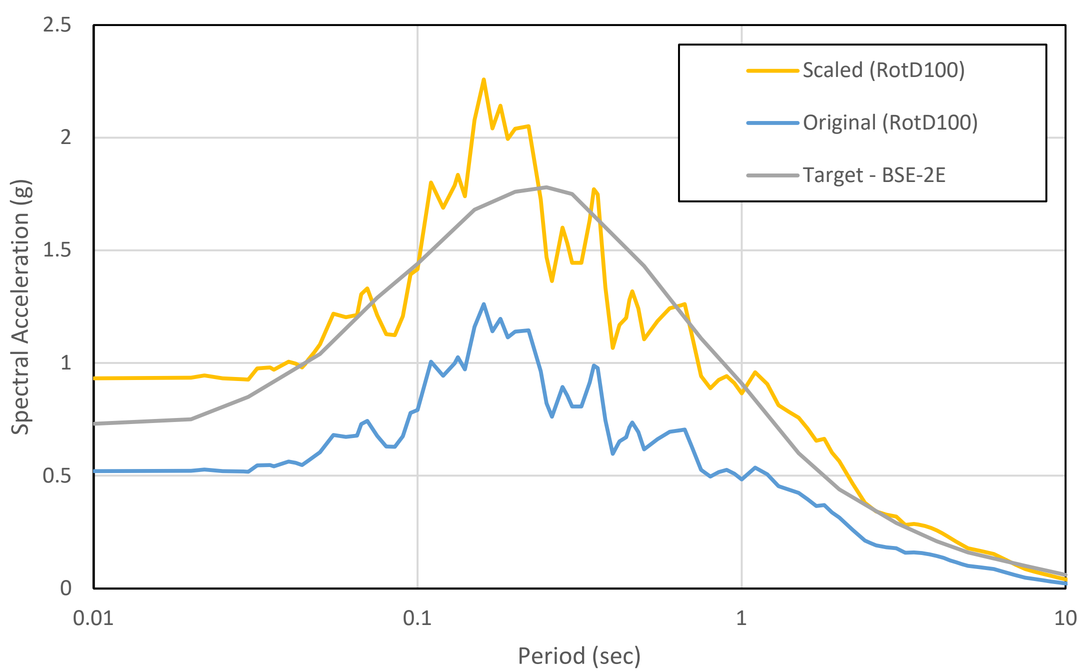
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-14b

Response Spectra (5% damping)

Station: Saratoga - Aloha Ave Earthquake: Loma Prieta

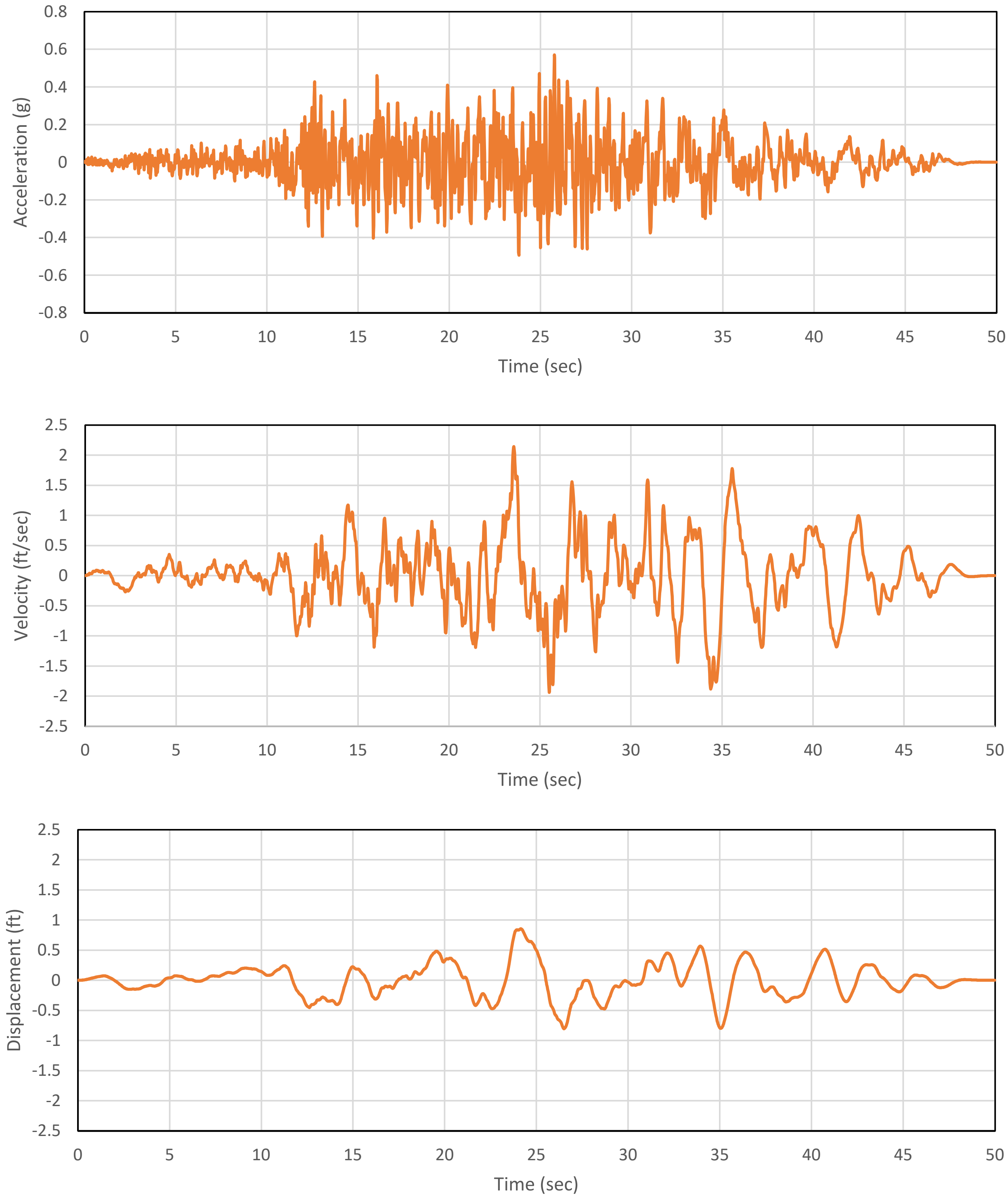




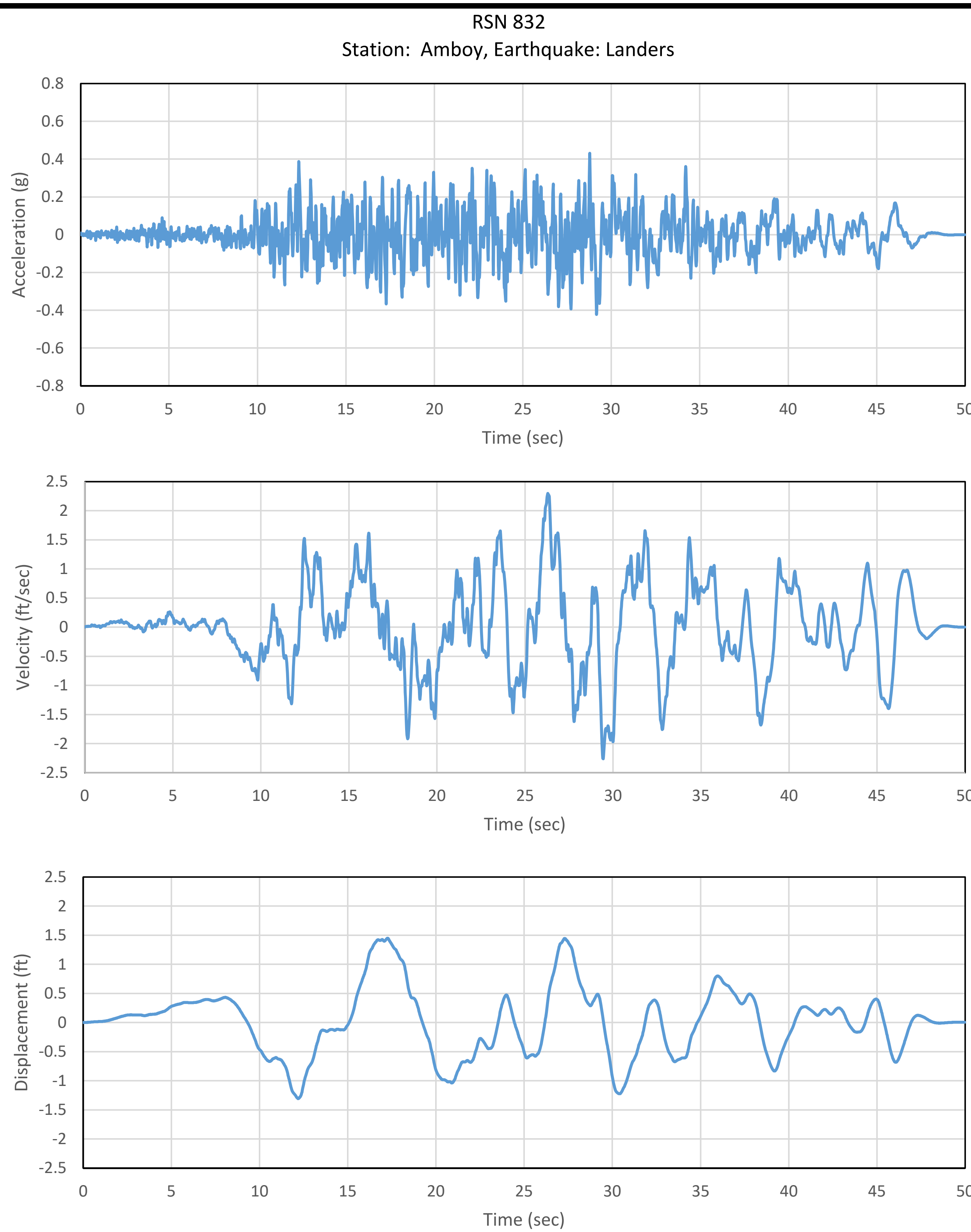
GROUP DELTA	**Response Spectra - RSN 802**		Project Number: LA-1417 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-14c**

RSN 832

Station: Amboy, Earthquake: Landers



GROUP DELTA	**Scaled Time Histories - RSN 832 (FN)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-15a**



GROUP DELTA

Scaled Time Histories - RSN 832 (FP)

Project Number: LA-1474

Date: 12/18/2020

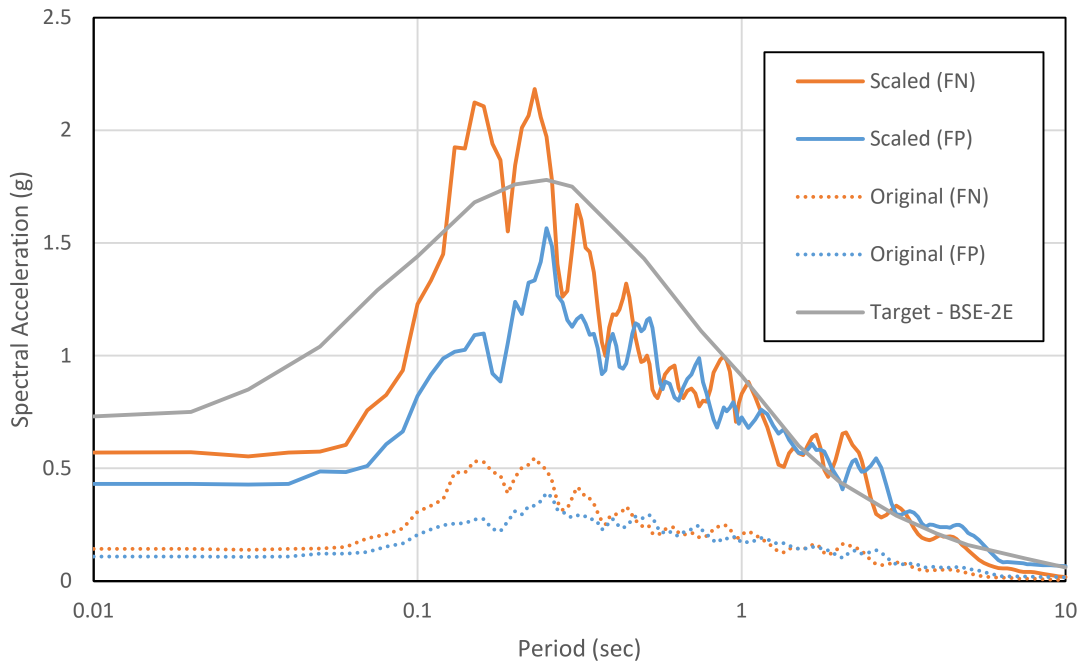
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

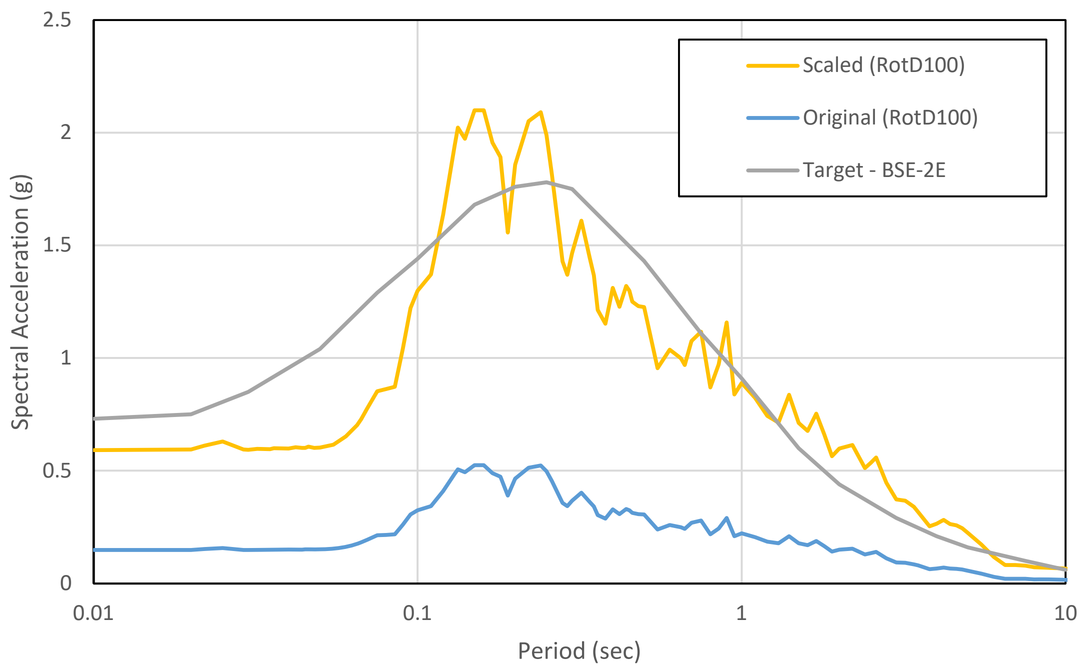
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-15b

Response Spectra (5% damping)

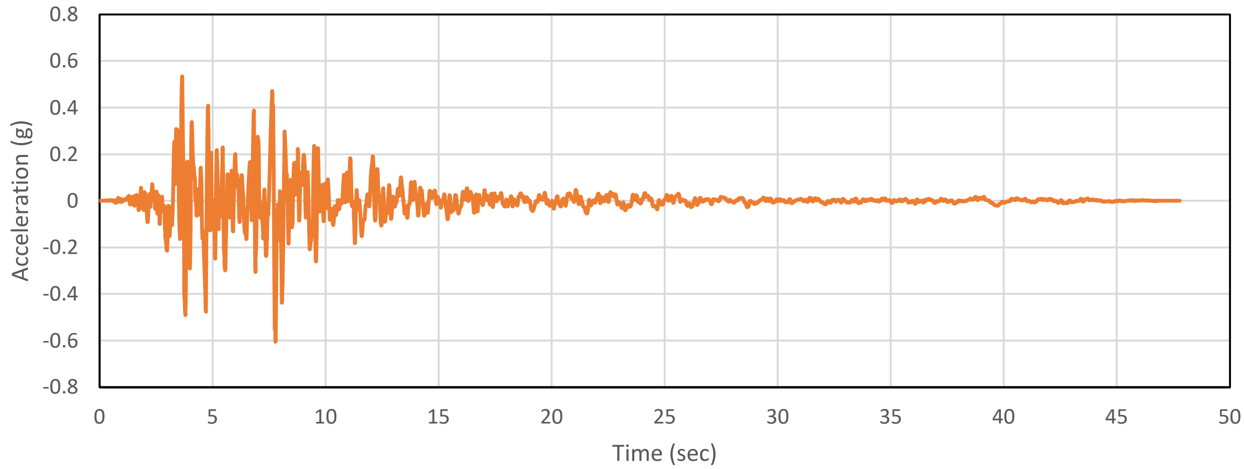
Station: Amboy, Earthquake: Landers

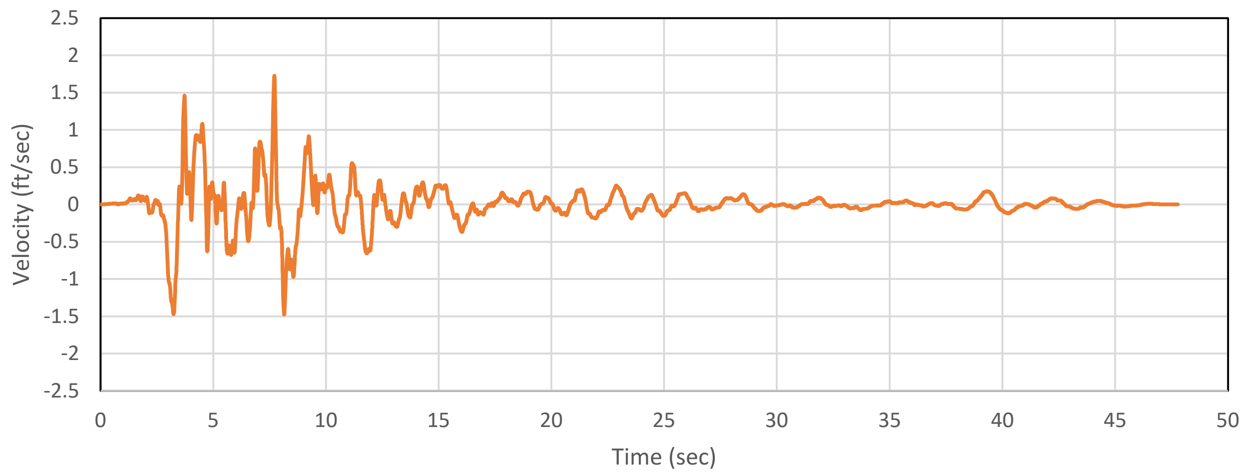


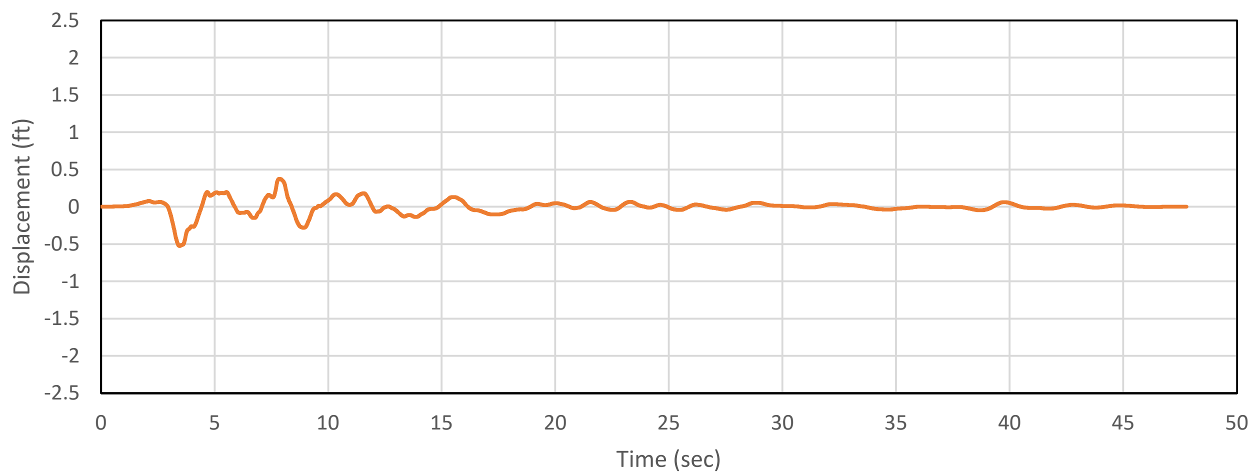


GROUP DELTA	Response Spectra - RSN 832		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-15c**

RSN 1004
Station: LA - Sepulveda VA Hospital, Earthquake: Northridge-01



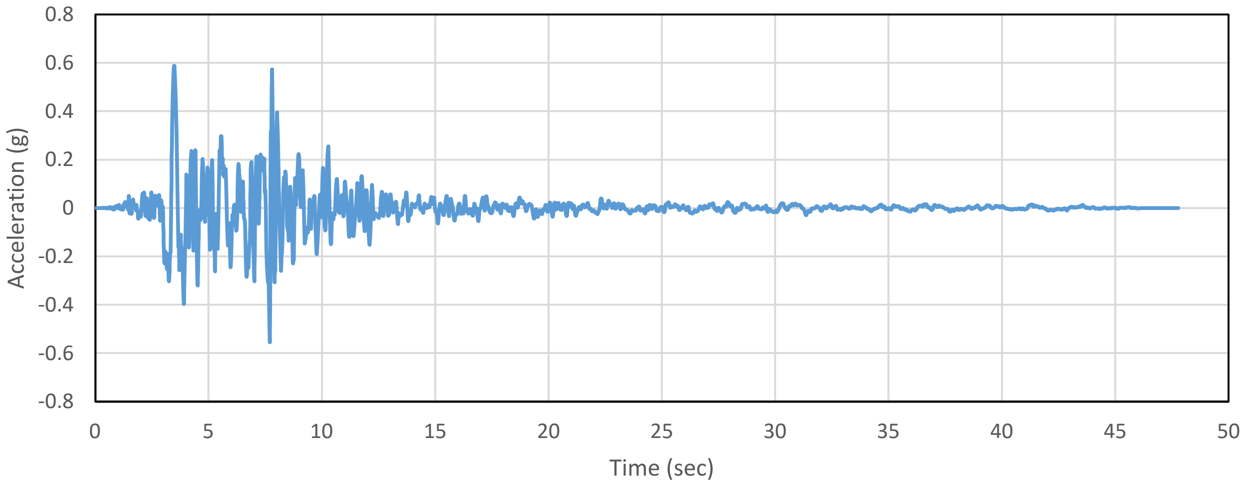


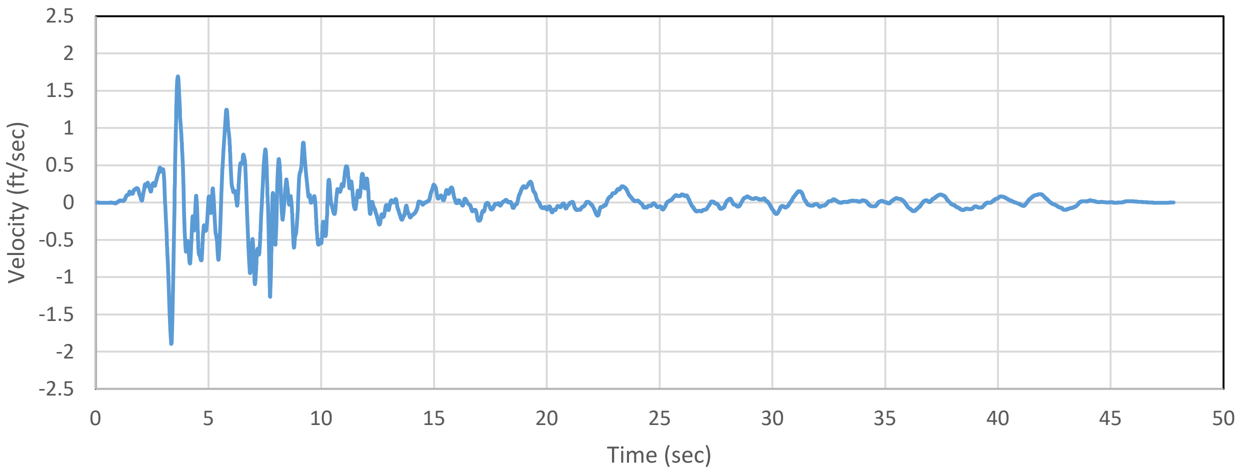


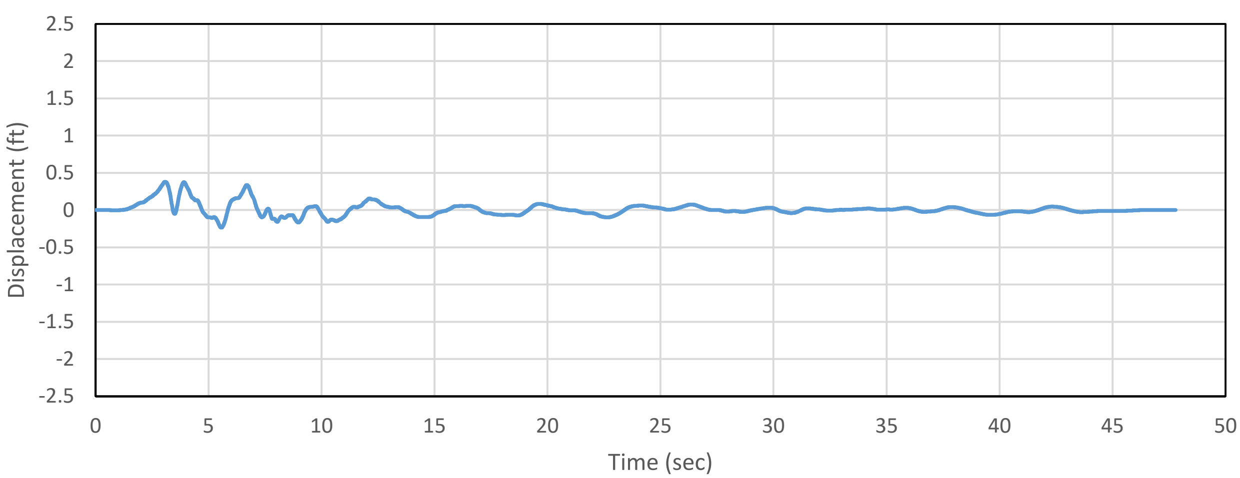
GROUP DELTA	**Scaled Time Histories - RSN 1004 (FN)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-16a**

RSN 1004

Station: LA - Sepulveda VA Hospital, Earthquake: Northridge-01



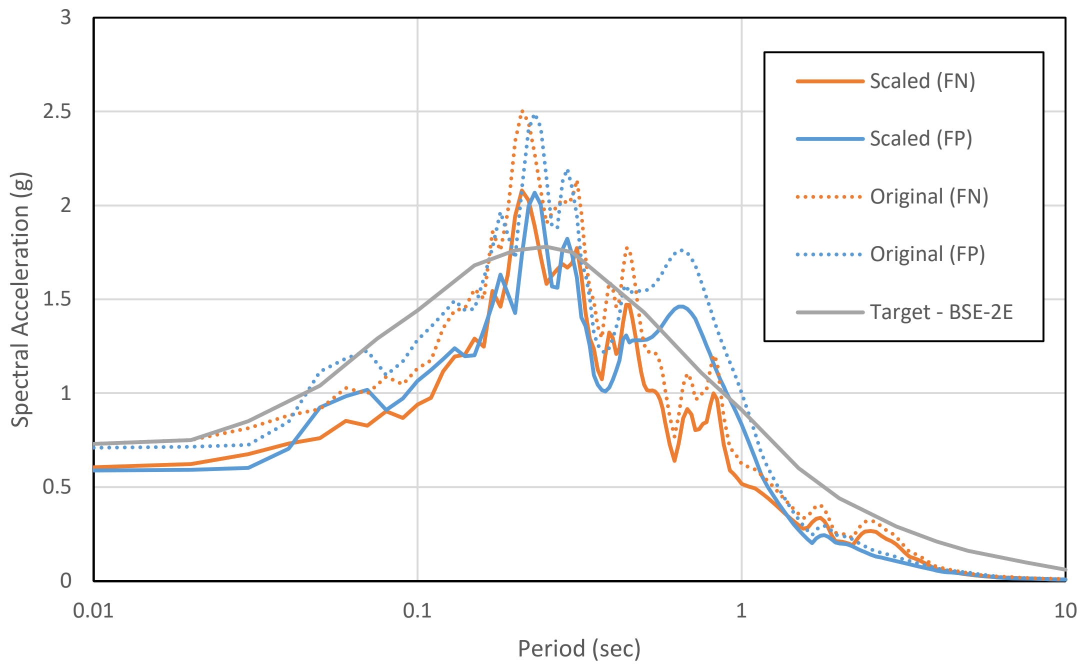


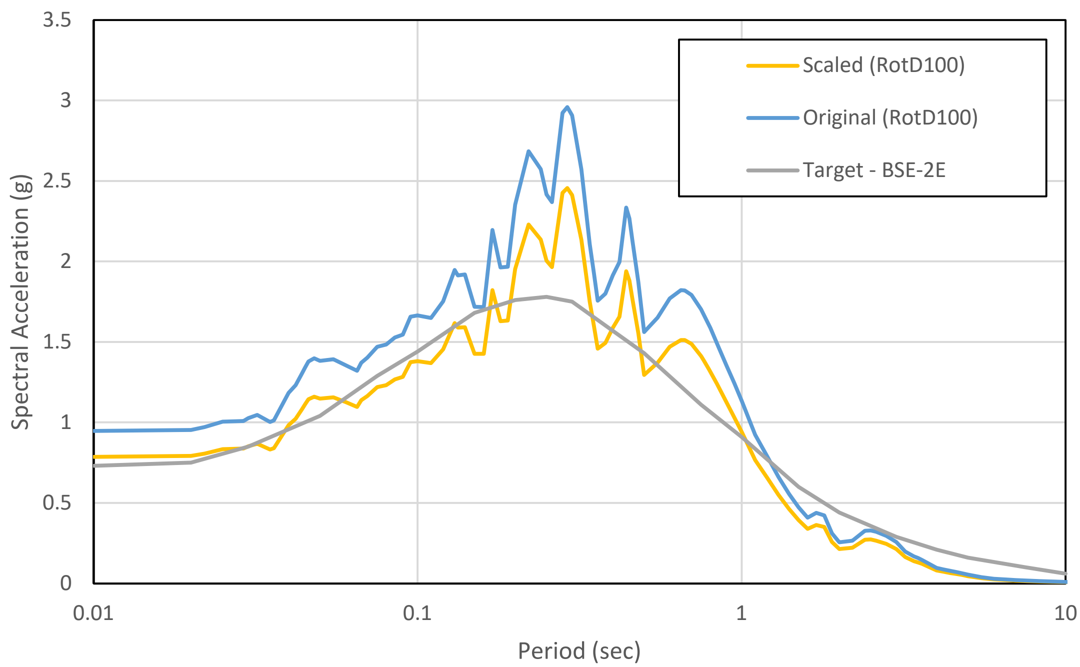


GROUP DELTA	**Scaled Time Histories - RSN 1004 (FP)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-16b**

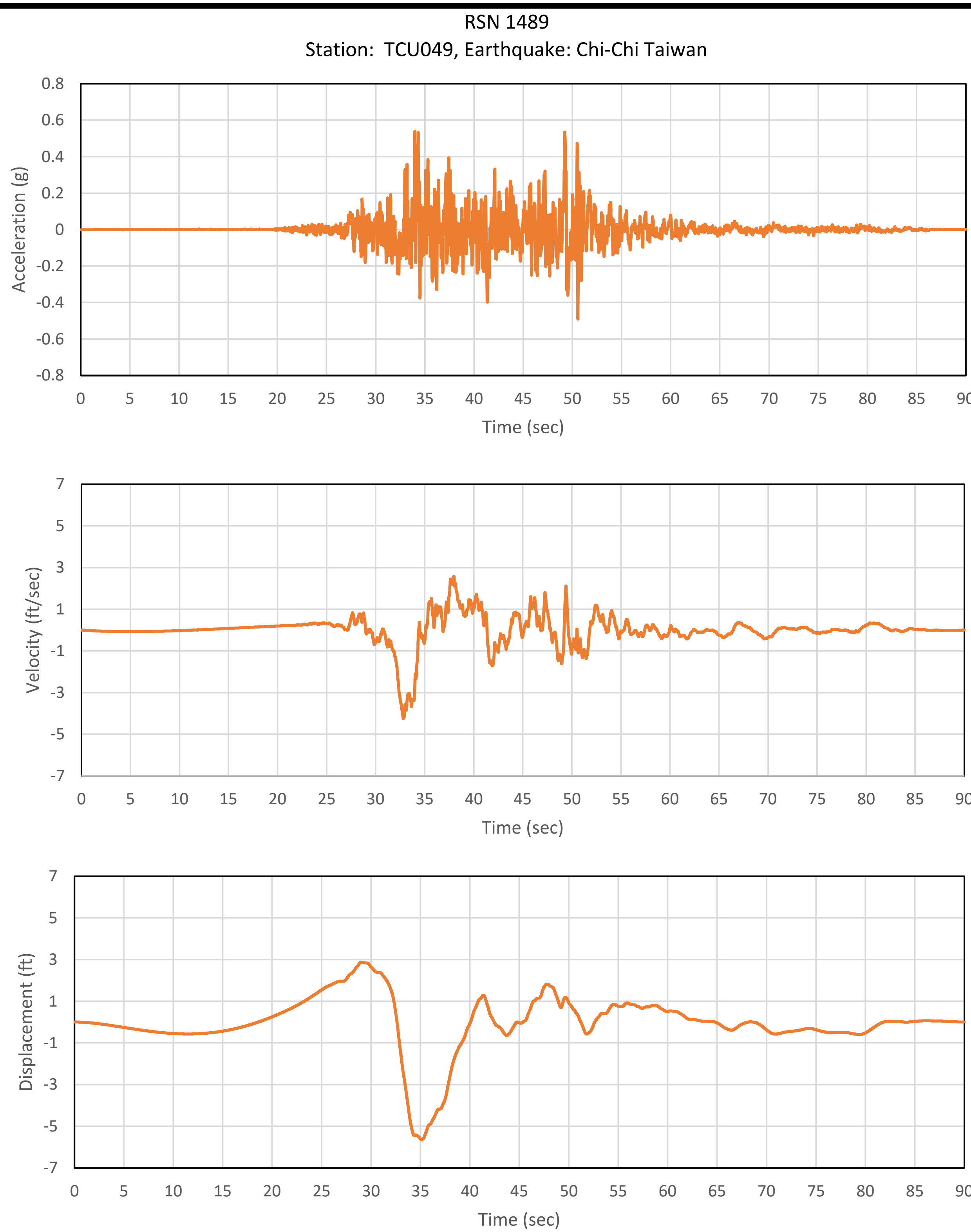
Response Spectra (5% damping)

Station: LA - Sepulveda VA Hospital, Earthquake: Northridge-01

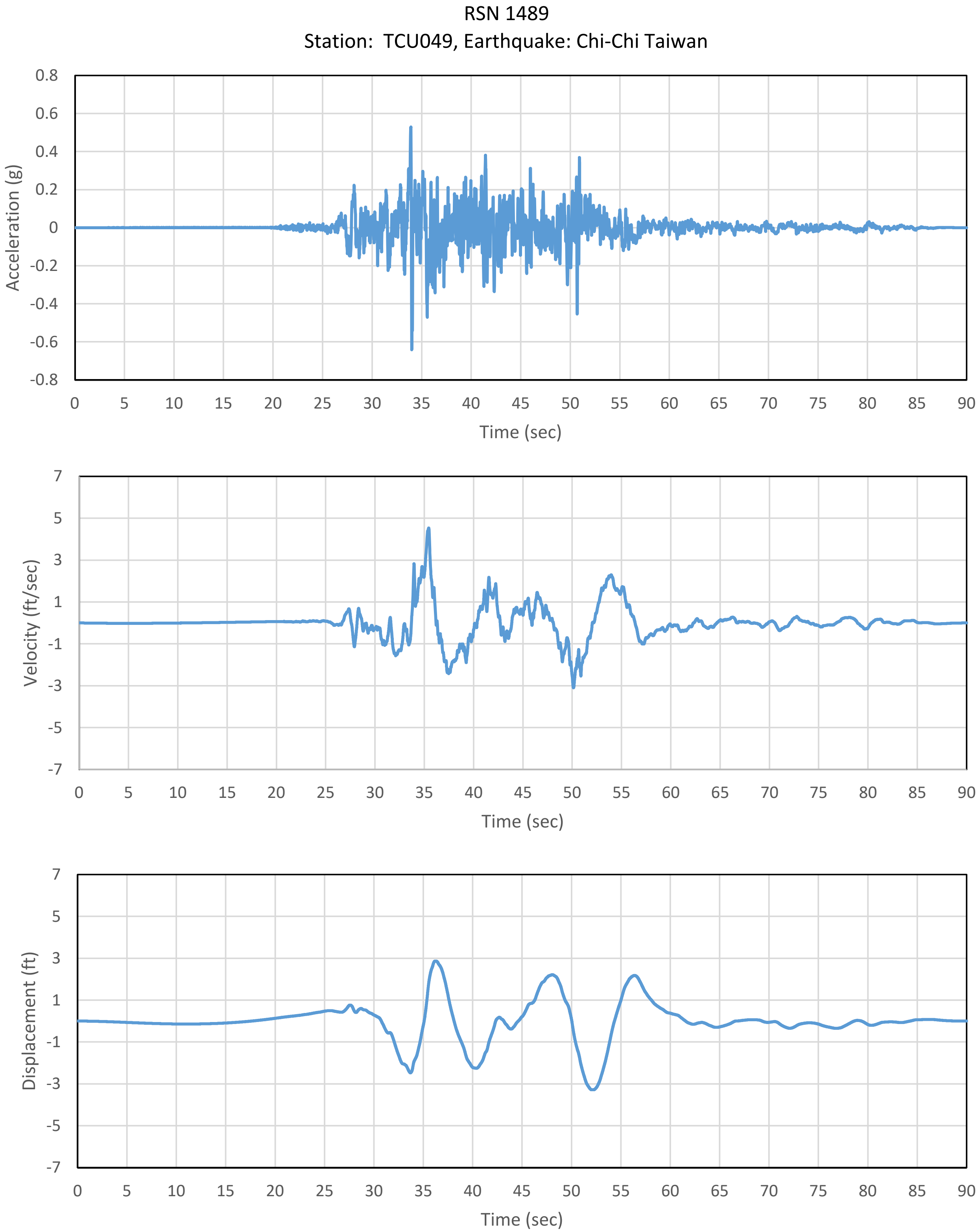




GROUP DELTA	Response Spectra - RSN 1004		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-16c**



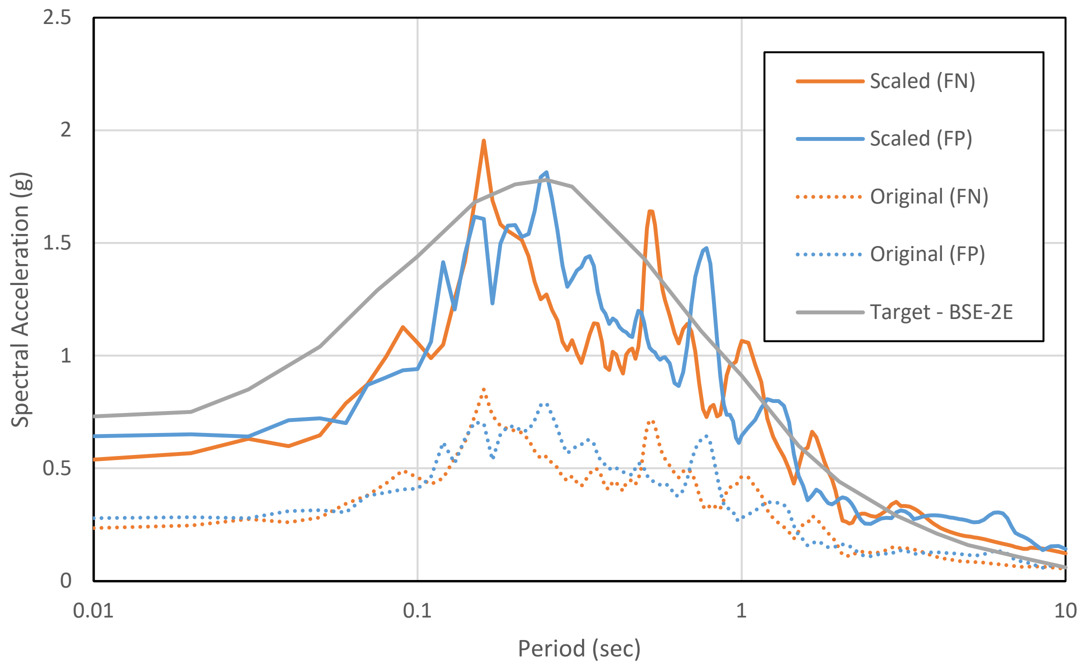
GROUP DELTA	Scaled Time Histories - RSN 1489 (FN)		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-17a**

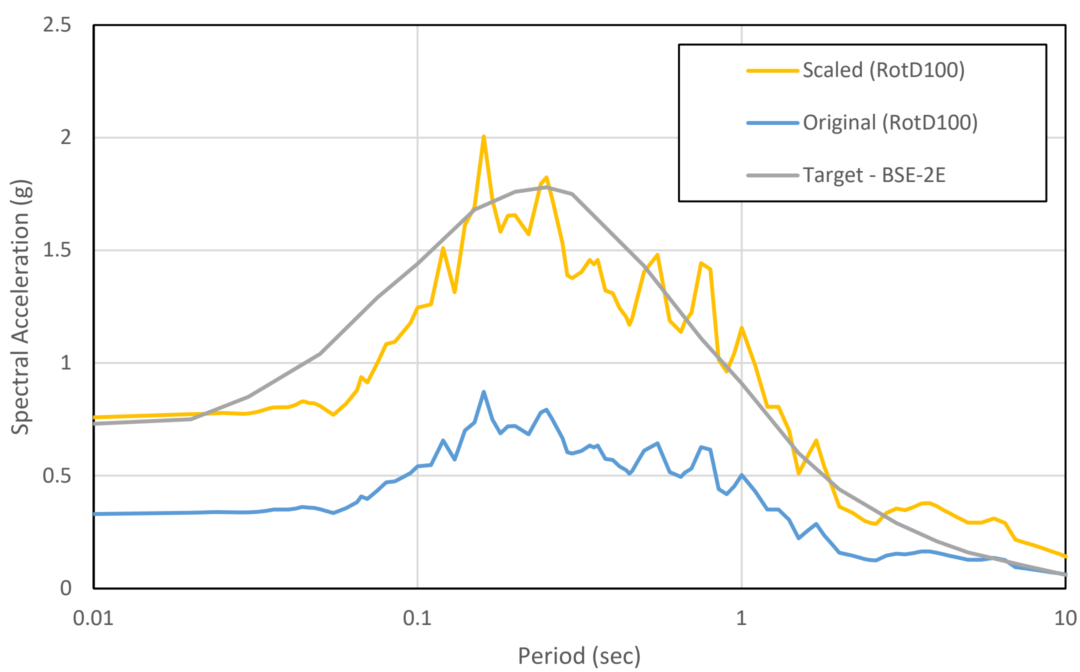


GROUP DELTA	**Scaled Time Histories - RSN 1489 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-17b**

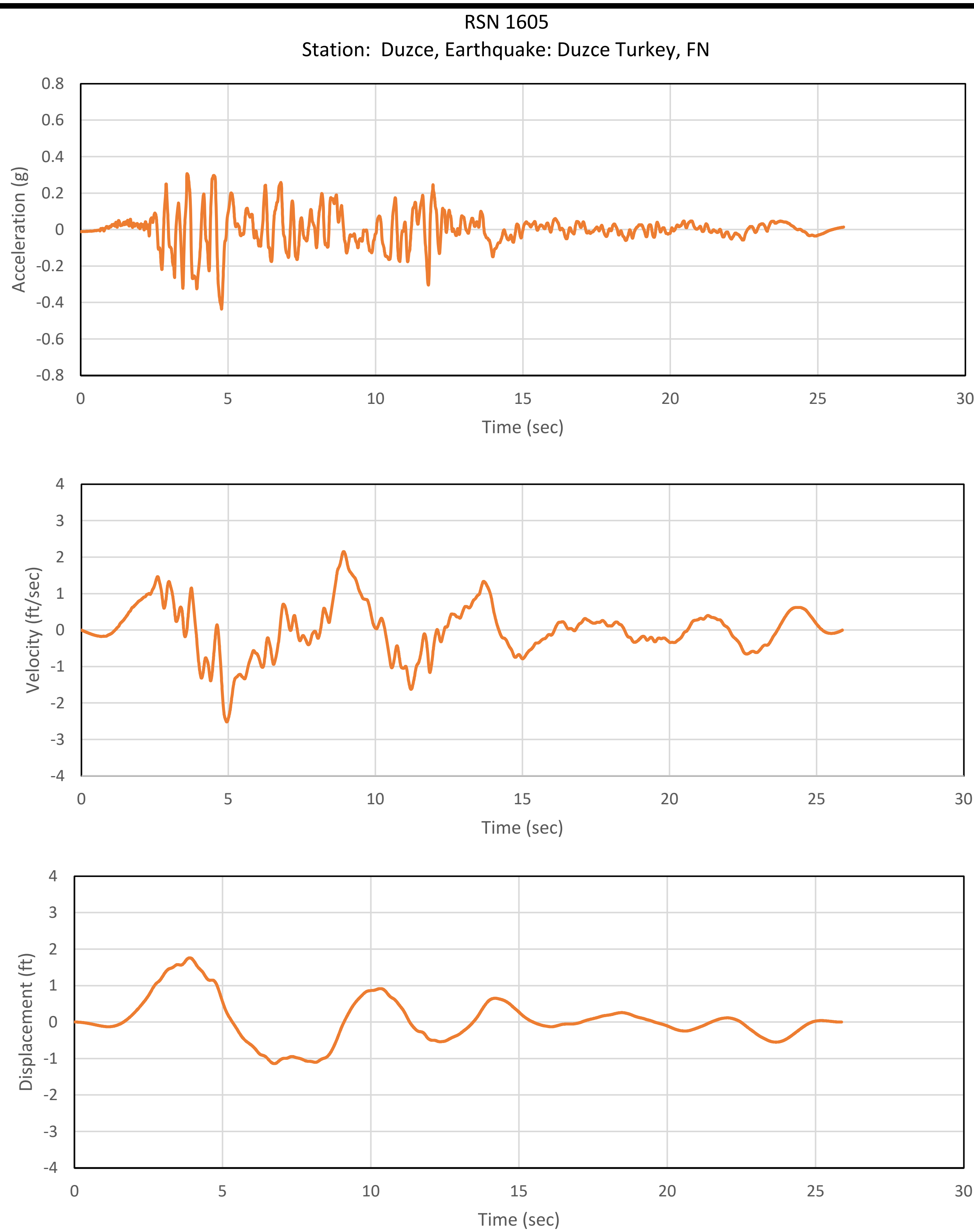
Response Spectra (5% damping)

Station: TCU049, Earthquake: Chi-Chi Taiwan





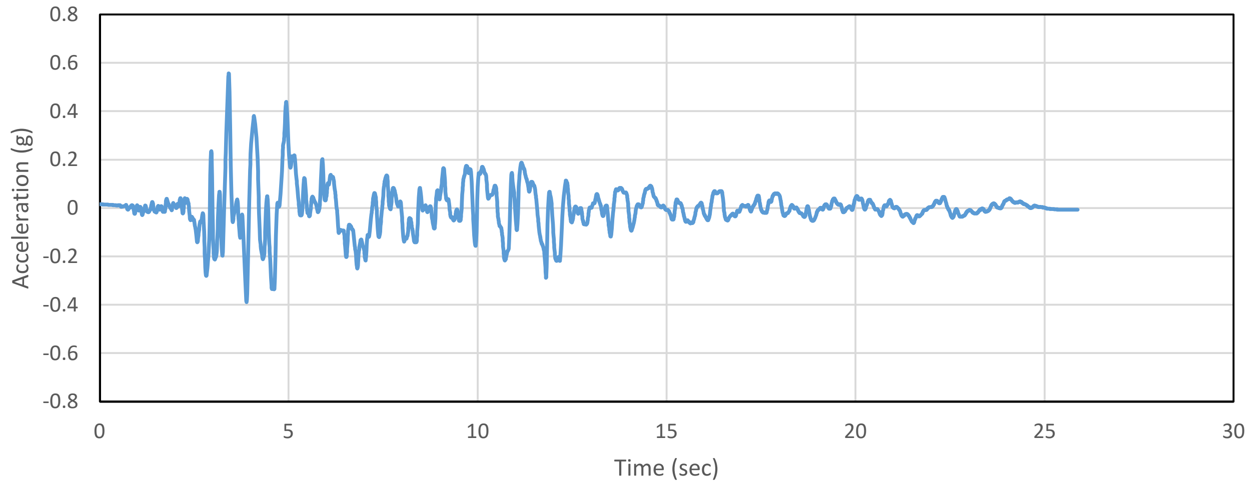
GROUP DELTA	**Response Spectra - RSN 1489**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-17c**

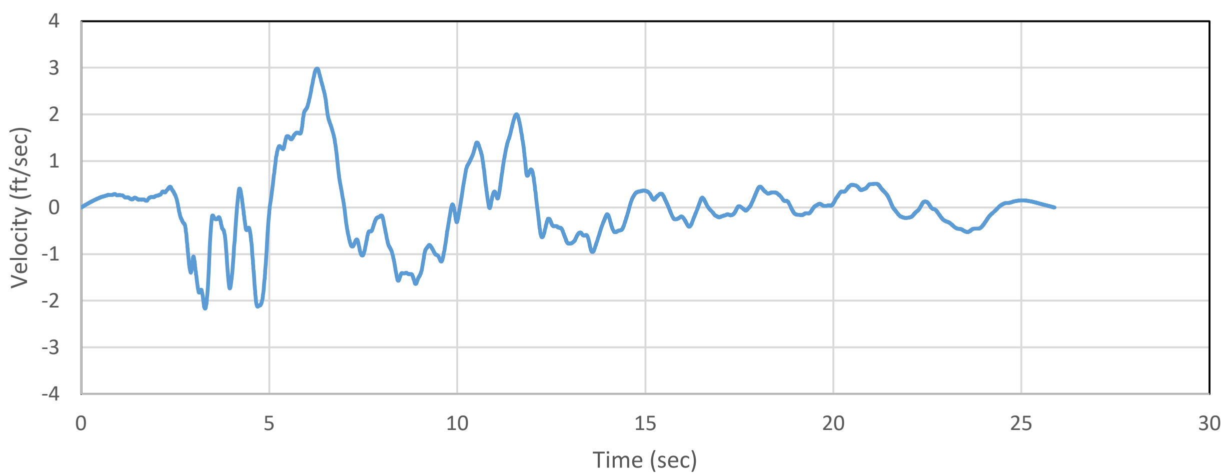


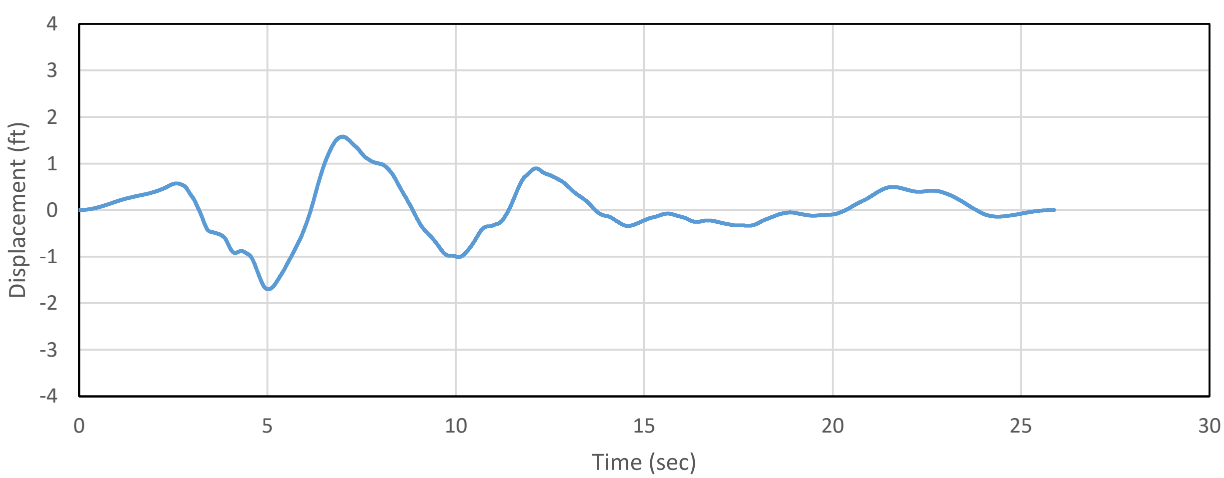
GROUP DELTA	**Scaled Time Histories - RSN 1605 (FN)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-18a**

RSN 1605

Station: Duzce, Earthquake: Duzce Turkey, FN



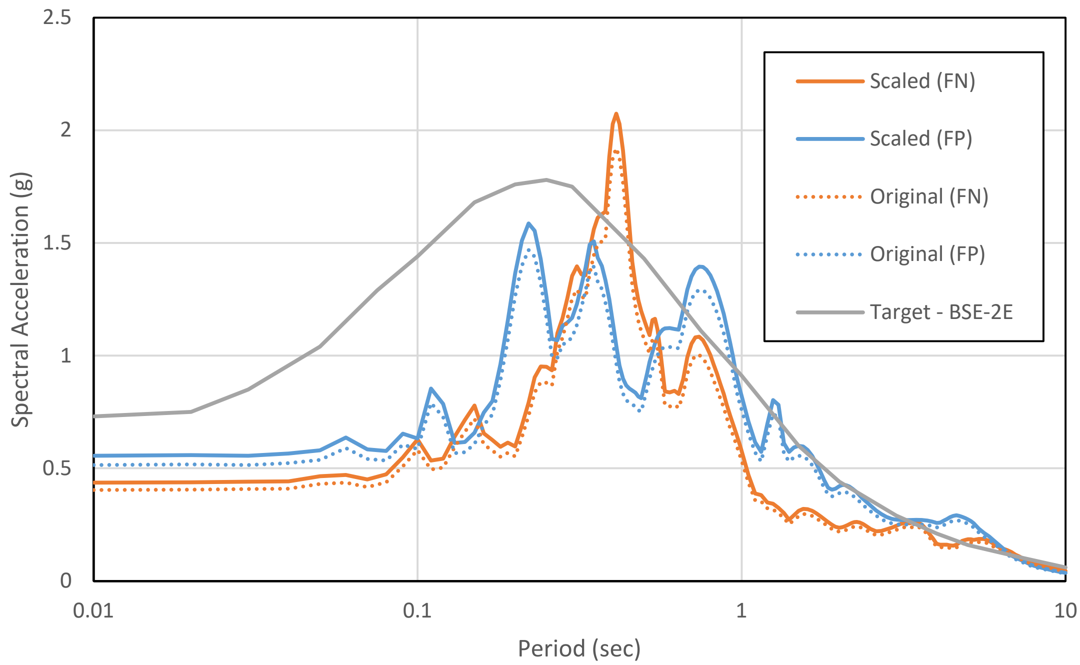


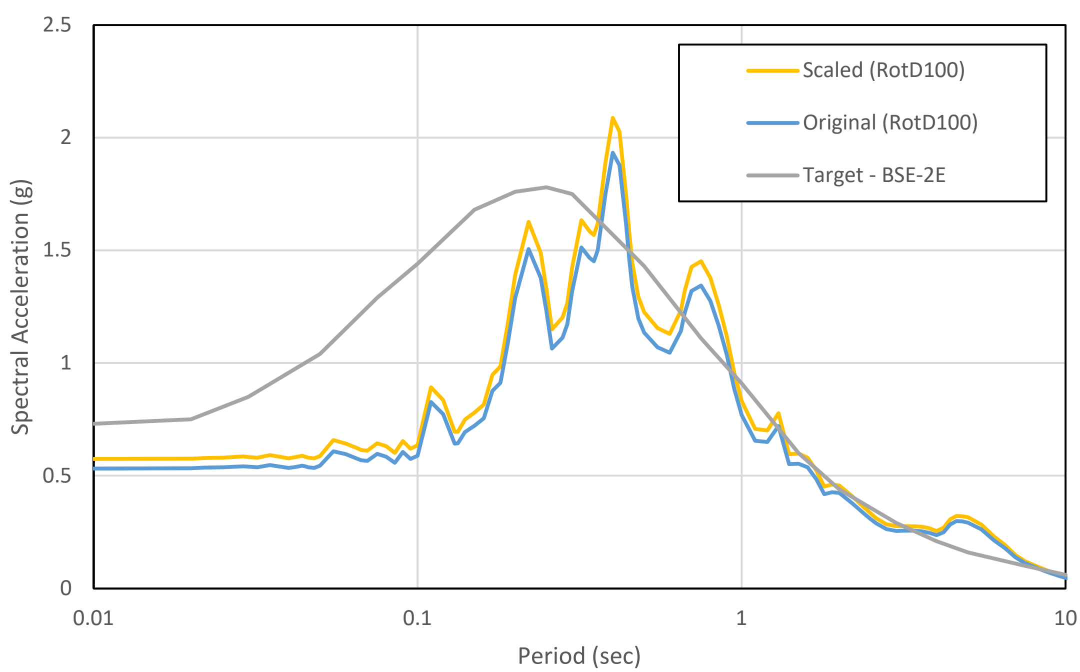


GROUP DELTA	**Scaled Time Histories - RSN 1605 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-18b**

Response Spectra (5% damping)

Station: Duzce, Earthquake: Duzce Turkey, FN

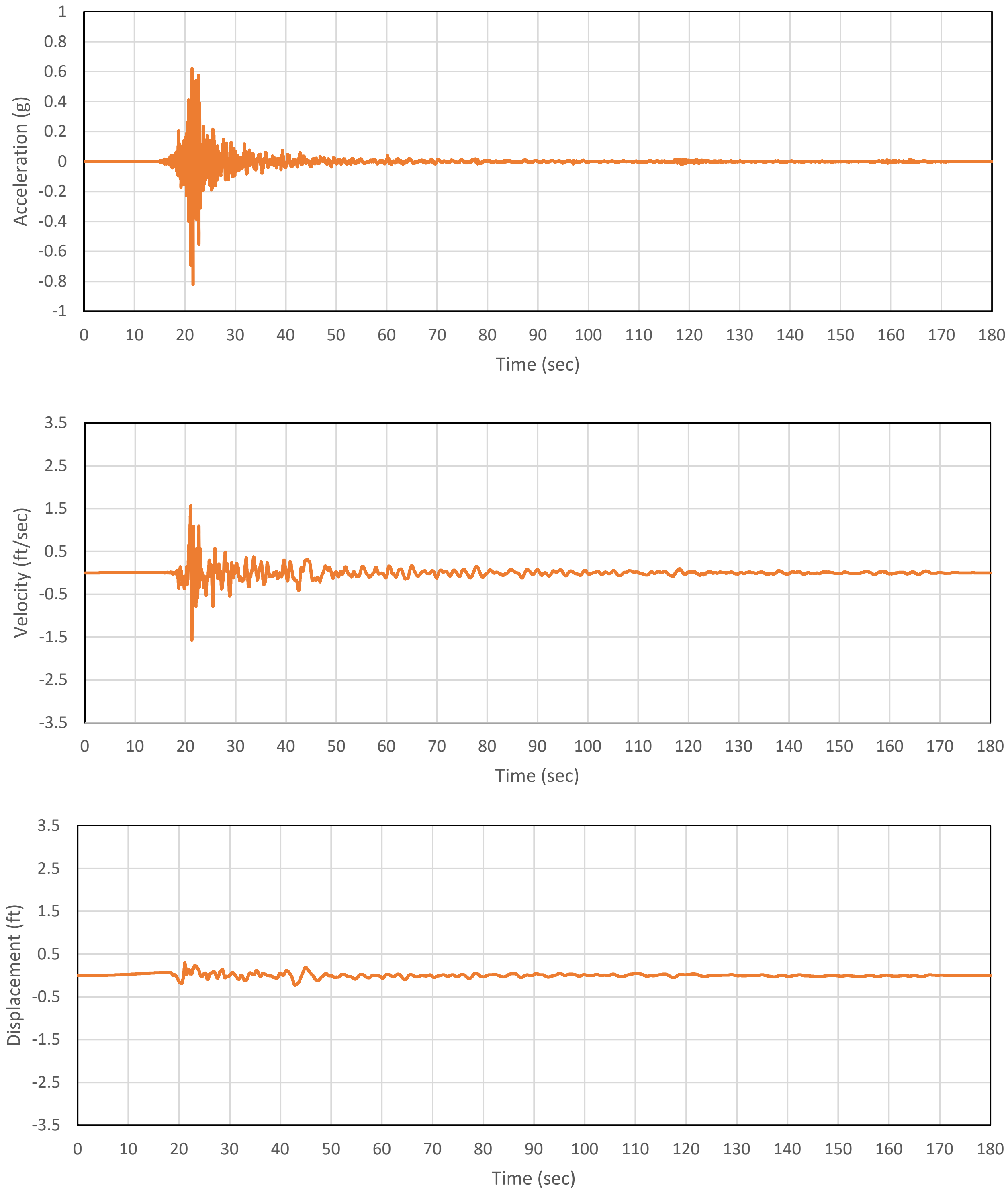




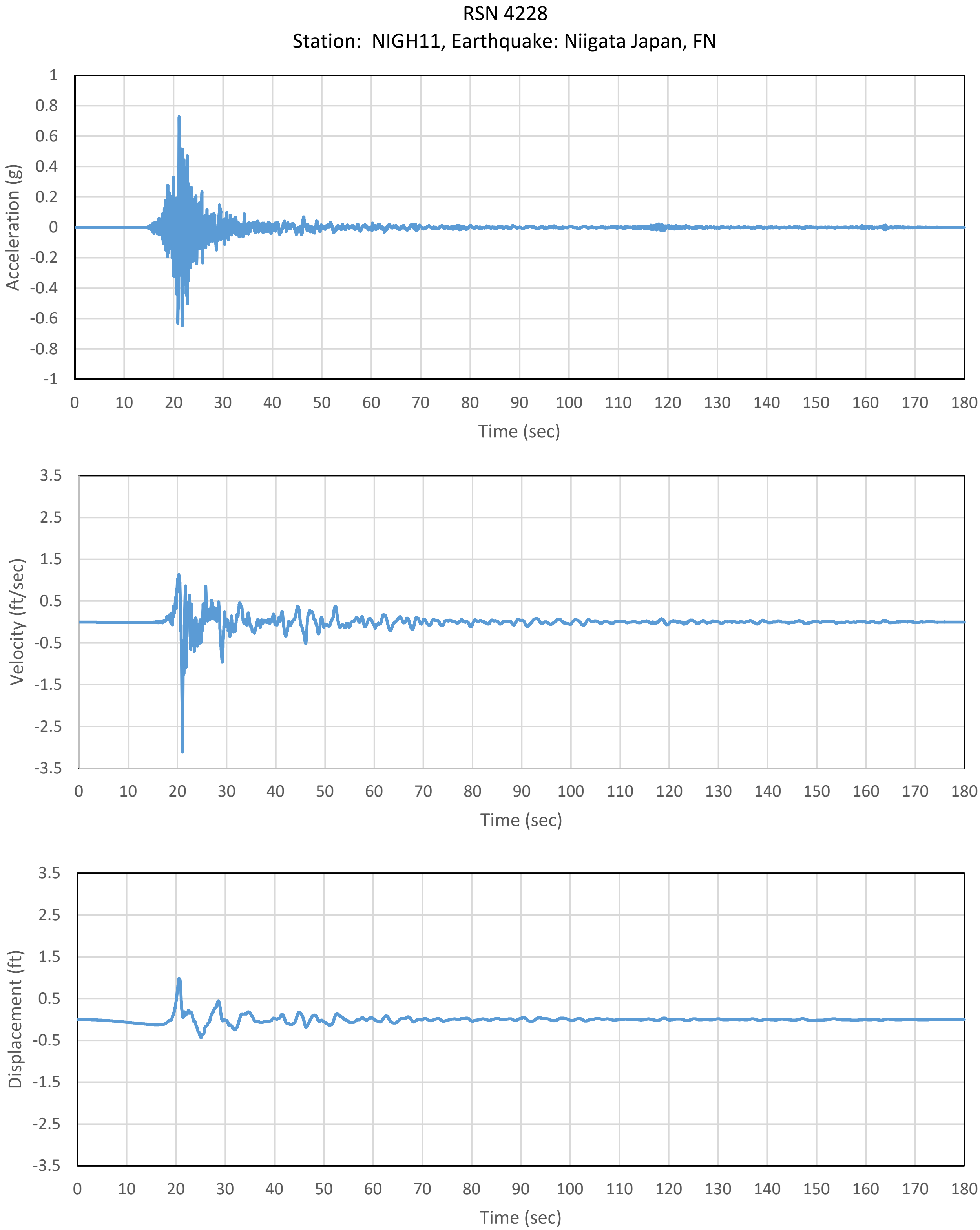
GROUP DELTA	**Response Spectra - RSN 1605**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-18c**

RSN 4228

Station: NIGH11, Earthquake: Niigata Japan, FN



GROUP DELTA	**Scaled Time Histories - RSN 4228 (FN)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-19a**



GROUP DELTA

Scaled Time Histories - RSN 4228 (FP)

Project Number: LA-1474

Date: 12/18/2020

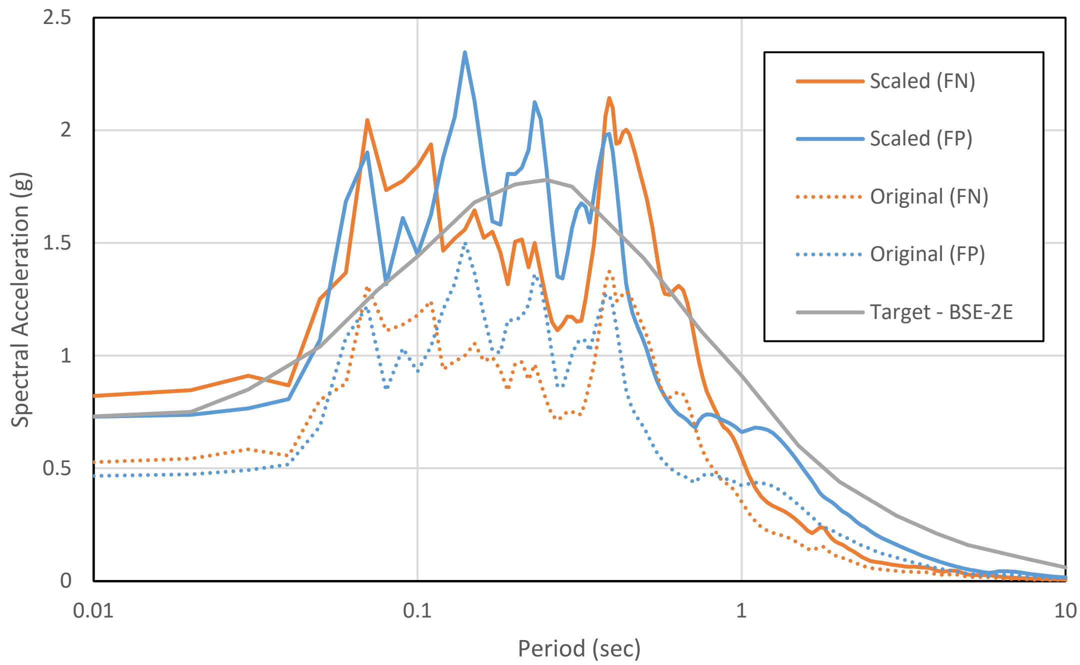
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

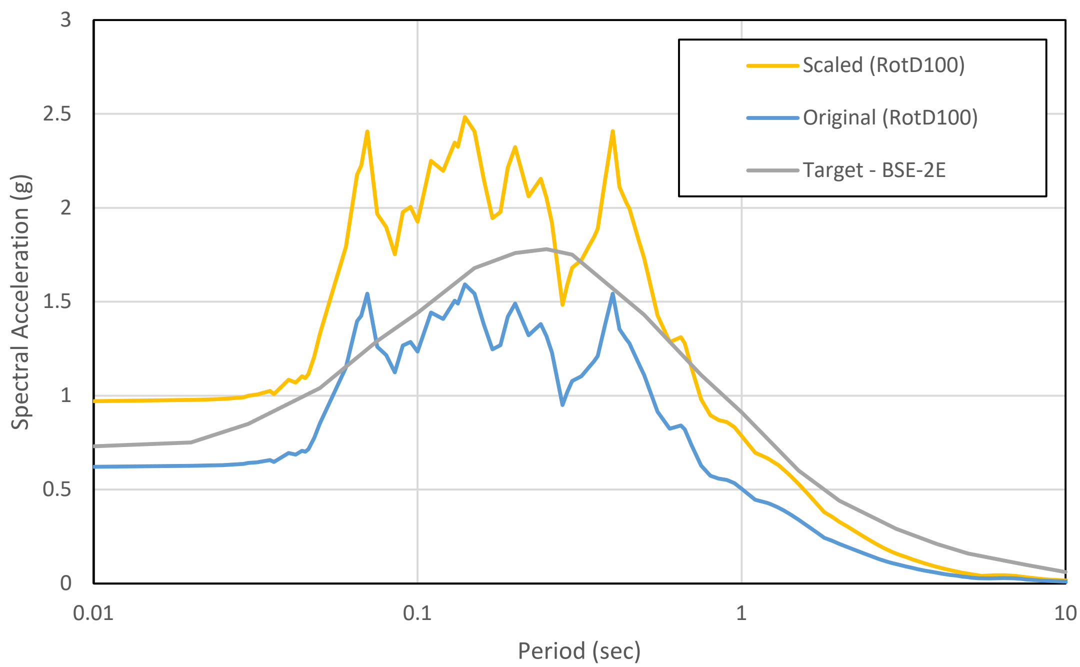
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-19b

Response Spectra (5% damping)

Station: NIGH11, Earthquake: Niigata Japan, FN

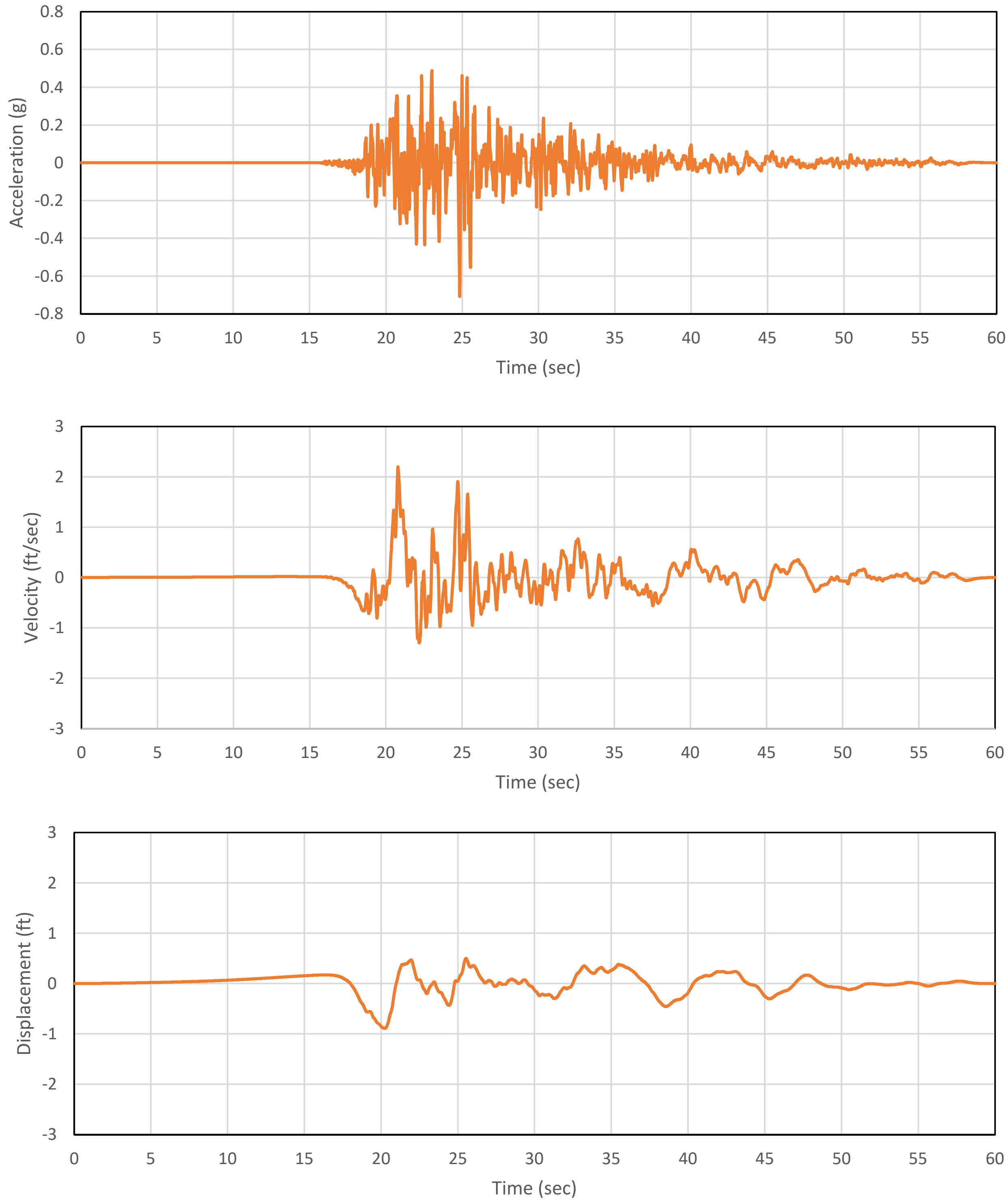




GROUP DELTA	Response Spectra - RSN 4228		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME: City of Inglewood Seismic Retrofits**	**Figure C-19c**

RSN 4866

Station: Kawanishi Izumozaki, Earthquake: Chuetsu-oki, FN



GROUP DELTA

Scaled Time Histories - RSN 4866 (FN)

Project Number: LA-1474

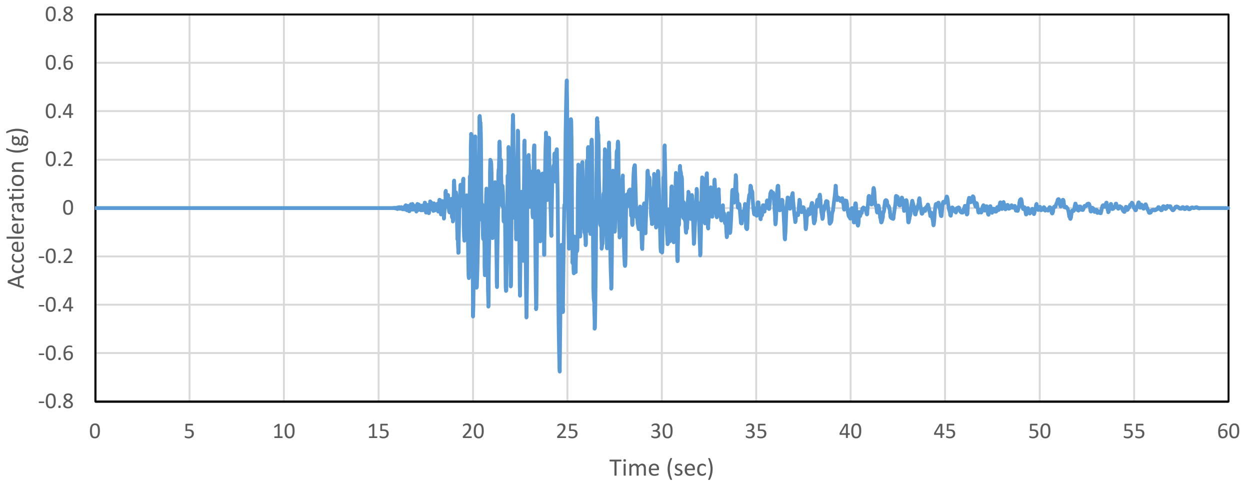
Date: 12/18/2020

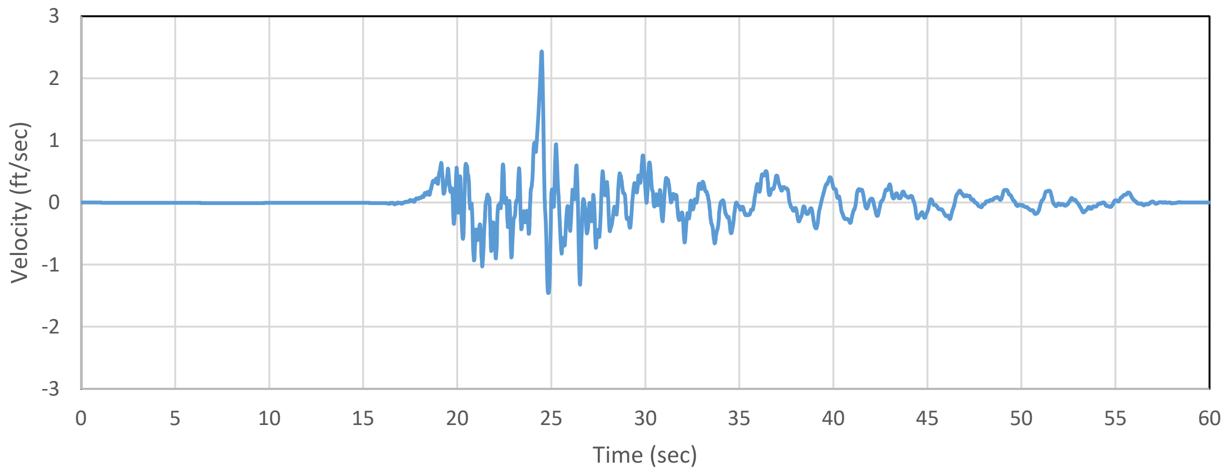
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

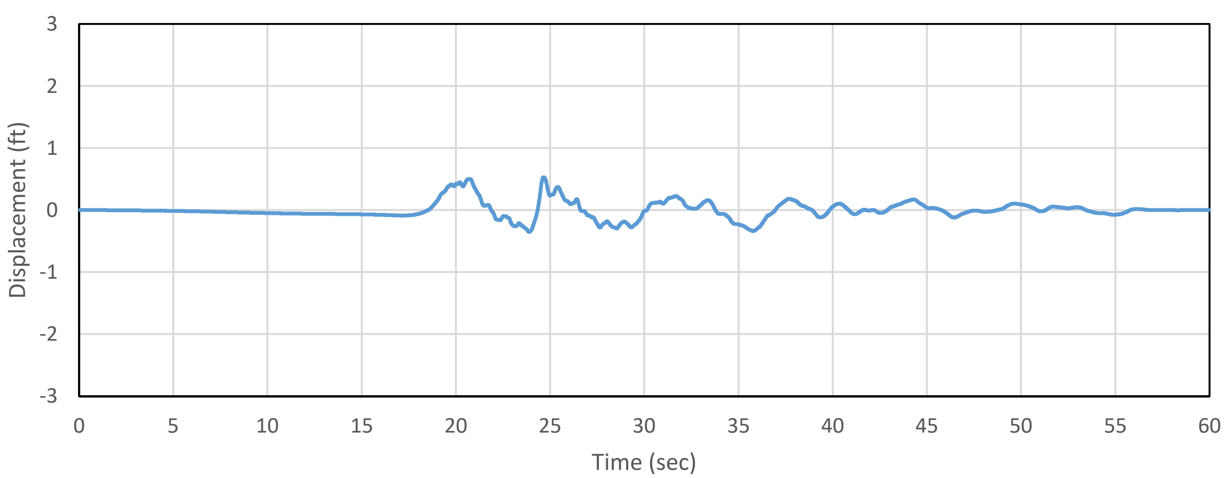
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-20a

RSN 4866
Station: Kawanishi Izumozaki, Earthquake: Chuetsu-oki, FN



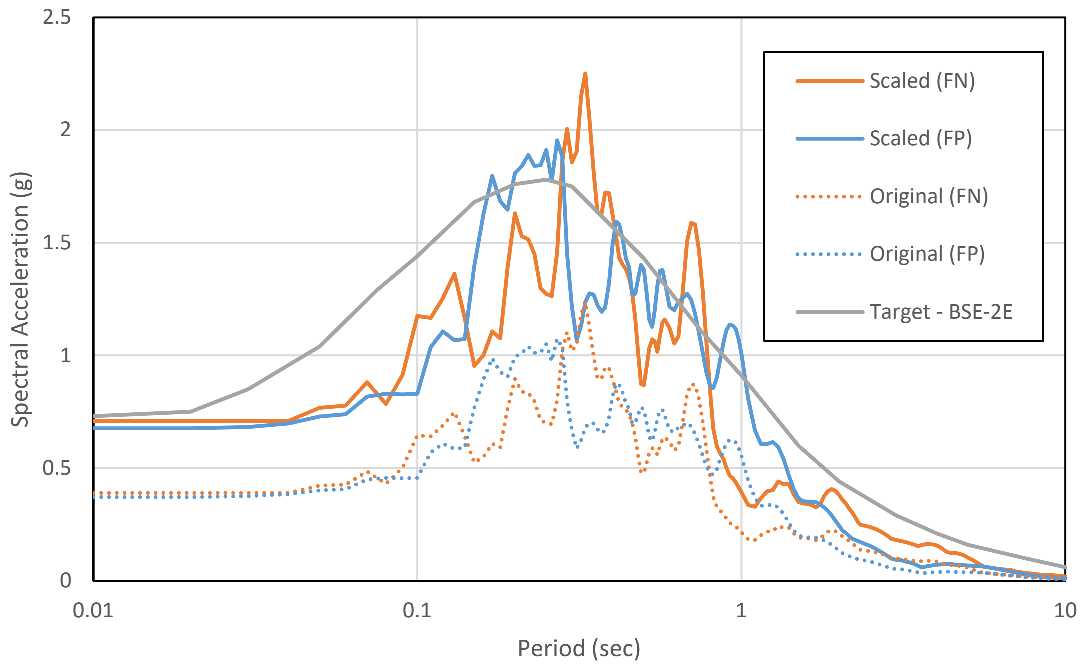


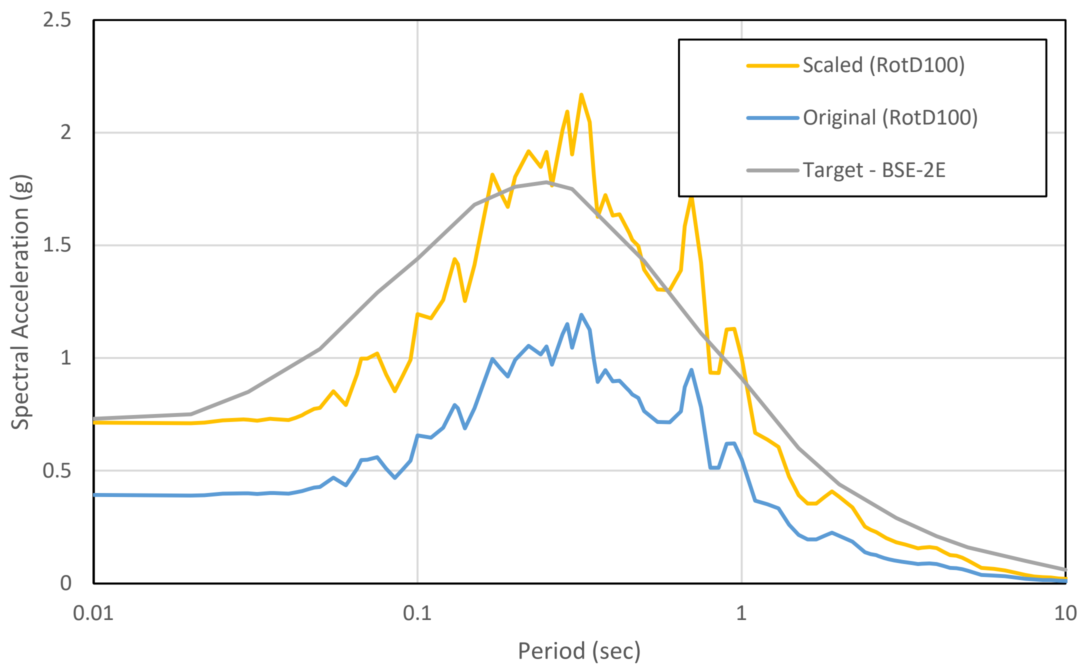


GROUP DELTA	**Scaled Time Histories - RSN 4866 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-20b**

Response Spectra (5% damping)

Station: Kawanishi Izumozaki, Earthquake: Chuetsu-oki, FN

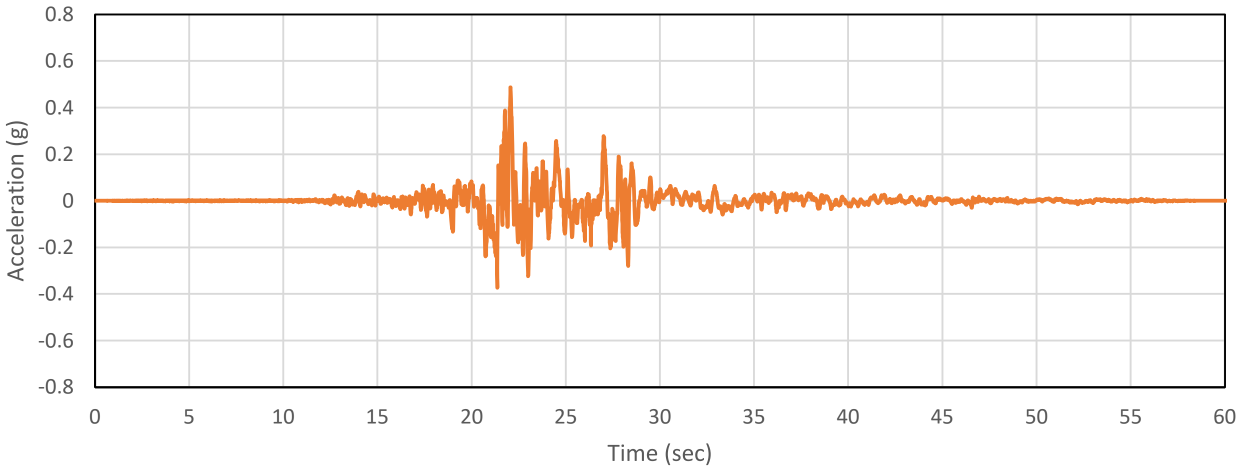


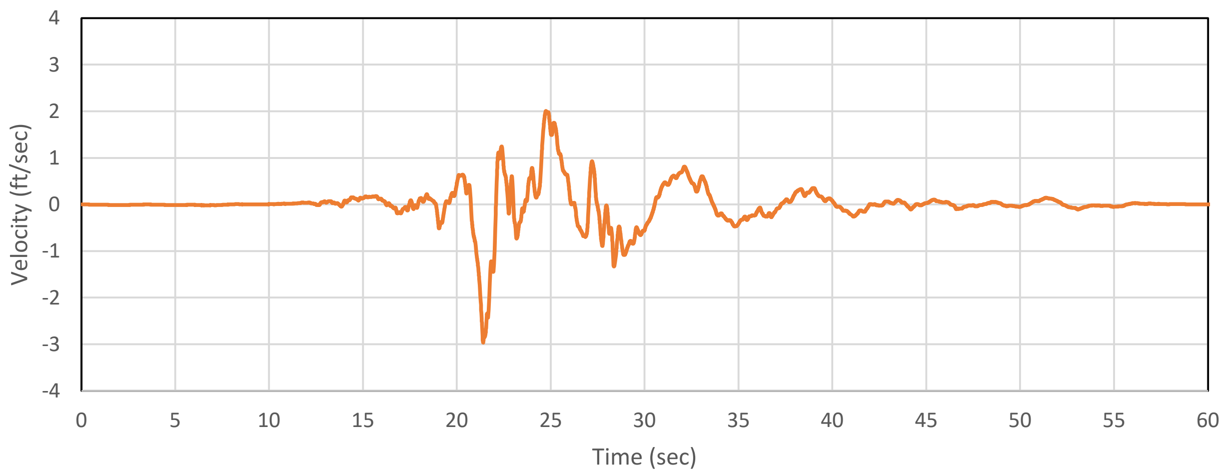


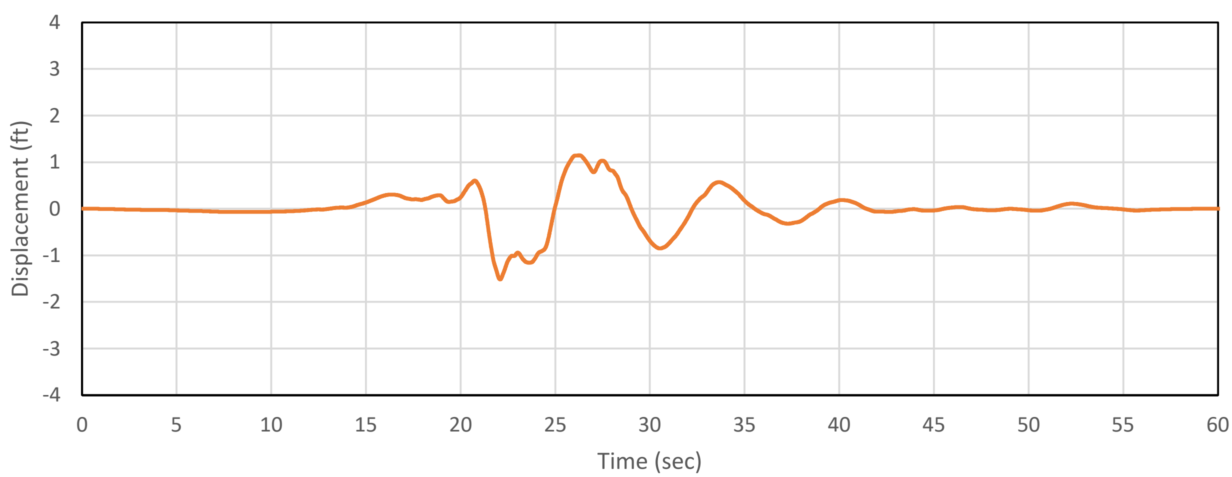
GROUP DELTA	**Response Spectra - RSN 4866**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-20c**

RSN 6911

Station: HORC, Earthquake: Darfield New Zealand, FN



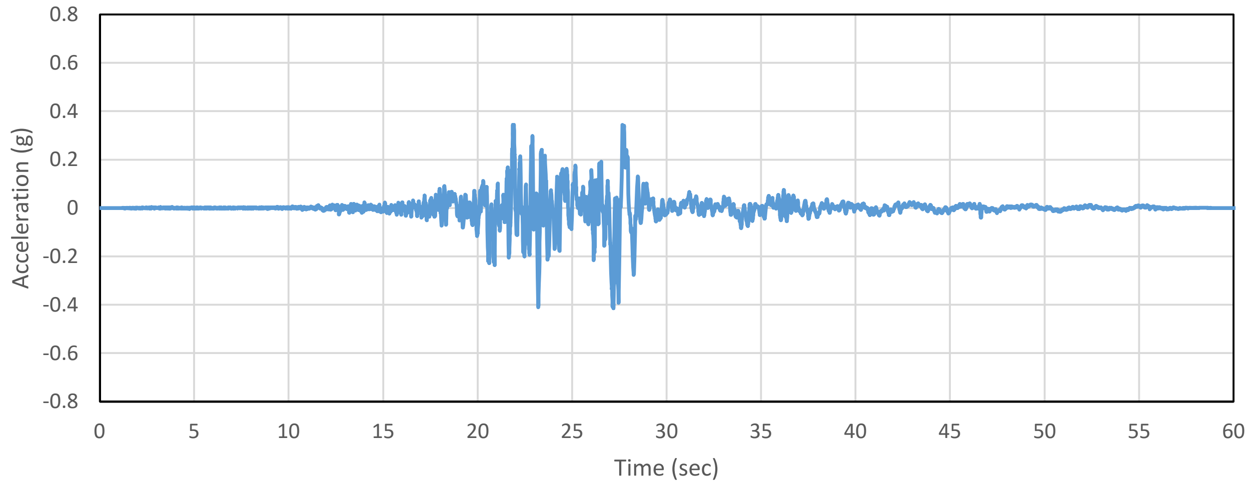


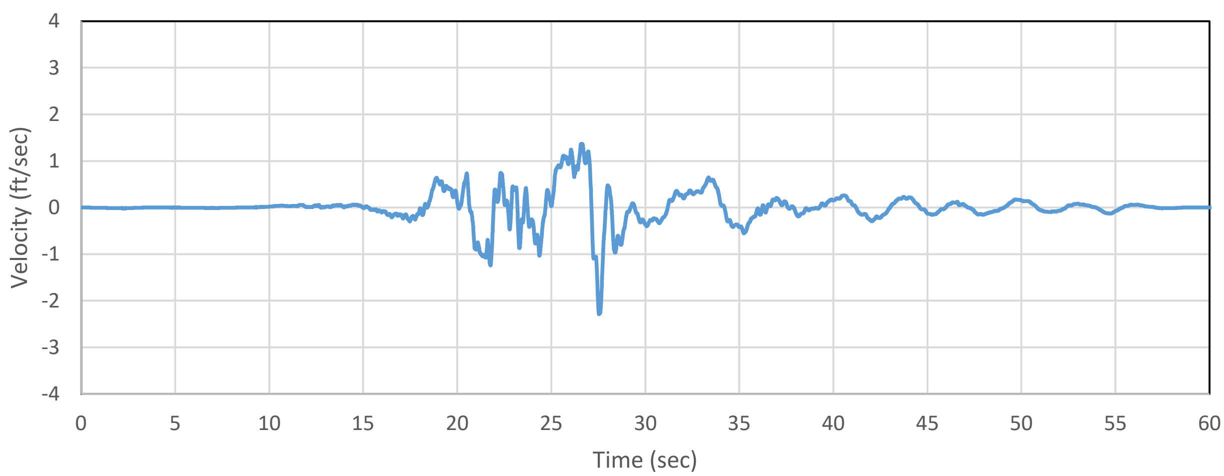


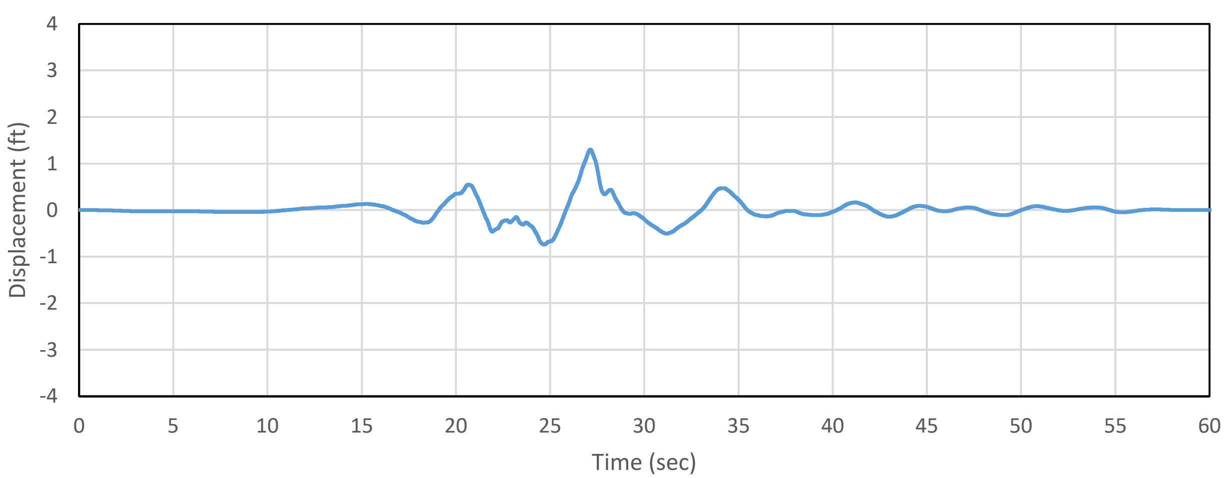
GROUP DELTA	**Scaled Time Histories - RSN 6911 (FN)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-21a**

RSN 6911

Station: HORC, Earthquake: Darfield New Zealand, FN







GROUP DELTA

Scaled Time Histories - RSN 6911 (FP)

Project Number: LA-1474

Date: 12/18/2020

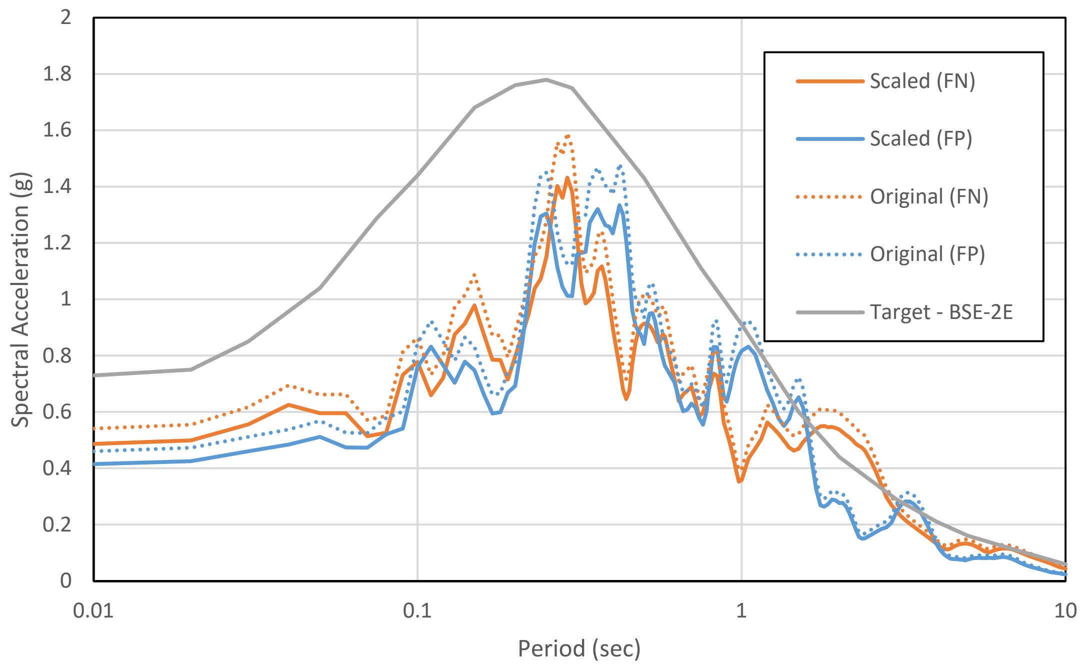
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

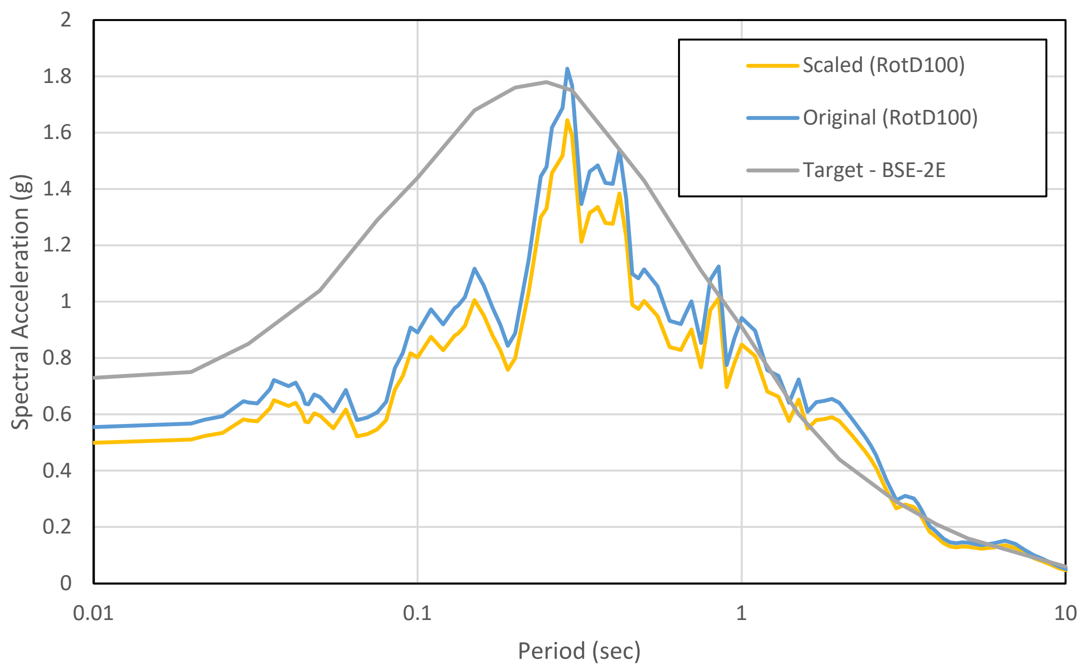
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-21b

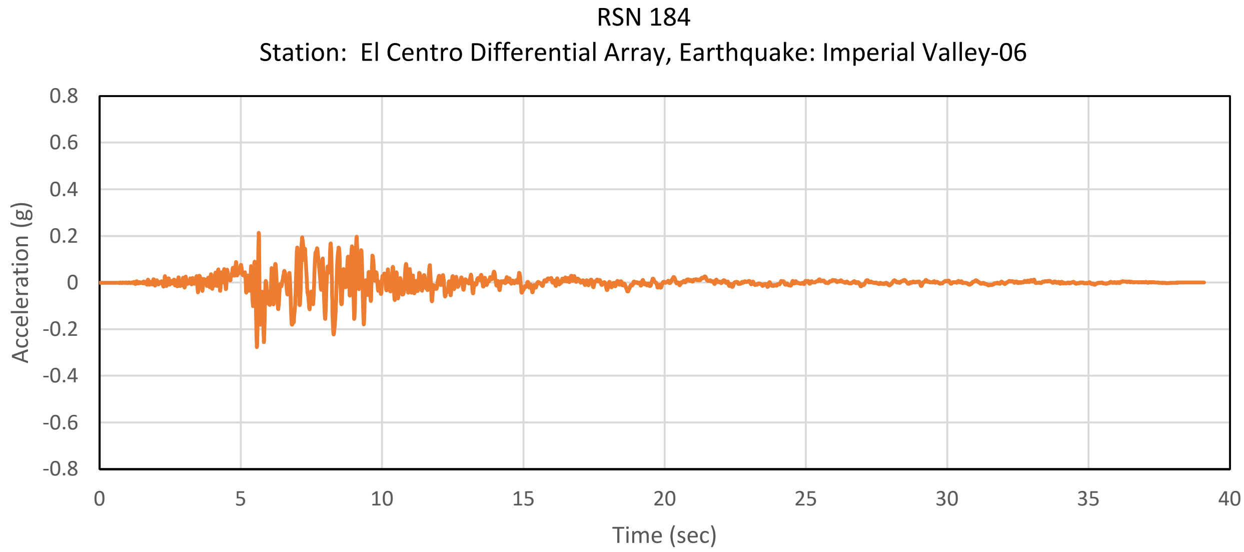
Response Spectra (5% damping)

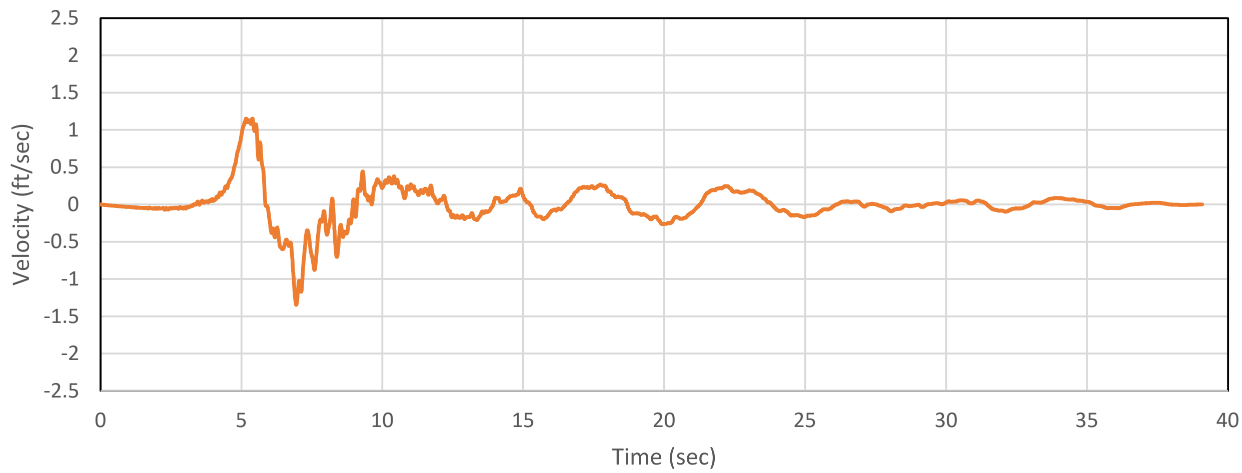
Station: HORC, Earthquake: Darfield New Zealand, FN

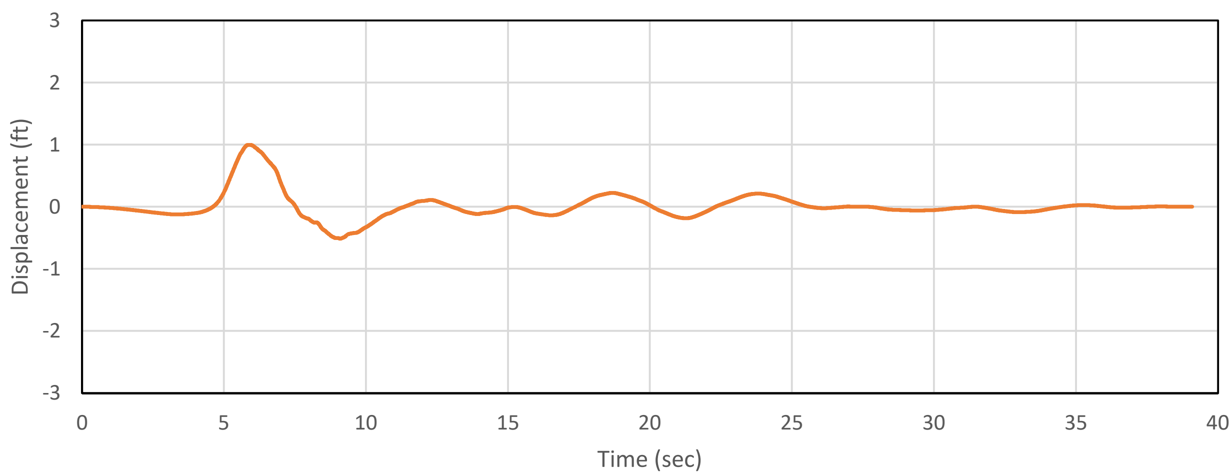




GROUP DELTA	Response Spectra - RSN 6911		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-21c**

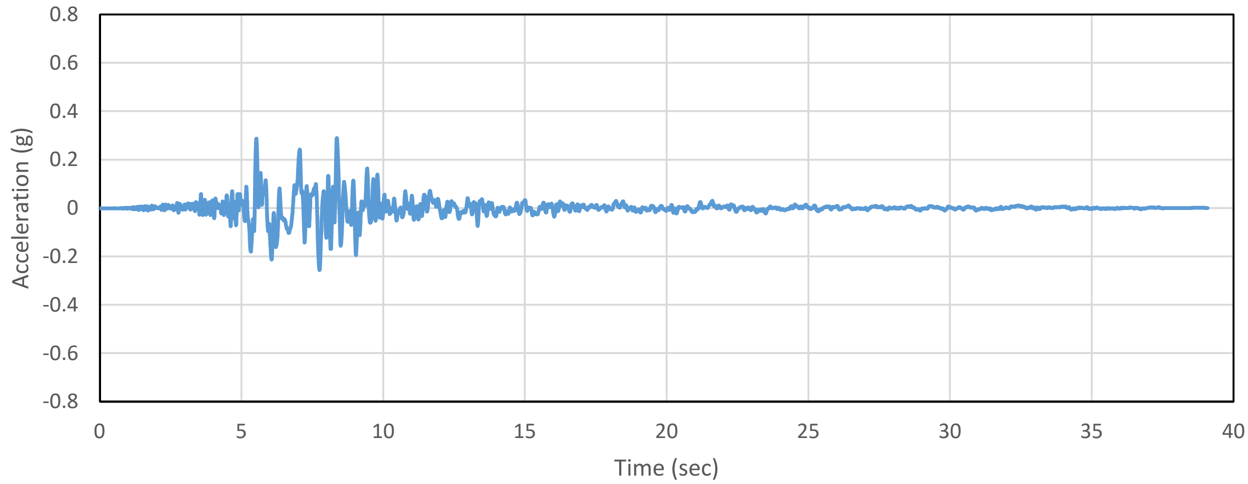


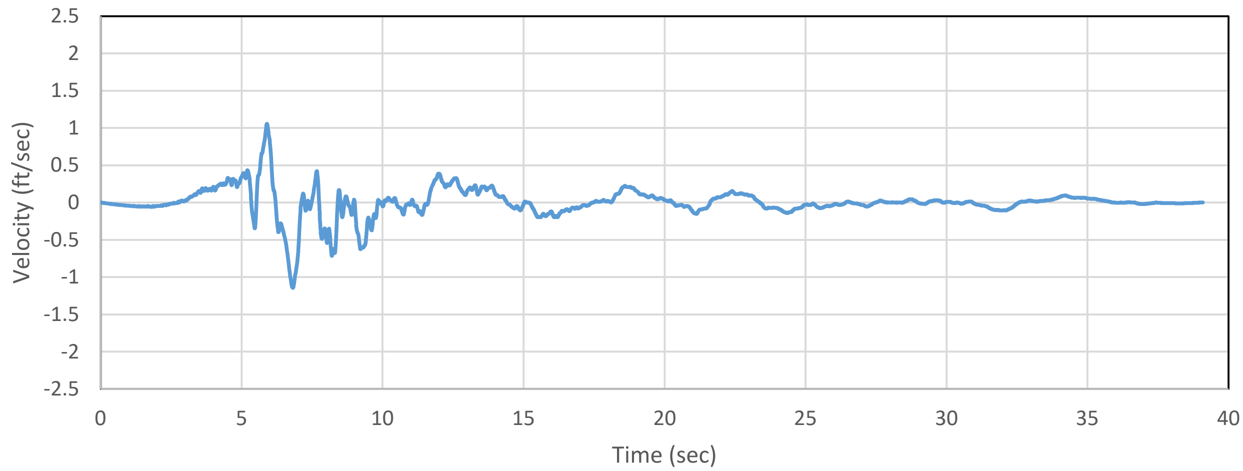


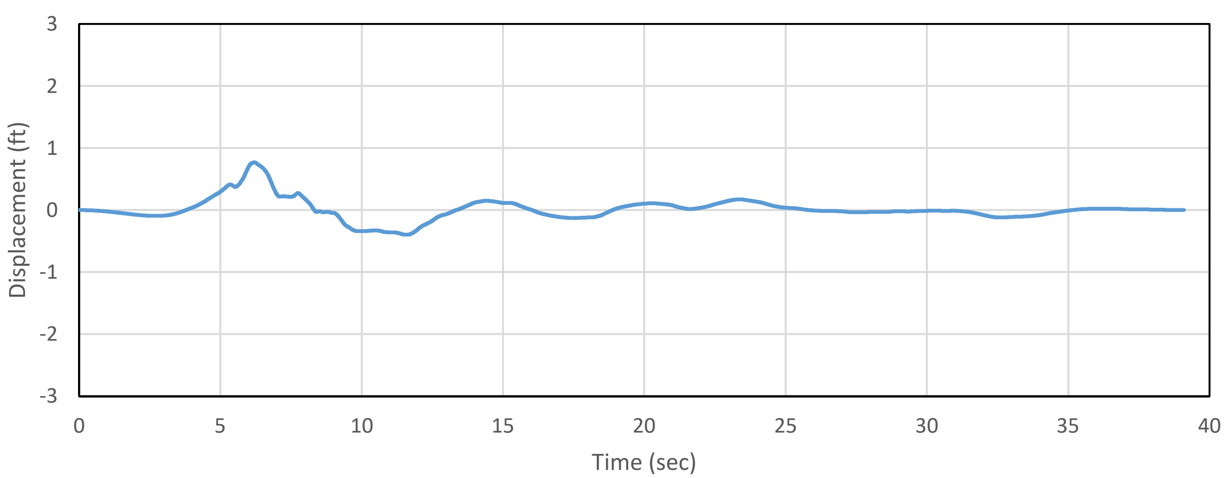


GROUP DELTA	Scaled Time Histories - RSN 184 (FN)		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-22a**

RSN 184
Station: El Centro Differential Array, Earthquake: Imperial Valley-06



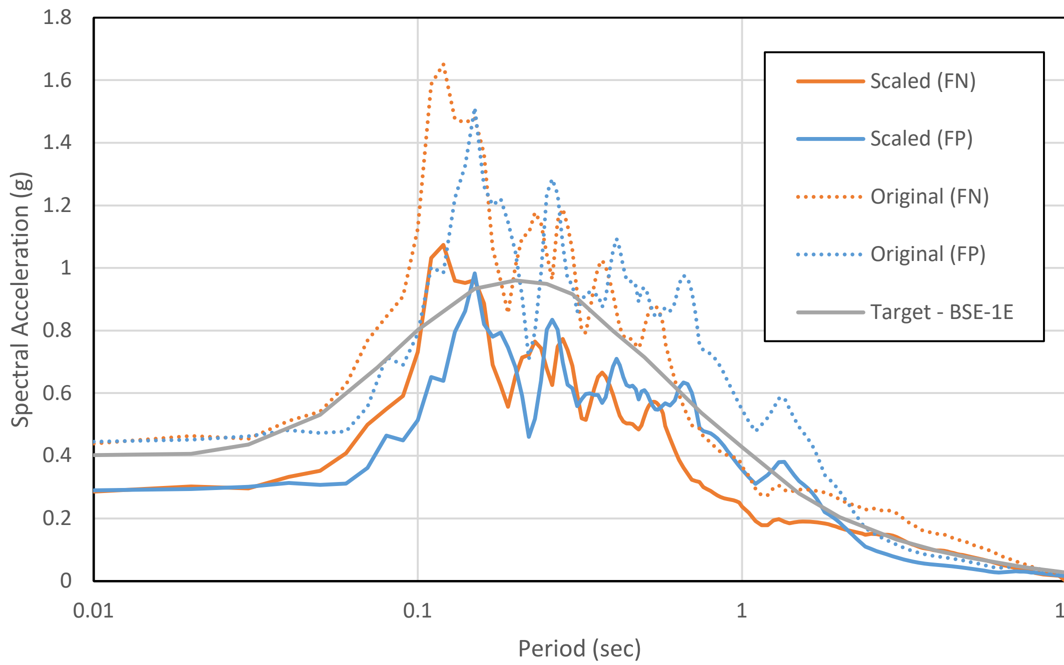


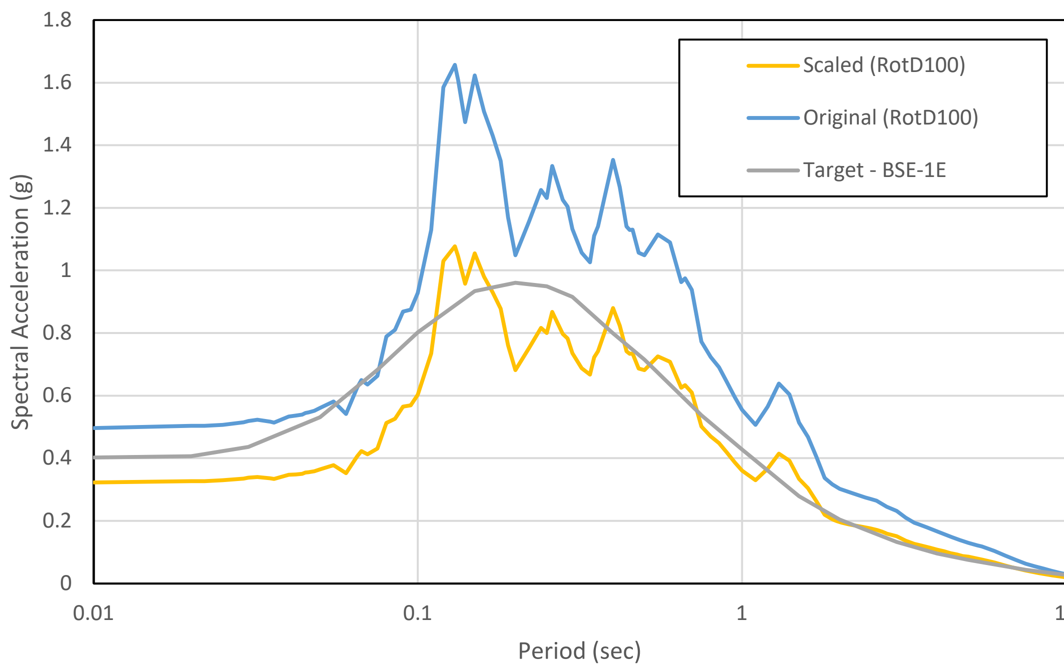


GROUP DELTA	Scaled Time Histories - RSN 184 (FP)		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-22b**

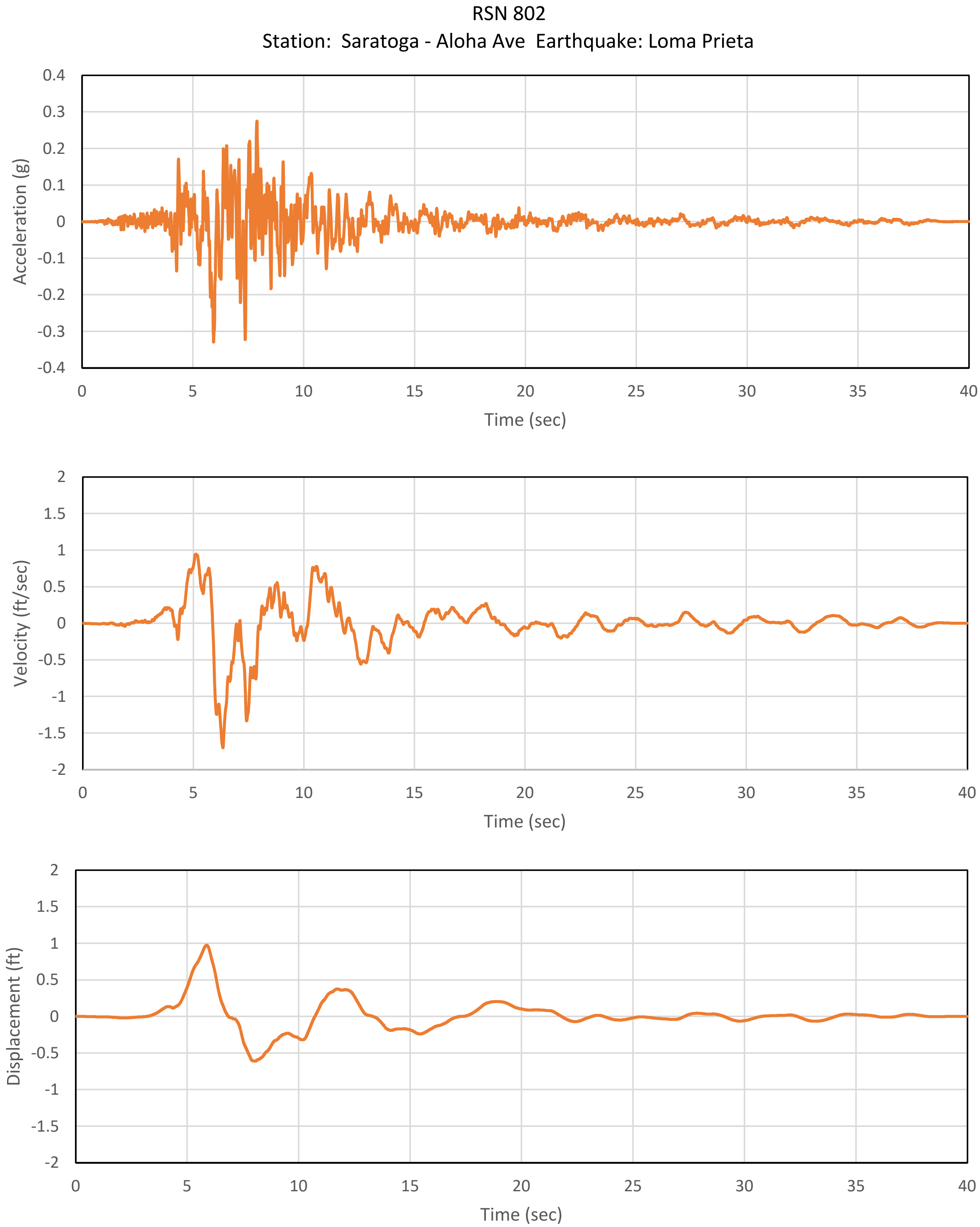
Response Spectra (5% damping)

Station: El Centro Differential Array, Earthquake: Imperial Valley-06





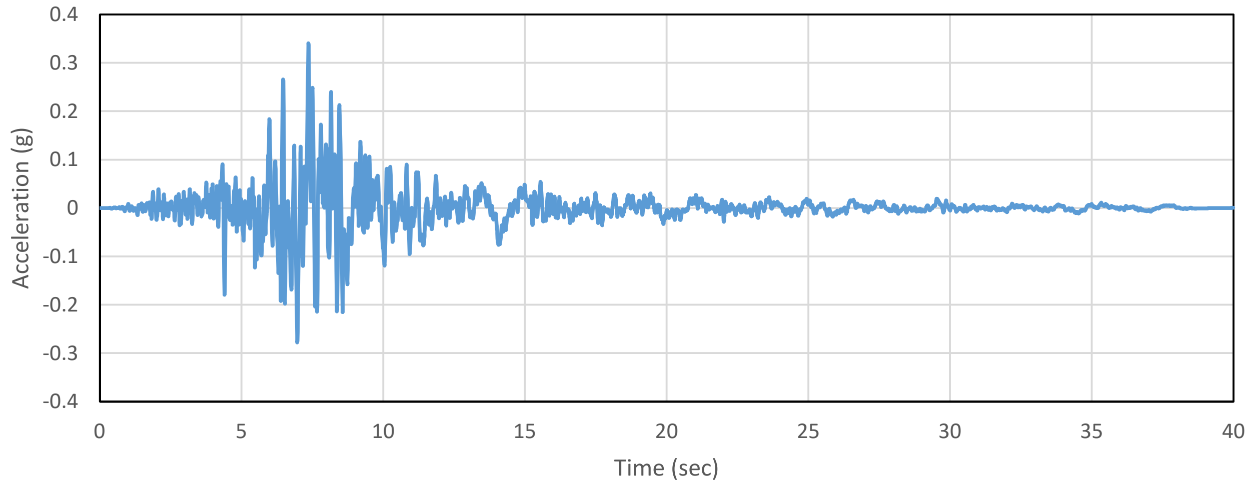
GROUP DELTA	Response Spectra - RSN 184		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-22c**

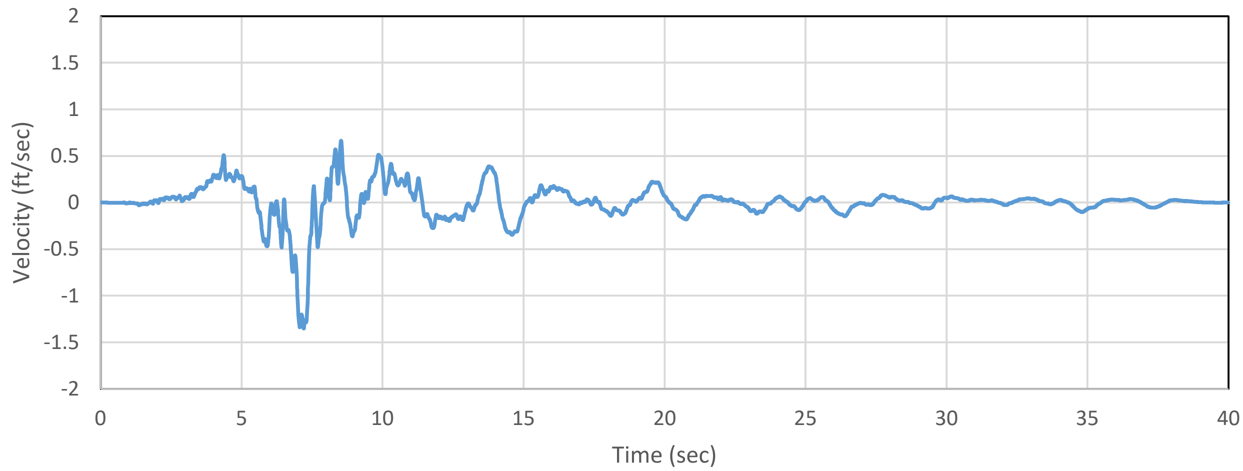


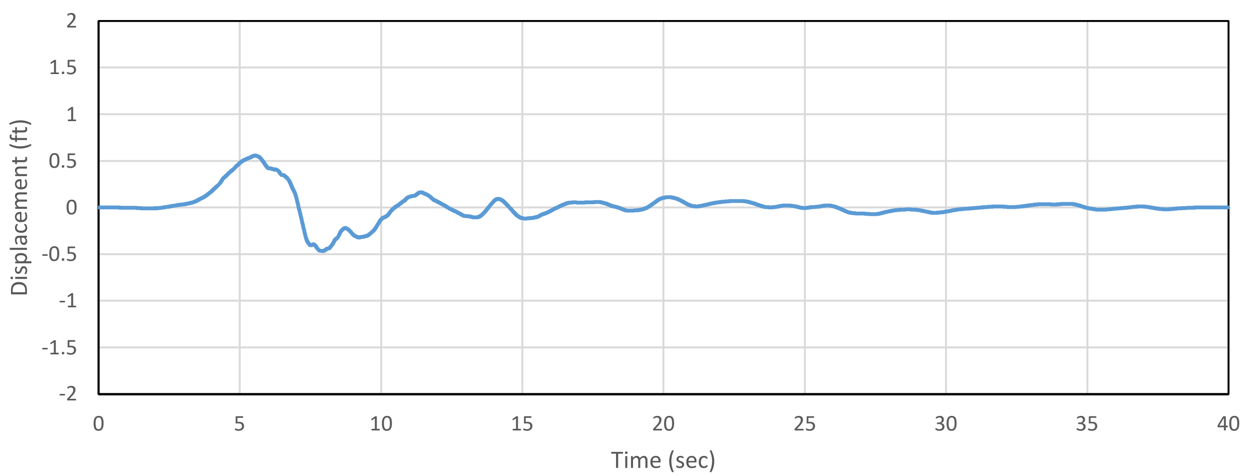
GROUP DELTA	**Scaled Time Histories - RSN 802 (FN)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-23a**

RSN 802

Station: Saratoga - Aloha Ave Earthquake: Loma Prieta

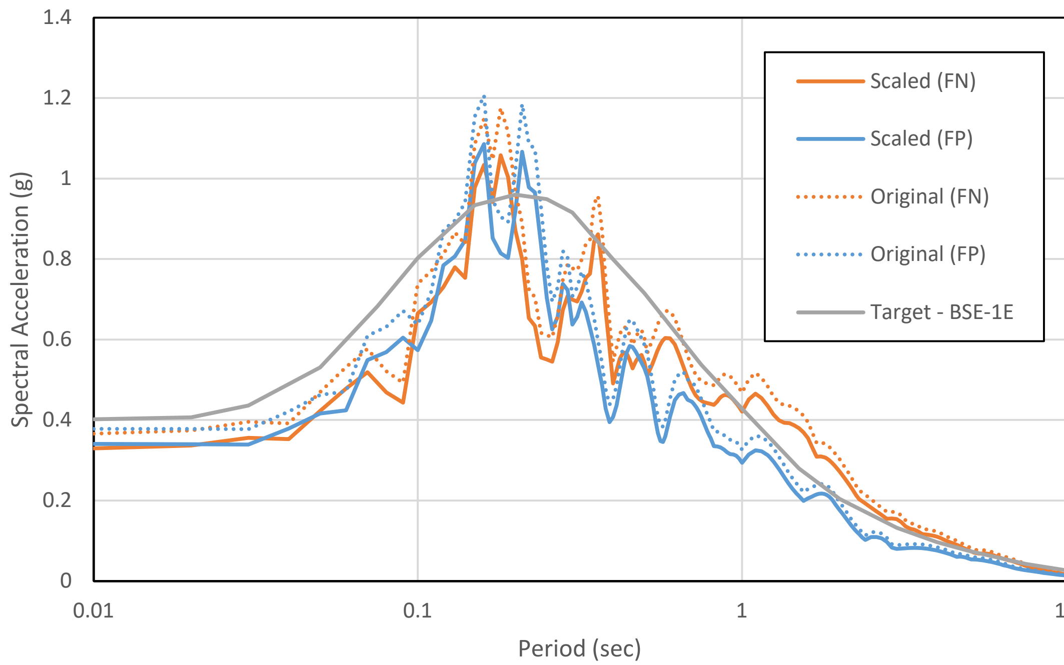


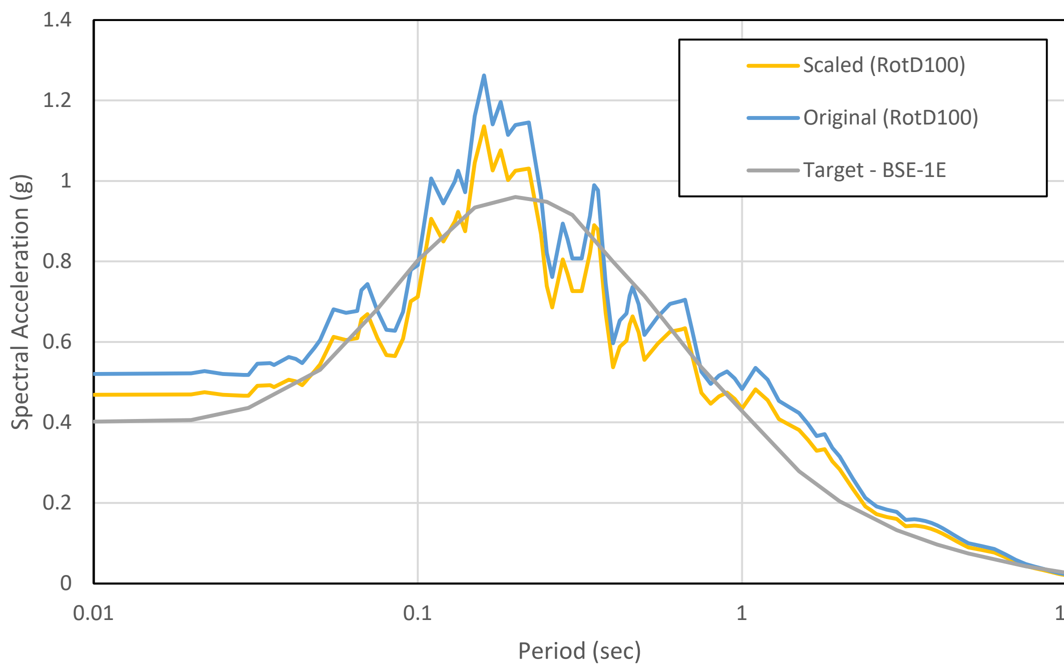




GROUP DELTA	**Scaled Time Histories - RSN 802 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-23b**

Response Spectra (5% damping)
Station: Saratoga - Aloha Ave Earthquake: Loma Prieta

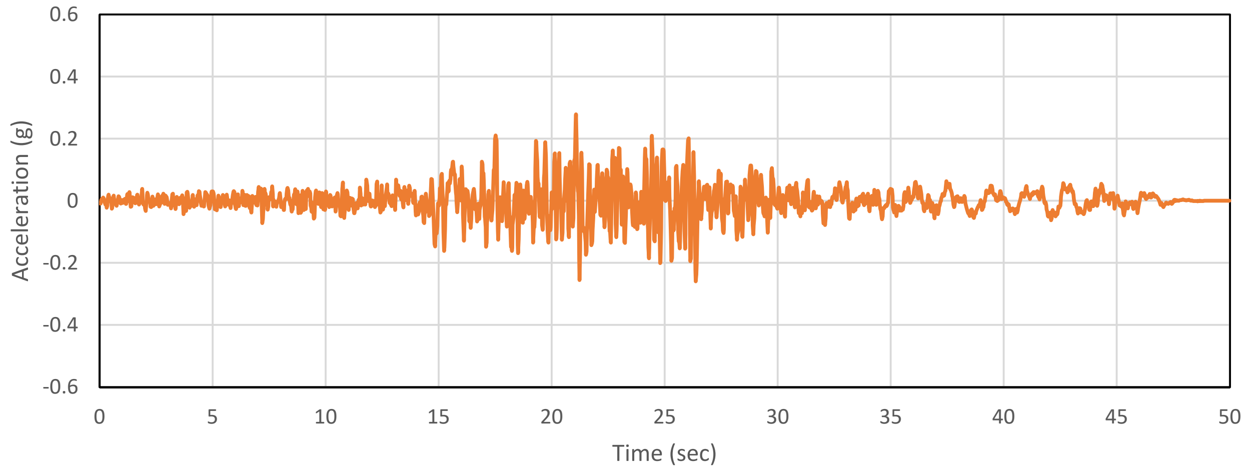


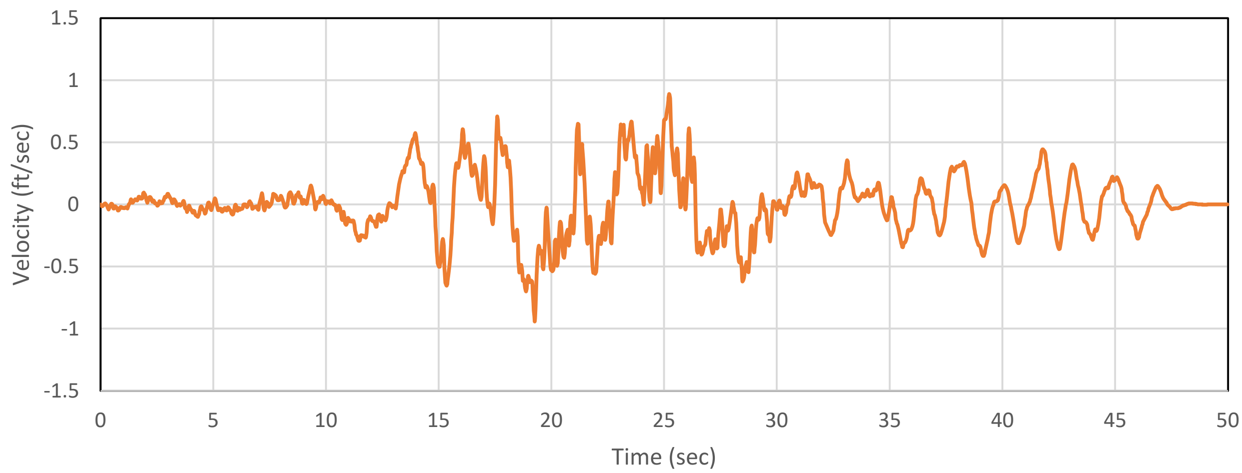


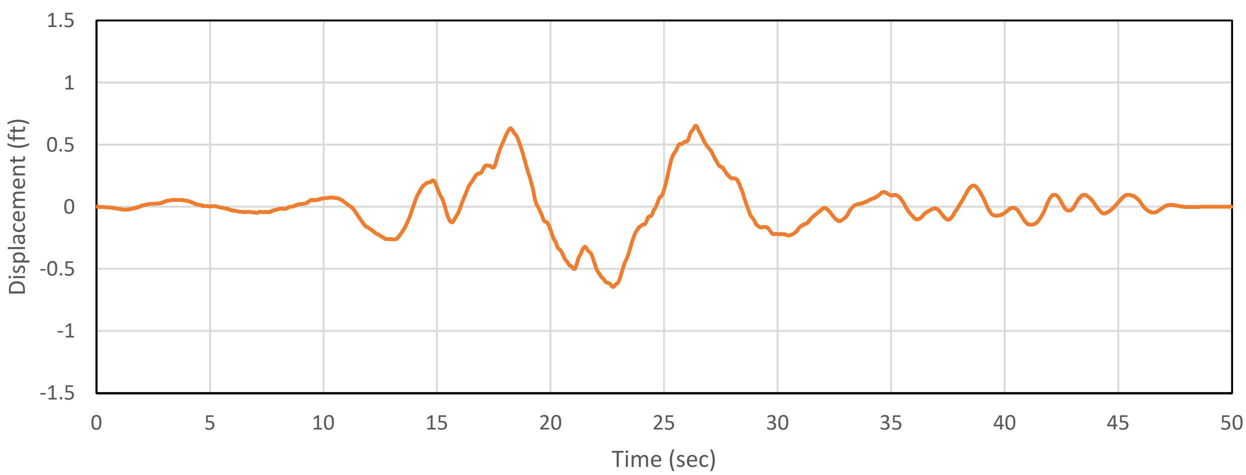
GROUP DELTA	Response Spectra - RSN 802		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-23c**

RSN 836

Station: Baker Fire Station, Earthquake: Landers



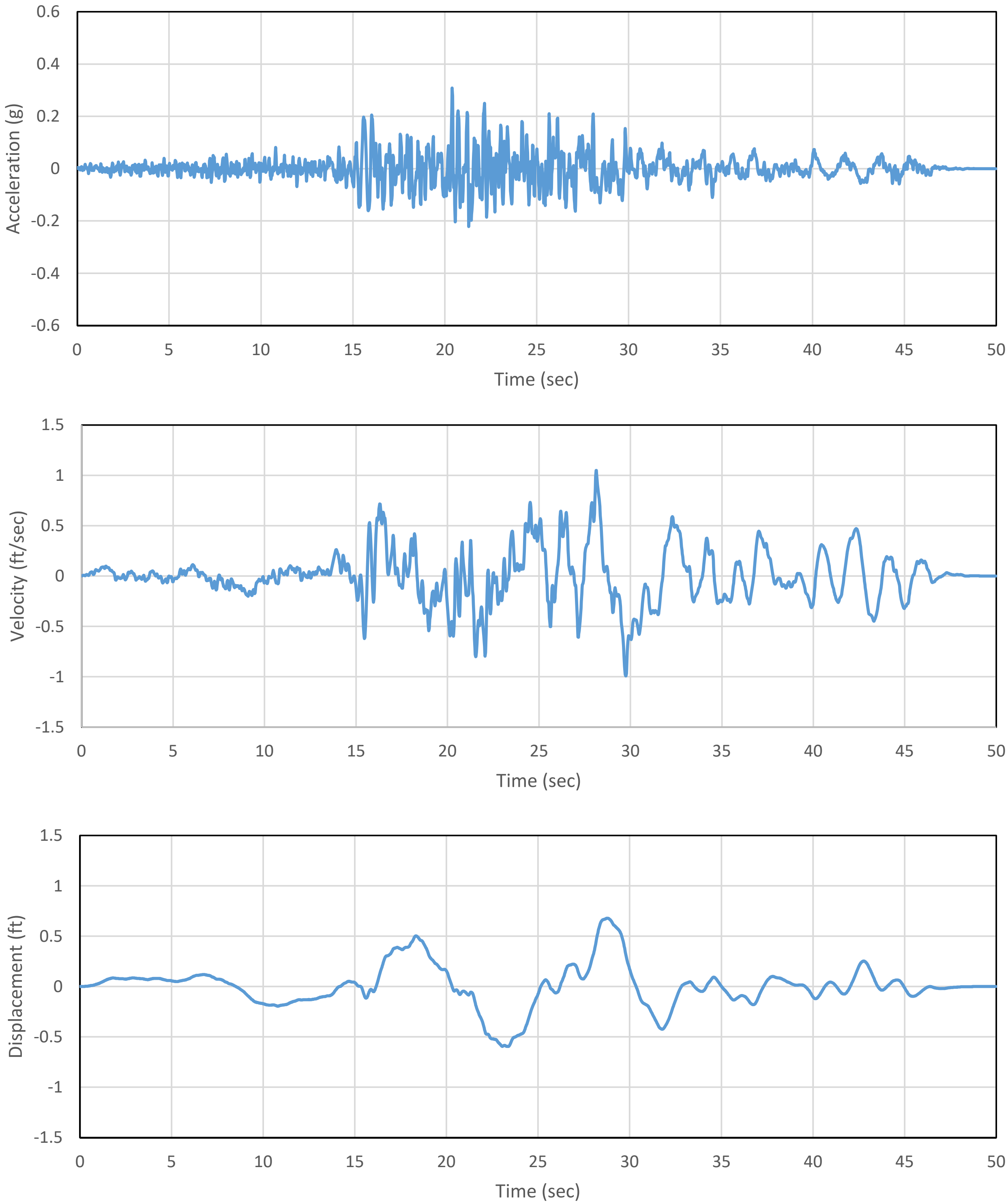




GROUP DELTA	**Scaled Time Histories - RSN 836 (FN)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-24a**

RSN 836

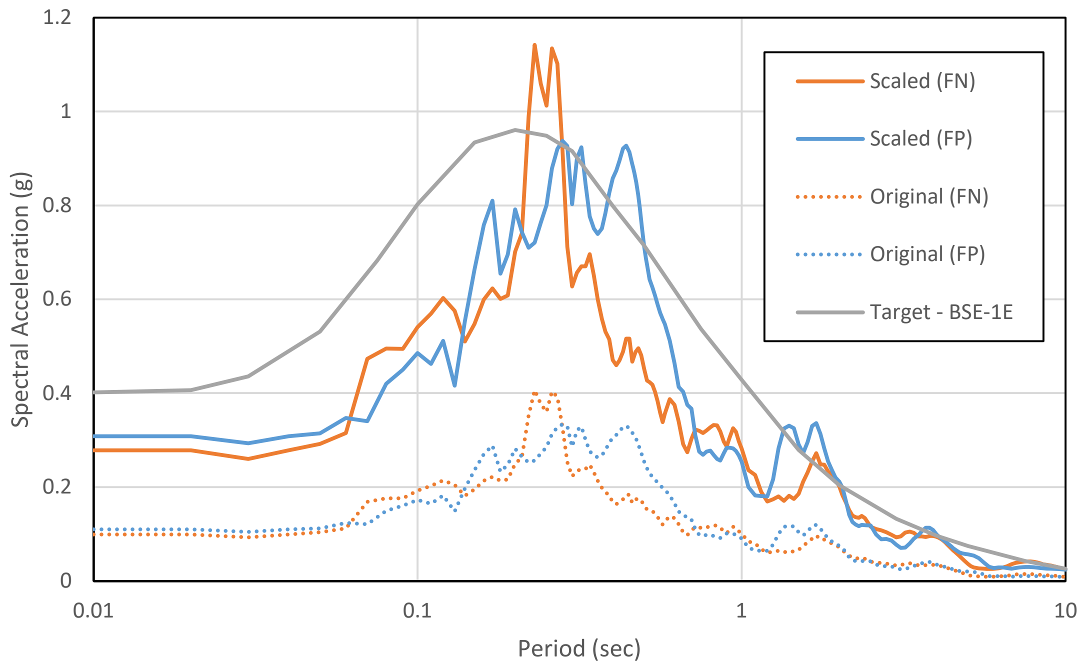
Station: Baker Fire Station, Earthquake: Landers

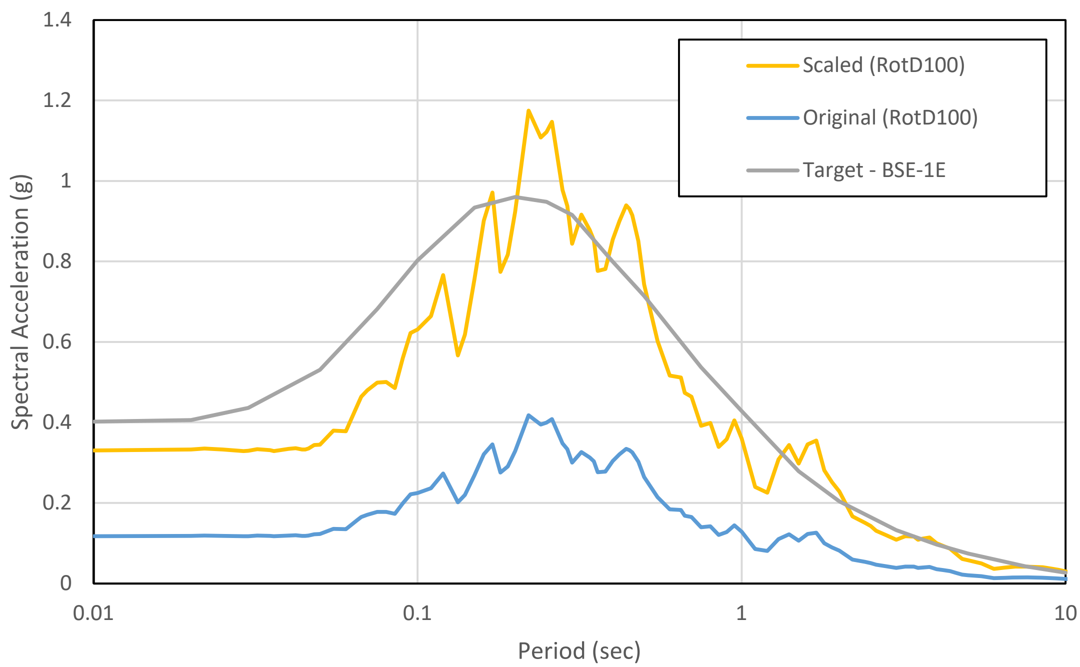


GROUP DELTA	Scaled Time Histories - RSN 836 (FP)		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-24b**

Response Spectra (5% damping)

Station: Baker Fire Station, Earthquake: Landers

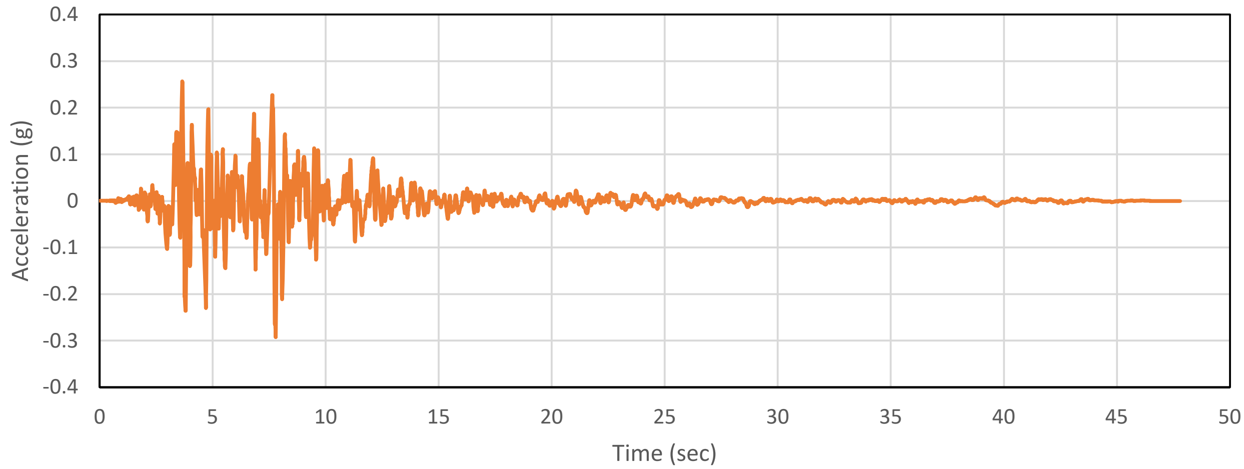


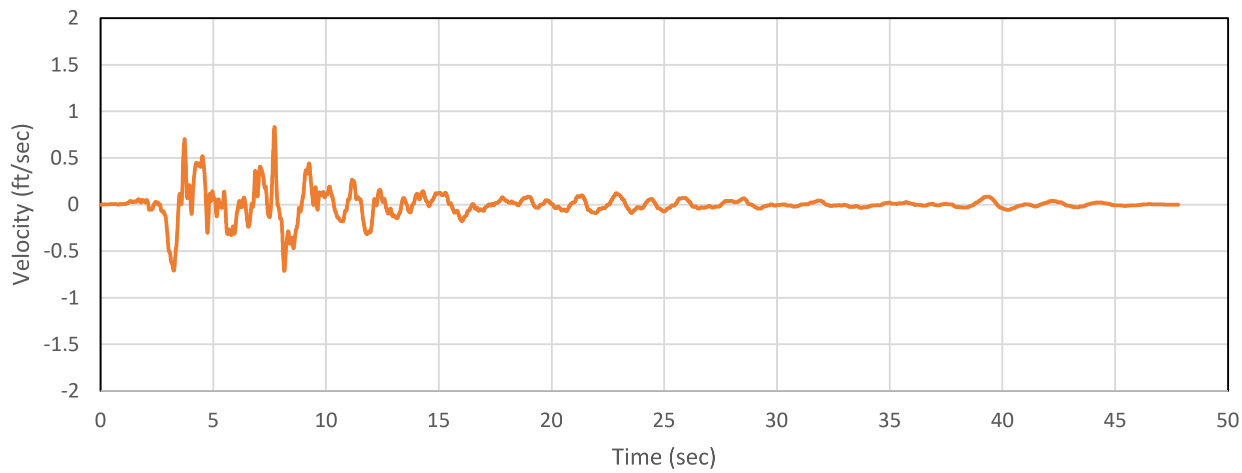


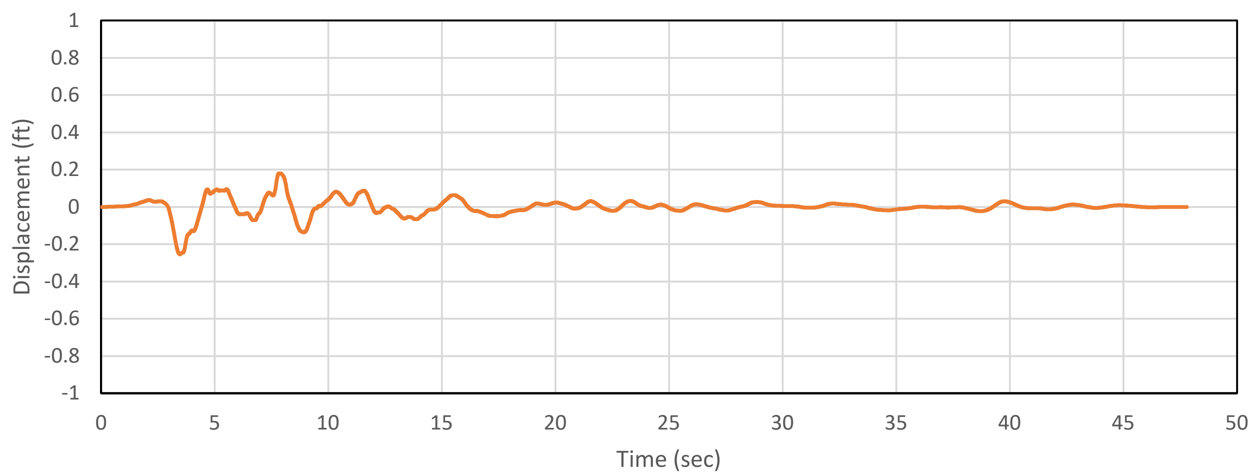
GROUP DELTA	Response Spectra - RSN 836		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-24c**

RSN 1004

Station: LA - Sepulveda VA Hospital, Earthquake: Northridge-01







GROUP DELTA

Scaled Time Histories - RSN 1004 (FN)

Project Number: LA-1474

Date: 12/18/2020

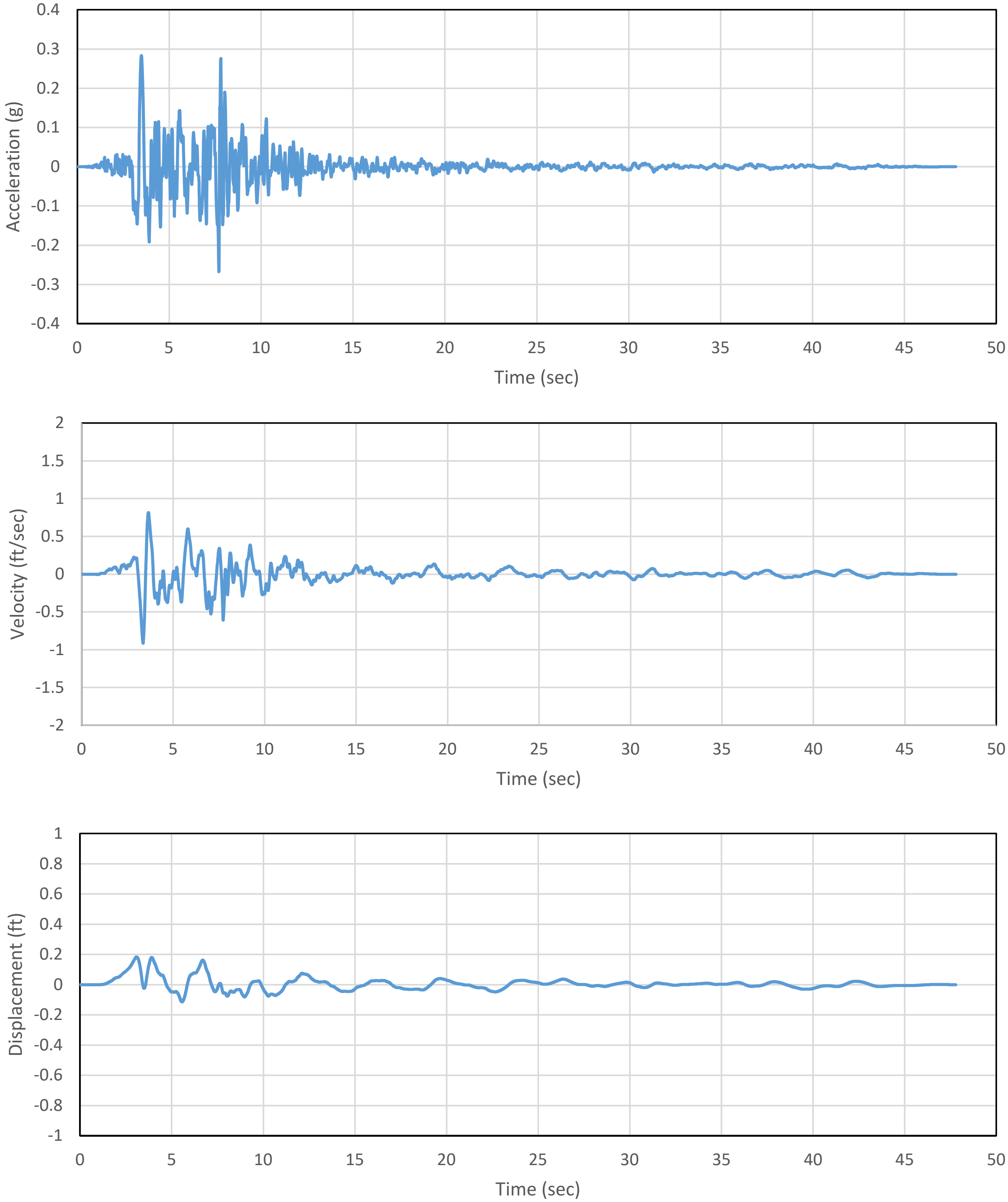
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-25a

RSN 1004

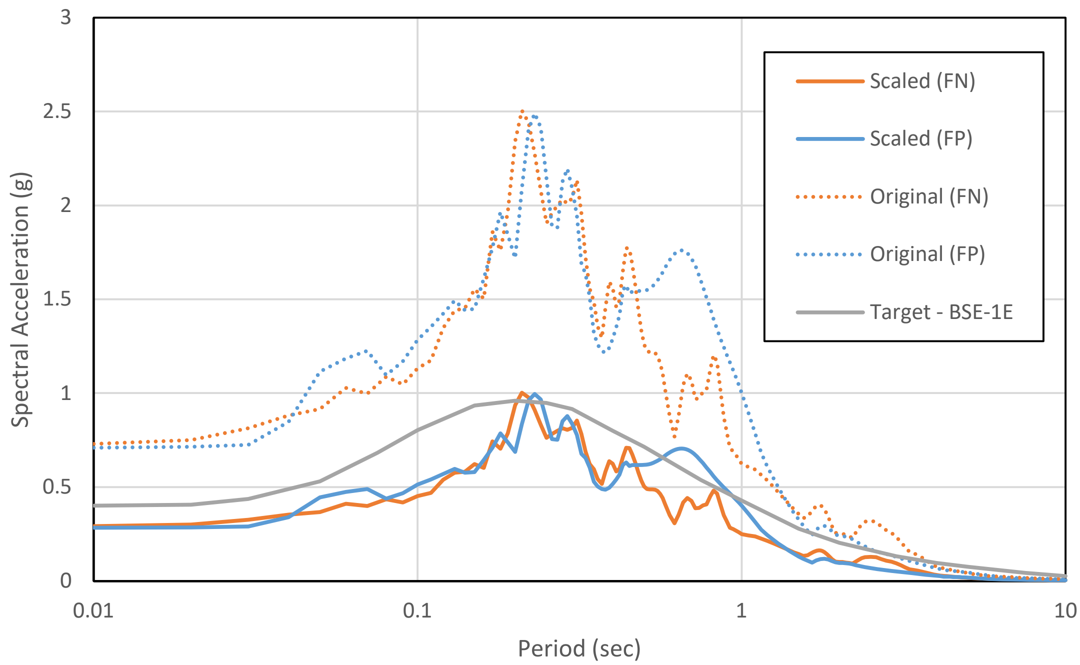
Station: LA - Sepulveda VA Hospital, Earthquake: Northridge-01

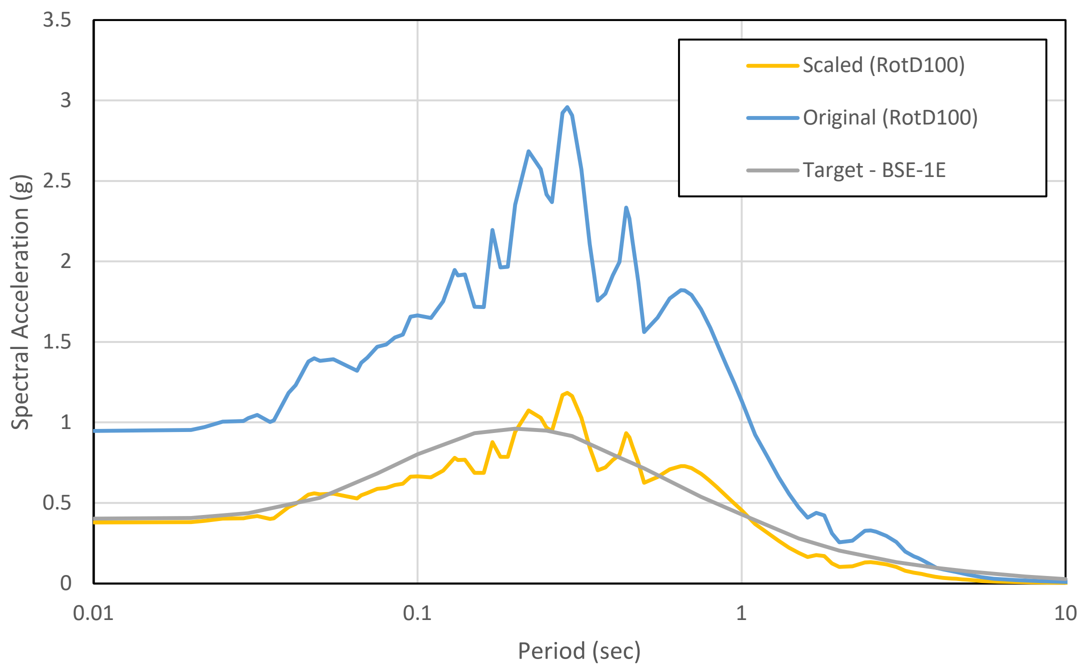


GROUP DELTA	**Scaled Time Histories - RSN 1004 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-25b**

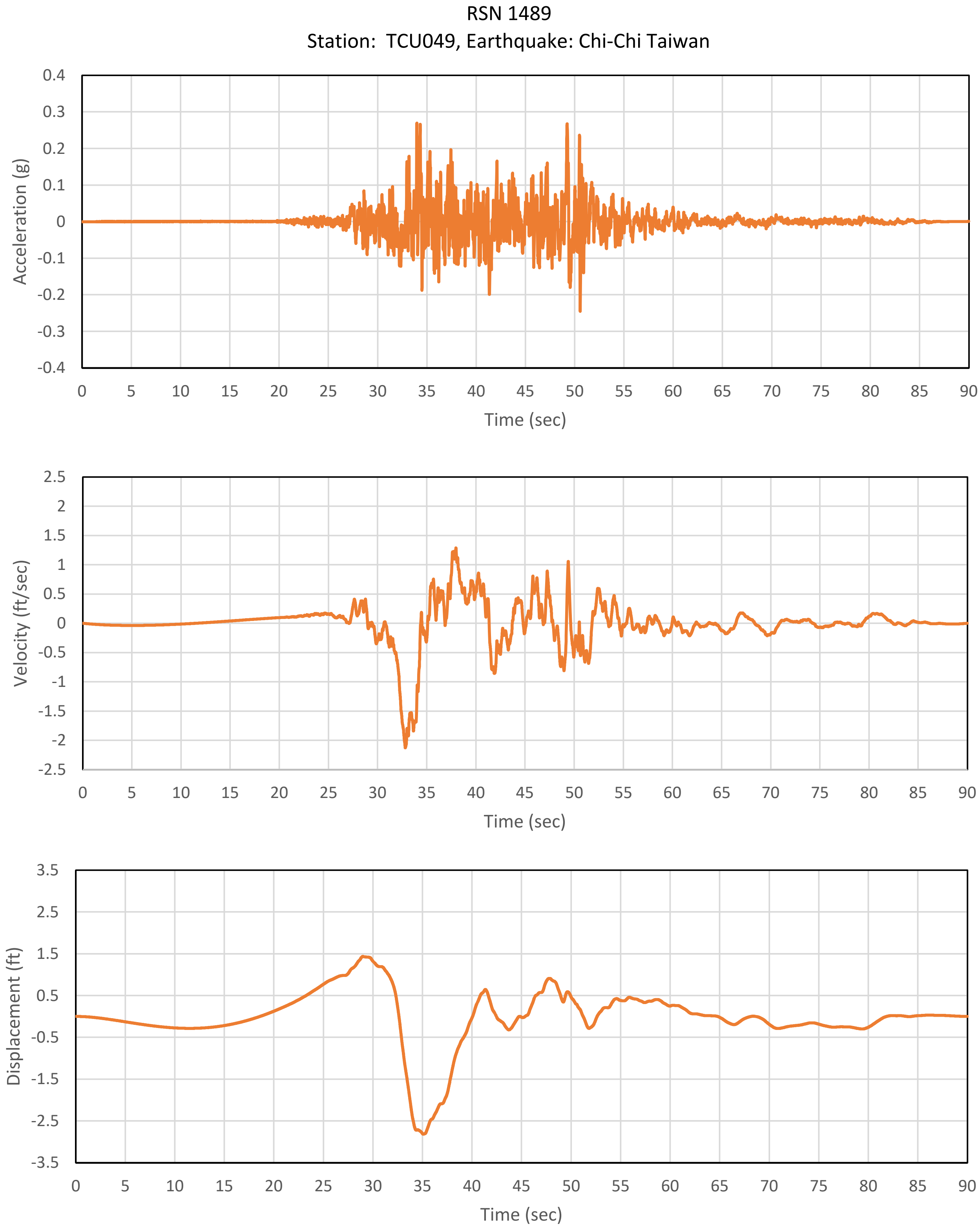
Response Spectra (5% damping)

Station: LA - Sepulveda VA Hospital, Earthquake: Northridge-01

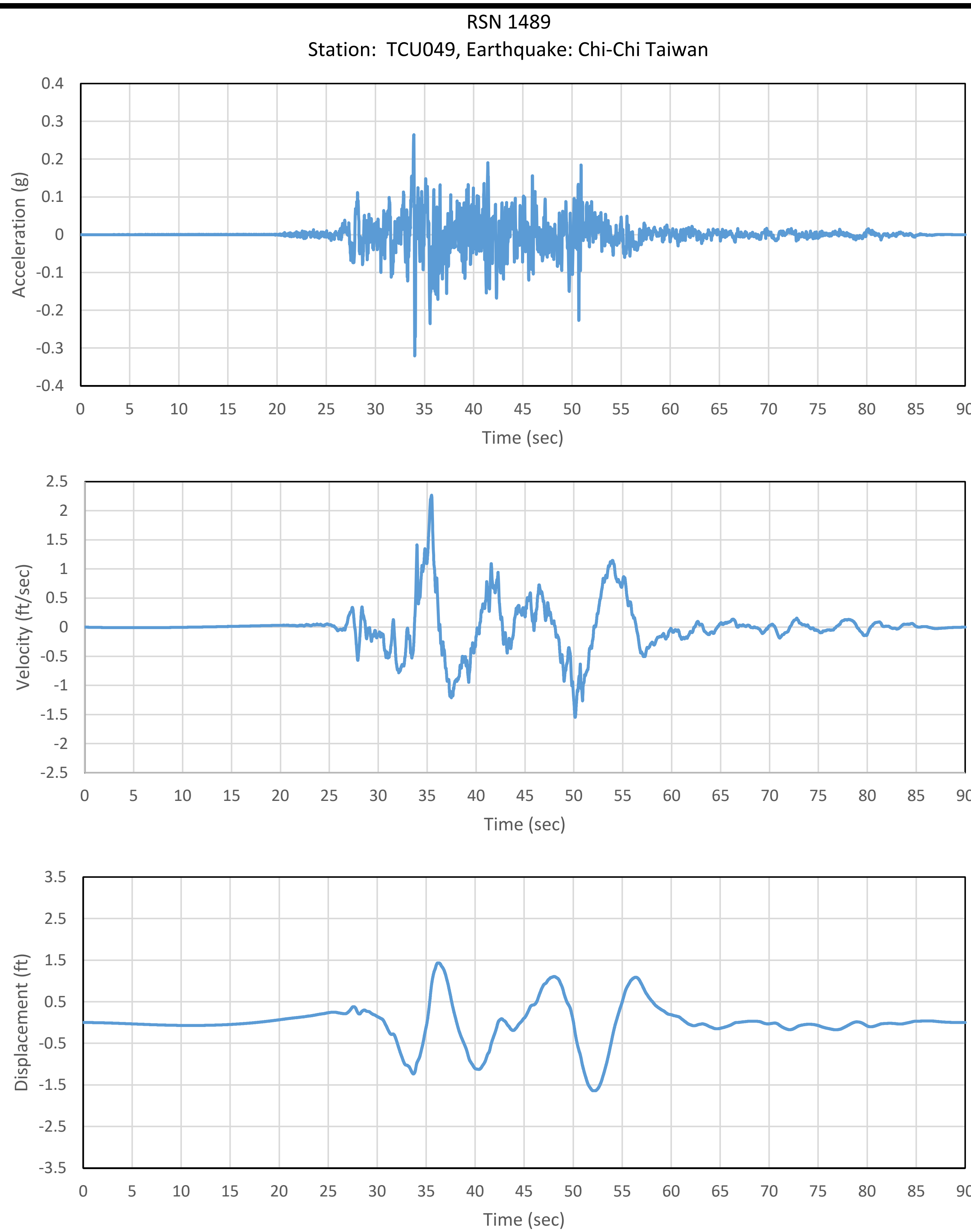




GROUP DELTA	**Response Spectra - RSN 1004**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-25c**



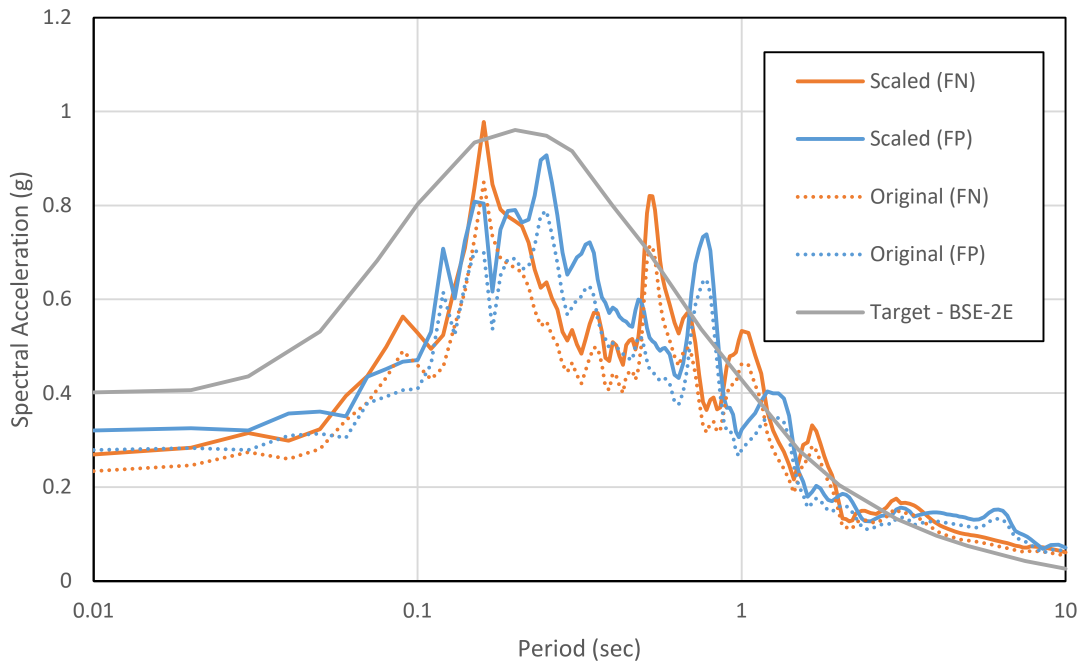
GROUP DELTA	Scaled Time Histories - RSN 1489 (FN)		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-26a**

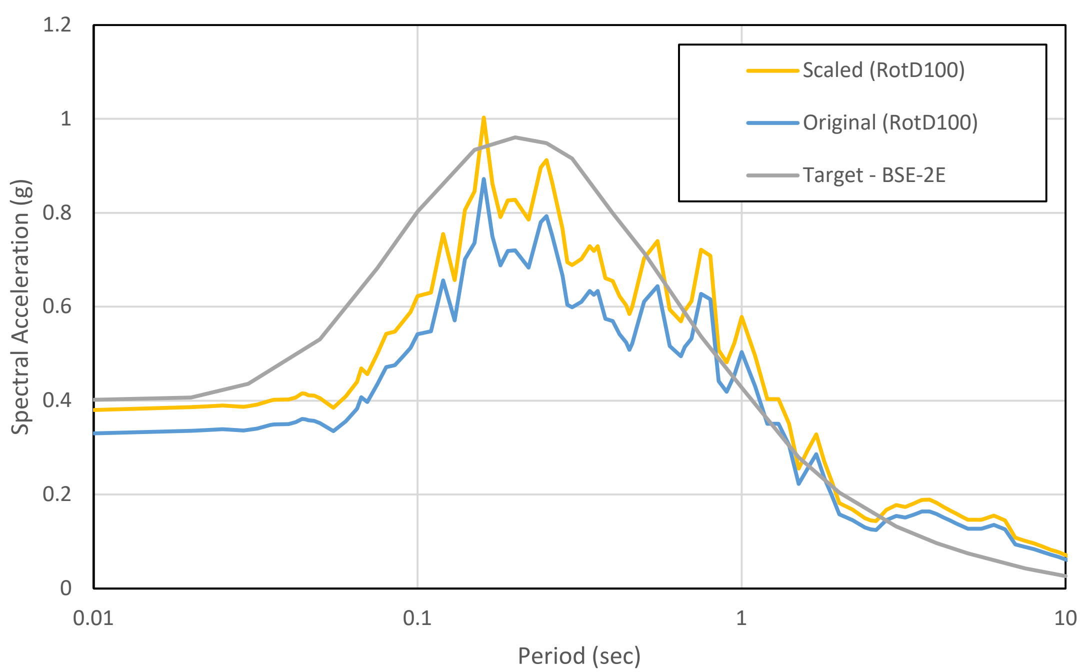


GROUP DELTA	**Scaled Time Histories - RSN 1489 (FP)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-26b**

Response Spectra (5% damping)

Station: TCU049, Earthquake: Chi-Chi Taiwan





GROUP DELTA

Response Spectra - RSN 1489

Project Number: LA-1474

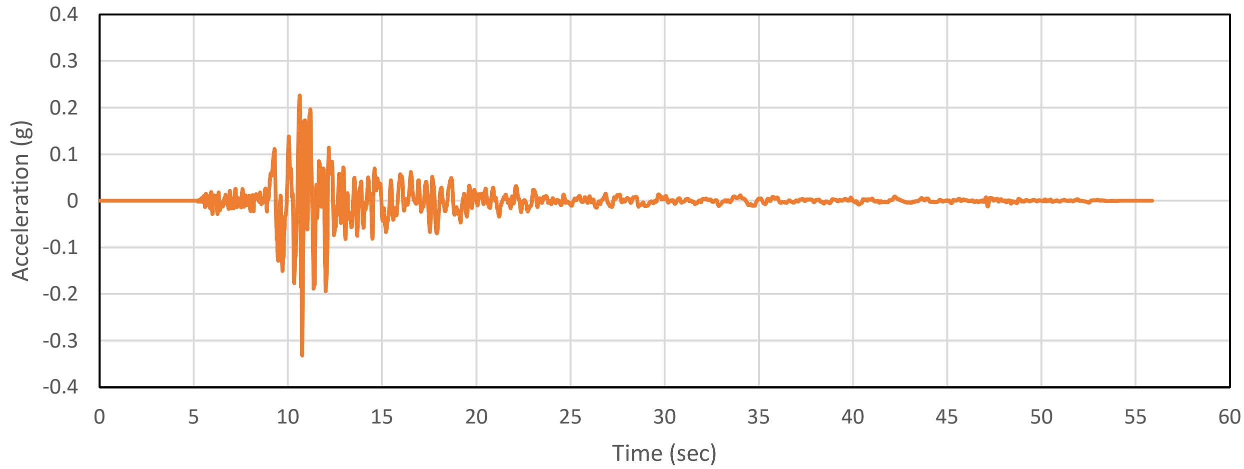
Date: 12/18/2020

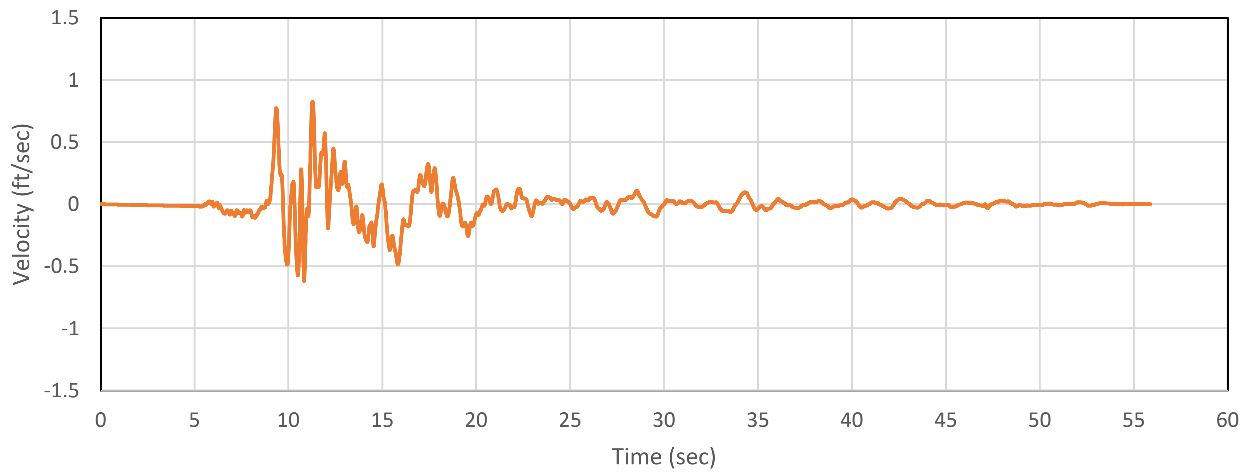
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

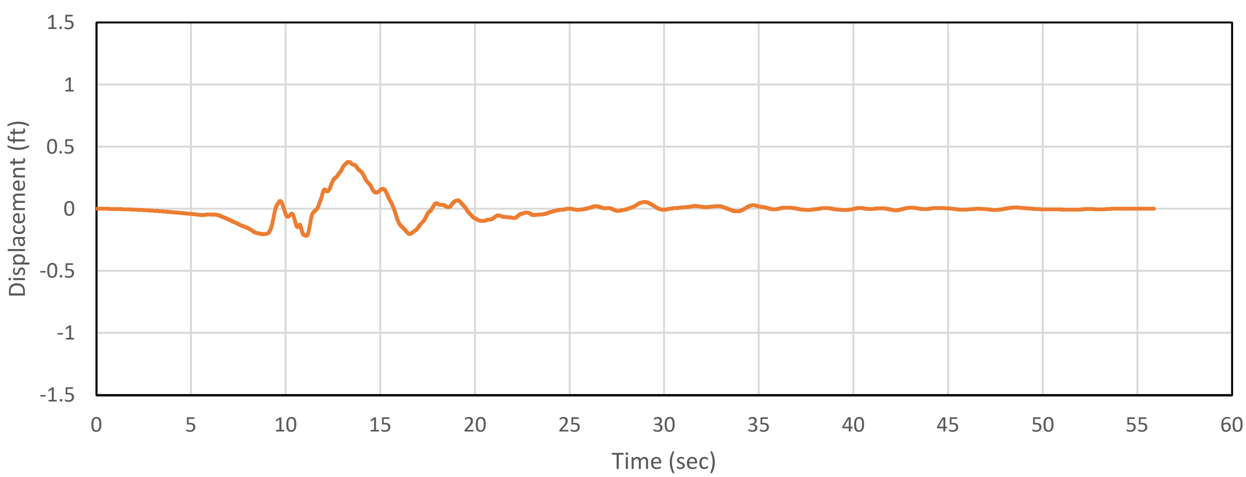
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-26c

RSN 1602
Station: Bolu, Earthquake: Duzce Turkey



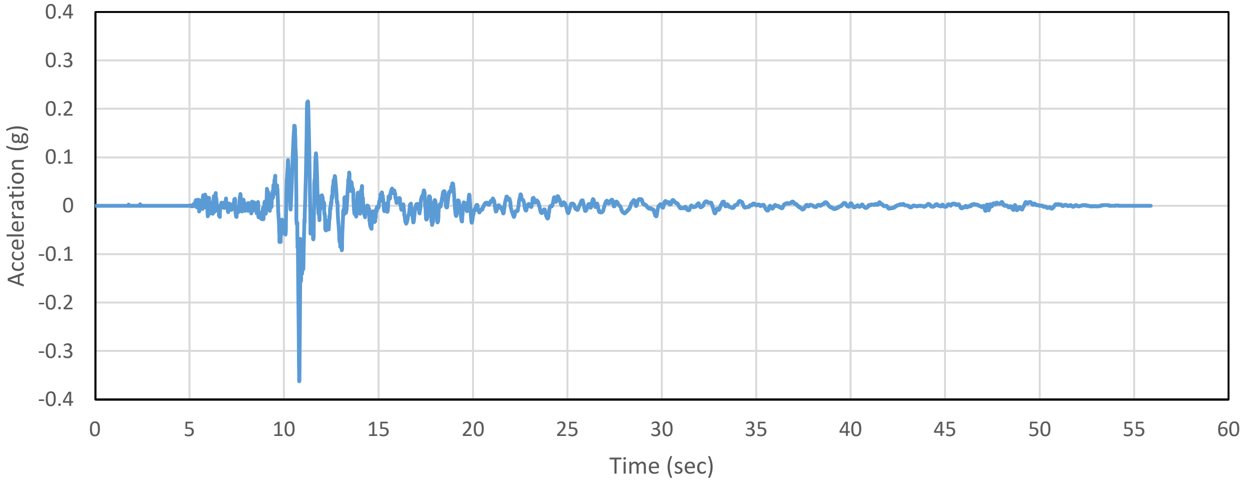


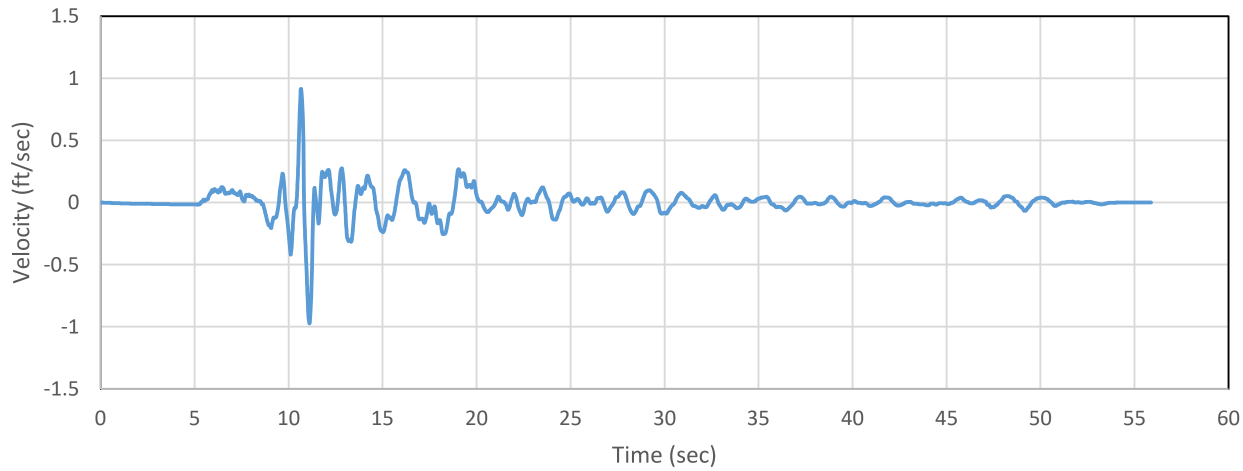


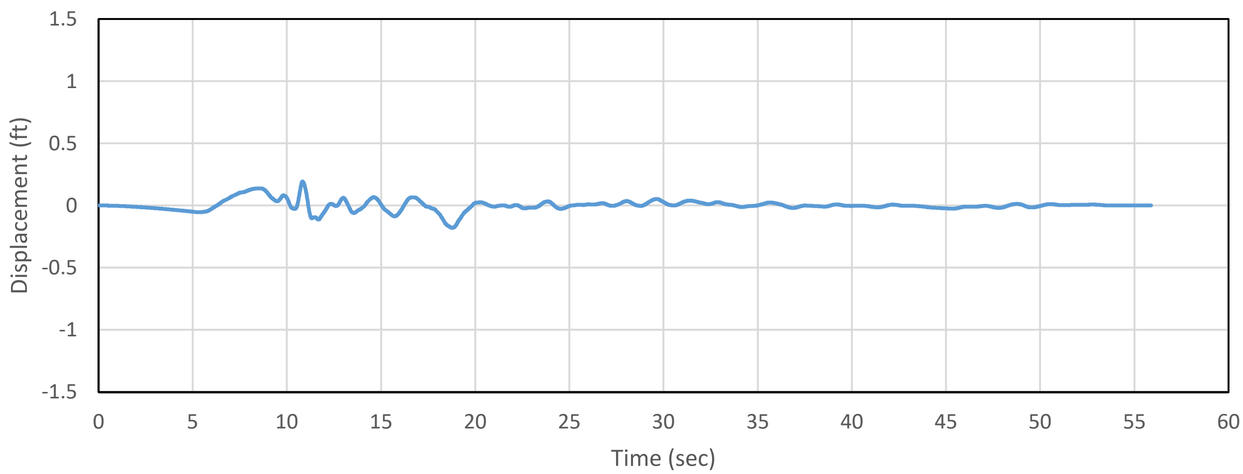
GROUP DELTA	**Scaled Time Histories - RSN 1602 (FN)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-27a**

RSN 1602

Station: Bolu, Earthquake: Duzce Turkey

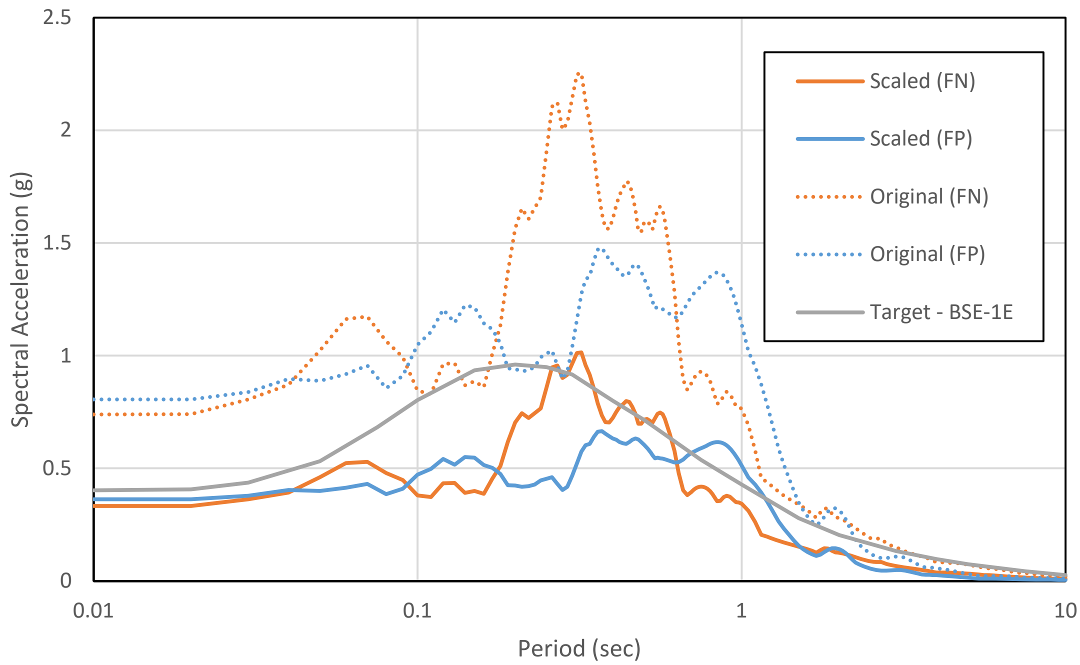


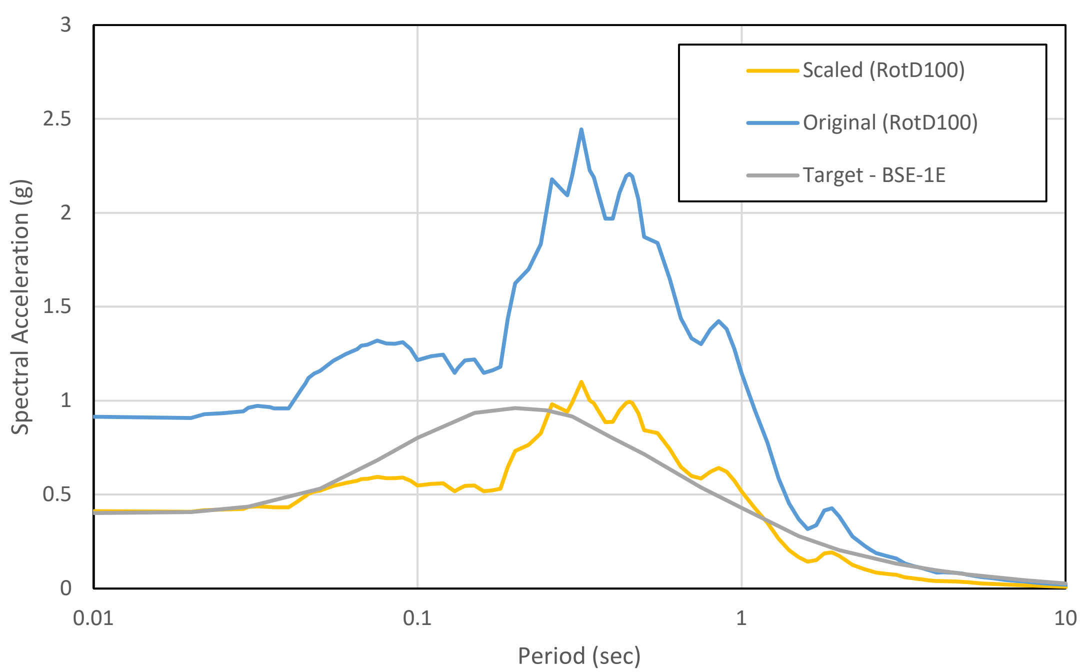




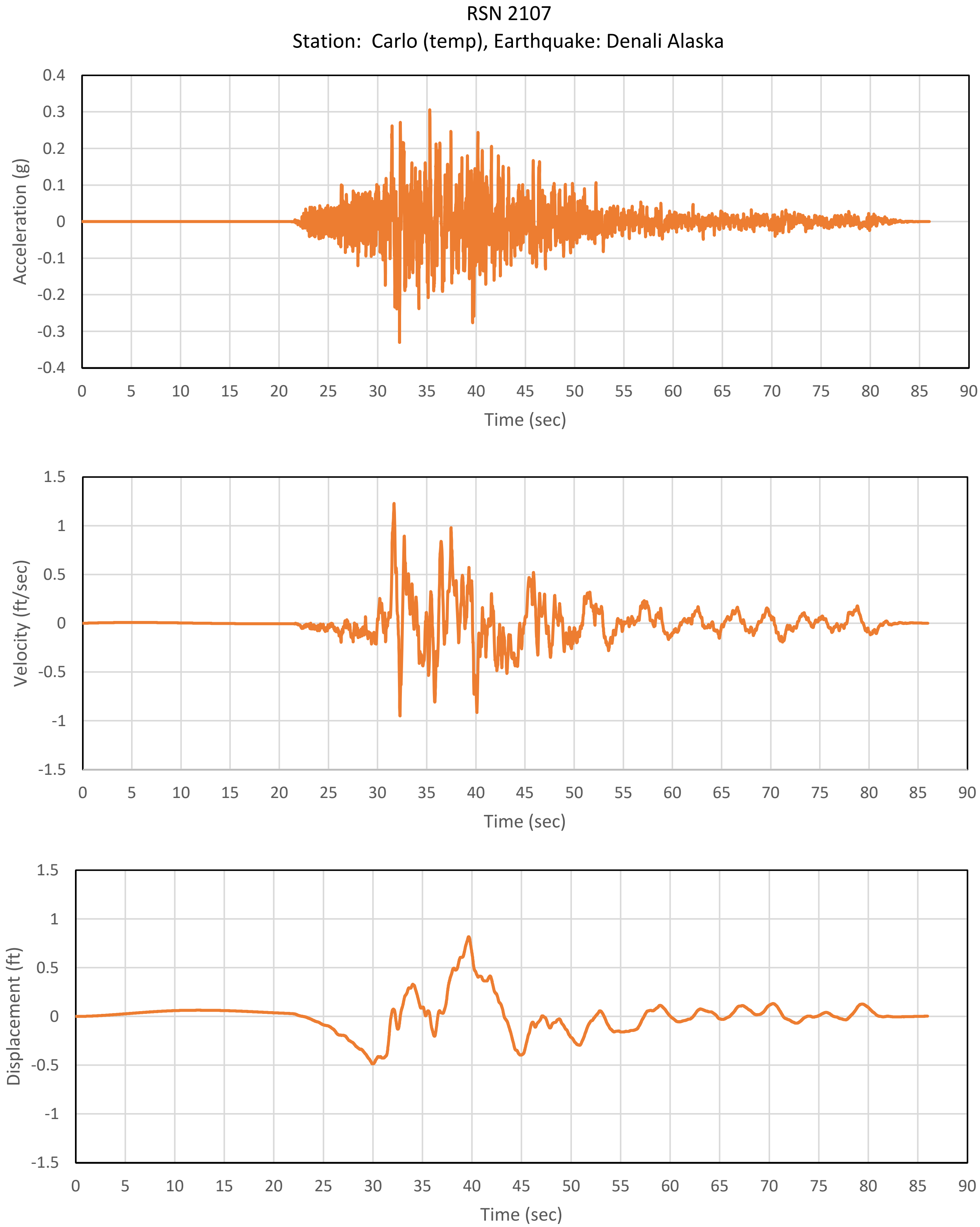
GROUP DELTA	**Scaled Time Histories - RSN 1602 (FP)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-27b**

Response Spectra (5% damping)
Station: Bolu, Earthquake: Duzce Turkey





GROUP DELTA	**Response Spectra - RSN 1602**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-27c**



GROUP DELTA

Scaled Time Histories - RSN 2107 (FN)

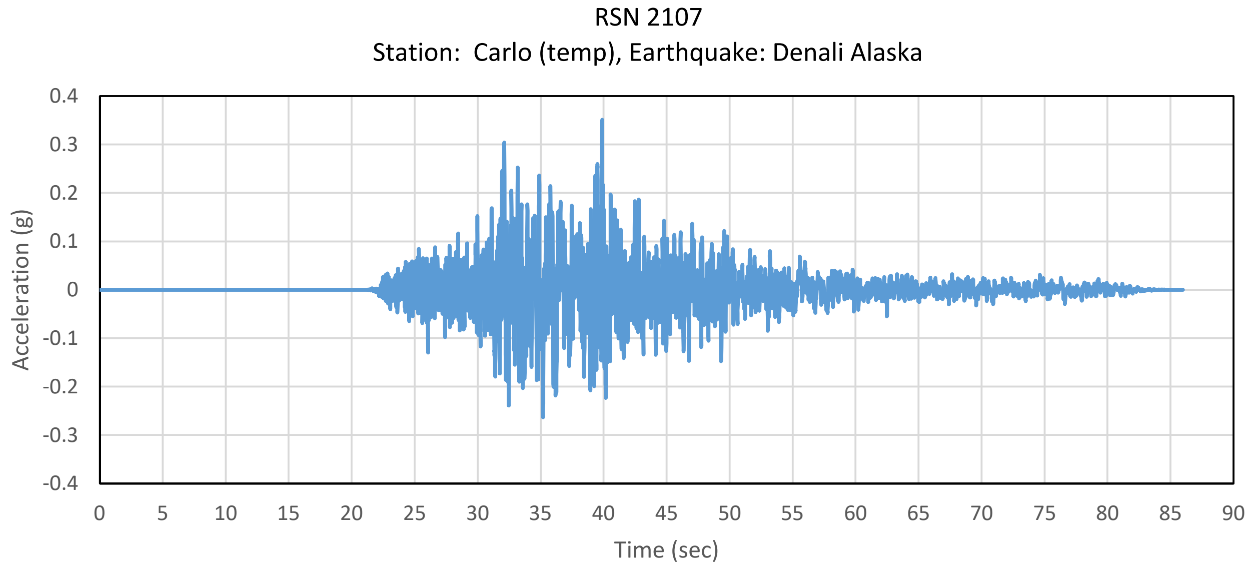
Project Number: LA-1474

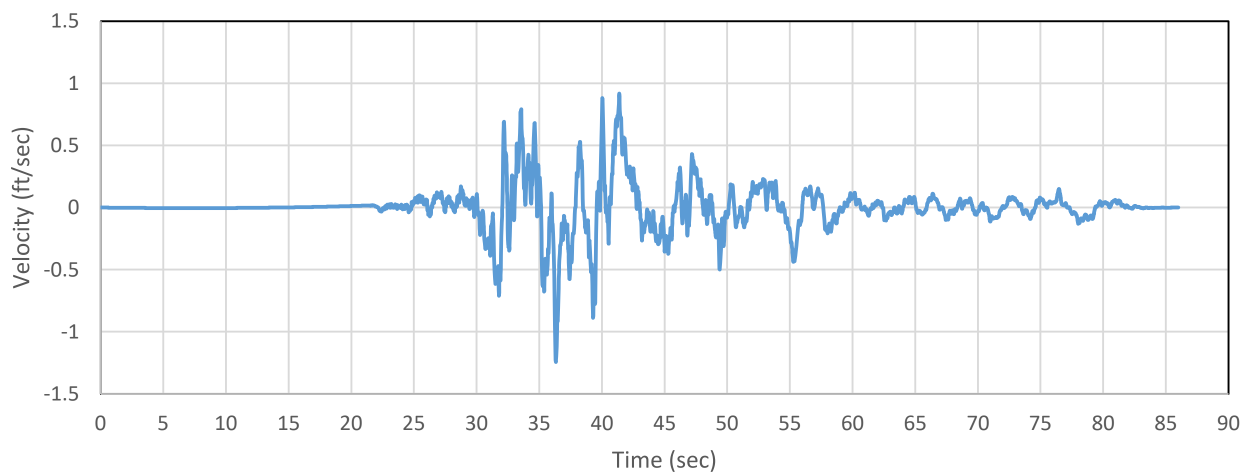
Date: 12/18/2020

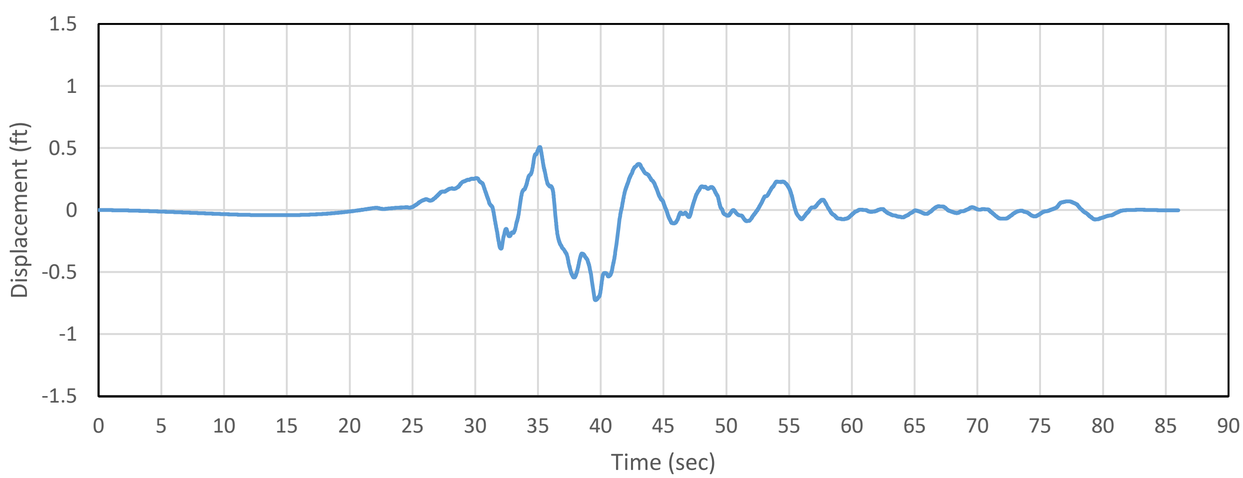
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-28a



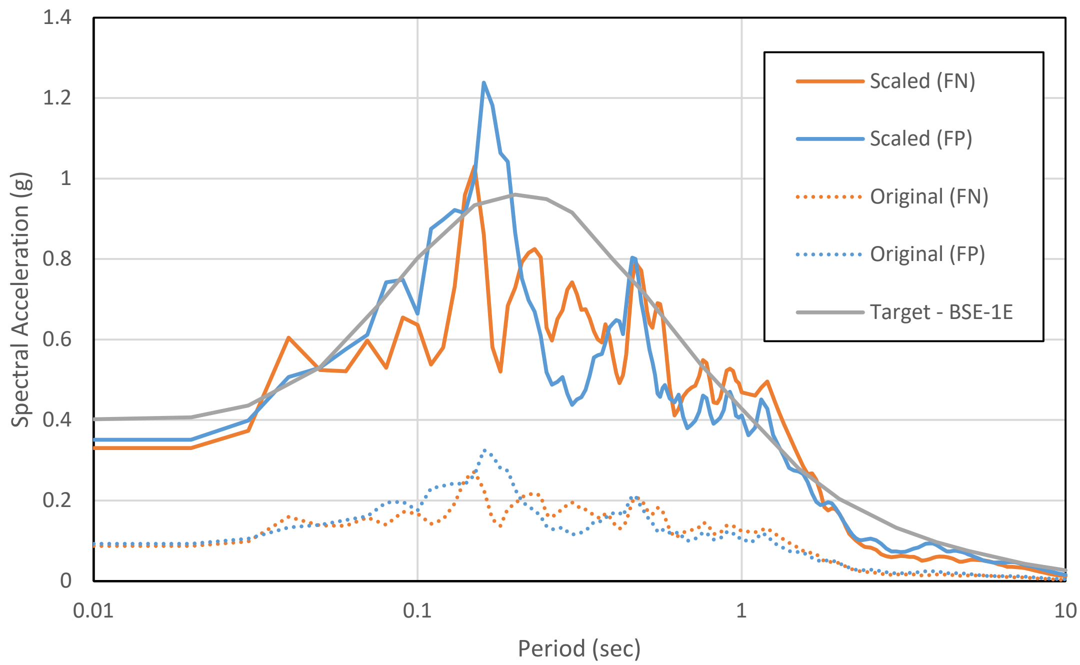


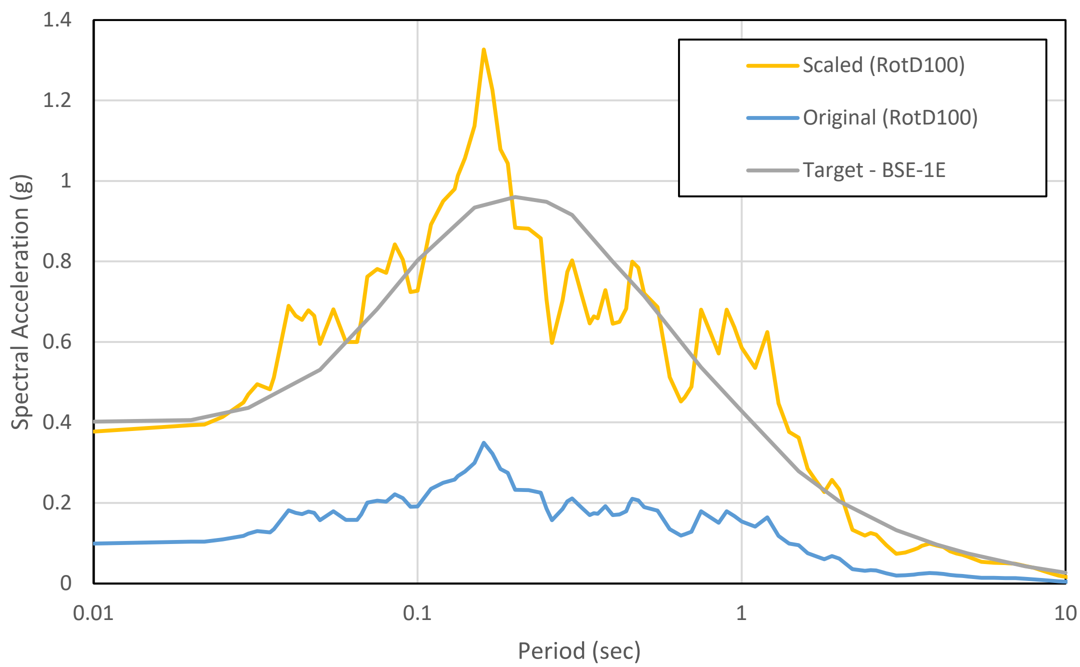


GROUP DELTA	**Scaled Time Histories - RSN 2107 (FP)**		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-28b**

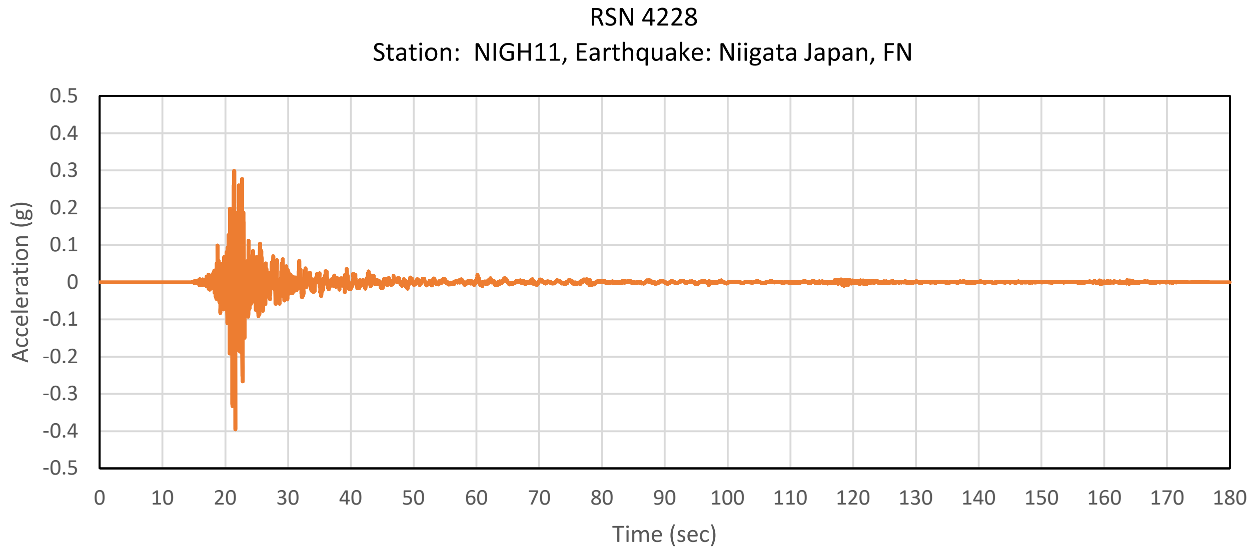
Response Spectra (5% damping)

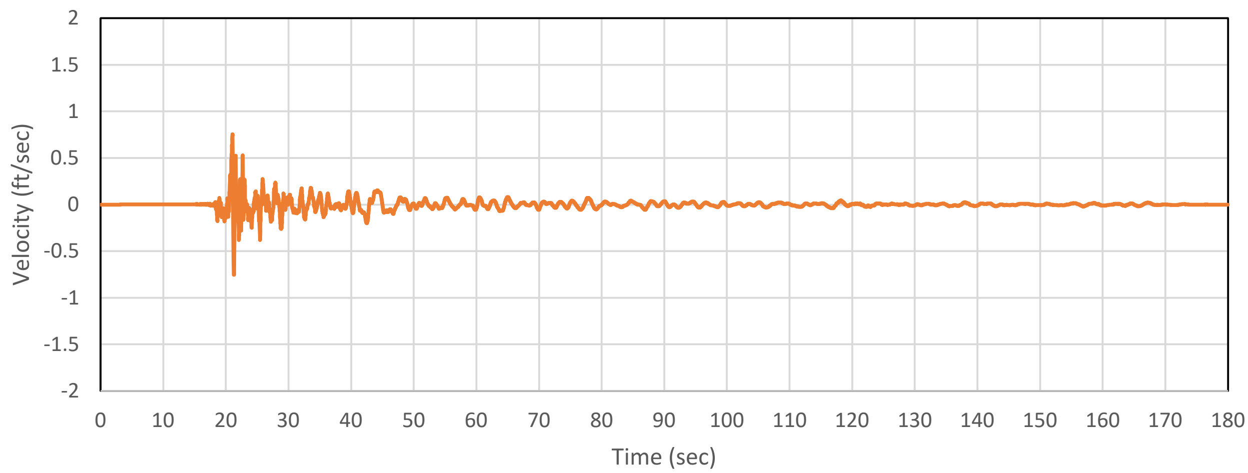
Station: Carlo (temp), Earthquake: Denali Alaska

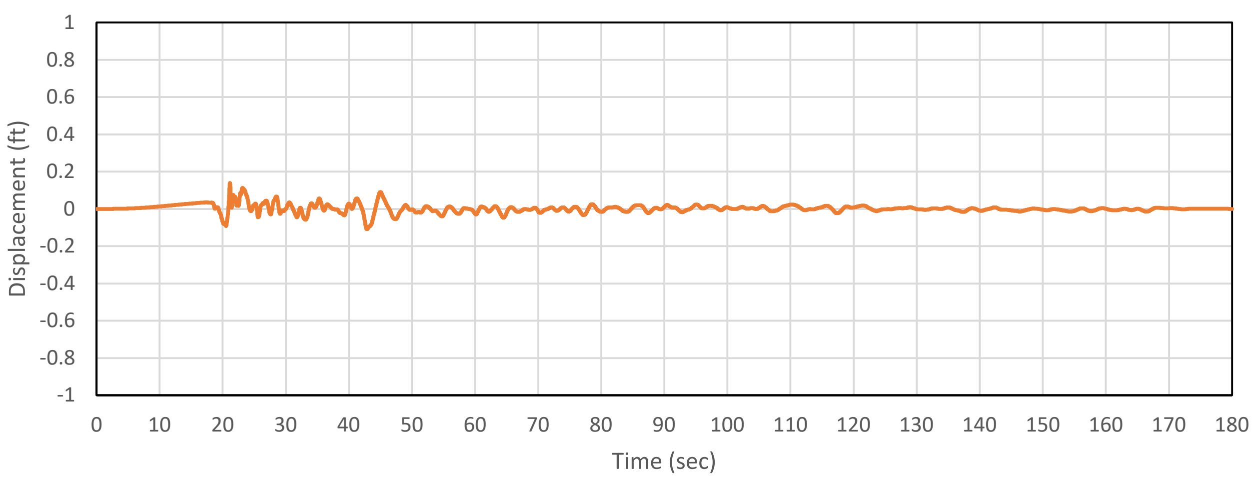




GROUP DELTA	**Response Spectra - RSN 2107**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-28c**







GROUP DELTA

Scaled Time Histories - RSN 4228 (FN)

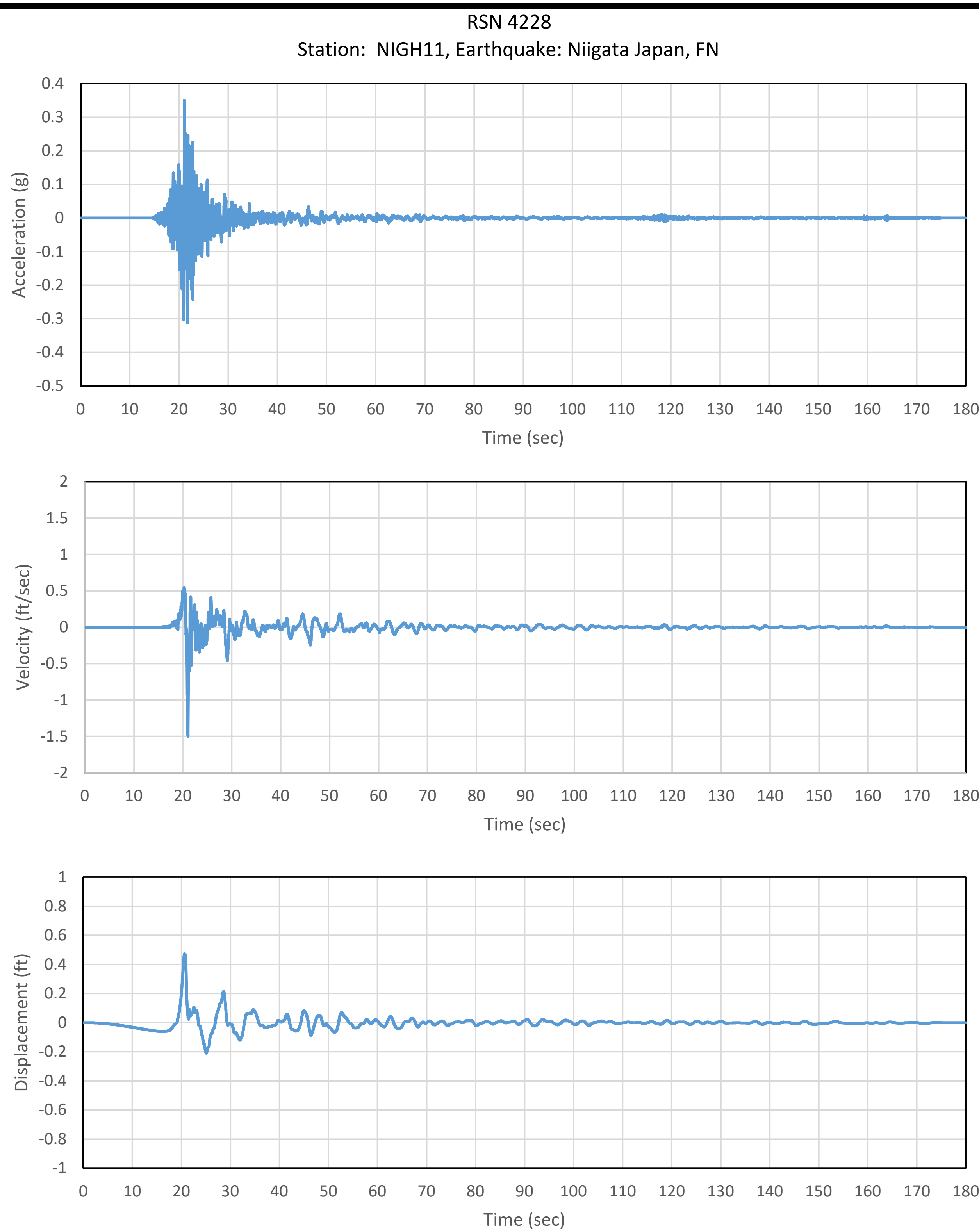
Project Number: LA-1474

Date: 12/18/2020

GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

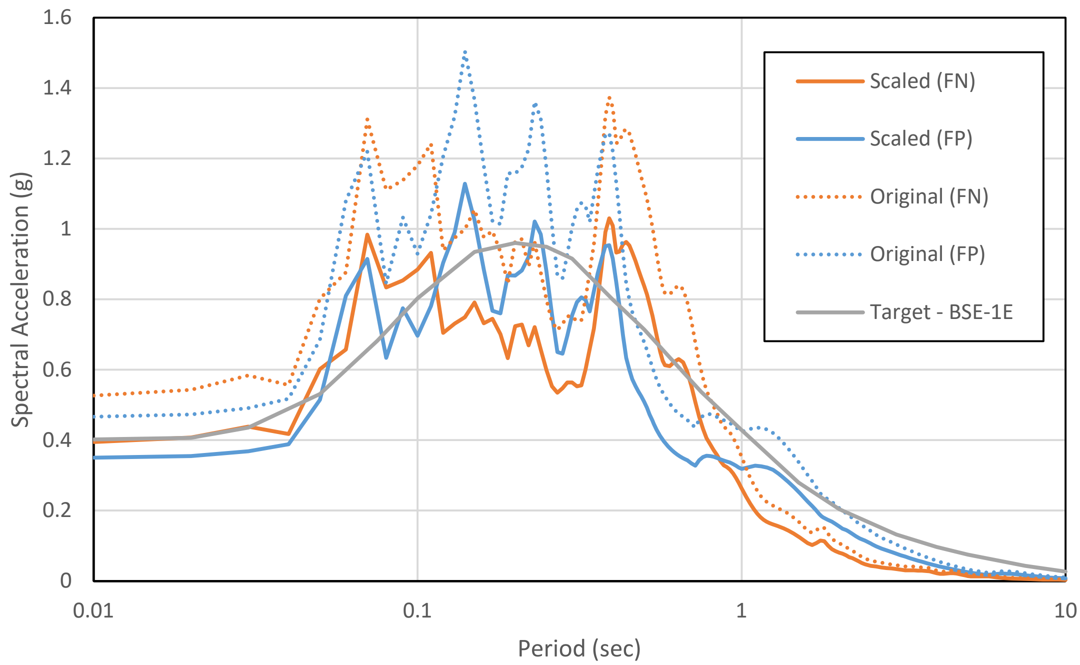
PROJECT NAME:
City of Inglewood Seismic Retrofits

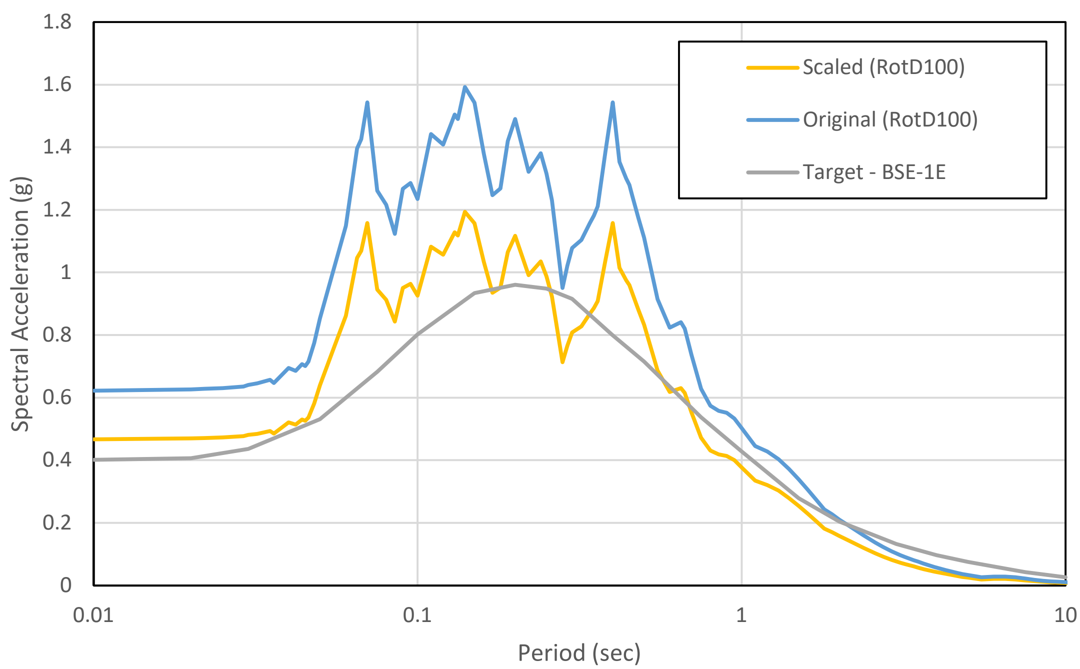
Figure C-29a



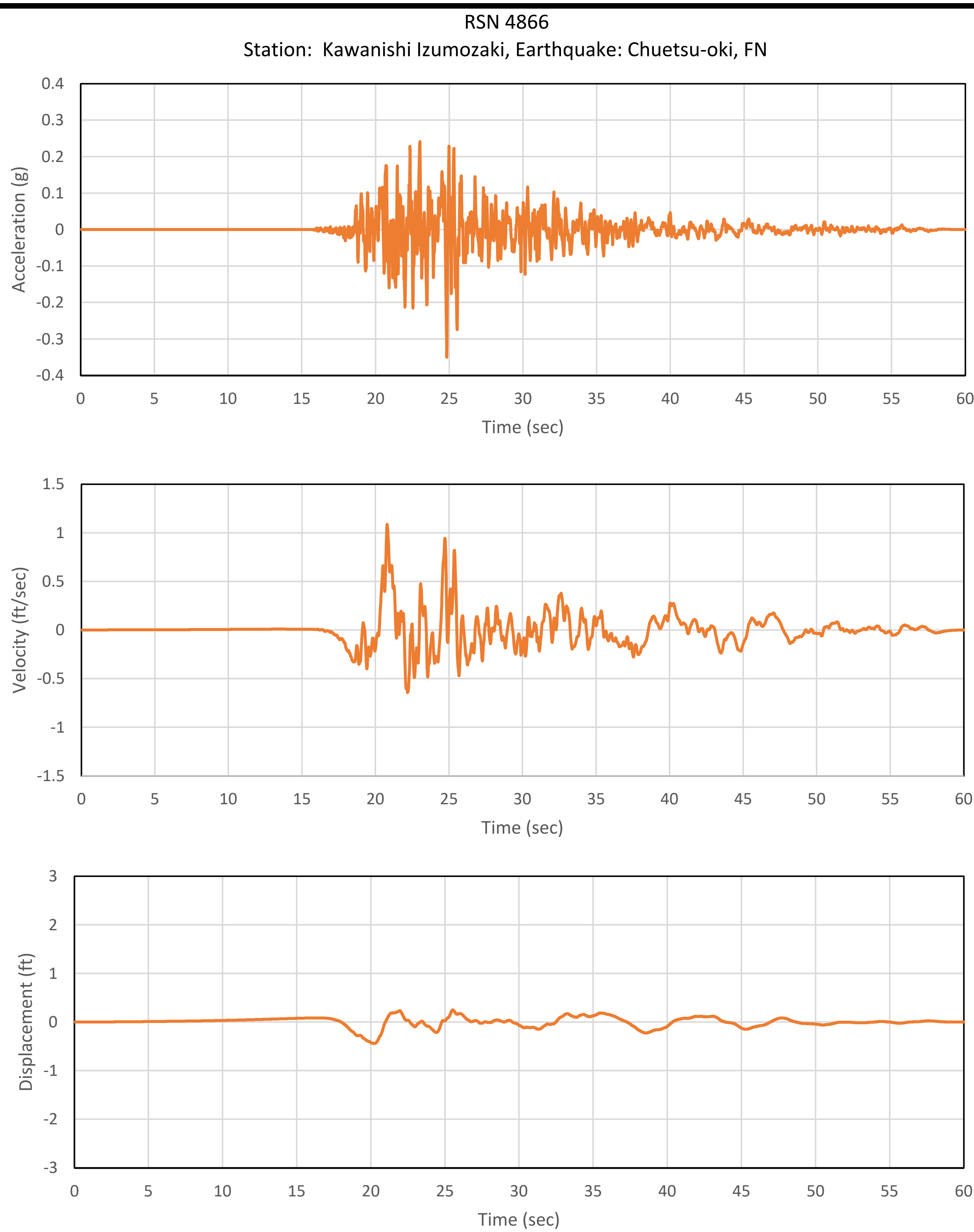
GROUP DELTA	**Scaled Time Histories - RSN 4228 (FP)**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-29b**

Response Spectra (5% damping)
Station: NIGH11, Earthquake: Niigata Japan, FN





GROUP DELTA	**Response Spectra - RSN 4228**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-29c**



GROUP DELTA

Scaled Time Histories - RSN 4866 (FN)

Project Number: LA-1474

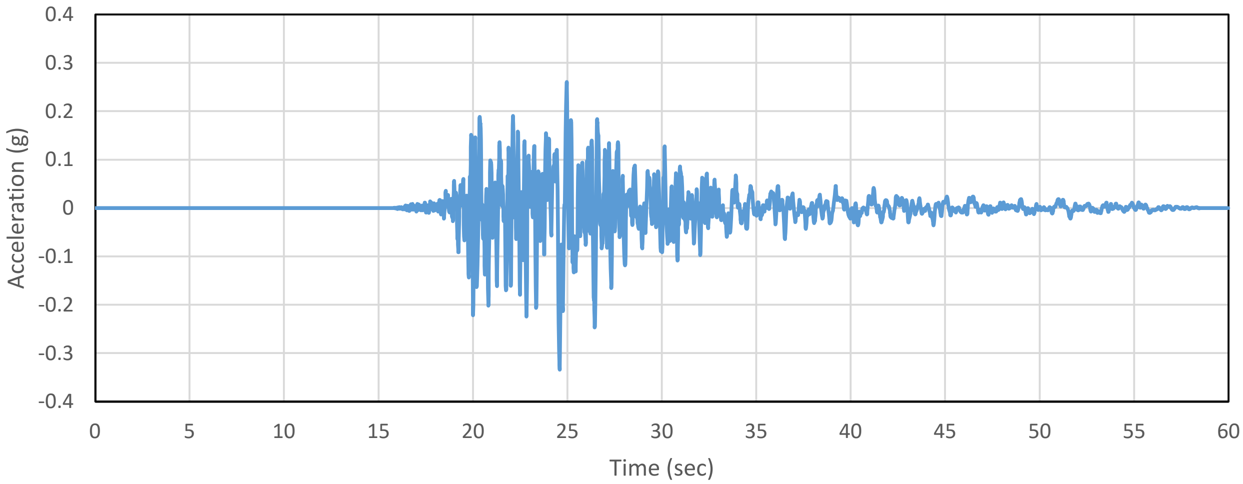
Date: 12/18/2020

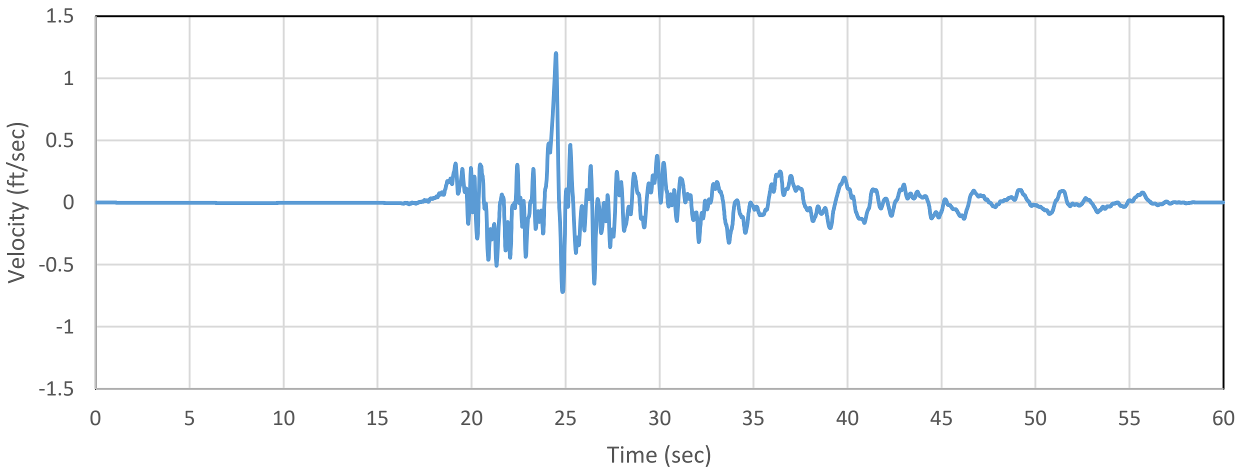
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

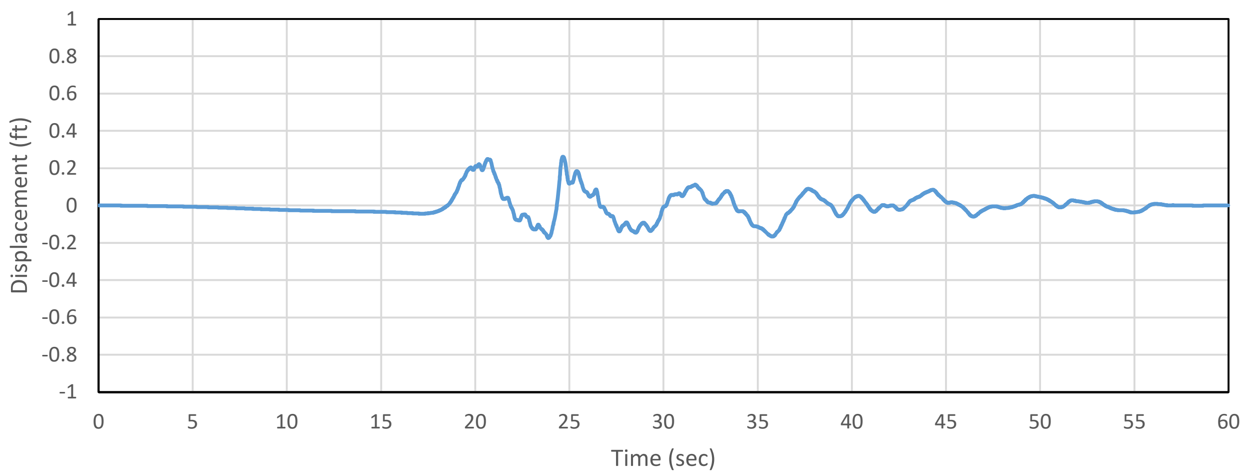
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-30a

RSN 4866
Station: Kawanishi Izumozaki, Earthquake: Chuetsu-oki, FN



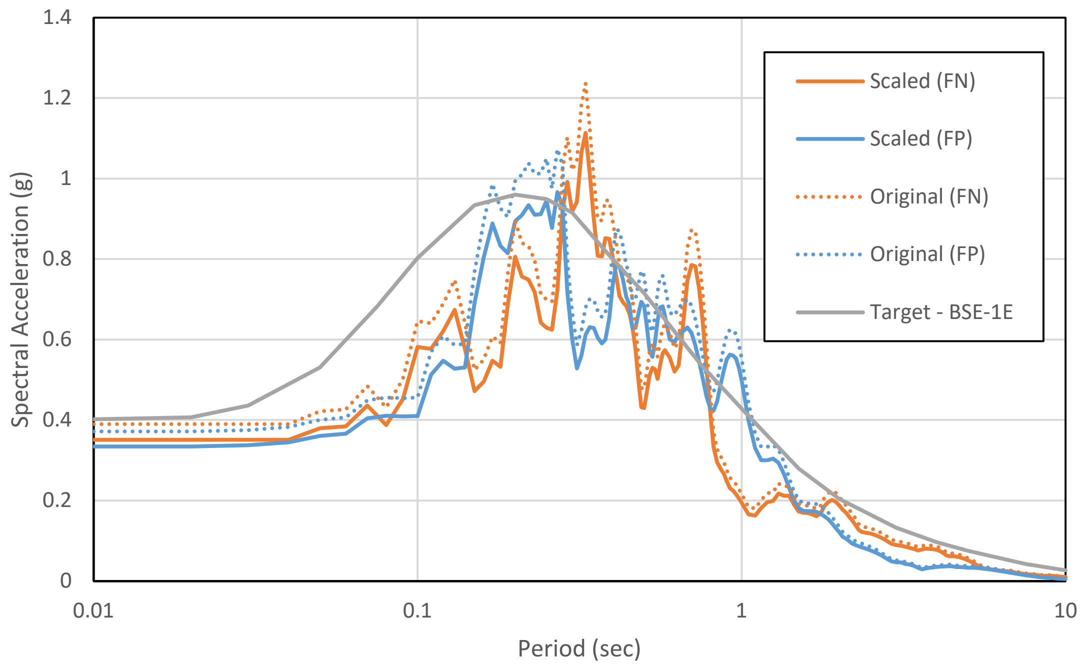


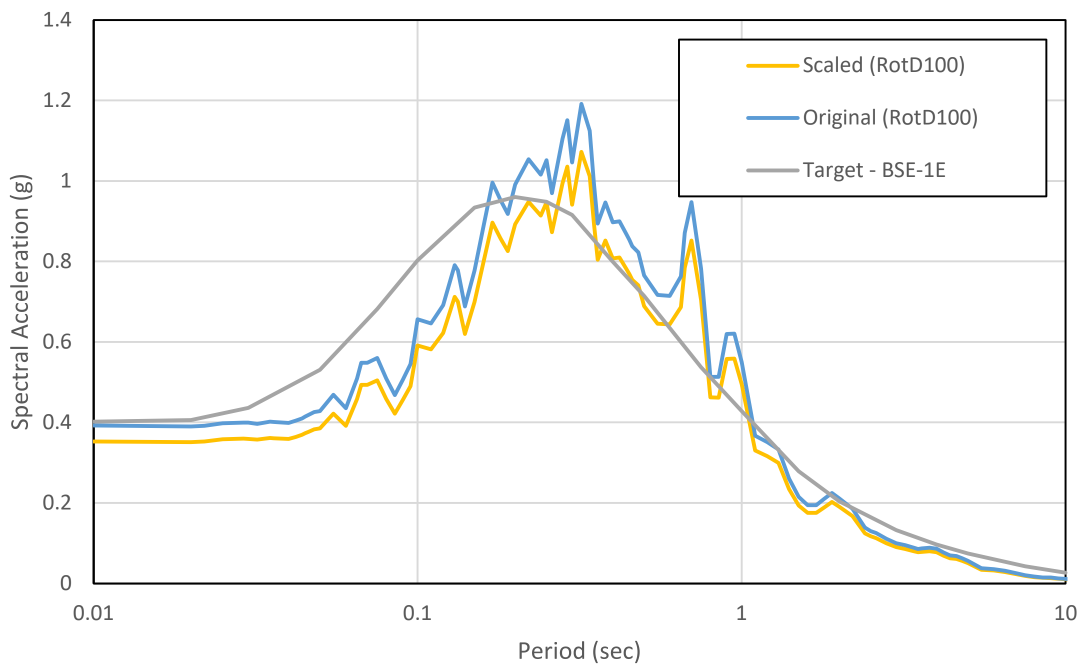


GROUP DELTA	Scaled Time Histories - RSN 4866 (FP)		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-30b**

Response Spectra (5% damping)

Station: Kawanishi Izumozaki, Earthquake: Chuetsu-oki, FN

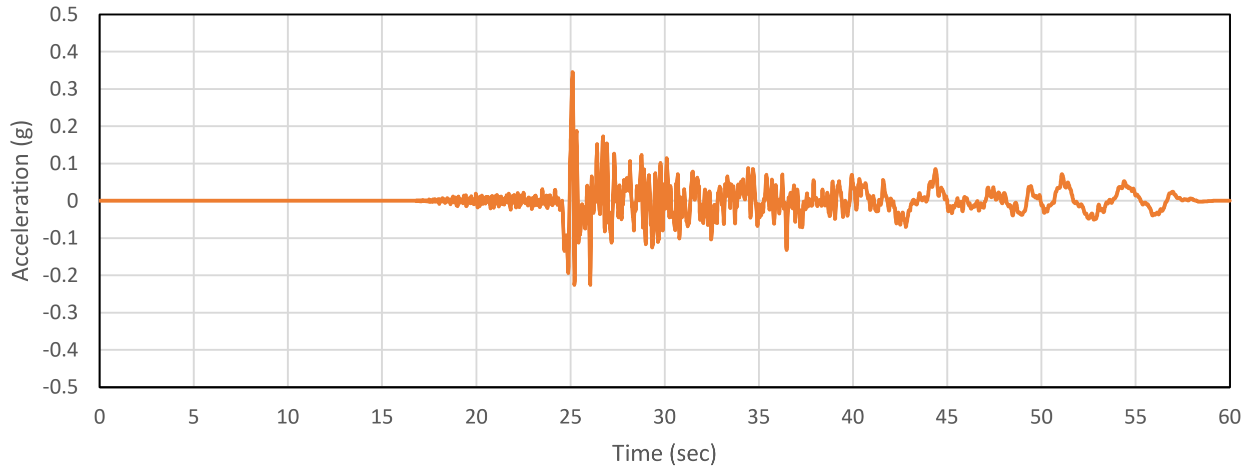


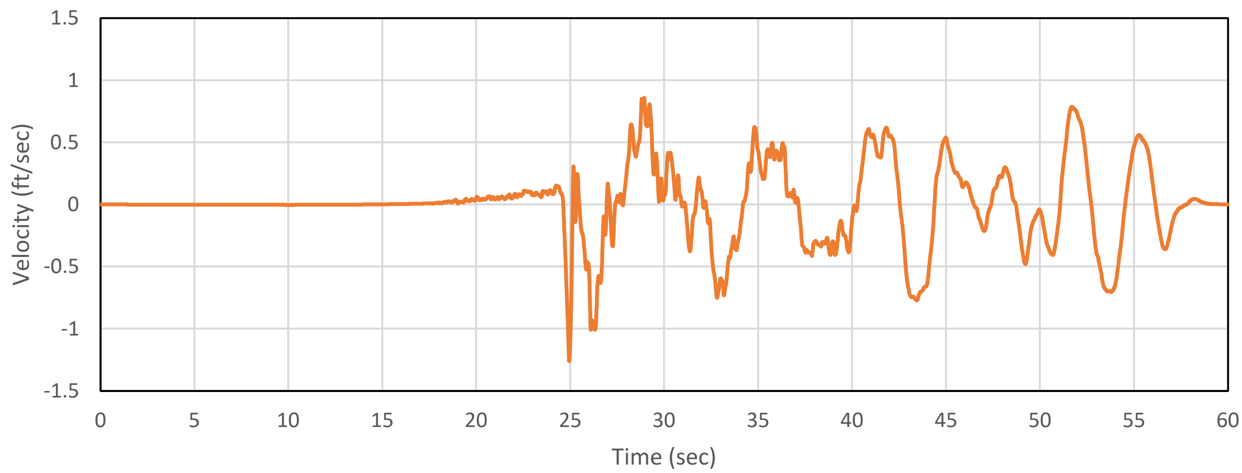


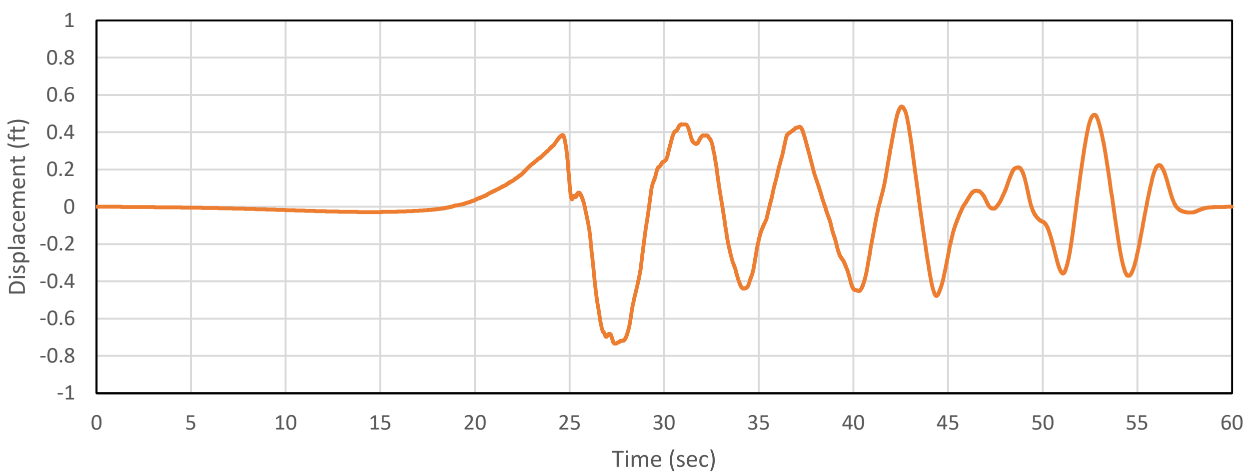
GROUP DELTA	**Response Spectra - RSN 4866**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-30c**

RSN 5779

Station: Sanbongi Osaki City, Earthquake: Iwate







GROUP DELTA

Scaled Time Histories - RSN 5779 (FN)

Project Number: LA-1474

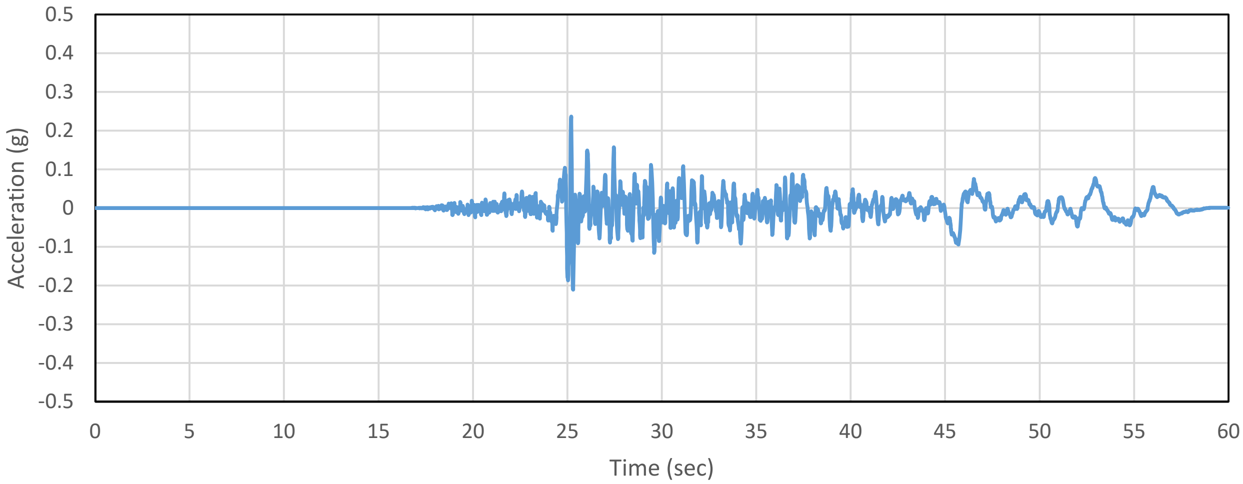
Date: 12/18/2020

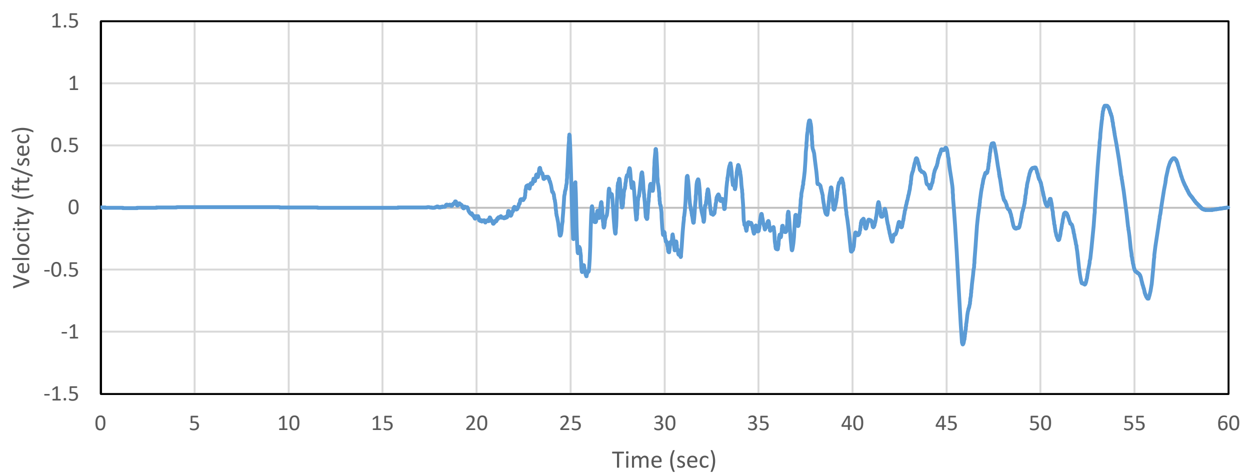
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

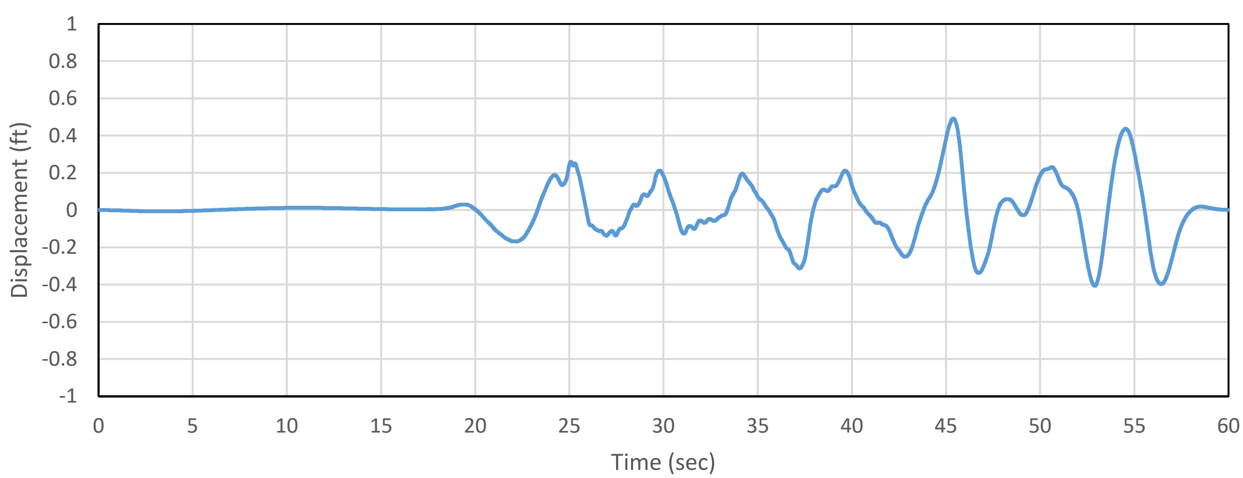
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-31a

RSN 5779
Station: Sanbongi Osaki City, Earthquake: Iwate



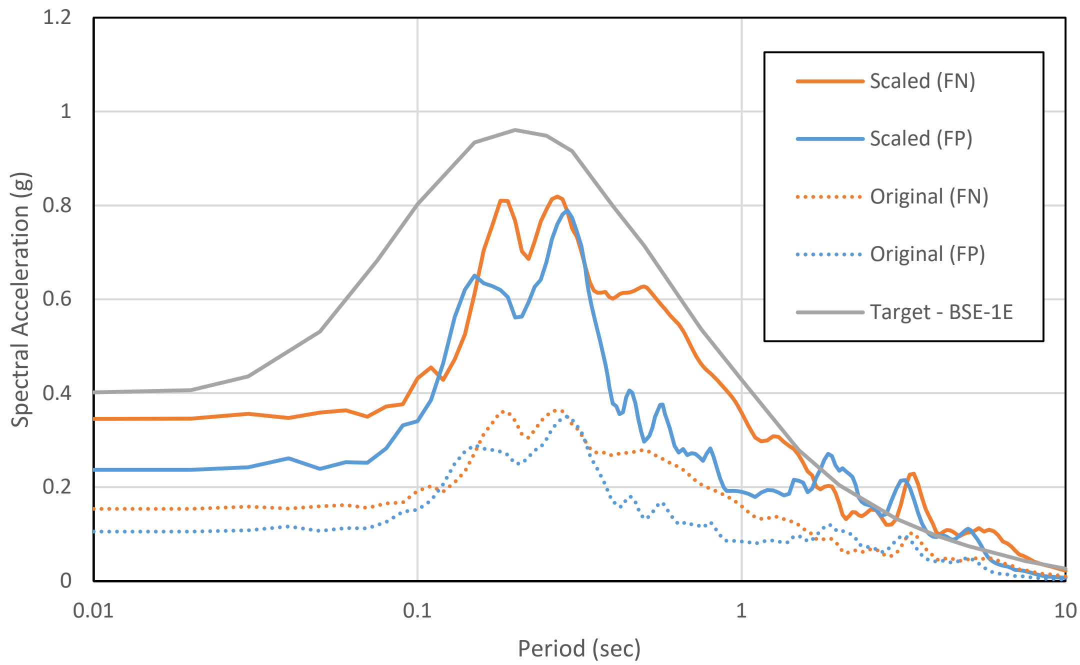


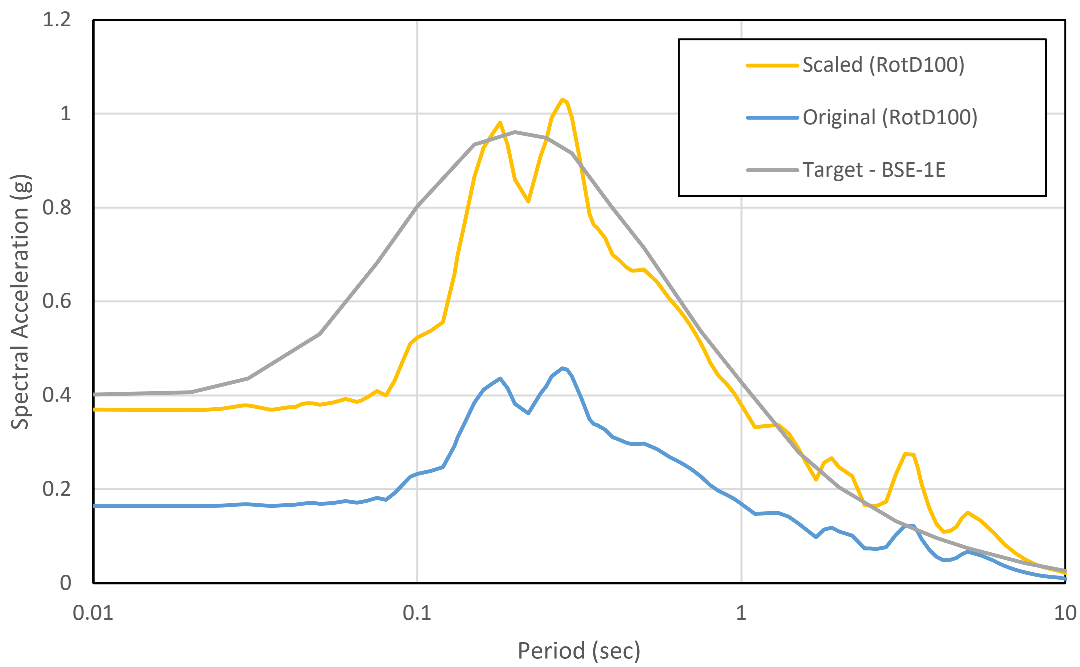


GROUP DELTA	Scaled Time Histories - RSN 5779 (FP)		Project Number: LA-1474 Date: 12/18/2020
	GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-31b**

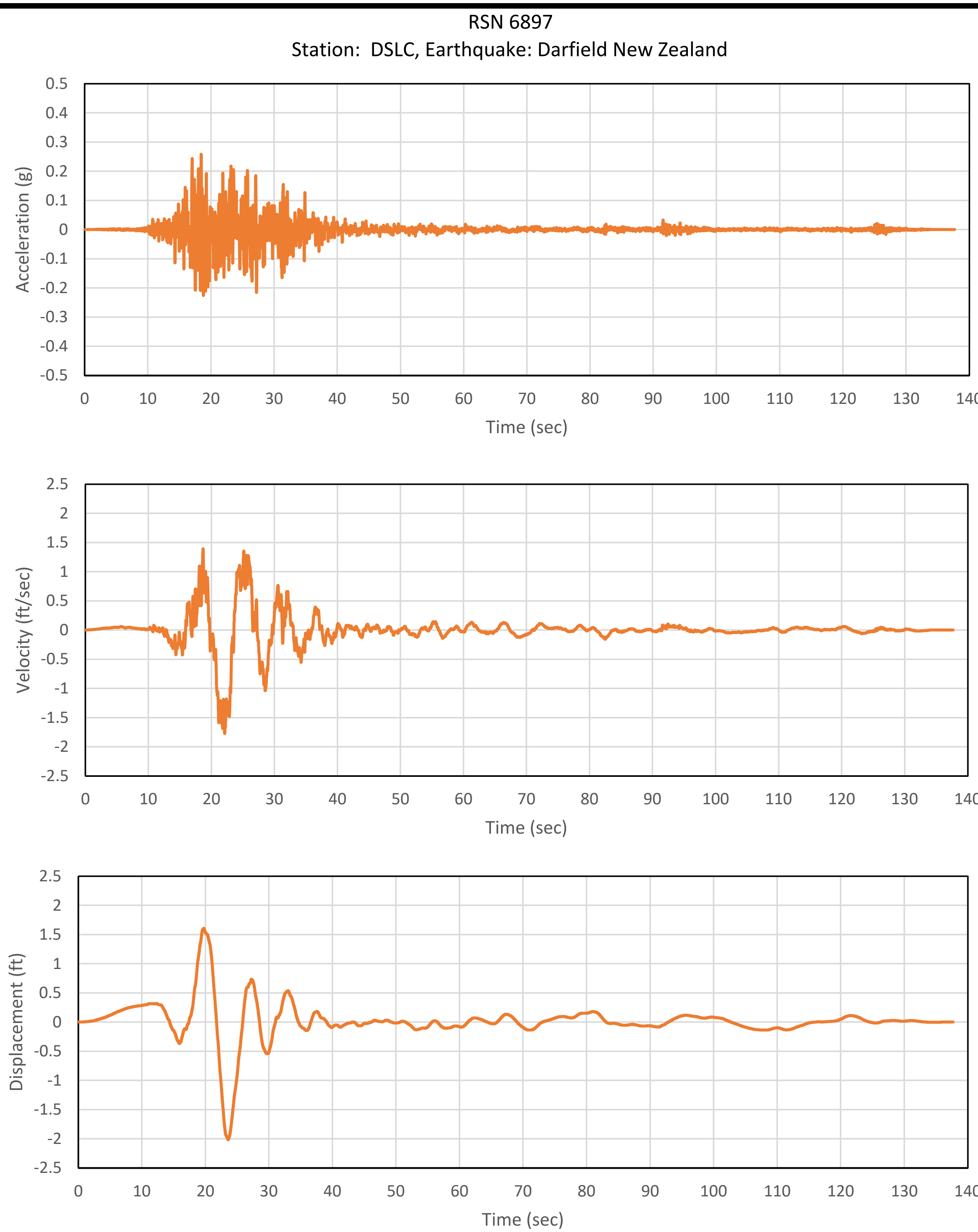
Response Spectra (5% damping)

Station: Sanbongi Osaki City, Earthquake: Iwate





GROUP DELTA	**Response Spectra - RSN 5779**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-31c**



GROUP DELTA

Scaled Time Histories - RSN 6897 (FN)

Project Number: LA-1474

Date: 12/18/2020

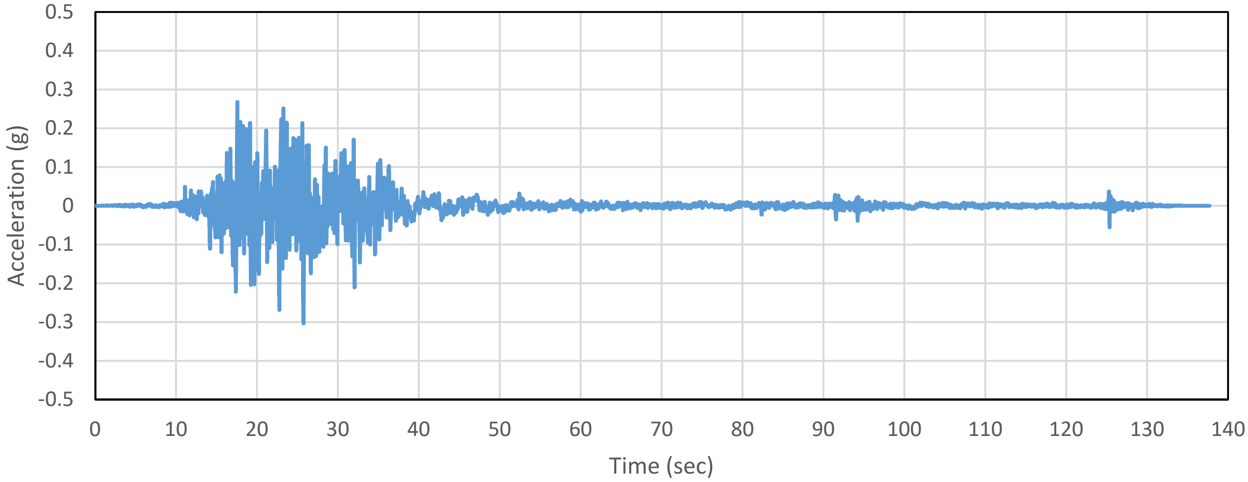
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

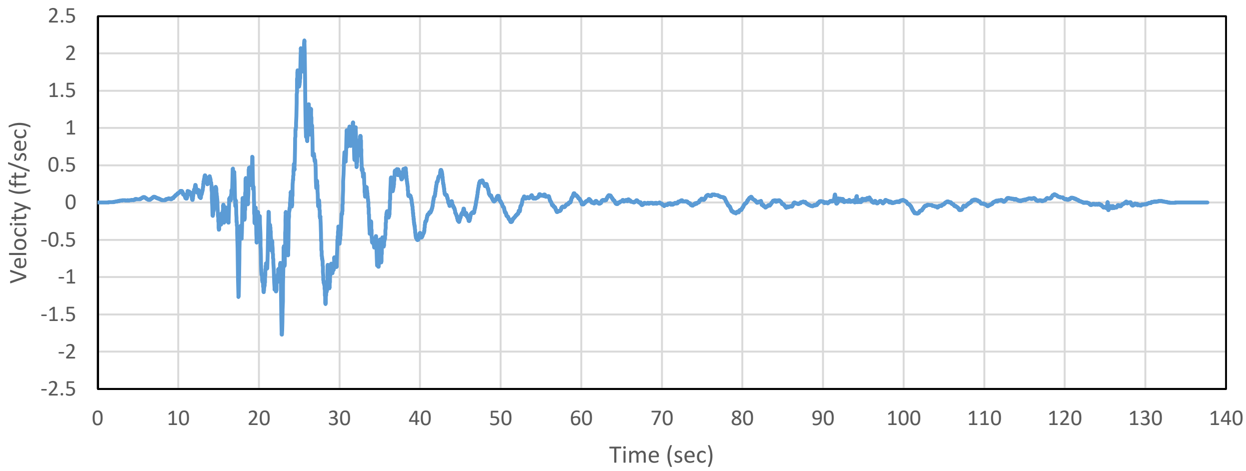
PROJECT NAME:
City of Inglewood Seismic Retrofits

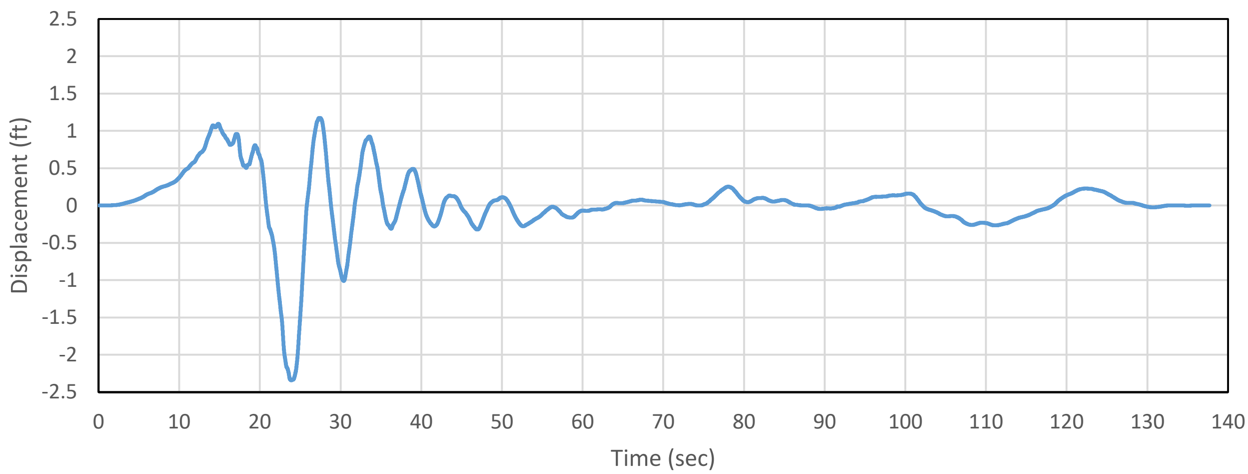
Figure C-32a

RSN 6897

Station: DSLC, Earthquake: Darfield New Zealand







GROUP DELTA

Scaled Time Histories - RSN 6897 (FP)

Project Number: LA-1474

Date: 12/18/2020

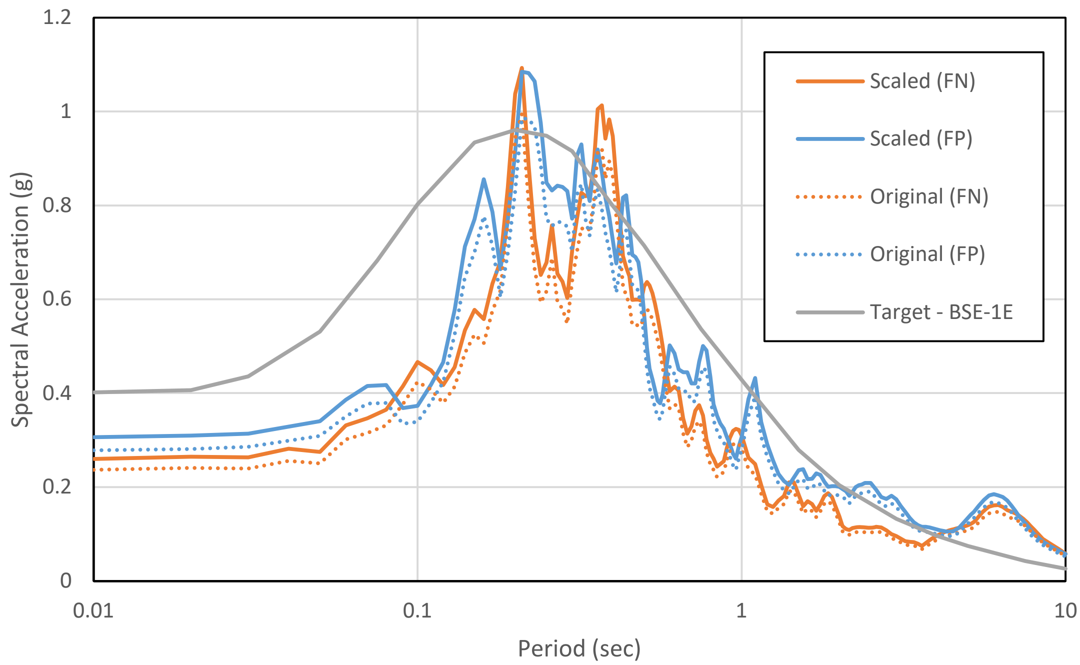
GROUP DELTA CONSULTANTS, INC.
370 Amapola Ave., Suite 212
Torrance, CA 90501

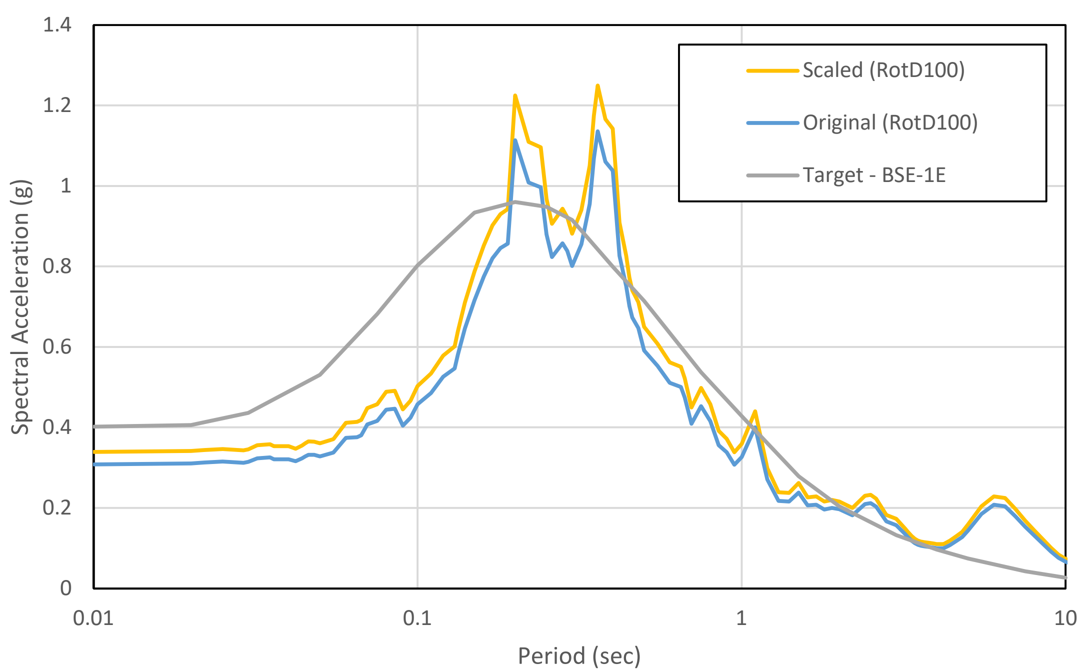
PROJECT NAME:
City of Inglewood Seismic Retrofits

Figure C-32b

Response Spectra (5% damping)

Station: DSLC, Earthquake: Darfield New Zealand





GROUP DELTA	**Response Spectra - RSN 6897**	Project Number: LA-1474 Date: 12/18/2020
GROUP DELTA CONSULTANTS, INC. 370 Amapola Ave., Suite 212 Torrance, CA 90501	**PROJECT NAME:** **City of Inglewood Seismic Retrofits**	**Figure C-32c**

APPENDIX D

LIMITED FAULT DISPLACEMENT HAZARD EVALUATION

D.1 Introduction

Group Delta performed a limited evaluation of the potential displacements associated with surface fault rupture from the Newport-Inglewood Fault for the City of Inglewood Civic Center. A simplified probabilistic evaluation and a deterministic evaluation of fault displacements were performed for the Newport-Inglewood Fault that is mapped at the site. The purpose of this evaluation is to support the City of Inglewood's understanding of this risk.

Details of the assumptions, methods, and fault parameters used in the analyses are described in the following sections.

D.2 Evaluation of Fault Displacement Hazard Calculation

The mapped trace of the Newport-Inglewood fault at the site is part of the north Los Angeles Basin section according to the USGS Quaternary Fault Database. Fault geometry and parameters were adopted from the Uniform Earthquake Rupture Forecast, Version 3 (UCERF3) (Field et al., 2013) for the Newport-Inglewood Alternative 1 fault, which is the fault adopted for the development of the National Seismic Hazard Maps (NSHM) in 2014 (Petersen et al., 2014). According to UCERF3, the best estimate slip rate for the Newport-Inglewood fault is 1 mm/year, and the site-specific geologic slip rate data at the nearby Inglewood Oil Fields is reported to be about 0.6 mm/year. The following parameters adopted by UCERF3 for the Newport-Inglewood Alt. 1 fault, listed in Table D-1, were also adopted for the fault displacement hazard evaluation.

Table D-1: Newport-Inglewood Alt. 1 Fault Geometry and Parameters

Parameter	Value
Slip Rate (mm/year)	1.0
Fault Width (km)	15
Fault Length (km)	65.4
Fault Dip (degrees)	88
Aseismic Factor	0.1

The procedure for a simplified probabilistic evaluation of surface fault rupture hazard developed by Caltrans (Shantz, 2013) was used to develop a fault displacement hazard curve for the Newport-Inglewood fault. As recommended by the Caltrans procedure, the magnitude-area relationship by Hanks and Bakun (2008) was used to estimate the characteristic earthquake magnitude. The proposed relationship for average fault displacement for strike-slip earthquakes by Wells and Coppersmith (1994) was also used. The resulting recurrence interval for the Newport-Inglewood fault is 1663.3 years. The fault displacement hazard curve developed following the Caltrans procedure is provided in Figure D-1. Based on this methodology, the fault displacement at the BSE-1E and BSE-2E hazard levels (225-year and 975-year return periods, respectively) is negligible.

Note that the deterministic evaluation of average fault displacement using the Wells and Coppersmith (1994) relationship estimates about 0.95 meters (about 3.1 feet).

D.3 Limitations

This limited fault displacement hazard evaluation is for information only, and not intended for design purposes. If structural mitigation for the surface fault rupture hazard is desired, a more comprehensive study, such as additional field investigation and site-specific fault displacement evaluation, may be required.

D.4 References

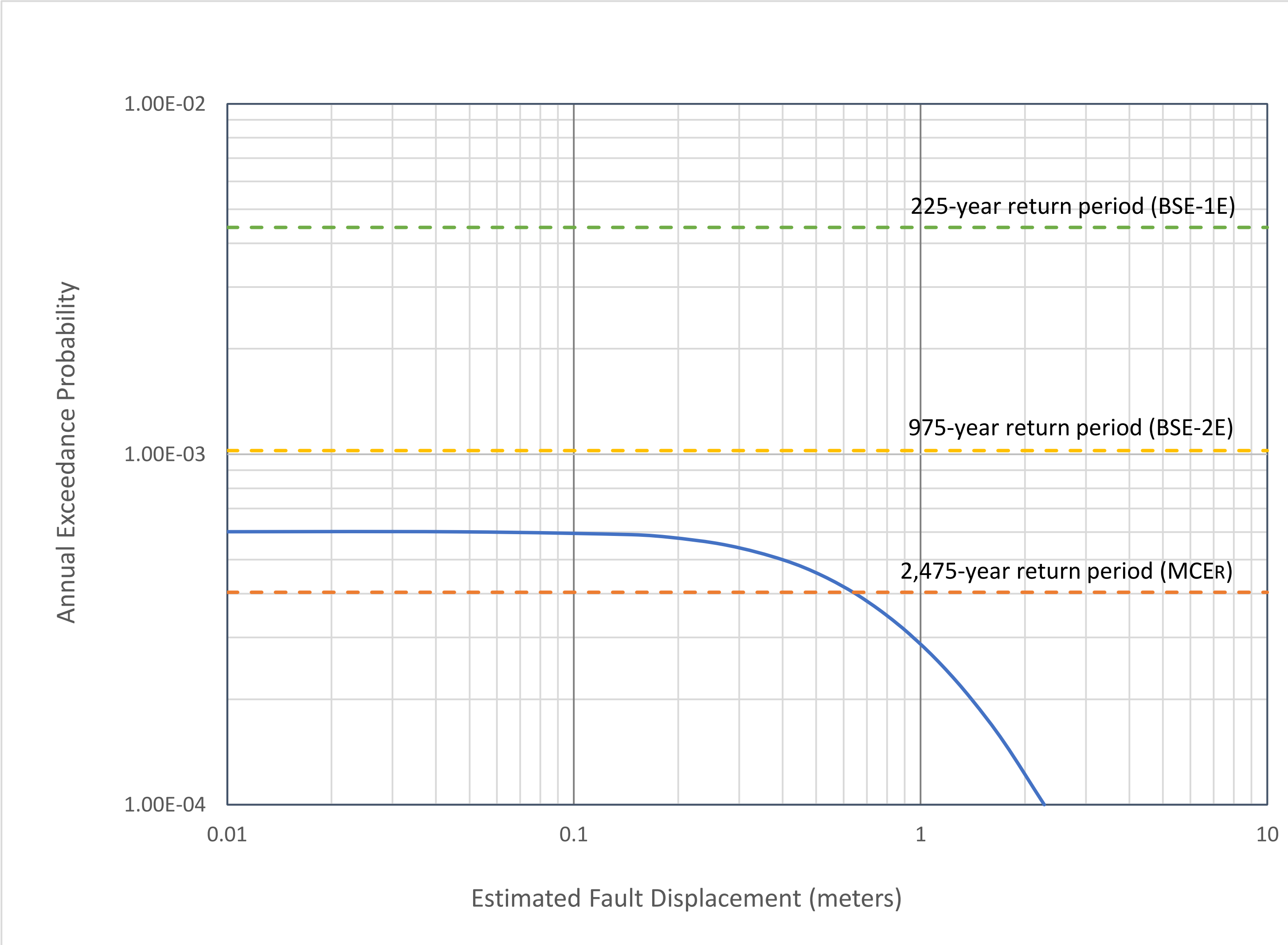
Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., and Zeng, Y. (2013). *Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) – The Time-Independent Model*, U.S. Geological Survey Open-File Report 2013-1165, U.S. Geological Survey, Reston, Virginia, 2013.

Hanks, T.C., and Bakun, W.H. (2008). "M-logA Observations for Recent Large Earthquakes", *Bulletin of the Seismological Society of America*, February 2008, V. 98, No. 1, p. 490-494.

Petersen, M.D., M. P. Moschetti, P. M. Powers, C. S. Mueller, K. M. Haller, A. D. Frankel, Y. Zeng, S. Rezaeian, S. C. Harmsen, O. S. Boyd, N. Field, R. Chen, K. S. Rukstales, N. Luco, R. L. Wheeler, R. A. Williams, and A. H. Olsen (2014). Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014–1091, 243 p., http://dx.doi.org/10.3133/ofr20141091 .

Shantz, Tom (2013). Caltrans Procedures for Calculation of Fault Rupture Hazard. Caltrans Division of Research and Innovation, February 2013.

Wells, D.L., Coppersmith, K.J. (1994). "New Empirical Relationships among Magnitude, Rupture Length, Rupture Width, Rupture Area, and Surface Displacement", *Bulletin of the Seismological Society of America*, v. 84, No. 4, p. 974-1002.



PROJECT NAME	FIGURE NAME	GROUP DELTA	
City of Inglewood Seismic Retrofits **1 W Manchester Blvd, Inglewood, CA 90301**	**Fault Displacement Hazard Curve**	PROJECT NUMBER LA-1474	FIGURE NUMBER D-1