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**TRANSPORTATION IMPACT ANALYSIS**

# FAIRFIELD MARRIOTT

**San Diego, California**
**March 19, 2018**

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# EXECUTIVE SUMMARY

This Transportation Impact Analysis has been prepared for the Fairfield Marriott Hotel project. Currently, the site is occupied by the 46-room Rodeway Inn and a previous business that has been vacant since March 2017. The project proposes the demolition of the existing hotel and the other building to construct a 112-room[1] hotel, yielding a net increase of 66 rooms. The project site is located at 4345 Mission Bay Drive at the southeast corner of Mission Bay Drive and Glendora Street in the City of San Diego. Access to the Project site will be provided via the two existing, unsignalized driveways: one full-access located on Glendora Street and a right-in / right-out driveway on Mission Bay Drive.

Several intersections and roadway segments within the study area were analyzed to determine project related transportation impacts. Existing weekday AM and PM peak hour traffic volumes were collected at key area intersections and segments to capture peak commuter activity. The AM and PM peak hour manual turning movement counts and daily counts were conducted on Wednesday, April 19, 2017 when area schools were in session.

The proposed project is calculated to generate 594 new net daily trips[2], with 48 AM peak hour trips (19 inbound, 29 outbound) and 54 PM peak hour trips (21 inbound, 33 outbound).

LLG coordinated with City staff and reviewed other planned projects in the vicinity. One (1) cumulative project was identified in the immediate project vicinity and included in the traffic analysis.

The project is calculated with a significant direct and cumulative impact at Mission Bay Drive / Rosewood Street intersection under the Existing + Project, Near-Term (Opening Day 2020) + Project and Year 2035 (Horizon Year) + Project scenarios. Mitigation measures are explained in detail in *Section 11.0.*

According to the *City of San Diego Municipal Code, Chapter 14, Article 2, Division 5, Page 27, Table 142-05G*, “visitor accommodations” (i.e. hotel rooms) require 1 parking space per guest room. The project proposes 106 rooms, requiring 106 parking spaces. The project proposes to provide 106 onsite parking spaces, meeting the City’s minimum requirement.

[1] Since the preparation of the TIA, the project description was revised to include a total of 106 rooms, yielding a net increase of 60 rooms. However, given that the reduction in room count was minor, the analysis was left unchanged as the transportation impact analysis was based on a higher room count of 112 rooms. Therefore, the findings and conclusions presented in this study are conservative.

[2] The trip generation and impact analyses in this study are based on a higher room count of 112 rooms and therefore, the findings and conclusions presented in this study are conservative.

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# 1.0 INTRODUCTION

This Transportation Impact Analysis has been prepared for the Fairfield Marriott Hotel project. Currently, the site is occupied by the 46-room Rodeway Inn and a previous business that has been vacant since March 2017. The project proposes the demolition of the existing hotel and the other building to construct a 112-room[3] hotel, yielding a net increase of 66 rooms. The project site is located at 4345 Mission Bay Drive at the southeast corner of Mission Bay Drive and Glendora Street in the City of San Diego.

Several intersections and roadway segments within the study area were analyzed to determine project related transportation impacts, as set forth in the following sections. The following items are included in this report:

- Project Description
- Existing Conditions
- Analysis Approach and Methodology
- Significance Criteria
- Analysis of Existing Conditions
- Project Trip Generation / Distribution / Assignment
- Cumulative Projects
- Near-Term Analysis
- Long-Term Analysis
- Access Analysis
- Alternative Modes
- Parking
- Signal Warrants
- Significance of Impacts and Conclusions

***Figure 1-1*** shows the vicinity map. ***Figure 1-2*** depicts a more detailed project area map.

[3] Since the preparation of the TIA, the project description was revised to include a total of 106 rooms, yielding a net increase of 60 rooms. However, given that the reduction in room count was minor, the transportation impact analysis was left unchanged as it was based on a higher room count of 112 rooms. Therefore, the findings and conclusions presented in this study are conservative.

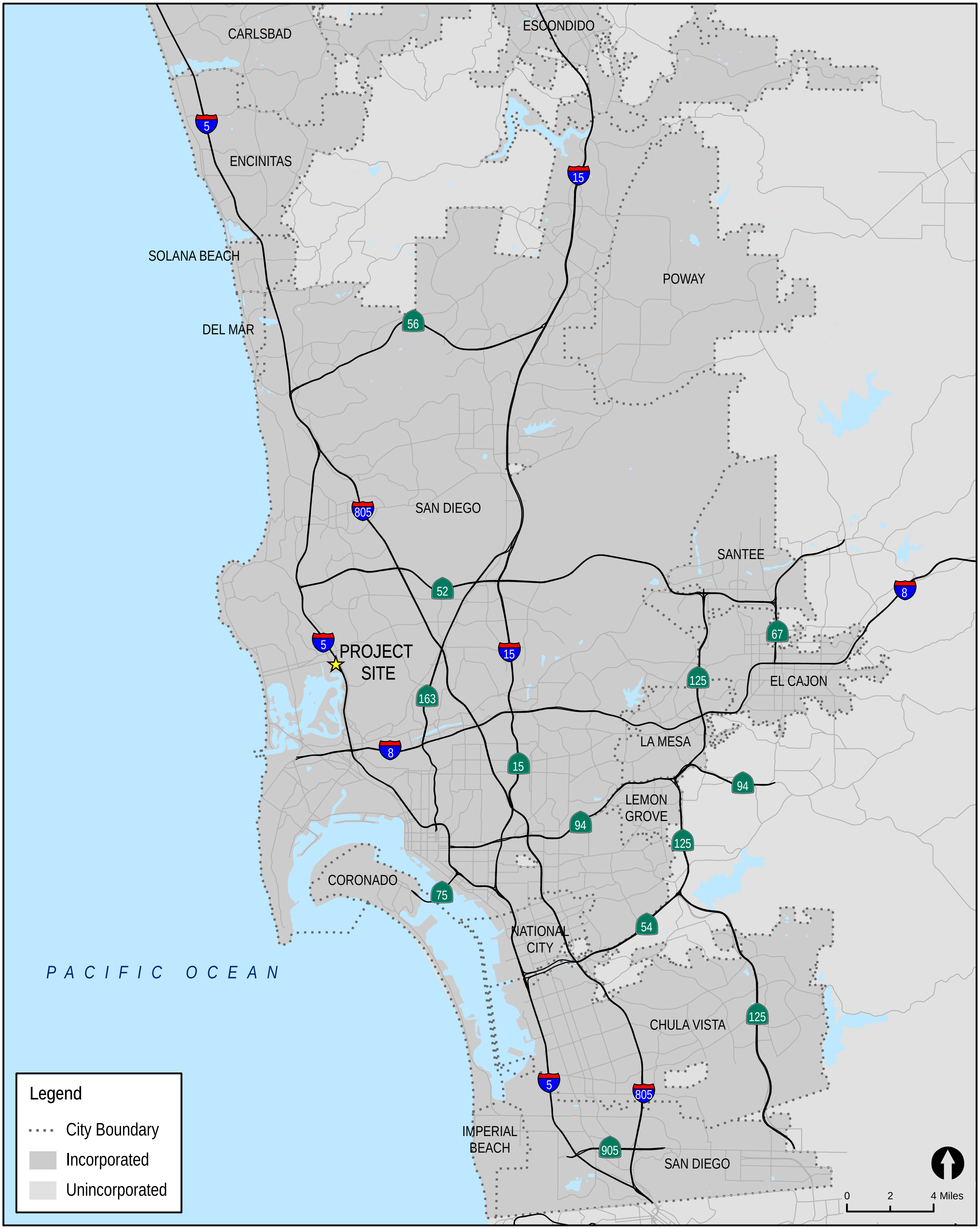


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**Figure 1-1**

## Vicinity Map

FAIRFIELD MARRIOTT

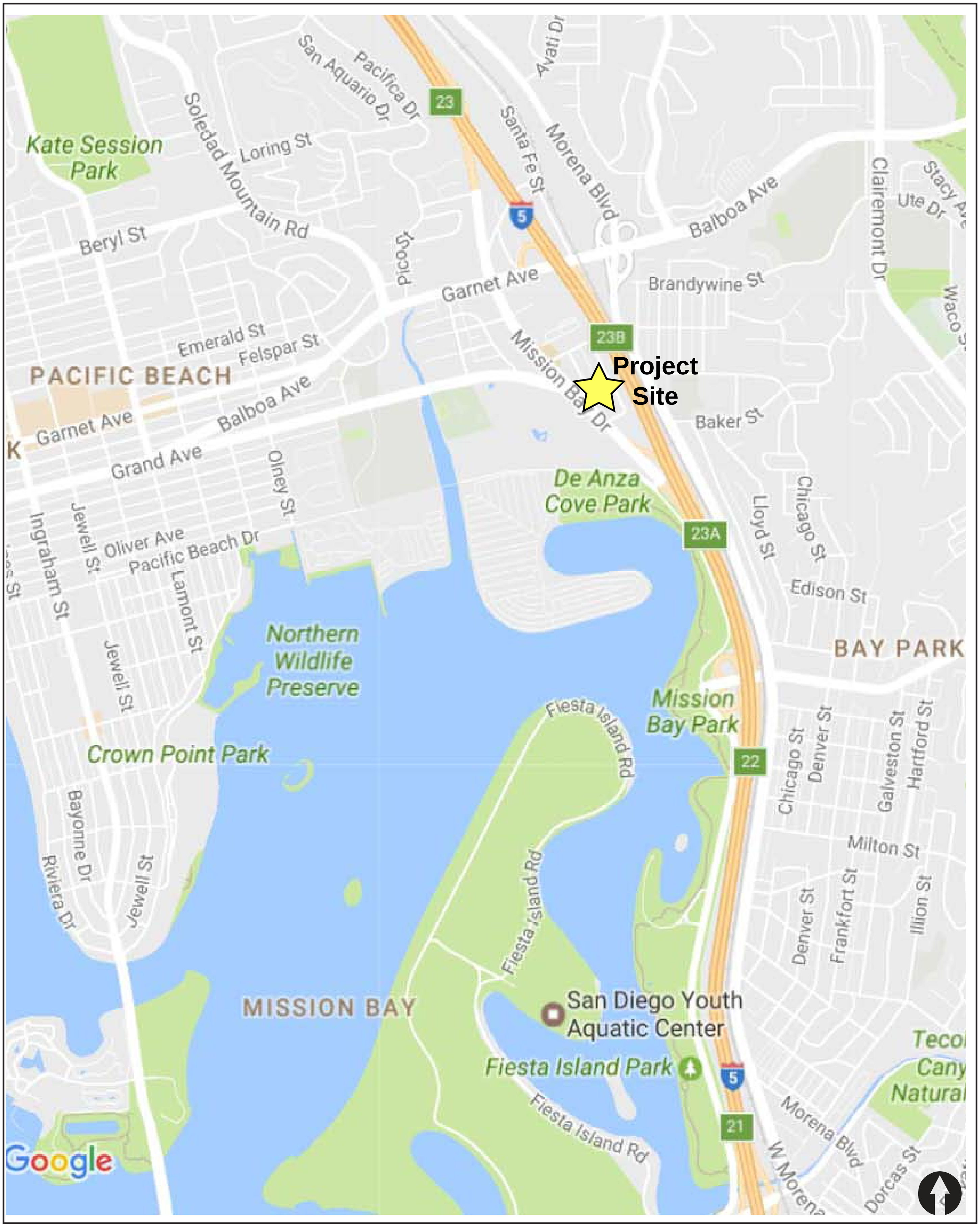


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**Figure 1-2**

**Project Area Map**

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# 2.0 PROJECT DESCRIPTION

## 2.1 Project Location

The project site is located at 4345 Mission Bay Drive at the southeast corner of Mission Bay Drive and Glendora Street in the City of San Diego.

## 2.2 Project Description

Currently, the site is occupied by the 46-room Rodeway Inn and a previous business that has been vacant since March 2017. The project proposes the demolition of the existing hotel to construct a 112-room hotel[4], yielding a net increase of 66 rooms.

## 2.3 Project Access

Access to the Project site will be provided via the two existing, unsignalized driveways: one full-access located on Glendora Street and a right-in / right-out driveway on Mission Bay Drive.

***Figure 2–1*** shows a conceptual site plan.

[4] Since the preparation of the TIA, the project description was revised to include a total of 106 rooms, yielding a net increase of 60 rooms. However, given that the reduction in room count was minor, the transportation impact analysis was left unchanged as it was based on a higher room count of 112 rooms. Therefore, the findings and conclusions presented in this study are conservative.

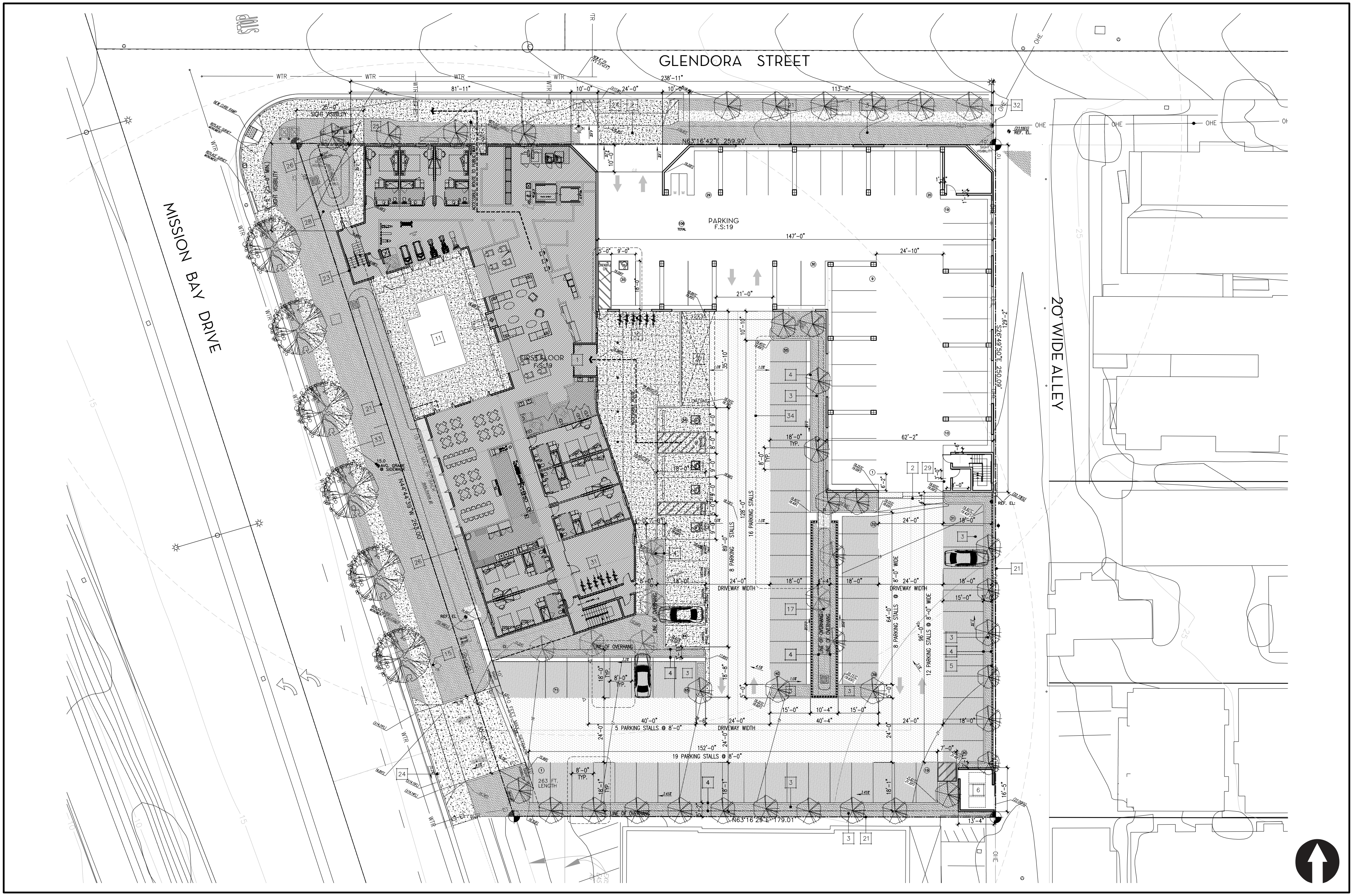


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**Figure 2-1**

## Site Plan

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# 3.0 EXISTING CONDITIONS

Effective evaluation of the traffic impacts associated with the proposed project requires an understanding of the existing transportation system within the project area.

## 3.1 Study Area

Based on discussions with City staff, the following study area was determined using the City's 50 peak hour trip threshold as a guideline. No freeway segments or ramp meters were analyzed as a part of this study as the project does not add 150 peak hour trips to the freeway segments or 20 peak hour trips to the ramp meters in the project area:

1. Mission Bay Drive / Garnet Avenue
2. Mission Bay Drive / Bunker Hill Street
3. Mission Bay Drive / Grand Avenue
4. Mission Bay Drive / Glendora Street
5. Mission Bay Drive / Project Driveway A
6. Mission Bay Drive / Rosewood Street
7. Mission Bay Drive / I-5 SB On-Ramp
8. Glendora Street / Project Driveway B

**Street Segments**

The specific study area includes the following street segments:

*Grand Avenue*

- Figueroa to Mission Bay Drive

*Mission Bay Drive*

- Garnet Avenue to Bunker Hill Street
- Bunker Hill Street to Grand Avenue
- Grand Avenue to Rosewood Street
- Rosewood Street to N. Mission Bay Drive

*Revere Avenue*

- Bunker Hill Street to Glendora Street

*Bunker Hill Street*

- Mission Bay Drive to Del Rey Street

*Glendora Street*

- Mission Bay Drive to Revere Avenue

*Rosewood Street*

- Mission Bay Drive to Del Rey Street

## 3.2 Existing Roadway Conditions

The following is a description of the roadways in the project area. ***Figure 3–1*** illustrates the existing street network.

**Grand Avenue** is currently built as a 4-lane Major Arterial between Balboa Avenue and Figueroa Boulevard; and built as a 3-lane road between Figueroa Boulevard and Mission Bay Drive. According to the current Pacific Beach Community Plan, Grand Avenue is classified as 4-lane Major Arterial between Balboa Avenue and Mission Bay Drive. The posted speed limit is 45 mph. Bike lanes are provided on both sides, and on-street parking is prohibited in the project vicinity.

**Mission Bay Drive** is currently built as a 4-lane Major Arterial between Garnet Avenue and Glendora Street; and as a 5-lane Major Arterial (3 northbound lanes, 2 southbound lanes) between Glendora Avenue and I-5. According to the current Pacific Beach Community Plan, Mission Bay Drive is classified as a 4-lane Major Arterial from Garnet Avenue to I-5. It has a raised median with intermittent on-street parking on both sides. Bike lanes are not provided. Between Garnet Avenue and Grand Avenue, the posted speed limit on Mission Bay Drive is 35 mph. From Rosewood to the I-5 Ramps, the posted speed limit is 45 mph.

**Revere Avenue** is currently built as a 2-lane local roadway. It is unclassified in the Pacific Beach Community Plan. On-street parking is provided on both sides of the street. Bike lanes are not provided.

**Del Rey Street** is currently built as a 2-lane local roadway. It is unclassified in the Pacific Beach Community Plan. On-street parking is provided on both sides of the street. Bike lanes are not provided.

**Bunker Hill Street** is currently built as a 2-lane local roadway. It is unclassified in the Pacific Beach Community Plan. On-street parking is provided on both sides of the street. Bike lanes are not provided.

**Glendora Street** is currently built as a 2-lane local roadway. It is unclassified in the Pacific Beach Community Plan. On-street parking is provided on both sides of the street. Bike lanes are not provided.

**Rosewood Street** is currently built as a 2-lane local roadway. It is unclassified in the Pacific Beach Community Plan. A curve warning sign of 15mph is posted on Rosewood Street. On-street parking is provided on both sides of the street. Bike lanes are not provided.

## 3.3 Existing Traffic Volumes

Existing weekday AM and PM peak hour traffic volumes were collected at key area intersections to capture peak commuter activity. The AM and PM peak hour manual turning movement counts and daily counts were conducted on Wednesday, April 19, 2017 when area schools were in session.

***Table 3–1*** is a summary of the most recent available average daily traffic volumes (ADTs). ***Figure 3–2*** shows the existing AM and PM peak hour turning movement volumes and ADT volumes. ***Appendix A*** contains the manual count sheets.

TABLE 3–1
EXISTING SEGMENT VOLUMES

| Street Segment | ADT[a] |
|---|---|
| **Grand Avenue** | |
| Figueroa Boulevard to Mission Bay Drive | 40,860 |
| **Mission Bay Drive** | |
| Garnet Avenue to Bunker Hill Street | 31,210 |
| Bunker Hill Street to Grand Avenue | 31,360 |
| Grand Avenue to Rosewood Street | 60,410 |
| Rosewood Street to N. Mission Bay Drive | 60,430 |
| **Revere Avenue** | |
| Bunker Hill Street to Glendora Street | 750 |
| **Bunker Hill Street** | |
| Mission Bay Drive to Del Rey Street | 3,680 |
| **Glendora Street** | |
| Mission Bay Drive to Revere Avenue | 590 |
| **Rosewood Street** | |
| Mission Bay Drive to Del Rey Street | 780 |

***Footnotes:***

a. Average Daily Traffic Volumes.

***General notes:***

1. Traffic counts conducted on Wednesday, April 19, 2017.

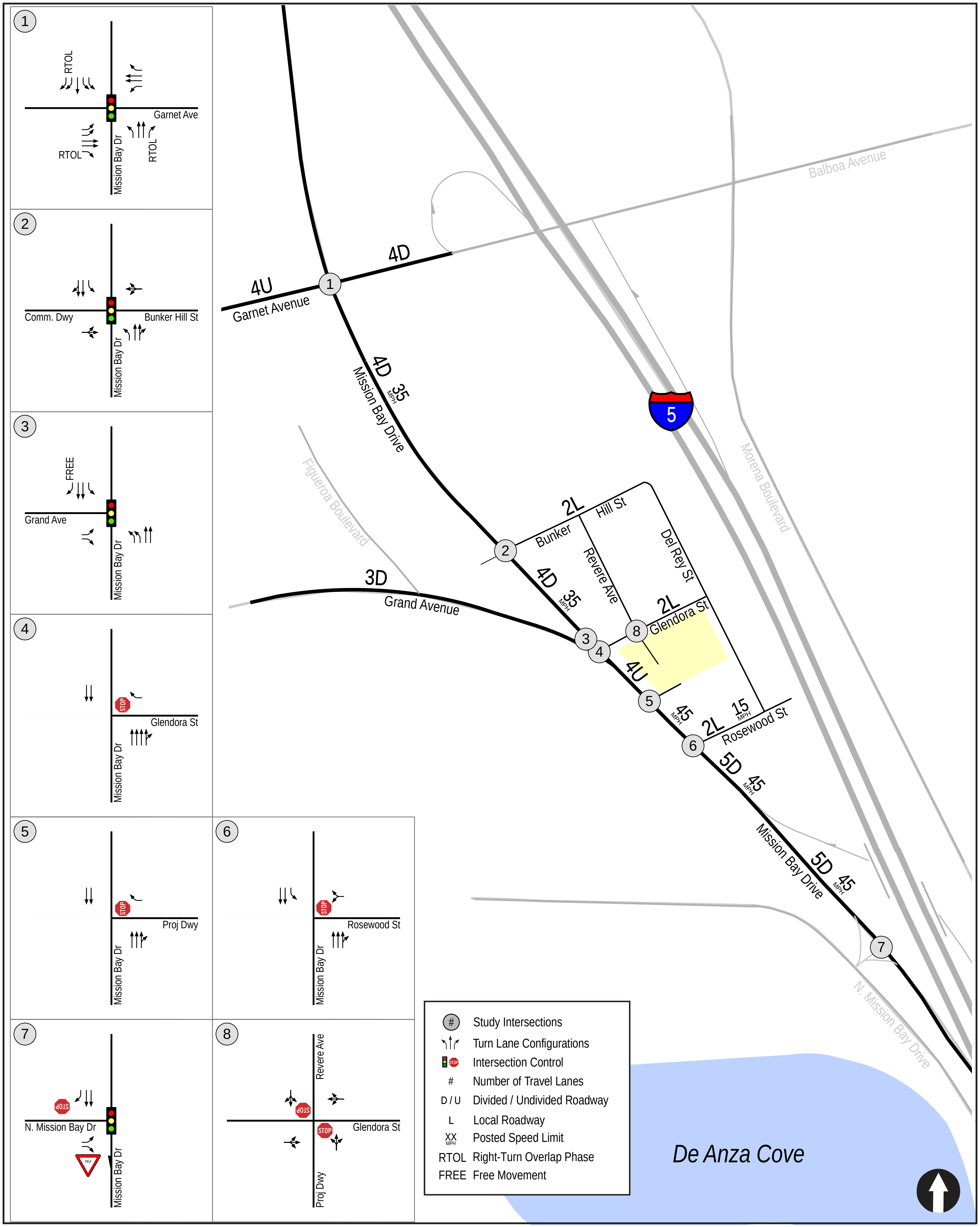


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**Figure 3-1**

# Existing Conditions Diagram

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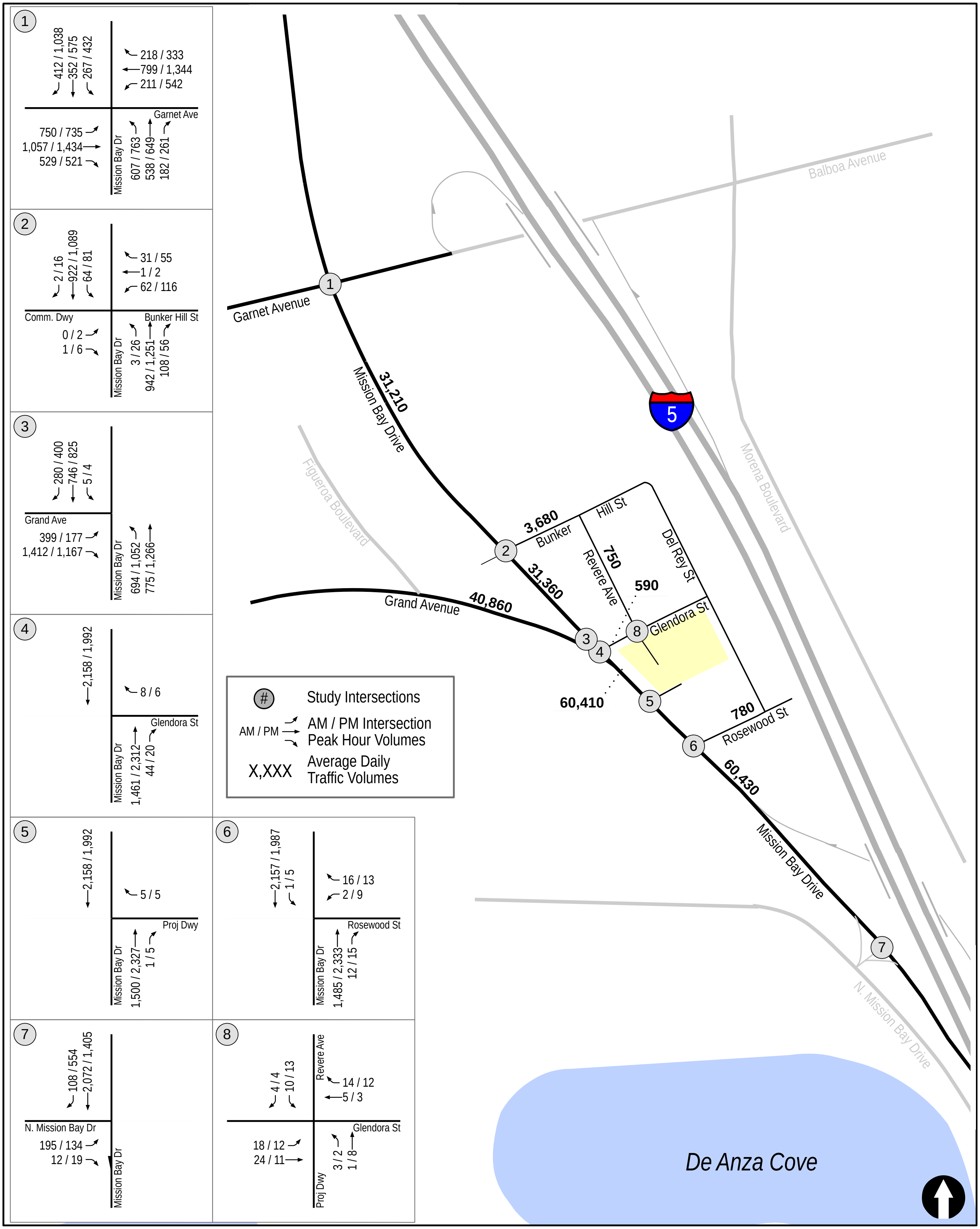


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**Figure 3-2**

# Existing Traffic Volumes

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# 4.0 ANALYSIS APPROACH AND METHODOLOGY

There are various methodologies used to analyze signalized intersections, unsignalized intersections, and street segments. The measure of effectiveness for intersection and segment operations is level of service (LOS), which denotes the operating conditions which occur at a given intersection or on a given roadway segment under various traffic volume loads.

LOS is a qualitative measure used to describe a quantitative analysis taking into account factors such as roadway geometries, signal phasing, speed, travel delay, freedom to maneuver, and safety. Level of service provides an index to the operational qualities of a roadway segment or an intersection. Levels of service designations range from A to F, with LOS A representing the best operating conditions and LOS F representing the worst. Level of service designation is reported differently for signalized and unsignalized intersections, as well as for roadway segments.

In the 2010 Highway Capacity Manual (HCM), Level of Service for signalized intersections is defined in terms of delay. The level of service analysis results in seconds of delay expressed in terms of letters A through F. Delay is a measure of driver discomfort, frustration, fuel consumption, and lost travel time.

There are three types of analyses conducted in this study:

- Signalized Intersections
- Unsignalized Intersections
- Street Segments (ADTs)

## 4.1 Intersections

***Table 4–1*** summarizes the signalized intersections levels of service descriptions. ***Table 4–2*** depicts the intersection LOS and corresponding delay ranges, which are based on overall intersection delay (signalized intersections) and the average control delay for any particular minor movement (unsignalized intersections), respectively. LOS relative to signalized and unsignalized intersection is further described below. Signal Timing plans are included in ***Appendix C***.

### 4.1.1 *Signalized Intersections*

Signalized intersections were analyzed under AM and PM peak hour conditions. Average vehicle delay was determined utilizing the methodology found in Chapter 18 of the *2010 Highway Capacity Manual (HCM),* with the assistance of the *Synchro* (version 10) computer software. The delay values (represented in seconds) were qualified with a corresponding intersection LOS. Signalized intersection calculation worksheets and a more detailed explanation of the methodology are attached in ***Appendix B***.

### 4.1.2 *Unsignalized Intersections*

Unsignalized intersections were analyzed under AM and PM peak hour conditions. Average vehicle delay and LOS was determined based upon the procedures found in Chapter 19 and 20 of the *2010 HCM,* with the assistance of the *Synchro* (version 10) computer software.

## TABLE 4–1
## INTERSECTION LEVEL OF SERVICE DESCRIPTIONS

| Level of Service | Description |
|---|---|
| A | Occurs when progression is extremely favorable and most vehicles arrive during the green phase. Most vehicles do not stop at all. Short cycle lengths may also contribute to low delay. |
| B | Generally occurs with good progression and/or short cycle lengths. More vehicles stop than for LOS A, causing higher levels of average delay. |
| C | Generally results when there is fair progression and/or longer cycle lengths. Individual cycle failures may begin to appear in this level. The number of vehicles stopping is significant at this level, although many still pass through the intersection without stopping. |
| D | Generally results in noticeable congestion. Longer delays may result from some combination of unfavorable progression, long cycle lengths, or high volume-to-capacity ratios. Many vehicles stop, and the proportion of vehicles not stopping declines. Individual cycle failures are noticeable. |
| E | Considered to be the limit of acceptable delay. These high delay values generally indicate poor progression, long cycle lengths, and high volume-to-capacity ratios. Individual cycle failures are frequent occurrences. |
| F | Considered to be unacceptable to most drivers. This condition often occurs with over saturation i.e. when arrival flow rates exceed the capacity of the intersection. It may also occur at high volume-to-capacity ratios below 1.00 with many individual cycle failures. Poor progression and long cycle lengths may also be major contributing causes to such delay levels |

## TABLE 4–2
## INTERSECTION LOS & DELAY RANGES

| LOS | Delay (seconds/vehicle) | |
|---|---|---|
| | Signalized Intersections | Unsignalized Intersections |
| A | ≤ 10.0 | ≤ 10.0 |
| B | 10.1 to 20.0 | 10.1 to 15.0 |
| C | 20.1 to 35.0 | 15.1 to 25.0 |
| D | 35.1 to 55.0 | 25.1 to 35.0 |
| E | 55.1 to 80.0 | 35.1 to 50.0 |
| F | ≥ 80.1 | ≥ 50.1 |

***Source:*** 2010 Highway Capacity Manual

## 4.2 Street Segments

Street segment ultimate classifications were taken from the Pacific Beach Community Plan Circulation Element. Street segment analysis is based upon the comparison of daily traffic volumes (ADTs) to the City of San Diego's *Roadway Classification, Level of Service, and ADT Table*. This table provides segment capacities for different street classifications, based on traffic volumes and roadway characteristics. A copy of the City of San Diego's roadway classification table and the Pacific Beach Community Plan Circulation Element is attached in ***Appendix C***.

## 5.0 SIGNIFICANCE CRITERIA

According to the City of San Diego's *Significance Determination Thresholds* report dated January 2011, a project is considered to have a significant impact if the new project traffic has decreased the operations of surrounding roadways by a City defined threshold. For projects deemed complete on or after January 1, 2011, the City defined threshold by roadway type or intersection is shown in ***Table 5–1***.

The impact is designated either a "direct" or "cumulative" impact. According to the City's *Significance Determination Thresholds* report,

> "*Direct* traffic impacts are those projected to occur at the time a proposed development becomes operational, including other developments not presently operational but which are anticipated to be operational at that time."
>
> "*Cumulative* traffic impacts are those projected to occur at some point after a proposed development becomes operational, such as during subsequent phases of a project and when additional proposed developments in the area become operational or when affected community plan area reaches full planned buildout."
>
> "It is possible that a project's near term impacts may be reduced in the long term, as future projects develop and provide additional roadway improvements (for instance, through implementation of traffic phasing plans). In such a case, the project may have direct impacts but not contribute considerably to a cumulative impact."
>
> "For intersections and roadway segments affected by a project, LOS D or better is considered acceptable under both direct and cumulative conditions."

If the project exceeds the thresholds in *Table 5–1*, then the project may be considered to have a significant "direct" or "cumulative" project impact. A significant impact can also occur if a project causes the LOS to degrade from D to E, even if the allowable increases in *Table 5–1* are not exceeded. A feasible mitigation measure will need to be identified to return the impact within the City thresholds, or the impact will be considered significant and unmitigated.

## TABLE 5–1
## CITY OF SAN DIEGO
## TRAFFIC IMPACT SIGNIFICANT THRESHOLDS

| Level of Service with Project [b] | Allowable Increase Due to Project Impacts [a] | |
|---|---|---|
| | Roadway Segments | Intersections |
| | V/C | Delay (sec.) |
| E | 0.02 | 2.0 |
| F | 0.01 | 1.0 |

***Footnotes:***

a. If a proposed project's traffic causes the values shown in the table to be exceeded, the impacts are determined to be significant. The project applicant shall then identify feasible improvements (within the Traffic Impact Study) that will restore/and maintain the traffic facility at an acceptable LOS.

b. All LOS measurements are based upon Highway Capacity Manual procedures for peak-hour conditions. However, V/C ratios for roadway segments are estimated on an ADT/24-hour traffic volume basis (using Table 2 of the City's Traffic Impact Study Manual).

***General Notes:***

1. Delay = Average control delay per vehicle measured in seconds for intersections.
2. LOS = Level of Service
3. V/C = Volume to Capacity Ratio (capacity at LOS E should be used)

# 6.0 Analysis of Existing Conditions

The analysis of existing conditions includes the assessment of the study area intersections and street segments using the methodologies described in *Section 4.0*.

## 6.1 Peak Hour Intersection Levels of Service

***Table 6–1*** summarizes the existing intersection Levels of Service. As seen in *Table 6–1*, the following intersections are calculated to currently operate at LOS E or F:

1. Mission Bay Drive / Garnet Ave (LOS E/F during the AM/PM peak hour, respectively)
6. Mission Bay Drive / Rosewood Street (LOS E/F during the AM/PM peak hour, respectively)

***Appendix D*** contains the existing intersection calculation sheets.

## 6.2 Street Segment Levels of Service

***Table 6–2*** summarizes the existing street segment operations. As shown in *Table 6–2*, the following segments are calculated to currently operate at LOS F:

- Grand Avenue: Figueroa Boulevard to Mission Bay Drive
- Mission Bay Drive: Grand Avenue to Rosewood Street
- Mission Bay Drive: Rosewood Street to N. Mission Bay Drive

## TABLE 6–1
## EXISTING INTERSECTION OPERATIONS

| Intersection | Control Type | Peak Hour | Existing Delay[a] | Existing LOS[b] |
|---|---|---|---|---|
| 1. Mission Bay Drive / Garnet Avenue | Signal | AM<br>PM | **76.7**<br>**161.2** | **E**<br>**F** |
| 2. Mission Bay Drive / Bunker Hill | Signal | AM<br>PM | 27.2<br>28.6 | C<br>C |
| 3. Mission Bay Drive / Grand Avenue | Signal | AM<br>PM | 23.8<br>30.1 | C<br>C |
| 4. Mission Bay Drive / Glendora Street | MSSC[c] | AM<br>PM | 19.1<br>25.3 | C<br>D |
| 5. Mission Bay Drive / Project Driveway A | MSSC[c] | AM<br>PM | 18.9<br>25.8 | C<br>D |
| 6. Mission Bay Drive / Rosewood Street | MSSC[c] | AM<br>PM | **41.0**<br>**>200** | **E**<br>**F** |
| 7. Mission Bay Drive / I-5 SB On-Ramp | Signal | AM<br>PM | 1.8<br>1.0 | A<br>A |
| 8. Glendora Street / Project Driveway B | MSSC[c] | AM<br>PM | 9.6<br>9.7 | A<br>A |

***Footnotes:***

a. Average delay expressed in seconds per vehicle.
b. Level of Service.
c. MSSC – Minor Street Stop-Controlled intersection. Minor street critical movement delay is reported.

| SIGNALIZED DELAY/LOS THRESHOLDS Delay | LOS | UNSIGNALIZED DELAY/LOS THRESHOLDS Delay | LOS |
|---|---|---|---|
| 0.0 ≤ 10.0 | A | 0.0 ≤ 10.0 | A |
| 10.1 to 20.0 | B | 10.1 to 15.0 | B |
| 20.1 to 35.0 | C | 15.1 to 25.0 | C |
| 35.1 to 55.0 | D | 25.1 to 35.0 | D |
| 55.1 to 80.0 | E | 35.1 to 50.0 | E |
| ≥ 80.1 | F | ≥ 50.1 | F |

### TABLE 6–2
### EXISTING STREET SEGMENT OPERATIONS

| Street Segment | Classification | Capacity (LOS E) [a] | ADT [b] | LOS [c] | V/C [d] |
|---|---|---|---|---|---|
| **Grand Avenue** | | | | | |
| Figueroa Boulevard to Mission Bay Drive | 3-lane Major Arterial | 30,000 | 40,860 | F | 1.362 |
| **Mission Bay Drive** | | | | | |
| Garnet Avenue to Bunker Hill Street | 4-lane Major Arterial | 40,000 | 31,210 | D | 0.780 |
| Bunker Hill Street to Grand Avenue | 4-lane Major Arterial | 40,000 | 31,360 | D | 0.784 |
| Grand Avenue to Rosewood Street | 4-lane Major Arterial | 40,000 | 60,410 | F | 1.510 |
| Rosewood Street to N. Mission Bay Drive | 4-lane Major Arterial | 40,000 | 60,430 | F | 1.511 |
| **Revere Avenue** | | | | | |
| Bunker Hill Street to Glendora Street | 2-Lane Local (multi-family) | 2,200 | 750 | Better than C[e] | |
| **Bunker Hill Street** | | | | | |
| Mission Bay Drive to Del Rey Street | 2-Lane Local (commercial-industrial fronting) | 8,000 | 3,680 | C | 0.460 |
| **Glendora Street** | | | | | |
| Mission Bay Drive to Revere Avenue | 2-Lane Local (multi-family) | 2,200 | 590 | Better than C[e] | |
| **Rosewood Street** | | | | | |
| Mission Bay Drive to Del Rey Street | 2-Lane Local (Residential fronting) | 2,200 (LOS C) | 780 | Better than C[e] | |

***Footnotes:***

a. Capacities based on the City of San Diego's Roadway Classification Table.

b. Average Daily Traffic Volumes.

c. Level of Service.

d. Volume to Capacity.

e. City of San Diego Roadway Classification does not specify a capacity for residential streets. Therefore, better (or worse) than LOS C has been used as a performance metric.

## 7.0 TRIP GENERATION/DISTRIBUTION/ASSIGNMENT

Project trips consist of vehicular trips on the street system which begin or end at the project site and are generated by the proposed development. LLG has prepared the trip generation in accordance with the trip generation rates outlined in the *City of San Diego Trip Generation Manual, May 2003*. The "motel" trip rate was used because the proposed hotel does not include a restaurant or a conference facility.

***Table 7–1*** summarizes the project trip generation calculations.

### 7.1 Trip Generation

The proposed project is calculated to generate 594 new net daily trips[5], with 48 AM peak hour trips (19 inbound, 29 outbound) and 54 PM peak hour trips (21 inbound, 33 outbound).

### 7.2 Trip Distribution/Assignment

The project trip distribution and assignment was derived based on existing traffic patterns, the site's proximity to major local and regional facilities (i.e. Mission Bay Drive, I-5, etc.), ingress/egress movements afforded by the project site and other factors.

***Figure 7–1*** depicts the project traffic distribution and assignment percentages. ***Figure 7–2*** depicts the project traffic assignment. ***Figure 7–3*** shows the Existing + Project traffic volumes.

TABLE 7–1
TRIP GENERATION

| Land Use | Size | Daily Trip Ends (ADTs) [a] | | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Rate [b] | Volume | % of ADT | In:Out Split | Volume In | Volume Out | % of ADT | In:Out Split | Volume In | Volume Out |
| ***Proposed Use*** | | | | | | | | | | | |
| Motel | 112 rooms | 9 /room | 1,008 | 8% | 40:60 | 32 | 49 | 9% | 40:60 | 36 | 55 |
| ***Existing Use*** | | | | | | | | | | | |
| Motel | 46 rooms | 9 /room | (414) | 8% | 40:60 | (13) | (20) | 9% | 40:60 | (15) | (22) |
| **Total Trips** | – | – | **594** | – | – | **19** | **29** | – | – | **21** | **33** |

***Footnotes:***

a. ADT = Average Daily Traffic.

b. Trip rate is based on the published *City of San Diego Municipal Code Land Development Code Trip Generation Manual.*

[5] The trip generation and impact analyses in this study are based on a higher room count of 112 rooms and therefore, the findings and conclusions presented in this study are conservative.

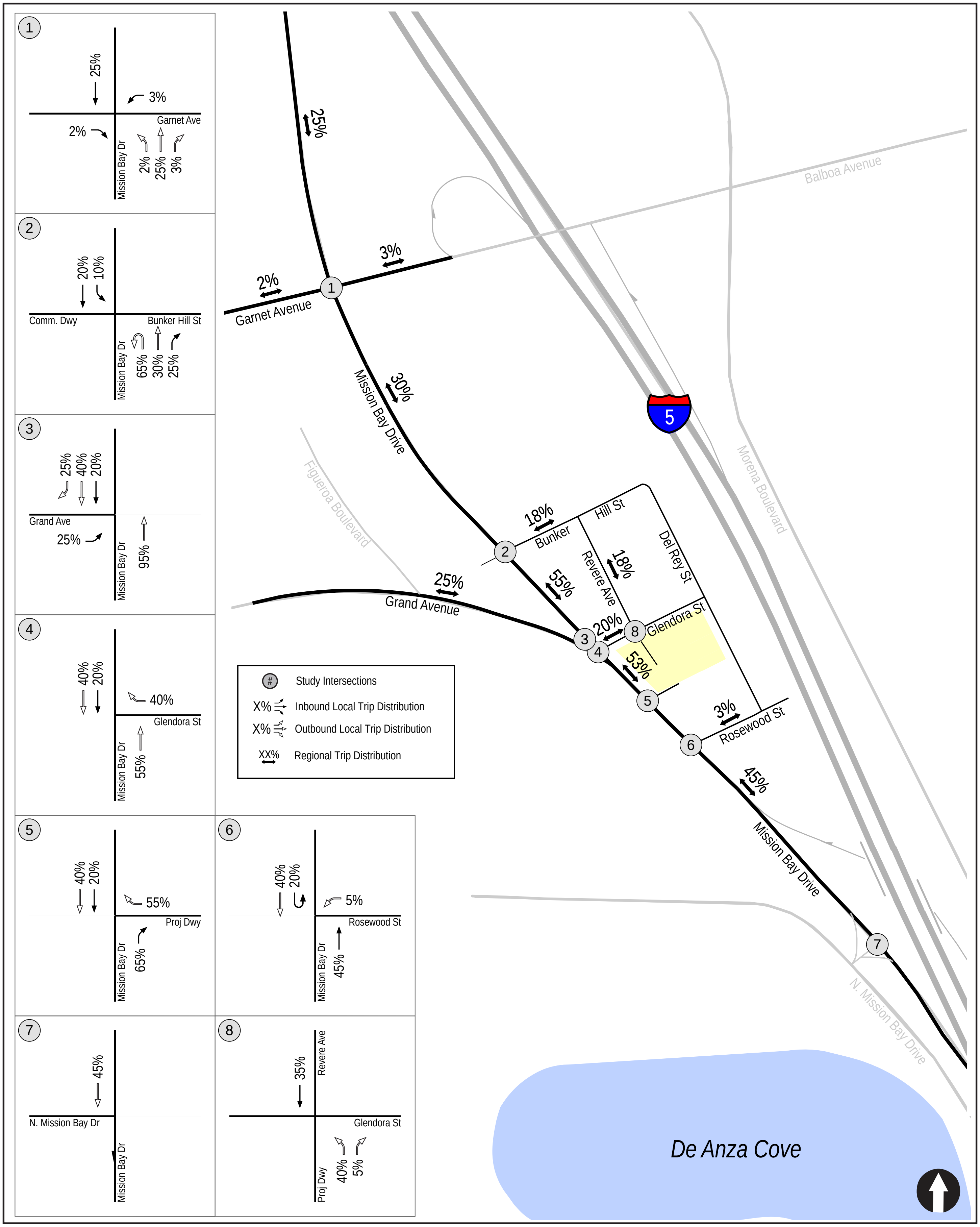


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**Figure 7-1**

## Project Traffic Distribution

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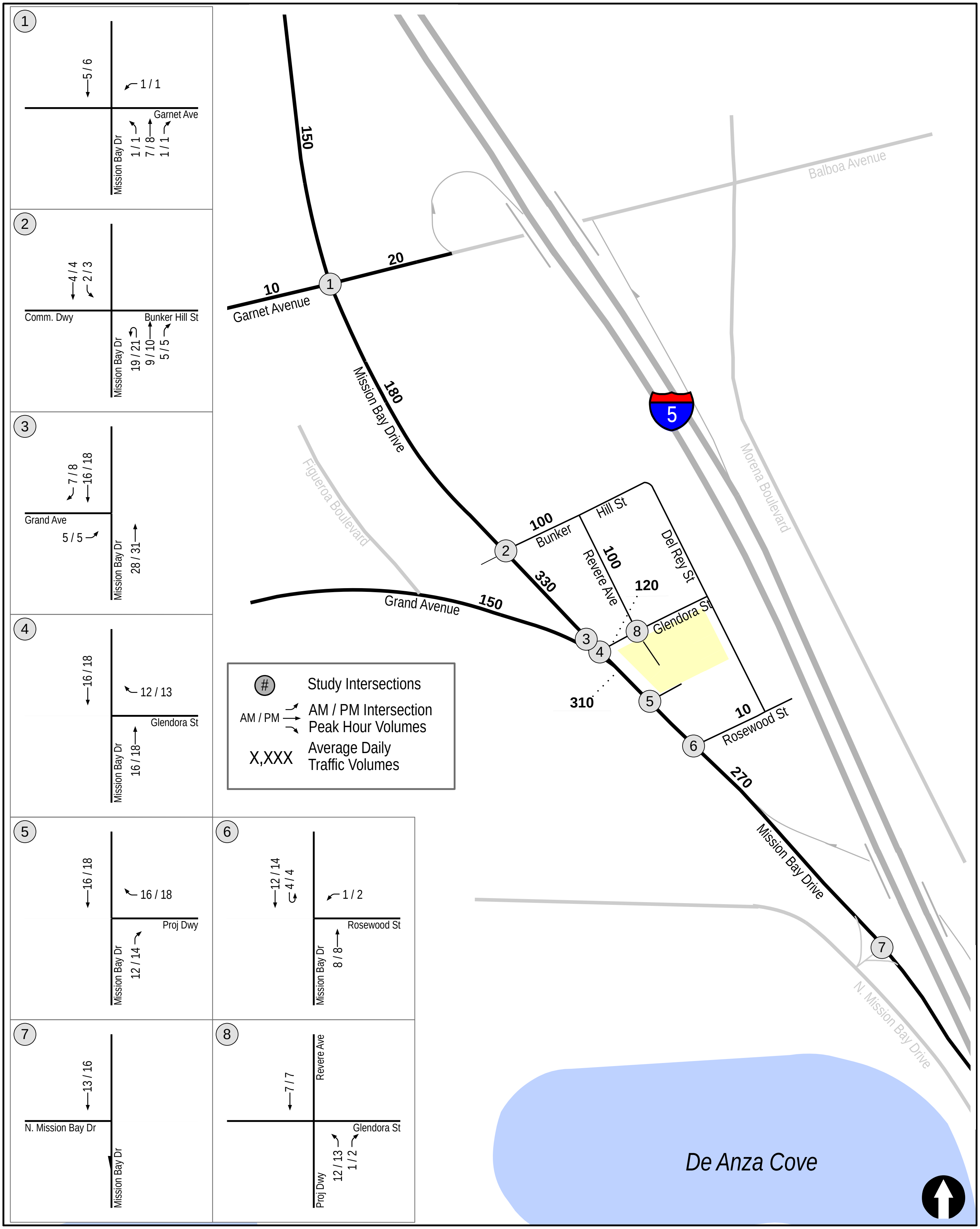


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**Figure 7-2**

## Project Traffic Volumes

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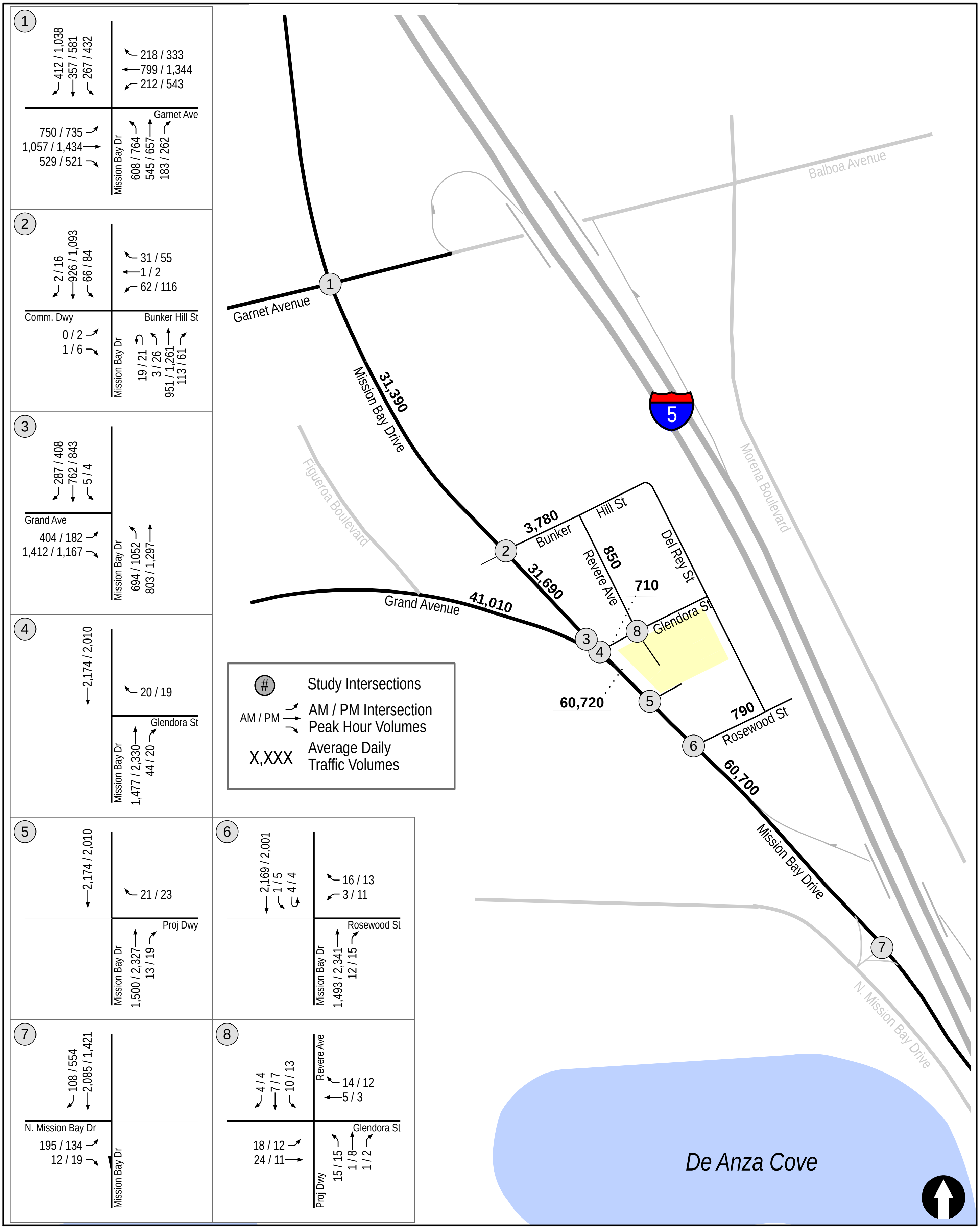


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**Figure 7-3**

## Existing + Project Traffic Volumes

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## 8.0 EXISTING + PROJECT ANALYSIS

The project traffic was added to the existing traffic volumes to obtain volumes for the Existing + Project analysis.

### 8.1.1 *Intersection Analysis*

***Table 8–1*** summarizes the peak hour intersection operations for the Existing + Project scenario. As seen in *Table 8–1*, the following intersections are calculated to continue operate at LOS E or F:

1. Mission Bay Drive / Garnet Ave (LOS E/F during the AM/PM peak hour, respectively)
6. Mission Bay Drive / Rosewood Street (LOS F/F during the AM/PM peak hour, respectively)

***Appendix F*** contains the Existing + Project conditions analysis worksheets.

Based on the City of San Diego's significance criteria, the project is calculated with a significant direct impact at the Mission Bay Drive / Rosewood Street intersection.

### 8.1.2 *Street Segment Analysis*

***Table 8–2*** summarizes the Existing + Project street segment operations. As shown in *Table 8–2*, the following segments are calculated to continue operate at LOS F:

- Grand Avenue: Figueroa Boulevard to Mission Bay Drive
- Mission Bay Drive: Grand Avenue to Rosewood Street
- Mission Bay Drive: Rosewood Street to N. Mission Bay Drive

Based on the City of San Diego's significance criteria, no significant impacts are identified on the above segments as the project contribution do not exceed the allowable thresholds.

## TABLE 8–1
## EXISTING + PROJECT INTERSECTION OPERATIONS

| Intersection | Control Type | Peak Hour | Existing | | Existing + Project | | Δ Delay | Significant Impact? |
|---|---|---|---|---|---|---|---|---|
| | | | Delay [a] | LOS [b] | Delay | LOS | | |
| 1. Mission Bay Drive / Garnet Avenue | Signal | AM | **76.7** | **E** | **76.8** | **E** | 0.1 | No |
| | | PM | **161.2** | **F** | **161.7** | **F** | 0.5 | No |
| 2. Mission Bay Drive / Bunker Hill Street | Signal | AM | 27.2 | C | 28.4 | C | 1.2 | No |
| | | PM | 28.6 | C | 30.2 | C | 1.6 | No |
| 3. Mission Bay Drive / Grand Avenue | Signal | AM | 23.8 | C | 24.2 | C | 0.4 | No |
| | | PM | 30.1 | C | 30.2 | C | 0.1 | No |
| 4. Mission Bay Drive / Glendora Street | MSSC[c] | AM | 19.1 | C | 20.0 | C | 0.9 | No |
| | | PM | 25.3 | D | 27.3 | D | 2.0 | No |
| 5. Mission Bay Drive / Project Driveway A | MSSC[c] | AM | 18.9 | C | 20.0 | C | 1.1 | No |
| | | PM | 25.8 | D | 28.2 | D | 2.4 | No |
| 6. Mission Bay Drive / Rosewood Street | MSSC[c] | AM | **41.0** | **E** | **54.3** | **F** | **13.3** | **Yes** |
| | | PM | **>200** | **F** | **>200** | **F** | **-** | **Yes** |
| 7. Mission Bay Drive / I-5 SB On-Ramp | Signal | AM | 1.8 | A | 1.9 | A | 0.1 | No |
| | | PM | 1.0 | A | 1.0 | A | 0.0 | No |
| 8. Glendora Street / Project Driveway B | MSSC[c] | AM | 9.6 | A | 9.6 | A | 0.0 | No |
| | | PM | 9.7 | A | 9.7 | A | 0.0 | No |

***Footnotes:***

a. Average delay expressed in seconds per vehicle.
b. Level of Service
c. Minor Street Stop Controlled intersection. Minor street critical movement delay is reported.

| SIGNALIZED | | UNSIGNALIZED | |
|---|---|---|---|
| DELAY/LOS THRESHOLDS | | DELAY/LOS THRESHOLDS | |
| Delay | LOS | Delay | LOS |
| 0.0 ≤ 10.0 | A | 0.0 ≤ 10.0 | A |
| 10.1 to 20.0 | B | 10.1 to 15.0 | B |
| 20.1 to 35.0 | C | 15.1 to 25.0 | C |
| 35.1 to 55.0 | D | 25.1 to 35.0 | D |
| 55.1 to 80.0 | E | 35.1 to 50.0 | E |
| ≥ 80.1 | F | ≥ 50.1 | F |

TABLE 8–2
EXISTING + PROJECT STREET SEGMENT OPERATIONS

| Street Segment | Capacity (LOS E) [a] | Existing | | | Existing + Project | | | Δ Delay | Significant Impact? |
|---|---|---|---|---|---|---|---|---|---|
| | | ADT [b] | LOS [c] | V/C [d] | ADT | LOS | V/C | | |
| **Grand Avenue** | | | | | | | | | |
| Figueroa Boulevard to Mission Bay Drive | 30,000 | 40,860 | F | 1.362 | 41,010 | F | 1.367 | 0.005 | No |
| **Mission Bay Drive** | | | | | | | | | |
| Garnet Avenue to Bunker Hill Street | 40,000 | 31,210 | D | 0.780 | 31,390 | D | 0.785 | 0.005 | No |
| Bunker Hill Street to Grand Avenue | 40,000 | 31,360 | D | 0.784 | 31,690 | D | 0.792 | 0.008 | No |
| Grand Avenue to Rosewood Street | 40,000 | 60,410 | F | 1.510 | 60,720 | F | 1.518 | 0.008 | No |
| Rosewood Street to N. Mission Bay Drive | 40,000 | 60,430 | F | 1.511 | 60,700 | F | 1.518 | 0.007 | No |
| **Revere Avenue** | | | | | | | | | |
| Bunker Hill Street to Glendora Street | 2,200 | 750 | Better than C | | 850 | Better than C | | – | No |
| **Bunker Hill Street** | | | | | | | | | |
| Mission Bay Drive to Del Rey Street | 8,000 | 3,680 | C | 0.460 | 3,780 | C | 0.473 | 0.013 | No |
| **Glendora Street** | | | | | | | | | |
| Mission Bay Drive to Revere Avenue | 2,200 | 590 | Better than C | | 710 | Better than C | | – | No |
| **Rosewood Street** | | | | | | | | | |
| Mission Bay Drive to Del Rey Street | 2,200 (LOS C) | 780 | better than C | | 790 | better than C | | – | No |

***Footnotes:***
a. Capacities based on City of San Diego's Roadway Classification & LOS table *(See Appendix C).*
b. Average Daily Traffic
c. Level of Service
d. Volume to Capacity ratio

## 9.0 CUMULATIVE PROJECTS

Cumulative projects are other reasonably foreseeable projects in the study area that will add traffic to the nearby circulation system and would be expected to be open and operating by the project expected opening day of 2020. LLG coordinated with City staff and reviewed other planned projects in the vicinity. One (1) cumulative project was identified in the immediate project vicinity.

Jefferson Pacific Beach project includes the construction of approximately 15,500 SF of mixed use retail/office/restaurant and 172 apartment units on a former car dealership site. The project is located on Mission Bay Drive on the west side of Interstate 5 (I-5), just south of Rosewood Street in the community of Pacific Beach. This project was approved in August of 2016 and is currently under construction with an expected opening in 2019.

In addition to Jefferson Pacific Beach project, a general growth rate of 2% (1% per year for two years) was added to the existing counts to accommodate potential increase to the background traffic or other cumulative projects that were not accounted for.

***Figure 9–1*** depicts the total cumulative projects traffic volumes, ***Figure 9–2*** shows the Near-Term (existing + cumulative projects) traffic volumes, and ***Figure 9–3*** shows the Near-Term + Project traffic volumes. ***Appendix E*** contains the cumulative project trip assignment.

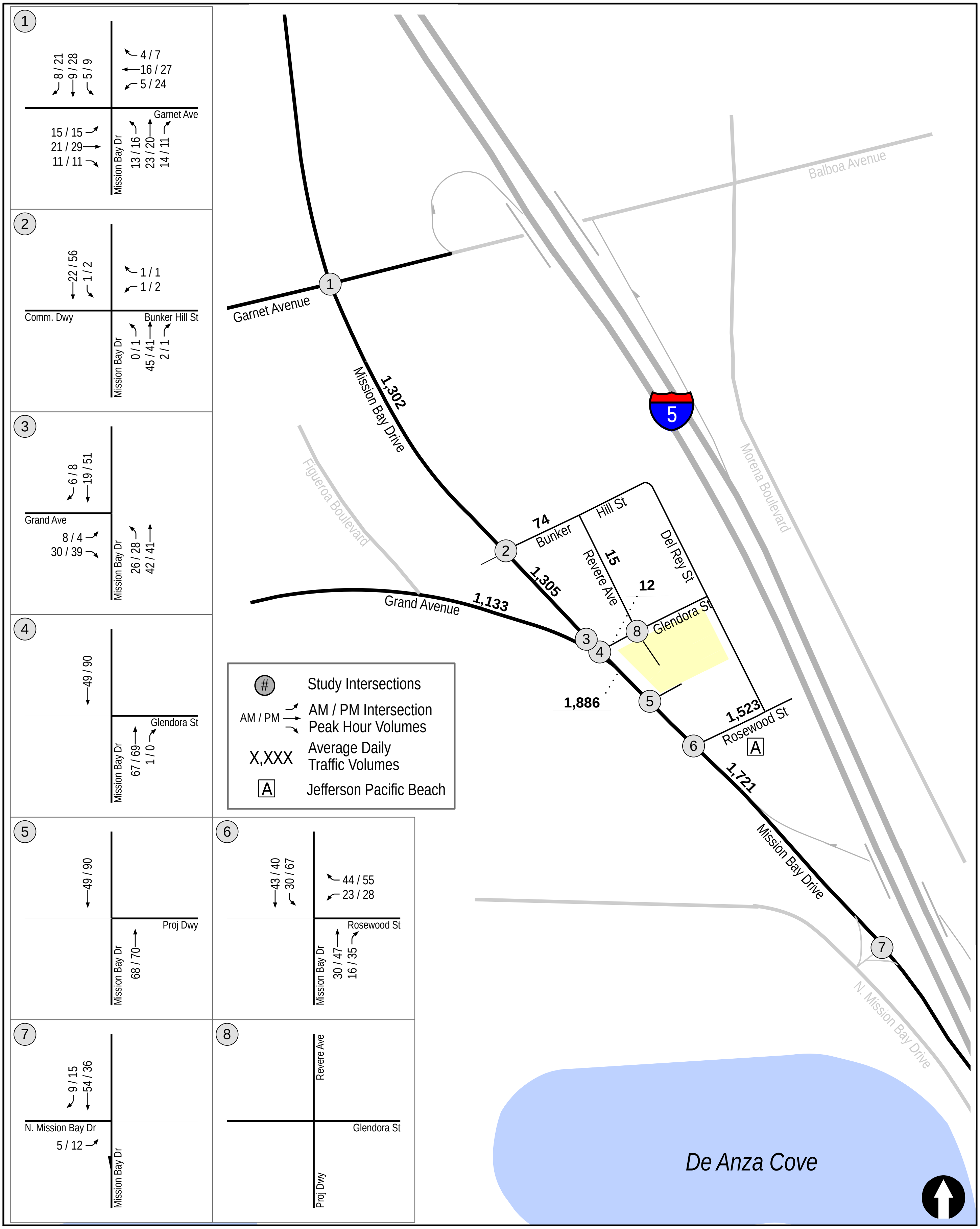


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**Figure 9-1**

## Cumulative Projects Traffic Volumes

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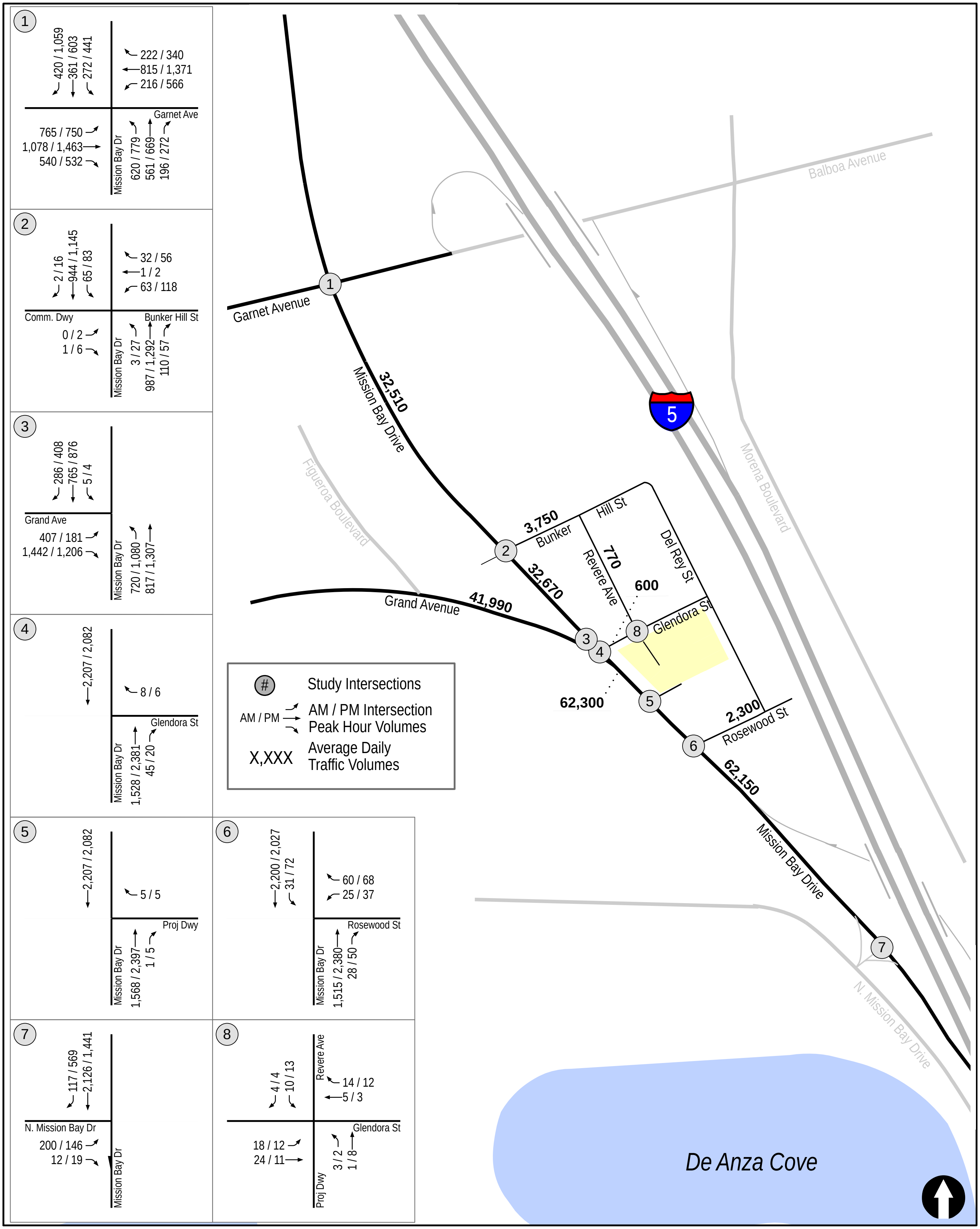


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**Figure 9-2**

## Near-Term Without Project Traffic Volumes

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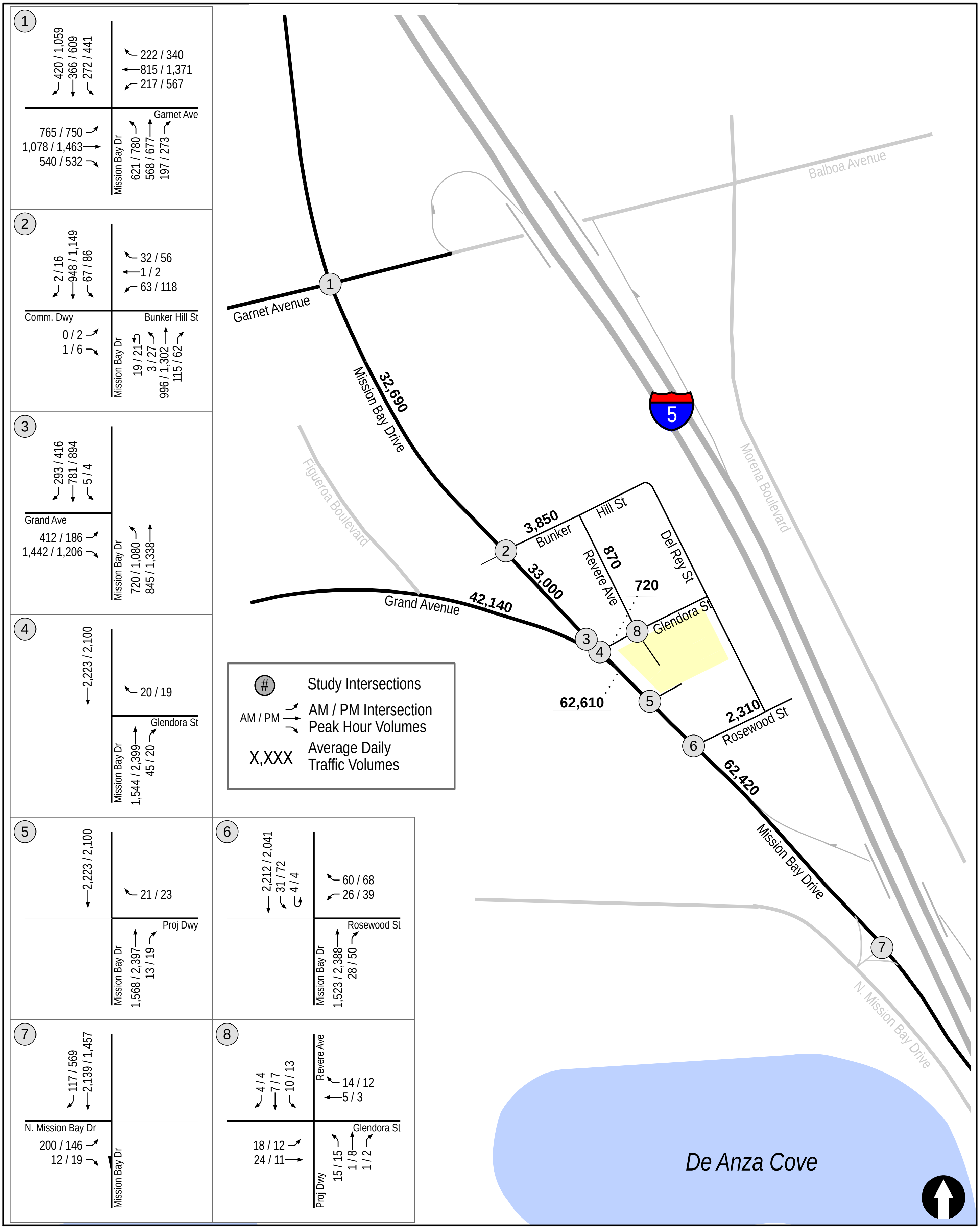


**Figure 9-3**

**Near-Term With Project Traffic Volumes**

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# 10.0 NEAR-TERM (OPENING DAY 2020) ANALYSIS

This section presents the Near-Term Analysis of the project. The scenarios analyzed include Near-Term without Project and Near-Term with Project. The proposed project's Opening Day (Year 2020) is expected to be after the opening day (Year 2019) of the Jefferson Beach project and therefore, the traffic generated from the Jefferson Beach project was assumed. However, the Jefferson Beach project's mitigation of installing a traffic signal at Rosewood Street / Mission Bay Drive was not assumed as a part of the Near-Term analyses. At the time of preparation of this report, the Jefferson Beach project is currently under construction. No other roadway network changes were assumed.

## 10.1 Near-Term

### 10.1.1 *Intersection Analysis*

***Table 10–1*** summarizes the peak hour intersection operations for the Near-Term scenario. As seen in *Table 10–1*, the following intersections are calculated to operate at LOS F:

1. Mission Bay Drive / Garnet Ave (LOS F/F during the AM/PM peak hours, respectively)
6. Mission Bay Drive / Rosewood Street (LOS F/F during the AM/PM peak hour, respectively)

***Appendix G*** contains the Near-Term intersection calculation sheets.

### 10.1.2 *Street Segment Analysis*

***Table 10–2*** summarizes the Near-Term street segment operations. As shown in *Table 10–2*, the following segments are calculated to operate at LOS F:

- Grand Avenue: Figueroa Boulevard to Mission Bay Drive
- Mission Bay Drive: Grand Avenue to Rosewood Street
- Mission Bay Drive: Rosewood Street to N. Mission Bay Drive

***Figure 10–1*** illustrates Near-Term traffic volumes on a peak hour and daily basis.

## 10.2 Near-Term + Project (Opening Day 2020)

### 10.2.1 *Intersection Analysis*

*Table 10–1* summarizes the peak hour intersection operations for the Near-Term + Project scenario. As seen in *Table 10–1*, the following intersections are calculated to continue to operate at LOS F:

1. Mission Bay Drive / Garnet Ave (LOS F during the AM/PM peak hours)
6. Mission Bay Drive / Rosewood Street (LOS F/F during the AM/PM peak hour, respectively)

Based on City of San Diego's significance thresholds, one (1) significant direct impact is identified at Mission Bay Drive / Rosewood Street as the project contribution to the intersection delay exceeds the allowable thresholds.

***Appendix H*** contains the Near-Term + Project intersection calculation sheets.

### 10.2.2 *Street Segment Analysis*

*Table 10–2* summarizes the Near-Term + Project street segment operations. As shown in *Table 10–2*, the following segments are calculated to continue to operate at LOS F:

- Grand Avenue: Figueroa Boulevard to Mission Bay Drive
- Mission Bay Drive: Grand Avenue to Rosewood Street
- Mission Bay Drive: Rosewood Street to N. Mission Bay Drive

Based on City of San Diego's significance thresholds, no significant direct impacts are identified to the above segments as the project contribution do not exceed the allowable thresholds.

## TABLE 10–1
## NEAR-TERM INTERSECTION OPERATIONS

| Intersection | Control Type | Peak Hour | Near-Term | | Near-Term (Opening Day 2020) + Project | | Δ [c] Delay | Significant Impact? |
|---|---|---|---|---|---|---|---|---|
| | | | Delay [a] | LOS [b] | Delay | LOS | | |
| 1. Mission Bay Drive / Garnet Avenue | Signal | AM | **80.9** | **F** | **81.1** | **F** | 0.2 | No |
| | | PM | **168.9** | **F** | **169.1** | **F** | 0.2 | No |
| 2. Mission Bay Drive / Bunker Hill Street | Signal | AM | 28.3 | C | 29.6 | C | 1.3 | No |
| | | PM | 29.0 | C | 33.2 | C | 4.2 | No |
| 3. Mission Bay Drive / Grand Avenue | Signal | AM | 25.6 | C | 26.3 | C | 0.7 | No |
| | | PM | 30.4 | C | 30.4 | C | 0.0 | No |
| 4. Mission Bay Drive / Glendora Street | MSSC[d] | AM | 19.9 | C | 21.0 | C | 1.1 | No |
| | | PM | 26.2 | D | 28.4 | D | 2.2 | No |
| 5. Mission Bay Drive / Project Driveway A | MSSC[d] | AM | 19.7 | C | 21.0 | C | 1.3 | No |
| | | PM | 27.0 | D | 28.4 | D | 1.4 | No |
| 6. Mission Bay Drive / Rosewood Street | MSSC[d] | AM | **>200.0** | **F** | **>200.0** | **F** | **>100.0** | **Yes** |
| | | PM | **>200.0** | **F** | **>200.0** | **F** | **>100.0** | **Yes** |
| 7. Mission Bay Drive / I-5 SB On-Ramp | Signal | AM | 2.0 | A | 2.0 | A | 0.0 | No |
| | | PM | 1.0 | A | 1.0 | A | 0.0 | No |
| 8. Glendora Street / Project Driveway B | MSSC[d] | AM | 9.6 | A | 9.6 | A | 0.0 | No |
| | | PM | 9.7 | A | 9.7 | A | 0.0 | No |

***Footnotes:***

a. Average delay expressed in seconds per vehicle.
b. Level of Service
c. Δ denotes the increase in delay due to Project.
d. Minor Street Stop Controlled intersection. Minor street critical movement delay is reported.

| SIGNALIZED | | UNSIGNALIZED | |
|---|---|---|---|
| DELAY/LOS THRESHOLDS | | DELAY/LOS THRESHOLDS | |
| Delay | LOS | Delay | LOS |
| 0.0 ≤ 10.0 | A | 0.0 ≤ 10.0 | A |
| 10.1 to 20.0 | B | 10.1 to 15.0 | B |
| 20.1 to 35.0 | C | 15.1 to 25.0 | C |
| 35.1 to 55.0 | D | 25.1 to 35.0 | D |
| 55.1 to 80.0 | E | 35.1 to 50.0 | E |
| ≥ 80.1 | F | ≥ 50.1 | F |

TABLE 10–2
NEAR-TERM STREET SEGMENT OPERATIONS

| Street Segment | Capacity (LOS E) [a] | Near-Term | | | Near-Term + Project | | | Δ [e] V/C | Sig Impact? |
|---|---|---|---|---|---|---|---|---|---|
| | | ADT [b] | LOS [c] | V/C [d] | ADT | LOS | V/C | | |
| **Grand Avenue** | | | | | | | | | |
| Figueroa Boulevard to Mission Bay Drive | 30,000 | 41,990 | F | 1.399 | 42,140 | F | 1.404 | 0.005 | No |
| **Mission Bay Drive** | | | | | | | | | |
| Garnet Avenue to Bunker Hill Street | 40,000 | 32,510 | D | 0.813 | 32,690 | D | 0.817 | 0.004 | No |
| Bunker Hill Street to Grand Avenue | 40,000 | 32,670 | D | 0.817 | 33,000 | D | 0.825 | 0.008 | No |
| Grand Avenue to Rosewood Street | 40,000 | 62,300 | F | 1.558 | 62,610 | F | 1.565 | 0.007 | No |
| Rosewood Street to N. Mission Bay Drive | 40,000 | 62,150 | F | 1.554 | 62,420 | F | 1.561 | 0.007 | No |
| **Revere Avenue** | | | | | | | | | |
| Bunker Hill Street to Glendora Street | 2,200 | 770 | Better than C | | 870 | Better than C | | - | No |
| **Bunker Hill Street** | | | | | | | | | |
| Mission Bay Drive to Del Rey Street | 8,000 | 3,750 | C | 0.469 | 3,850 | C | 0.481 | 0.012 | No |
| **Glendora Street** | | | | | | | | | |
| Mission Bay Drive to Revere Avenue | 2,200 | 600 | Better than C | | 720 | Better than C | | - | No |
| **Rosewood Street** | | | | | | | | | |
| Mission Bay Drive to Del Rey Street | 2,200 (LOS C) | 2,300 | worse than C | | 2,310 | worse than C | | - | No |

***Footnotes:***
a. Capacities based on City of San Diego's Roadway Classification & LOS table *(See Appendix C)*.
b. Average Daily Traffic
c. Level of Service
d. Volume to Capacity ratio
e. Δ denotes a Project-induced increase in the Volume to Capacity ratio

# 11.0 YEAR 2035 (HORIZON YEAR) ANALYSIS

The following section presents the analysis of study area intersections and street segments under Year 2035 (Horizon Year) conditions without and with the project.

## 11.1 Year 2035 (Horizon Year) Traffic Volumes

Year 2035 (Horizon Year) traffic volumes were obtained from the recently *approved Jefferson Pacific Beach project,* which used SANDAG Series 12 projections. ***Appendix I*** contains the Series 12 model outputs.

Based on the projected forecast ADT volumes, the Year 2035 (Horizon Year) peak hour volumes were calculated based on the existing relationship between ADT and peak hour volumes. The forecast volumes were also checked for consistency between intersections, where no driveways or roadways exist between intersections, and were compared to existing volumes for reasonableness.

***Figure 11–1*** shows the forecasted Year 2035 (Horizon Year) without Project traffic volumes. ***Figure 11–2*** shows the Year 2035 (Horizon Year) + Project traffic volumes.

## 11.2 Year 2035 (Horizon Year) without Project

### 11.2.1 *Intersection Analysis*

***Table 11–1*** summarizes the peak hour intersection operations for the Year 2035 (Horizon Year) without project scenario. As seen in *Table 11–1*, the following intersections are calculated to operate at LOS E or F:

1. Mission Bay Drive / Garnet Ave (LOS F during the AM/PM peak hours)
2. Mission Bay Drive / Bunker Hill Street (LOS E/F during the AM/PM peak hour, respectively)
6. Mission Bay Drive / Rosewood Street (LOS F/F during the AM/PM peak hour, respectively)

***Appendix J*** contains the Year 2035 (Horizon Year) intersection calculation sheets.

### 11.2.2 *Street Segment Analysis*

***Table 11–2*** summarizes the Year 2035 (Horizon Year) without project street segment operations. As shown in *Table 11–2*, the following segments are calculated to operate at LOS E or F:

- Grand Avenue: Figueroa Boulevard to Mission Bay Drive (LOS F)
- Mission Bay Drive: Garnet Avenue to Bunker Hill Street (LOS F)
- Mission Bay Drive: Bunker Hill Street to Grand Avenue (LOS E)
- Mission Bay Drive: Grand Avenue to Rosewood Street (LOS F)
- Mission Bay Drive: Rosewood Street to N. Mission Bay Drive (LOS F)

## 11.3 Year 2035 (Horizon Year) with Project

### 11.3.1 *Intersection Analysis*

*Table 11–1* summarizes the peak hour intersection operations for the Year 2035 (Horizon Year) with project scenario. As seen in *Table 11–1*, the following intersections are calculated to operate at LOS E or F:

1. Mission Bay Drive / Garnet Ave (LOS F/F during the AM/PM peak hours, respectively)
2. Mission Bay Drive / Bunker Hill Street (LOS E/F during the AM/PM peak hour, respectively)
6. Mission Bay Drive / Rosewood Street (LOS F/F during the AM/PM peak hour, respectively)

Based on City of San Diego's significance thresholds, one (1) significant cumulative impact is identified as the project contribution to the intersection delay exceeds the allowable thresholds.

***Appendix K*** contains the Year 2035 (Horizon Year) + Project intersection calculation sheets.

### 11.3.2 *Street Segment Analysis*

*Table 11–2* summarizes the Year 2035 (Horizon Year) with project street segment operations. As shown in *Table 11–2*, the following segments are calculated to operate at LOS E or F:

- Grand Avenue: Figueroa Boulevard to Mission Bay Drive (LOS F)
- Mission Bay Drive: Garnet Avenue to Bunker Hill Street (LOS F)
- Mission Bay Drive: Bunker Hill Street to Grand Avenue (LOS E)
- Mission Bay Drive: Grand Avenue to Rosewood Street (LOS F)
- Mission Bay Drive: Rosewood Street to N. Mission Bay Drive (LOS F)

Based on City of San Diego's significance thresholds, no significant cumulative impacts are identified to the above segments as the project contribution do not exceed the allowable thresholds.

TABLE 11–1
YEAR 2035 (HORIZON YEAR) INTERSECTION OPERATIONS

| Intersection | Control Type | Peak Hour | Year 2035 (Horizon Year) | | Year 2035 (Horizon Year) + Project | | Δ [c] Delay | Significant Impact? |
|---|---|---|---|---|---|---|---|---|
| | | | Delay [a] | LOS [b] | Delay | LOS | | |
| 1. Mission Bay Drive / Garnet Avenue | Signal | AM<br>PM | **120.2**<br>**>200** | **F**<br>**F** | **120.6**<br>**>200** | **F**<br>**F** | 0.4<br>0.6 | No<br>No |
| 2. Mission Bay Drive / Bunker Hill Street | Signal | AM<br>PM | **56.6**<br>**85.2** | **E**<br>**F** | **57.8**<br>**86.4** | **E**<br>**F** | 1.2<br>1.2 | No<br>No |
| 3. Mission Bay Drive / Grand Avenue | Signal | AM<br>PM | 42.7<br>36.1 | D<br>D | 43.4<br>36.6 | D<br>D | 0.7<br>0.5 | No<br>No |
| 4. Mission Bay Drive / Glendora Street | MSSC[d] | AM<br>PM | 23.7<br>32.2 | C<br>D | 25.4<br>34.0 | D<br>D | 1.7<br>1.8 | No<br>No |
| 5. Mission Bay Drive / Project Driveway A | MSSC[d] | AM<br>PM | 23.8<br>31.4 | C<br>D | 25.9<br>34.6 | D<br>D | 2.1<br>3.2 | No<br>No |
| 6. Mission Bay Drive / Rosewood Street | MSSC[d] | AM<br>PM | **>200.0**<br>**>200.0** | **F**<br>**F** | **>200.0**<br>**>200.0** | **F**<br>**F** | **>100.0**<br>**>100.0** | **Yes**<br>**Yes** |
| 7. Mission Bay Drive / I-5 SB On-Ramp | Signal | AM<br>PM | 2.0<br>1.0 | A<br>A | 2.1<br>1.0 | A<br>A | 0.1<br>0.0 | No<br>No |
| 8. Glendora Street / Project Driveway B | MSSC[d] | AM<br>PM | 10.0<br>10.1 | B<br>B | 10.3<br>10.4 | B<br>B | 0.3<br>0.3 | No<br>No |

***Footnotes:***
a. Average delay expressed in seconds per vehicle.
b. Level of Service
c. Δ denotes the increase in delay due to Project.
d. Minor Street Stop Controlled intersection. Minor street critical movement delay is reported.

| SIGNALIZED DELAY/LOS THRESHOLDS | | UNSIGNALIZED DELAY/LOS THRESHOLDS | |
|---|---|---|---|
| Delay | LOS | Delay | LOS |
| 0.0 ≤ 10.0 | A | 0.0 ≤ 10.0 | A |
| 10.1 to 20.0 | B | 10.1 to 15.0 | B |
| 20.1 to 35.0 | C | 15.1 to 25.0 | C |
| 35.1 to 55.0 | D | 25.1 to 35.0 | D |
| 55.1 to 80.0 | E | 35.1 to 50.0 | E |
| ≥ 80.1 | F | ≥ 50.1 | F |

## TABLE 11–2
## YEAR 2035 (HORIZON YEAR) STREET SEGMENT OPERATIONS

| Street Segment | Capacity (LOS E) [a] | Year 2035 (Horizon Year) | | | Year 2035 (Horizon Year) + Project | | | Δ [e] V/C | Sig Impact? |
|---|---|---|---|---|---|---|---|---|---|
| | | ADT [b] | LOS [c] | V/C [d] | ADT | LOS | V/C | | |
| **Grand Avenue** | | | | | | | | | |
| Figueroa Boulevard to Mission Bay Drive | 30,000 | 48,750 | F | 1.625 | 48,900 | F | 1.630 | 0.005 | No |
| **Mission Bay Drive** | | | | | | | | | |
| Garnet Avenue to Bunker Hill Street | 40,000 | 42,020 | F | 1.051 | 42,200 | F | 1.055 | 0.004 | No |
| Bunker Hill Street to Grand Avenue | 40,000 | 38,580 | E | 0.965 | 38,910 | E | 0.973 | 0.008 | No |
| Grand Avenue to Rosewood Street | 40,000 | 62,730 | F | 1.568 | 63,040 | F | 1.576 | 0.008 | No |
| Rosewood Street to N. Mission Bay Drive | 40,000 | 62,540 | F | 1.564 | 62,810 | F | 1.570 | 0.006 | No |
| **Revere Avenue** | | | | | | | | | |
| Bunker Hill Street to Glendora Street | 2,200 | 950 | Better than C | | 1,050 | Better Than C | | - | No |
| **Bunker Hill Street** | | | | | | | | | |
| Mission Bay Drive to Del Rey Street | 8,000 | 4,240 | C | 0.530 | 4,340 | C | 0.543 | 0.013 | No |
| **Glendora Street** | | | | | | | | | |
| Mission Bay Drive to Revere Avenue | 2,200 | 790 | Better than C | | 910 | Better than C | | - | No |
| **Rosewood Street** | | | | | | | | | |
| Mission Bay Drive to Del Rey Street | 2,200 (LOS C) | 2,410 | worse than C | | 2,420 | worse than C | | - | No |

***Footnotes:***

a. Capacities based on City of San Diego's Roadway Classification & LOS table *(See Appendix C)*.
b. Average Daily Traffic
c. Level of Service
d. Volume to Capacity ratio
e. Δ denotes a Project-induced increase in the Volume to Capacity ratio

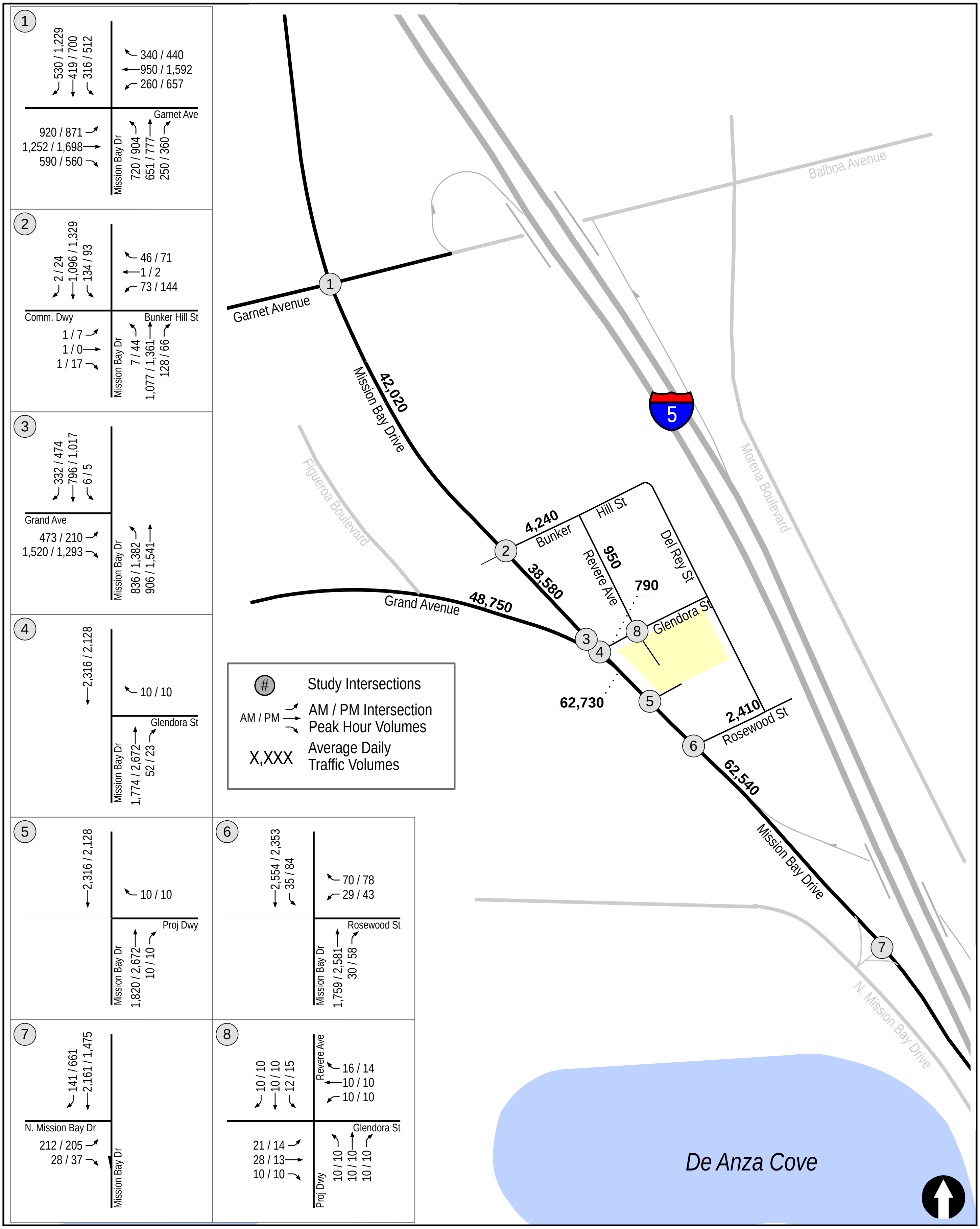


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**Figure 11-1**

## Year 2035 Without Project Traffic Volumes

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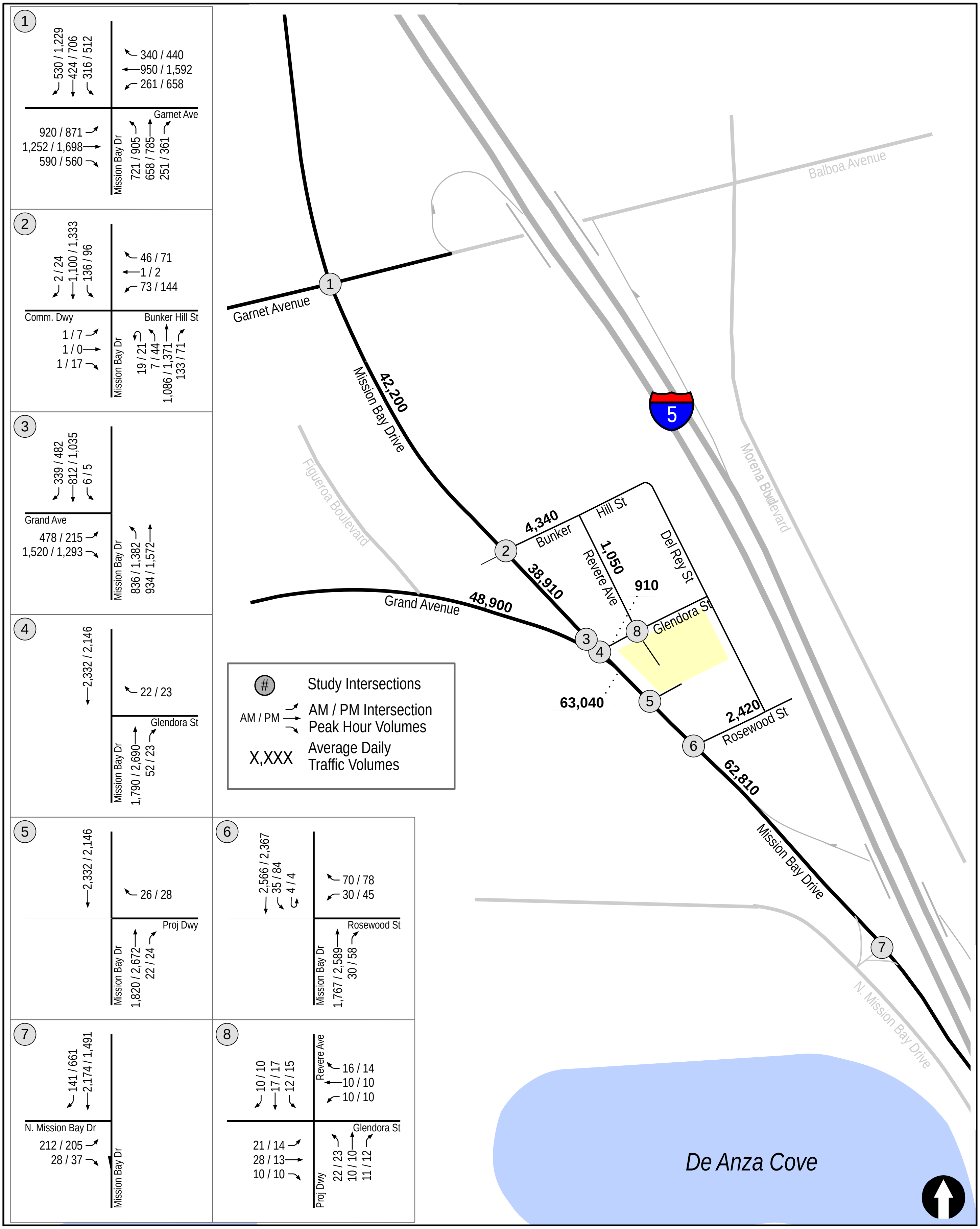


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**Figure 11-2**

## Year 2035 With Project Traffic Volumes

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## 12.0 ACCESS ANALYSIS

The project site is currently served by driveways on Mission Bay Drive and Glendora Street. The project proposes to maintain the existing access scheme to the site.

Both driveways are unsignalized. The driveway on Mission Bay Drive is limited to allow right in and right out movements only by the existing raised median along the project frontage. The unsignalized driveway on Glendora Street is full-access and allows all movements. The project does not propose to change the existing access to the site.

As shown in *Sections 10.0 and 11.0*, both driveways are calculated to operate at acceptable levels of service with the addition of the proposed project traffic.

# 13.0 ALTERNATIVE MODES

## 13.1 Pedestrian

Pedestrian access is provided via sidewalks along both access streets: Mission Bay Drive and Glendora Street. The project proposes non-contiguous sidewalks on Mission Bay Drive and Glendora Street along the project frontage.

## 13.2 Bicycle

A Class II bike lane already exists on southbound Mission Bay Drive, south of Grand Avenue. The Pacific Beach Community Plan includes bike lanes on northbound Mission Bay Drive. Along the project frontage, the northbound side of Mission Bay Drive currently is 4 lanes and 48 feet wide. This width is sufficient for two left turn lanes (each 10 foot wide), a through lane (11 feet wide), a through lane (12 feet wide) and a bike lane (5 feet wide).

## 13.3 Transit

MTS Bus routes 30 and 27 currently serve this area on Grand Avenue.

Route 30 runs between Downtown to UTC. A westbound bus stop is located approximately 50 feet northwest of Mission Bay Drive/Grand Avenue intersection and an eastbound stop is located approximately 600 feet northwest of Mission Bay Drive/Grand Avenue intersection. Weekday and weekend headways are approximately every 15 minutes.

Route 27 runs between Pacific Beach and Kearny Mesa. A westbound bus stop is located approximately 280 feet east of Mission Bay Drive/Garnet Avenue intersection and an eastbound bus stop is located approximately 580 feet east of Mission Bay Drive/Garnet Avenue intersection. Weekday headways are approximately every 30 minutes. Saturday headways are hourly with no service provided on Sundays.

Additionally, a new trolley station will be constructed at the southeast corner of the Balboa Avenue / I-5 interchange as part of the Mid-Coast Trolley Line. Access to the trolley station will be provided via sidewalks on Mission Bay Drive and Garnet Avenue.

## 14.0 PARKING

According to the *City of San Diego Municipal Code, Chapter 14, Article 2, Division 5, Page 27, Table 142-05G*, "visitor accommodations" (i.e. hotel rooms) require 1 parking space per guest room.

The project proposes 106[6] rooms, requiring 106 parking spaces. The project proposes to provide 107 onsite parking spaces, meeting the City's minimum requirement.

[6] Since the preparation of the TIA, the project description has been revised to include a total of 106 rooms. The project proposes to provide 106 onsite parking spaces, meeting the City's minimum requirement.

## 15.0 SIGNAL WARRANTS

A signal warrant assessment was conducted for the Mission Bay Drive / Rosewood Street intersection to determine if a traffic signal is warranted.

Traffic signal warrant analyses were conducted in accordance with the *Manual on Uniform Traffic Control Devices (MUTCD),* for Warrant 3–Peak Hour. The Near-Term (Opening Day 2020) + Project and Year 2035 (Horizon Year) + Project traffic volumes were used in this analysis. The calculation worksheets are attached in ***Appendix L***.

Based on the MUTCD Warrant 3 criteria, ***the results indicate that a traffic signal is warranted at the Mission Bay Drive / Rosewood Street intersection at the project's expected opening day (2020).***

## 16.0 SIGNIFICANCE OF IMPACTS

Per the City's significance thresholds and the analysis methodology presented in this report, project related traffic is calculated to cause one (1) significant direct and cumulative impact within the study area. The following section identifies the significant of impact and recommended mitigation measures.

### 16.1 Significant Impact

Under *Existing + Project, Near-Term (Opening Day 2020) + Project and Year 2035 (Horizon Year) + Project* conditions, project related traffic is calculated to cause one (1) significant direct and cumulative impact within the study area, as summarized below in ***Tables 16–1*** and ***16–2***, respectively.

TABLE 16–1
SIGNIFICANT DIRECT IMPACT

| Facility Type | Location |
|---|---|
| Intersections | • Mission Bay Drive / Rosewood Street<br>*(LOS F during the AM and PM peak hours)* |
| Street Segments | • None |

TABLE 16–2
SIGNIFICANT CUMULATIVE IMPACT

| Facility Type | Location |
|---|---|
| Intersections | • Mission Bay Drive / Rosewood Street<br>*(LOS F during the AM and PM peak hours)* |
| Street Segments | • None |

## 16.2 Mitigation Measures

This section discusses the proposed mitigation measures for the impacted facilities under all scenarios.

### 16.2.1 *Direct Impact Mitigation Measure*

Under Existing + Project and Near-Term (Opening Day 2020) + Project conditions, the project is calculated to cause a significant direct impact at one (1) intersection. The following summarizes the recommended mitigation measures to reduce the project's significant direct impact to below a level of significance:

*Intersection Mitigation*

The following intersection improvements are identified to mitigate the Project's significant direct impact. The intersection calculation sheets are contained in ***Appendix M***. ***Table 16–3*** shows the Existing + Project post mitigation intersection analysis. ***Table 16–4*** shows the Near-Term (Opening Day 2020) + Project post mitigation intersection analysis.

***Mission Bay Drive / Rosewood Street***

- Install a traffic signal.
- The installation of a traffic signal at Mission Bay Drive / Rosewood Street intersection is also a mitigation measure for the Jefferson Pacific Beach Project (*project No: 327976*). The opening day (Year 2019) of the Jefferson Pacific Beach project is anticipated prior to the opening day (Year 2020) of the proposed project since the Jefferson Pacific Beach project is under construction.
- If the Jefferson Pacific Beach does not install the traffic signal, then the proposed project shall install the traffic signal at Mission Bay Drive / Rosewood Street intersection. This improvement shall be assured by permit and bond satisfactory to the City Engineer prior to the issuance of the first building permit and constructed prior to the issuance of the first certificate of occupancy.
- The installation of a traffic signal by the project's opening day would mitigate the project's direct and cumulative impact to below a level of significance.

### TABLE 16–3
### EXISTING + PROJECT INTERSECTION MITIGATION ANALYSIS

| Intersection | Control Type | Peak Hour | Existing | | Existing + Project | | | Existing + Project With Mitigation | | Mitigation |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Delay[a] | LOS[b] | Delay | LOS | Δ[c] | Delay | LOS | |
| 6. Mission Bay Drive / Rosewood Street | MSSC[d]/ Signal | AM | **41.0** | **E** | **54.3** | **F** | **13.3** | 3.5 | A | Install a traffic signal. |
| | | PM | **>200** | **F** | **>200** | **F** | **–** | 3.2 | A | |

### TABLE 16–4
### NEAR-TERM (OPENING DAY 2020) + PROJECT INTERSECTION MITIGATION ANALYSIS

| Intersection | Control Type | Peak Hour | Near-Term (Opening Day 2020) | | Near-Term (Opening Day 2020) + Project | | | Near-Term (Opening Day 2020) + Project With Mitigation | | Mitigation |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Delay[a] | LOS[b] | Delay | LOS | Δ[c] | Delay | LOS | |
| 6. Mission Bay Drive / Rosewood Street | MSSC[d]/ Signal | AM | **>200.0** | **F** | **>200.0** | **F** | **>100.0** | 7.7 | A | Install a traffic signal. |
| | | PM | **>200.0** | **F** | **>200.0** | **F** | **>100.0** | 24.6 | C | |

### 16.2.2 *Cumulative Impact Mitigation Measure*

Under Year 2035 (Horizon Year) + Project conditions, the project is calculated to cause a significant cumulative impact at one (1) intersection. The following summarizes the recommended mitigation measures to reduce the project's significant cumulative impact to below a level of significance:

#### *Intersection Mitigation*

The following intersection improvements are identified to mitigate the Project's significant cumulative impact. The intersection calculation sheets are contained in ***Appendix M***. ***Table 16–5*** shows the Year 2035 + Project post mitigation intersection analysis.

***Mission Bay Drive / Rosewood Street***

- The direct impact mitigation of installing a traffic signal would mitigate the project's cumulative impact as well. It is also worth noting that if the opening day of the Jefferson Pacific Beach occurs *prior* to the opening day of the proposed project, no significant project cumulative impact would be calculated at Mission Bay Drive / Rosewood Street intersection as a traffic signal would already be installed by the Jefferson Beach project to meet their project's condition of approval obligations.

TABLE 16–5
YEAR 2035 (HORIZON YEAR) + PROJECT INTERSECTION MITIGATION ANALYSIS

| Intersection | Control Type | Peak Hour | Year 2035 (Horizon Year) | | Year 2035 (Horizon Year) + Project | | | Year 2035 (Horizon Year) + Project With Mitigation | | Mitigation |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | **Delay[a]** | **LOS[b]** | **Delay** | **LOS** | **Δ[c]** | **Delay** | **LOS** | |
| 6. Mission Bay Drive / Rosewood Street | MSSC[d]/ Signal | AM<br>PM | **>200.0**<br>**>200.0** | **F**<br>**F** | **>200.0**<br>**>200.0** | **F**<br>**F** | **>100.0**<br>**>100.0** | 13.3<br>44.5 | B<br>D | Install a traffic signal. |

***Footnotes:***

a. Average delay expressed in seconds per vehicle.
b. Level of Service
c. Δ denotes the increase in delay due to Project.
d. Minor Street Stop Controlled intersection. Minor street critical movement delay is reported.

| SIGNALIZED DELAY/LOS THRESHOLDS Delay | LOS | UNSIGNALIZED DELAY/LOS THRESHOLDS Delay | LOS |
|---|---|---|---|
| 0.0 ≤ 10.0 | A | 0.0 ≤ 10.0 | A |
| 10.1 to 20.0 | B | 10.1 to 15.0 | B |
| 20.1 to 35.0 | C | 15.1 to 25.0 | C |
| 35.1 to 55.0 | D | 25.1 to 35.0 | D |
| 55.1 to 80.0 | E | 35.1 to 50.0 | E |
| ≥ 80.1 | F | ≥ 50.1 | F |

# GEOTECHNICAL EVALUATION AND LIQUEFACTION ANALYSIS 4345 MISSION BAY DRIVE SAN DIEGO, CALIFORNIA

**PREPARED FOR:**
R&S Hospitality
521 Roosevelt Avenue
National City, California 91950

**PREPARED BY:**
Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
5710 Ruffin Road
San Diego, California 92123

March 4, 2016
Project No. 108107001

March 4, 2016
Project No. 108107001

Mr. Nilesh Patel
R&S Hospitality
521 Roosevelt Avenue
National City, California 91950

Subject: Geotechnical Evaluation and Liquefaction Analysis
4345 Mission Bay Drive
San Diego, California

Dear Mr. Patel:

In accordance with your request and authorization, we have prepared this geotechnical evaluation and liquefaction analysis for the construction of the proposed hotel at 4345 Mission Bay Drive in San Diego, California. This report presents our geotechnical findings, conclusions, and recommendations regarding the proposed development. We appreciate the opportunity to be of service on this project.

Respectfully submitted,
**NINYO & MOORE**

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CAT/JTK/JMM/KHM/gg

Distribution: (1) Addressee

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## 1. INTRODUCTION

In accordance with our proposal dated January 21, 2016, we have prepared this geotechnical evaluation and liquefaction analysis report for the proposed project. The project is located at 4345 Mission Bay Drive in San Diego (Figure 1). This report presents the results of our background review, subsurface evaluation, geotechnical laboratory testing, geotechnical analyses, and conclusions regarding the geotechnical and geologic conditions at the project site. Also presented are recommendations for design and construction of the proposed project.

## 2. SCOPE OF SERVICES

Our scope of services for this study included the following:

- Reviewing background information including available geologic and fault maps, stereoscopic aerial and historical photographs, and provided site plans (Architects CK, 2015).
- Performing a geologic reconnaissance of the subject site.
- Siting and staking of exploratory soil boring and cone penetration test (CPT) locations for clearance of potential conflicts with existing member utilities by Underground Service Alert (USA).
- Obtaining boring and CPT permits for our subsurface evaluation from the County of San Diego Department of Environmental Health (DEH). The work was conducted under DEH permit number LMWP-002128.
- Performing a geotechnical subsurface exploration consisting of the drilling, sampling, and logging of three exploratory borings and the advancement of three CPT soundings. Bulk and relatively undisturbed soil samples were obtained from the borings and transported to our in-house geotechnical laboratory for testing.
- Performing geotechnical laboratory testing on representative soil samples to evaluate soil characteristics, design parameters, and the potential for liquefaction of onsite soils.
- Compiling and analyzing the data obtained from our background research, subsurface evaluation, and geotechnical laboratory testing.
- Preparing this report presenting our findings, conclusions, and recommendations regarding and geotechnical design and construction aspects of the project.

## 3. SITE AND PROJECT DESCRIPTION

The project site is an approximately 1.25 acre lot located at 4345 Mission Bay Drive in the Mission Bay area of the City of San Diego (Figure 1). The site fronts on Mission Bay Drive to the west, is bounded by Glendora Street to the north and by residential and hotel properties to the east and south. Current development at the site includes one- and two-story hotel buildings and a two-story charter bus company building. Additional existing improvements include asphalt concrete (AC) parking areas and concrete flatwork with some landscaping. Per the survey included in the project plans (Howell, 2015), elevations at the site range from approximately 16 feet above mean sea level (MSL) along the western portion of the site to approximately 23 feet above MSL in the northeastern portion.

Based on our review of the referenced project plans (Architects CK, 2015), we understand that the proposed project will include the demolition of the existing improvements and the construction of a new three-story hotel building. The new building will be T-shaped and will be situated near the northern and western portions of the parcel. Further improvements will include a pool area, a patio, parking areas, a trash enclosure, and underground utilities.

## 4. GEOTECHNICAL SUBSURFACE EXPLORATION

Our geotechnical subsurface exploration was conducted on February 2 and February 8, 2016, and consisted of the drilling, logging, and sampling of three small-diameter exploratory borings (B-1 through B-3) and the advancement of three CPT soundings (CPT-1 through CPT-3). Exploratory boring B-1 was excavated to a depth of approximately 76½ feet using a truck-mounted drill rig equipped with 8-inch diameter, hollow-stem augers. Borings B-2 and B-3 were manually excavated to depths of approximately 4 feet. The CPTs were advanced using a truck-mounted sounding system to depths up to approximately 46 feet. Figure 2 is a map showing the locations of our borings and CPTs. The boring and CPT logs are presented in Appendix A.

## 5. GEOTECHNICAL LABORATORY TESTING

During the performance of our geotechnical subsurface exploration, relatively undisturbed and bulk soil samples were collected at selected depths from within the borings. These samples were then transported to our in-house laboratory for geotechnical testing. Geotechnical laboratory testing included an evaluation in-situ moisture content and dry density, gradation, Atterberg Limits, shear strength, expansion index, soil corrosivity, and R-value. The results of the in-situ moisture content and dry density tests are shown at the corresponding sample depth on the boring logs in Appendix A. The results of the other laboratory tests performed are presented in Appendix B.

## 6. GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional geology, site geology, and groundwater conditions are provided in the following sections.

### 6.1. Regional Geologic Setting

The project area is located in the western San Diego County section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 1998). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The portion of the province in San Diego County that includes the project area consists generally of Quaternary-age surficial deposits, underlain by Tertiary- and Cretaceous-age sedimentary rocks. Figure 3 is a map showing the geology of the project area.

### 6.2. Site Geology

The geologic units encountered during our subsurface exploration included fill and materials mapped as Quaternary-age old paralic deposits (Kennedy & Tan, 2008). Generalized descriptions of the units encountered are provided in the subsequent sections. Additional descriptions are provided on the boring logs in Appendix A. Geologic cross sections of the site are presented on Figures 4 and 5.

### 6.2.1. Fill

Fill soils were encountered in our borings from the ground surface or underlying the existing pavement sections and extending to depths up to approximately 5 feet. Fill materials generally consisted of brown and dark brown, moist, medium dense, silty and clayey sand and stiff, sandy clay. Scattered gravel was encountered in the fill materials.

### 6.2.2. Old Paralic Deposits ($Qop_6$)

Materials mapped as unit 6 of the old paralic deposits (Kennedy & Tan, 2008) were encountered underlying the fill and extending to the total depths explored. As encountered, the old paralic deposits generally consisted of various shades of brown and gray, moist to wet, medium dense to very dense, silty to clayey sand and sandy silt and poorly graded sand with silt. These deposits also included stiff to hard, clayey silt and silty to sandy clay and highly plastic clay. Scattered shells and gravel were encountered in the old paralic deposits.

## 6.3. Groundwater

Groundwater was encountered in boring B-1 at a depth of approximately 19½ feet. Based on the results from our explorations and review of nearby monitoring well data from the Geotracker (2016) website, design groundwater elevations should be based on historical data, which indicates an elevation of approximately 8½ feet above MSL. Fluctuations in the groundwater level may occur due to variations in tidal fluctuations, ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors. Additionally, seepage may be encountered at excavations that are performed at or near existing underground utility lines. Existing underground utility trenches may act as a conduit for subsurface water.

# 7. GEOLOGIC HAZARDS

The following sections describe potential geologic hazards at the site, including faulting and seismicity, strong ground motion, ground surface rupture, liquefaction, seismically-induced settlement, lateral spreading, tsunamis, seiches, and landsliding.

## 7.1. Faulting and Seismicity

The subject site is considered to be in a seismically active area. Our review of readily available published geological maps and literature indicates that traces of the Rose Canyon fault zone are mapped east of the site. However, there are no indications that there are known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively) underlying the site. Major known active faults in the region consist generally of en-echelon, northwest-striking, right-lateral, strike-slip faults. These include the San Andreas, Elsinore, and San Jacinto faults located northeast of the site, and the San Clemente, San Diego Trough, and Coronado Bank faults located to the west of the site (Figure 6).

The closest known active fault is the Rose Canyon fault zone, which can generate an earthquake of up to magnitude 6.9 (USGS, 2008). Quadrangle scale geologic mapping (Kennedy and Tan, 2008) depicts multiple concealed, approximate, and accurately located fault traces within the Rose Canyon fault zone east of the site (Figure 3). The City of San Diego maps a potentially active, inactive, presumed inactive, or activity unknown concealed fault strand within the Rose Canyon fault zone approximately 200 feet east of the site, and additional splays within the fault zone are mapped further to the east (Figure 7).

In general, hazards associated with seismic activity include strong ground motion, ground surface rupture, liquefaction, and seismically induced settlement. Discussion of these considerations is included in the following sections.

### 7.1.1. Ground Motion

The 2013 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake ($MCE_R$) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The $MCE_R$ ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the $MCE_R$ for the site was calculated as 0.50g using the United States Geological Survey (USGS, 2016) seismic design tool (web-based). Spectral response acceleration parameters, consistent with the 2013 CBC, are also provided in the recommendations section of this report for the evaluation of seismic loads on buildings and other structures.

The 2013 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean ($MCE_G$) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The $MCE_G$ peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The $MCE_G$ peak ground acceleration with adjustment for site class effects ($PGA_M$) was calculated as 0.57g using the USGS (USGS, 2016) seismic design tool that yielded a mapped $MCE_G$ peak ground acceleration of 0.57g for the site and a site coefficient ($F_{PGA}$) of 1.00 for Site Class D.

### 7.1.2. Ground Surface Rupture

Based on our review of background data, it is our opinion that active faults do not cross the subject site and, therefore, the potential for ground surface rupture due to active faulting is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

### 7.1.3. Liquefaction

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 60 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The project site is located in an area mapped as having a high potential for liquefaction on the City of San Diego Seismic Safety Study map (City of San Diego, 2009). Accordingly, liquefaction potential of subsurface soils was evaluated using the CPT data and the historic high groundwater elevation of approximately 8½ above mean seal level. Our site- analysis used a peak ground acceleration of 0.57g based on the design seismic event. The liquefaction analysis was based on the NCEER procedure (Youd et al., 2001) developed from the methods originally recommended by Seed and Idriss (1982) using the computer program LiquefyPro (CivilTech Software, 2008). Our liquefaction analysis indicates that the granular soil layers occurring below the historic high groundwater level and up to a depth of approximately 35 feet below the ground surface are susceptible to liquefaction during the design seismic event.

### 7.1.4. Seismically Induced Settlement

As a result of liquefaction, the proposed structure may be subject to several hazards including liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) was used in which the seismically induced cyclic stress ratios and corrected tip resistance values are related to the volumetric strain of the soil. The amount of soil settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density and/or consistency of the soils.

Under the current conditions, a post-earthquake total settlement of up to approximately 1 inch is estimated for the liquefiable soils located below the historic high groundwater. Based on the guidelines presented in Special Publication 117A (CGS, 2008) and considering the subsurface stratigraphy across the site, we estimate differential settlement on the order of ¾ inch over a horizontal distance of 40 feet.

### 7.1.5. Lateral Spreading

Lateral spread of the ground surface during an earthquake usually takes place along weak shear zones that have formed within a liquefiable soil layer. Lateral spread has generally been observed to take place in the direction of a free-face (i.e., retaining wall, slope, channel, etc.) but has also been observed to a lesser extent on ground surfaces with very gentle slopes. An empirical model developed by Youd et al. (2002) is typically used to predict the amount of horizontal ground displacement within a site. For sites located in proximity to a free-face, the amount of lateral ground displacement is correlated with the distance of the site from the free-face. Other factors such as earthquake magnitude, distance from the causative fault, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also influence the amount of lateral ground displacement.

While the project site gently slopes to the southwest, the underlying old paralic deposits have corrected standard penetration test (SPT) sampler blow counts of more than 15, and the distance of the site from the nearest free-face is over 1,600 feet. Based on these considerations, it is our opinion that the potential for global occurrence of lateral spread at the site is low.

### 7.2. Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. Seiches are waves generated in a large enclosed body of water. The site is not within a mapped tsunami inundation area (CEMA, 2009) and it is our opinion that the potential for damage due to tsunamis and/or seiches is low.

### 7.3. Landslides

The site is located in an area classified as marginally susceptible to landslides (Tan, 1995). Based on our review of referenced geologic maps, literature, topographic maps, and aerial photographs, no landslides or related features underlie or are adjacent to the site. The potential for significant large-scale slope instability at the site is not a design consideration.

## 8. CONCLUSIONS

Based on the results of our subsurface evaluation, geotechnical laboratory testing, and data analysis, the proposed improvements are feasible from a geotechnical standpoint, provided the recommendations of this report are incorporated in the design and construction of the project. Geotechnical considerations include the following:

- The project site is underlain by fill materials and old paralic deposits. The existing fill is undocumented and is not considered suitable for structural support in its current condition. Recommendations for the remedial grading of this material are presented in the following sections.

- Based on our subsurface exploration, excavation of the subsurface materials should generally be feasible with heavy-duty excavation equipment in good working condition. The contractor should be prepared to mitigate caving conditions that could be encountered during construction.

- Based on our review of mapping, it is our opinion that active faults do not cross the subject property. However, the project site is located adjacent to the active Rose Canyon fault zone. Accordingly, the potential for relatively strong seismic ground motions should be considered in the project design.

- The results of our geotechnical evaluation indicate that the project site is underlain by soils susceptible to liquefaction. Our analysis of the subsurface data indicates that up to 1 inch of seismically induced settlement could occur during a major seismic event (Appendix C).
- Our evaluation indicates that groundwater may be encountered at an elevation of approximately 8½ feet above MSL. Due to variations within the subsurface soil, fluctuations in the groundwater table and seepage conditions, groundwater may be encountered at shallower depths. Excavations that extend into or near the groundwater table should anticipate pumping or yielding soils conditions.
- Based on the proximity of the site to a marine environment, the project site soils should be considered corrosive.

## 9. RECOMMENDATIONS

Based on our understanding of the project, the following recommendations are provided for the design and construction of the proposed project.

### 9.1. Earthwork

In general, earthwork should be performed in accordance with the recommendations presented in this report. Ninyo & Moore should be contacted for questions regarding the recommendations or guidelines presented herein.

#### 9.1.1. Site Preparation

We understand that the existing structures will be demolished or removed as part of this project. Site preparation should begin with the removal of the existing structures, as well as vegetation, utility lines, asphalt, concrete, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area. Underground utilities or other structures located within the proposed limits of the construction should be removed or abandoned, capped off or relocated so as not to interfere with earthwork operations.

### 9.1.2. Temporary Excavations

For temporary excavations, we recommend that the following Occupational Safety and Health Administration (OSHA) soil classifications be used:

| | |
|---|---|
| *Fill* | *Type C* |
| *Old Paralic Deposits* | *Type B* |

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

### 9.1.3. Excavation Characteristics

The results of our field exploration program indicate that the project site is underlain by fill materials and old paralic deposits. Based on our subsurface exploration of the site, excavation of the materials underlying the site should be generally feasible with heavy-duty excavation equipment in good working condition. Excavations that extend near or wet and clayey soils were encountered above the groundwater table that may also exhibit soft and yielding subgrade conditions. In general, the unstable bottom condition may be mitigated by an overexcavation and replacement with gravel, wrapped in a non-woven geotextile fabric. However, specific recommendations for stabilizing excavation bottoms should be based on evaluation in the field by Ninyo & Moore at the time of construction.

### 9.1.4. Remedial Grading – Building Pad

We recommend that the existing soils be overexcavated to a depth of 5 feet below finished building pad subgrade or 2 feet below the bottom of foundations, whichever is deeper. This overexcavation should extend to the horizontal limits of the building pad. For the purposes of this report, the building pad is defined as the structural footprint (including foundations for attached overhangs, canopies, and other building appurtenances) plus a horizontal distance of 5 feet. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed. The resultant overexcavation surface should be scarified to a depth of approximately 8 inches, moisture conditioned and recompacted to a relative compaction of 90 percent as evaluated by the ASTM International (ASTM) Test Method D 1557 prior to placing new fill. The resulting excavation should then be backfilled with generally granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less).

### 9.1.5. Remedial Grading – Pavements and Flatwork

In the proposed pavement and flatwork areas, we recommend that the on-site soils be overexcavated to a depth of 1 foot below the planned finished subgrade elevation. The proposed overexcavations should extend outward horizontally 2 feet from the horizontal limits of the pavement or flatwork. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed. The resulting surface should be scarified 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. The overexcavation should then be filled with engineered fill. The engineered fill should be moisture conditioned to near optimum moisture content and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557 beneath flatwork and 95 percent beneath vehicular pavements.

### 9.1.6. Materials for Fill and Backfill

On-site soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight) and a gradation in accordance with the following are suitable for re-use as engineered fill. In general, fill material should not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ¾ inch in diameter. Oversize materials should be separated from material to be used for fill and removed from the site. Fill placed within the upper 5 feet of the building pad area and within the upper 2 feet of concrete flatwork should be low expansion (i.e., an Expansion Index of 50 or less) materials.

Utility trench backfill material should not contain rocks or lumps over approximately 3 inches in diameter in general. Soils classified as silts or clays should not be used for backfill in the pipe zone. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite.

Imported fill material, if needed for the project, should possess an organic content of less than approximately 3 percent by volume (or 1 percent by weight), be granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less as evaluated by the ASTM D 4829), and meet the following gradation. The imported select fill material should be granular, not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ¾ inch in diameter. Import material should also be non-corrosive in accordance with the Caltrans (2012) corrosion guidelines and ACI 318. A non-corrosive soil is defined as having an electrical resistivity of 1,000 ohm-centimeters (ohm-cm) or more, less than 500 ppm of chlorides, less than 0.1 percent sulfates, and a pH less than 5.5. Import material should also be non-corrosive in accordance with the Caltrans (2012) corrosion guidelines. Materials for use as fill should be evaluated by Ninyo & Moore's representative prior to filling or importing.

### 9.1.7. Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify this office and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill materials should be moisture conditioned to generally above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods, to a relative compaction of 90 percent as evaluated by ASTM D 1557. The upper 12 inches of subgrade soils beneath vehicular pavements should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. The aggregate base materials beneath vehicular pavements should also be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

### 9.1.8. Pipe Bedding and Modulus of Soil Reaction (*E'*)

It is our recommendation that the new pipeline (pipes), where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report. Pipe bedding typically consists of graded aggregate with a coefficient of uniformity of three or more. The pipe bedding should conform to the specifications presented for pipe zone backfill materials.

Pipe bedding and pipe zone backfill should have a Sand Equivalent of 30 or more, and be placed around the sides and the crown of the pipe. In addition, the pipe zone backfill should extend 1 foot or more above the crown of the pipe. If open-graded gravel is used as pipe zone backfill, we recommend that the pipe bedding and pipe zone materials be wrapped in a non-woven geotextile fabric.

The modulus of soil reaction (*E'*) is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,600 pounds per square inch (psi) may be used for design provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

#### 9.1.9. Utility Trench Zone Backfill

Utility trench zone backfill material should not generally contain rocks or lumps greater than approximately 3 inches in diameter, and otherwise conform to the Materials for Fill section of this report. Materials for use as backfill should be evaluated by Ninyo & Moore's representative prior to filling or importing. Backfill should be moisture conditioned to generally at or above the laboratory optimum, placed, and compacted to a relative compaction of 90 percent as evaluated ASTM D 1557. In areas where pavements are to be constructed, the upper 12 inches of subgrade soils and base materials should be placed at a relative compaction of 95 percent as evaluated by ASTM D 1557. Wet soils should be allowed to dry to moisture contents near the optimum prior to their placement as backfill. Backfill lift thickness will be dependent upon the type of compaction equipment utilized. Backfill should generally be placed in lifts not exceeding 8 inches in loose thickness.

## 9.2. Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the sites in accordance with the CBC (2013) guidelines and adjusted $MCE_R$ spectral response acceleration parameters (USGS, 2016).

**Table 1 – 2013 California Building Code Seismic Design Criteria**

| Factors | Values |
|---|---|
| Site Class | D |
| Site Coefficient, $F_a$ | 1.0 |
| Site Coefficient, $F_v$ | 1.515 |
| Mapped Spectral Response Acceleration at 0.2-second Period, $S_s$ | 1.256g |
| Mapped Spectral Response Acceleration at 1.0-second Period, $S_1$ | 0.485g |
| Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, $S_{MS}$ | 1.256g |
| Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, $S_{M1}$ | 0.735g |
| Design Spectral Response Acceleration at 0.2-second Period, $S_{DS}$ | 0.838g |
| Design Spectral Response Acceleration at 1.0-second Period, $S_{D1}$ | 0.490g |

## 9.3. Shallow Foundations

The proposed building may be supported on shallow, spread, or continuous footings bearing on compacted fill. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

### 9.3.1. Bearing Capacity

Shallow, spread, or continuous footings supported on compacted fill may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf) based on the embedment depths described below. These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. From a geotechnical standpoint, spread or continuous footings should have an embedment depth of 18 inches. Continuous footings should have a width of 18 inches and isolated footings should be 24 inches in width. The footings should be reinforced in accordance with the recommendations of the structural engineer.

#### 9.3.2. Lateral Earth Pressures

For resistance of footings to lateral loads, we recommend an allowable passive pressure of 300 psf of depth be used with a value of up to 3,000 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.35 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

#### 9.3.3. Static Settlement

We estimate that the proposed apartment building, designed and constructed as recommended herein, will undergo total settlements of less than approximately 1 inch. Differential settlement on the order of ½ inch over a horizontal span of 40 feet should be expected. Note, in addition to the static settlement, as described earlier, the site is susceptible to seismically induced settlements due to liquefaction during a major seismic event on the order of 1 inch.

### 9.4. Swimming Pool Recommendations

Detailed design plans were not available for our review. However, we anticipate that the pool will consist of a gunite shell or concrete reinforced walls and floor. Design recommendations are presented below.

### 9.4.1. Swimming Pool Foundation and Pool Bottom

To reduce the potential for differential settlement of the pool, the pool bottom should rest wholly on competent old paralic deposits and not on a transition between compacted fill and old paralic deposits. Due to the potential for varying pool depths along with the varying depths to old paralic deposits, the planned elevations for the pool bottom may not coincide with the depth to old paralic deposits. In order to provide a uniform bearing surface and reduce the potential for differential settlements, we recommend that the portions of the pool that do not extend into old paralic deposits, be constructed on a controlled low-strength material (CLSM) foundation, such as two-sack cement-sand slurry foundation. Specifically, at the portions of the pool where the excavation does not extend into old paralic deposits, that portion of the excavation should be deepened such that competent old paralic deposits are exposed. Subsequent to that removal, the resulting overexcavation should be backfilled with two-sack cement-sand slurry to the bottom of the pool.

### 9.4.2. Bearing Capacity

As noted above, we recommend that the pool be founded wholly on competent old paralic deposits. Structures bearing on competent old paralic deposits may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf). The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. The pool wall and floor should be reinforced in accordance with the recommendations of the project structural engineer.

### 9.4.3. Lateral Resistance

For resistance of pool footings to lateral loads, we recommend an allowable passive pressure of 300 psf per foot of depth be used with a value of up to 3,000 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.35 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

#### 9.4.4. Lateral Earth Pressures

Swimming pool walls bordered by concrete decking (level conditions) may be designed using an at-rest earth pressure represented by an equivalent fluid weight of 60 pounds per cubic foot (pcf). Active and passive earth pressure represented by equivalent fluid weights of 40 and 300 pcf, respectively, may also be used for design. Pool walls should also be designed to resist lateral surcharge pressures imposed by any adjacent footings or structures in addition to the above lateral earth pressures. In the event the swimming pool is constructed to depths near or below the groundwater table, the pool should be designed to resist buoyant forces as shown on Figure 8.

#### 9.4.5. Stability of Temporary Pool Excavations

Temporary excavations in site soils may be performed with near-vertical sidewalls up to a depth of 4 feet, and at an inclination of 1:1 or flatter for slopes ranging in depth from 4 to 10 feet. Temporary excavations deeper than 10 feet should be performed at a slope inclination of 1.5:1 or flatter. Some surficial sloughing may, however, occur depending on the excavation depths and actual soil conditions encountered. Temporary slope excavations should be evaluated in the field by Ninyo & Moore. Forming of the pool walls may be required.

Slope setback requirements of the governing jurisdictions and applicable building codes should be followed during pool excavation operations. Any cuts exposed to seasonal precipitation or uncontrolled surface runoff may be easily eroded. Excavations should be performed in accordance with OSHA's regulations.

After the swimming pool walls are constructed, the backfill placed between the walls and temporary excavated slopes should be compacted. Backfill materials should be placed in uniform lifts not exceeding 8 inches in loose thickness, moisture conditioned as appropriate to achieve in-place moisture contents slightly above the laboratory optimum, and then mechanically compacted to a relative compaction of 90 percent or more as evaluated by the latest edition of ASTM D 1557. Flooding or jetting of the backfill should be avoided.

### 9.4.6. Temporary Access Ramps

Backfill materials placed within temporary access ramps extending into the pool excavations should be properly compacted and tested. This will mitigate excessive settlement of the backfill and subsequent damage to pool decking or other structures placed on the backfill.

### 9.4.7. Pool Decking

To reduce the potential for differential movement between the edge of the pool and the adjacent pool decking, we recommend that the pool decking within 10 of the edge of the pool, be doweled into the sidewalls of the pool. From a geotechnical standpoint, we recommend that the pool decking be 5 inches or more thick. The dowel sizing and spacing should be evaluated by the project structural engineer.

For pool decking and general site sidewalks, to reduce the potential for shrinkage cracking, the pool decking should be 5 inches thick. Crack control joints should be provided at an interval of every 6 feet or less. As a further measure to reduce cracking of pool decking, the subgrade soils to a depth of approximately 12 inches below the pool decking and general sidewalks should be compacted to a relative compaction of 90 percent or more in accordance with the latest edition of ASTM D 1557 at moisture contents generally above the laboratory optimum. The subgrade soils should be shaped to provide a minimum gradient of one percent away from the pool shell and towards a subsurface drainage system.

### 9.4.8. Plumbing Fixtures

Leakage from the swimming pool or the appurtenant plumbing fixtures could create adverse saturated conditions of the surrounding subgrade soils. Areas of over-saturation can lead to differential settlement of the subgrade soils and subsequent shifting of pool decking. Therefore, it is recommended that the plumbing and pool fixtures be inspected and maintained during the design life of the project. For similar reasons, drainage from the pool deck areas should be directed to area drains and/or swales designed to carry runoff water to suitable discharge locations.

## 9.5. Interior Building Slabs-on-Grade

We recommend that conventional, interior building slab-on-grade floors, underlain by compacted fill materials of generally very low to low expansion potential, be 5 inches in thickness and be reinforced with No. 3 reinforcing bars spaced 18 inches on center each way. The reinforcing bars should be placed near the middle of the slab height. As a means to help reduce shrinkage cracks, we recommend that the slabs be provided with crack control joints at intervals of approximately 12 feet each way. The slab reinforcement and expansion joint spacing should be designed by the project structural engineer.

If moisture sensitive floor coverings are to be used, we recommend that slabs be underlain by a vapor retarder and capillary break system consisting of a 10-mil polyethylene (or equivalent) membrane placed over 4 inches of medium to coarse, clean sand or pea gravel and overlain by an additional 2 inches of sand to help protect the membrane from puncture during placement and to aid in concrete curing. The exposed subgrade should be moistened just prior to the placement of concrete.

## 9.6. Retaining Walls

Retaining walls may be supported on a continuous footing bearing on compacted fill. Allowable bearing capacities of 3,000 psf may be used for the design of retaining wall foundations. The allowable bearing capacity may be increased by one-third when considering loads of short duration, such as wind or seismic forces.

For the design of a yielding retaining wall that is not restrained against movement by rigid corners or structural connections, lateral pressures are presented on Figure 9. Restrained walls (non-yielding) may be designed for lateral pressures presented on Figure 10. These pressures assume low-expansive backfill consisting of imported select fill as described earlier in this report and free draining conditions. Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. A drain should be provided behind the retaining wall as shown on Figure 11. The drain should be connected to an appropriate outlet.

### 9.7. Flexible Vehicular Pavements

Laboratory testing by Ninyo & Moore of the subgrade soils indicated an R-value of less than 5. However, we have used a design R-value of 5 for the preliminary basis for design of flexible pavements at the project site. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The recommended preliminary pavement sections for on-site areas should be as presented in Table 2. Off-site pavements should be constructed in accordance with the City of San Diego guidelines.

**Table 2 – Recommended Preliminary Flexible Pavement Sections**

| Traffic Index | Design R-Value | Asphalt Concrete (in) | Class 2 Aggregate Base (in) |
|---|---|---|---|
| 5 | 5 | 3 | 10 |
| 6 | 5 | 3½ | 13 |
| 7 | 5 | 4 | 16 |

These values assume traffic indices of seven or less for site pavements. In addition, we recommend that the upper 12 inches of the subgrade and aggregate base materials be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557. The AC materials should be compacted to a relative compaction of 95 percent as evaluated by the materials Hveem density. If traffic loads are different from those assumed, the pavement design should be re-evaluated.

We suggest that consideration be given to using Portland cement concrete pavements in areas where dumpsters will be stored and where refuse trucks will stop and load. Experience indicates that refuse truck traffic can significantly shorten the useful life of AC sections. We recommend that in these areas, 6 inches of 600 psi flexural strength Portland cement concrete be placed over 6 inches or more of Class 2 aggregate base compacted to 95 percent of its Proctor density (as evaluated by ASTM D 1557).

### 9.8. Exterior Pedestrian Concrete Flatwork

Exterior concrete flatwork should be 4 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on-center both ways. No vapor retarder is needed for exterior flatwork. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the project engineer. The subgrade soils should be scarified to a depth of 8 inches, moisture conditioned to generally above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Positive drainage should be established and maintained adjacent to flatwork.

### 9.9. Corrosive Soils

Laboratory testing to evaluate pH, electrical resistivity, soluble sulfate and chloride contents was performed on a representative sample of the near-surface soils. The pH and electrical resistivity tests were performed in accordance with California Test (CT) Method 643. Soluble sulfate and chloride content tests were performed in accordance with CT Methods 417 and 422, respectively. The results of the corrosivity tests are summarized below and presented in Appendix B.

The results of the corrosivity testing indicated a soil pH of approximately 7.5, an electrical resistivity on the order of 1,400 ohm-cm, a sulfate content of approximately 0.006 percent (i.e., 60 parts per million [ppm]), and a chloride content of 190 ppm for the tested sample. Based on the Caltrans (2012) criteria, ACI 318, and our experience with similar soils, the tested soils would not be classified as corrosive. A corrosive soil is defined as having an electrical resistivity of less than 1,000 ohm-cm, more than 500 ppm of chlorides, more than 0.1 percent sulfates, and/or a pH less than 5.5. However, due to the site's proximity to a marine environment, the site soils should be considered corrosive.

### 9.10. Concrete

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Laboratory testing indicated a sulfate content of the sample tested of 0.006 percent by weight. Based on ACI 318, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight. Thus, the sulfate exposure to concrete from near-surface site soils is considered negligible. However, we recommend that the use of Type II, V, or II/V cement be considered for the project due to potential variable soil conditions.

### 9.11. Site Drainage

Roof, pad, and slope drainage should be directed such that runoff water is diverted away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.). Positive drainage adjacent to structures should be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside the building perimeter, and further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained over the pad area and drainage patterns should be established to divert and remove water from the site to appropriate outlets.

Care should be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of final grading should be maintained for the life of the project. The property owner and the maintenance personnel should be made aware that altering drainage patterns might be detrimental to slope stability and foundation performance.

### 9.12. Infiltration Devices

Although specifics have not been provided to our office, we anticipate that the project may include the construction of pervious pavements or bio-retention swales. For the installation of infiltration devices, an elevation difference of 10 feet or more is recommended between the bottom of the infiltration device and the groundwater table. Due to a nearby groundwater elevation of approximately 8½ feet above MSL, the 10 feet elevation difference may not be feasible. If that is the case, the bottoms of infiltration devices should be lined with an impermeable layer. We recommend that infiltration systems be set back approximately 20 feet from future structures. Gravel reservoirs should generally be fully wrapped with a non-woven filter fabric (such as Mirafi 140N), to reduce the potential for fines to migrate to the voids in the gravel. In addition, site design may consider the use of pavement edge drains and cutoff curbs to reduce the potential for lateral migration of infiltration water from the gravel reservoir into adjacent subsurface soils beneath other improvements.

### 9.13. Pre-Construction Meeting

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the agency representatives, the architect, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the plans, the project, and the proposed construction schedule.

### 9.14. Plan Review and Construction Observation

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory excavations. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the owner with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

## 10. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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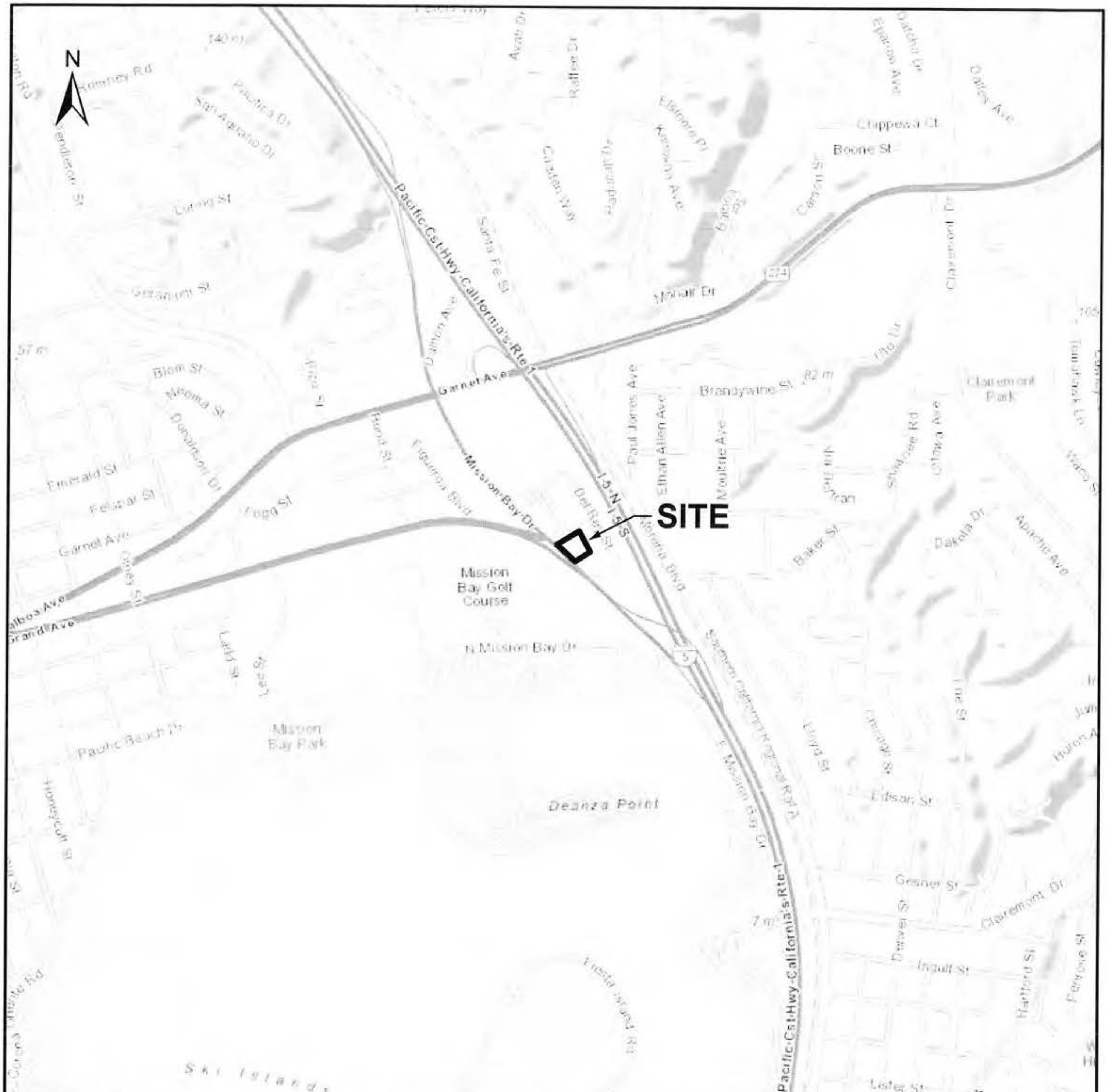
| AERIAL PHOTOGRAPHS | | | | |
|---|---|---|---|---|
| **Source** | **Date** | **Flight** | **Numbers** | **Scale** |
| USDA | March 31, 1953 | AXN-4M | 90 and 91 | 1:24,000 |




SOURCE: USGS, FAO, NPS, EPA, ESRI, DELORME, TANA, OTHER SUPPLIERS


MAP INDEX

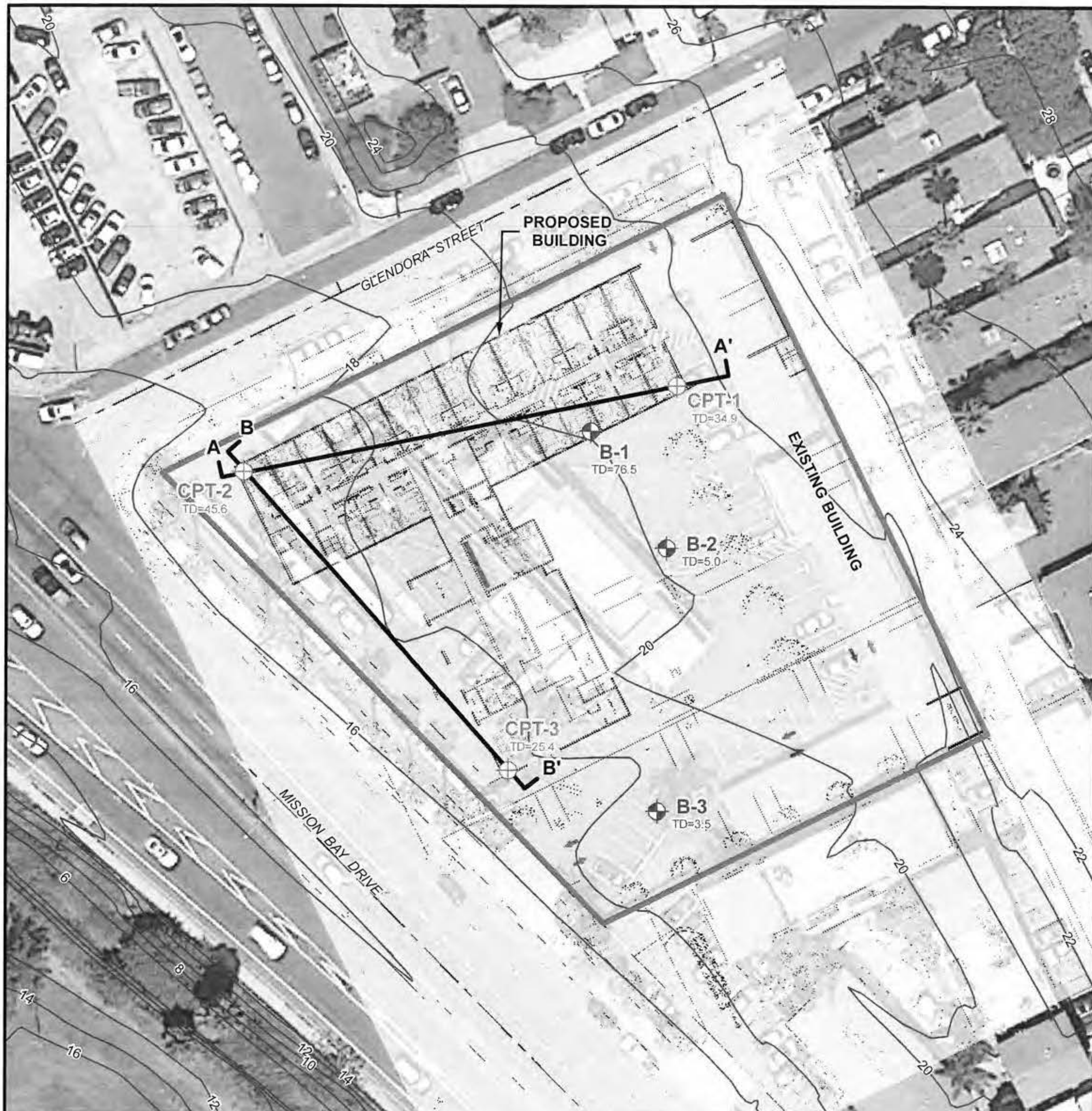
SCALE IN FEET

0 750 1,500 3,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE

| Ninyo & Moore | | SITE LOCATION | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 1 |
| 108107001 | 3/16 | | |

I_108107001_SL_93.mxd AOB




SOURCE: GOOGLE EARTH, 2016.

LEGEND

SITE BOUNDARY

B-3 TD=3.5 BORING
TD=TOTAL DEPTH IN FEET

CPT-3 TD=25.4 CONE PENETRATION TEST
TD=TOTAL DEPTH IN FEET

B B' GEOLOGIC CROSS SECTION

N

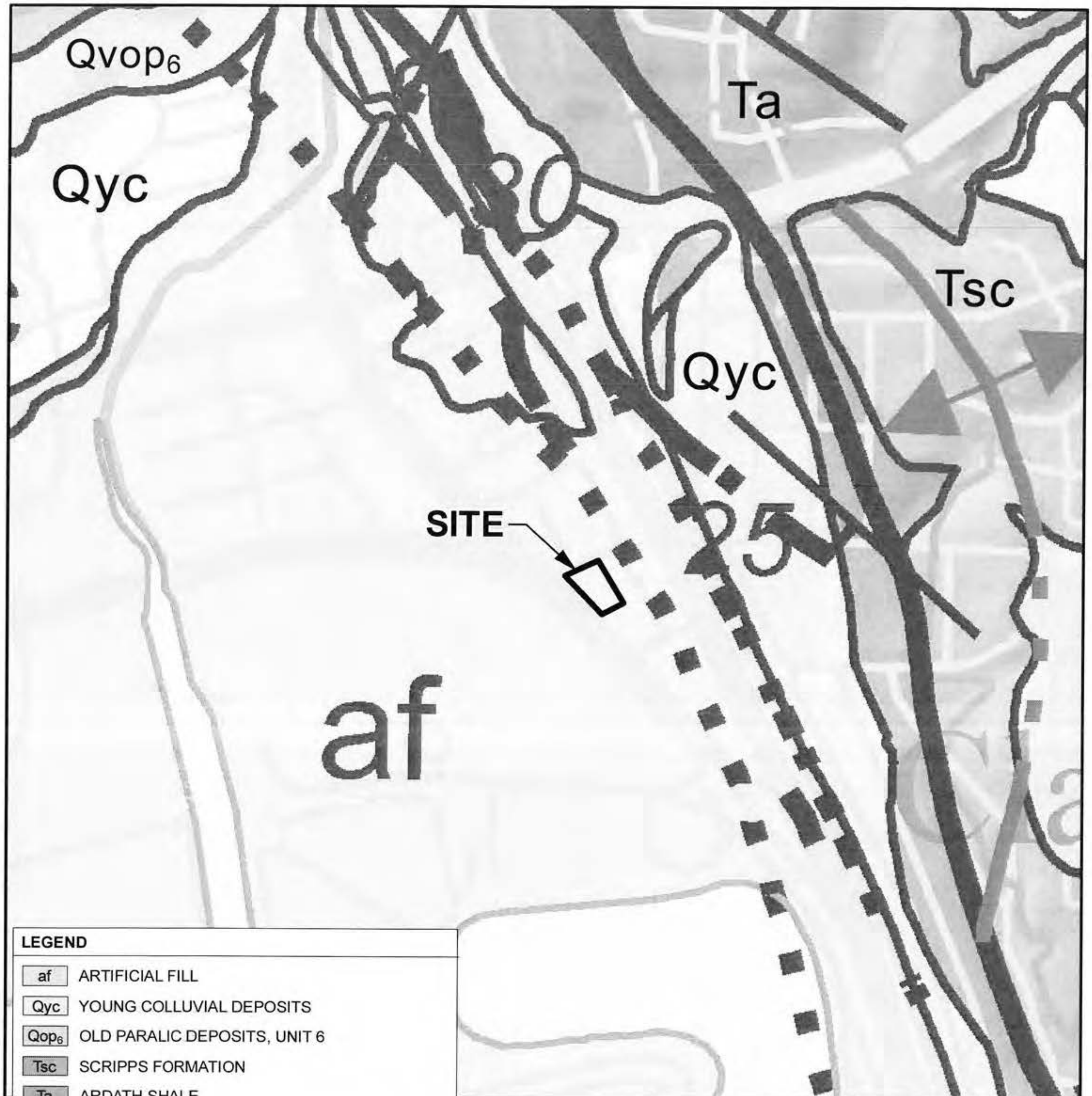
SCALE IN FEET

0 30 60 120

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE

| Ninyo & Moore | | EXPLORATION LOCATIONS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 2 |
| 108107001 | 3/16 | | |

2_108107001_EL.mxd AOB




**LEGEND**

| | |
|---|---|
| af | ARTIFICIAL FILL |
| Qyc | YOUNG COLLUVIAL DEPOSITS |
| $Qop_6$ | OLD PARALIC DEPOSITS, UNIT 6 |
| Tsc | SCRIPPS FORMATION |
| Ta | ARDATH SHALE |
| U/D | FAULT - SOLID WHERE ACCURATELY LOCATED, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED. ARROW AND NUMBER INDICATE DIRECTION AND ANGLE OF DIP OF FAULT PLANE |
| | ANTICLINE - SOLID WHERE ACCURATELY LOCATED, DOTTED WHERE CONCEALED. ARROW INDICATES DIRECTION OF AXIAL PLUNGE |
| 25 | STRIKE AND DIP OF BEDS, INCLINED |

REFERENCE: KENNEDY, M.P., AND TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30" X 60" QUADRANGLE, CALIFORNIA.

N

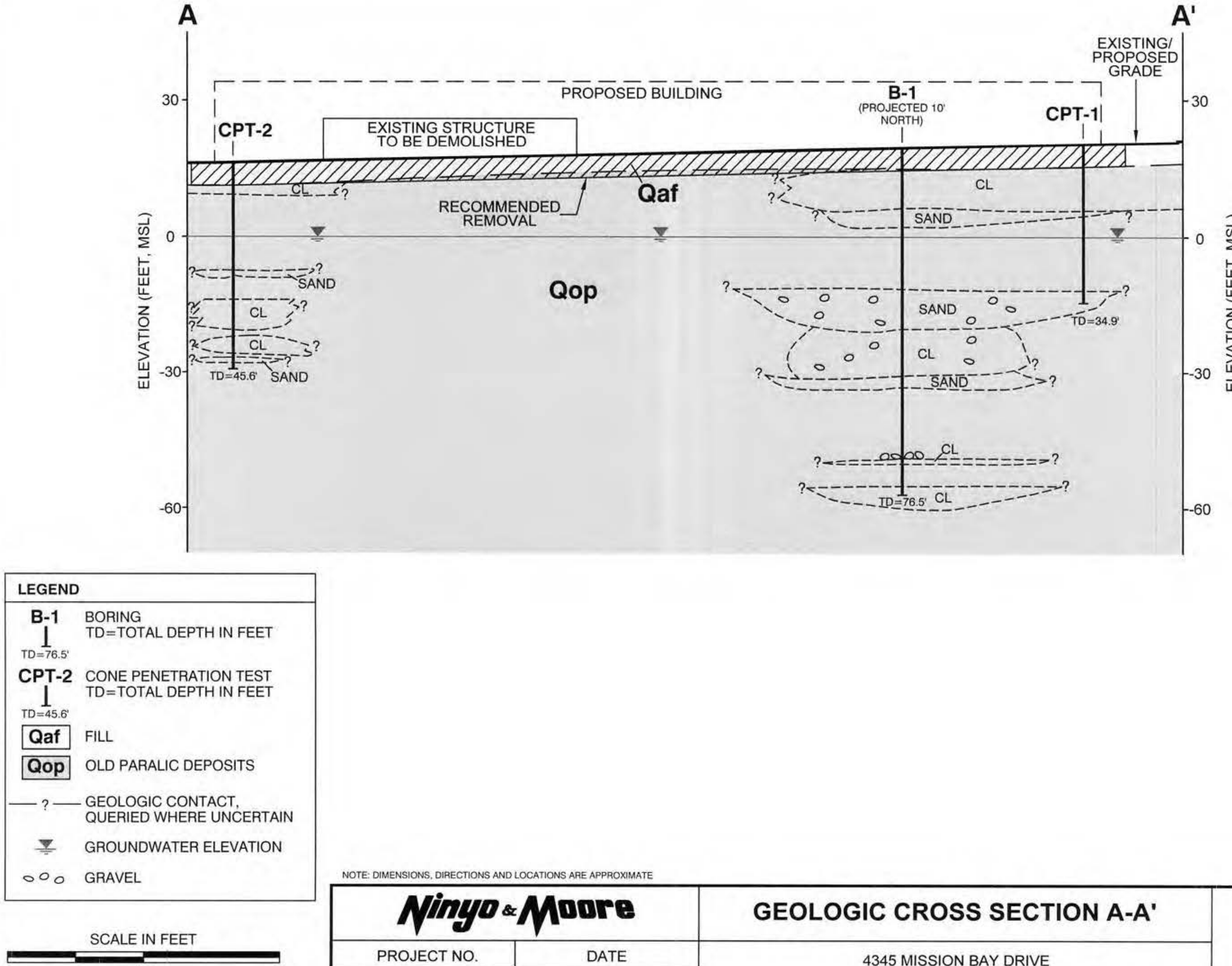
APPROXIMATE SCALE

0 800 1,600 FEET

NOTES: ALL DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE

3_108107001_G_93.mxd AOB

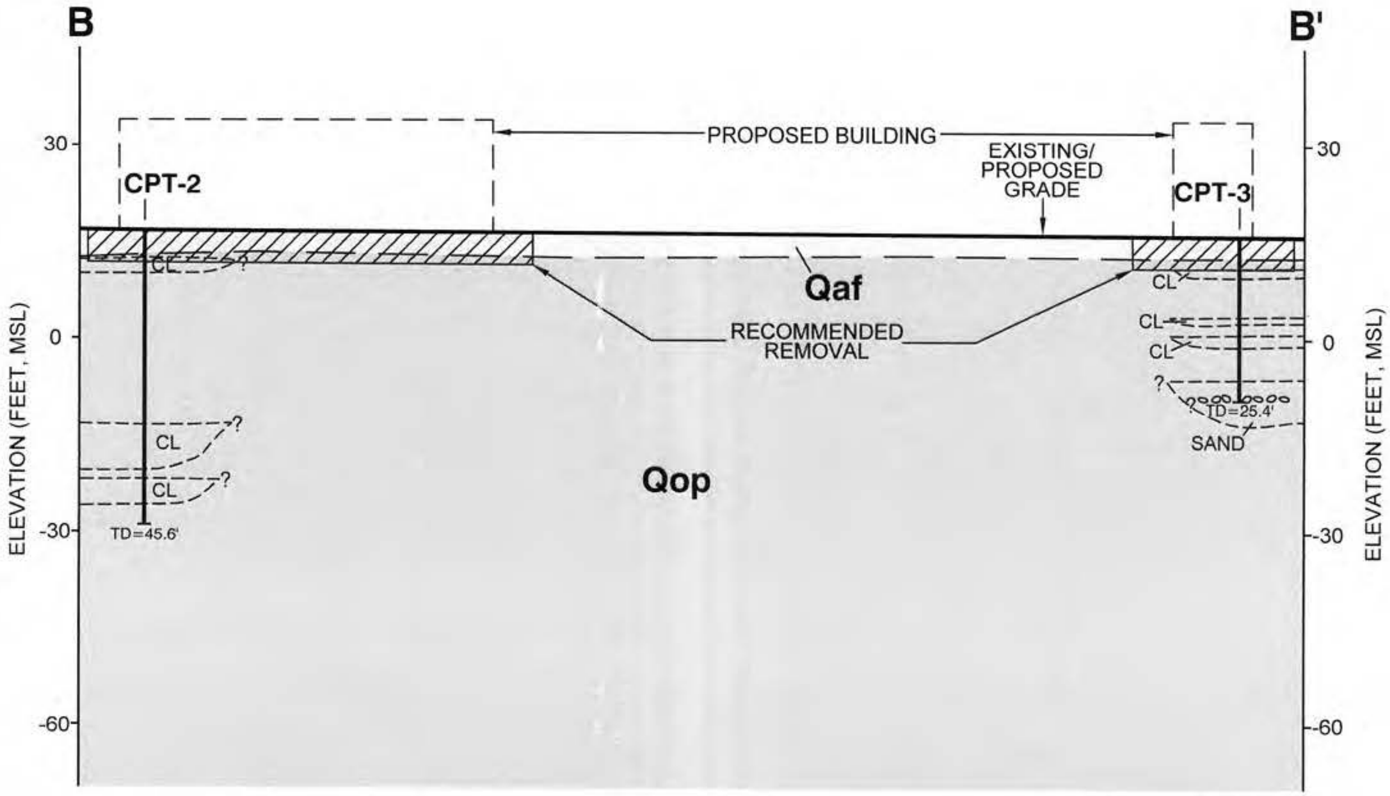
| Ninyo & Moore | | GEOLOGY | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 3 |
| 108107001 | 3/16 | | |




4 108107001 cs a-a'.dwg

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

| Ninyo & Moore | | GEOLOGIC CROSS SECTION A-A' | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 4 |
| 108107001 | 3/16 | | |




**LEGEND**

| | |
|---|---|
| **CPT-3** TD=25.4' | CONE PENETRATION TEST TD=TOTAL DEPTH IN FEET |
| **Qaf** | FILL |
| **Qop** | OLD PARALIC DEPOSITS |
| —— ? —— | GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN |
| ○ ○ ○ | GRAVEL |

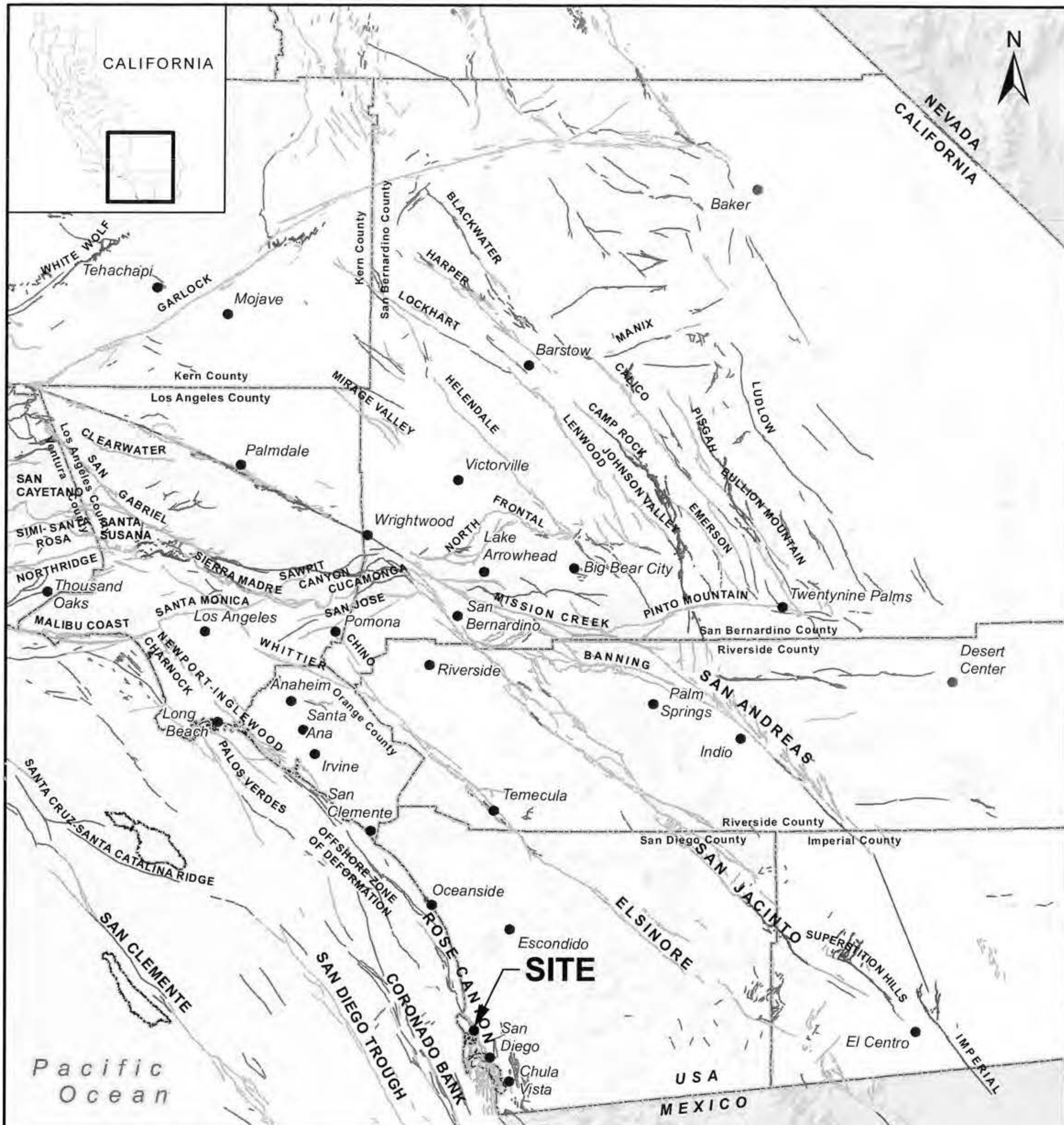
SCALE IN FEET

0 30 60

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

| Ninyo & Moore | | GEOLOGIC CROSS SECTION B-B' | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE SAN DIEGO, CALIFORNIA | **5** |
| 108107001 | 3/16 | | |

5 108107001 cs b-b'.dwg




LEGEND

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

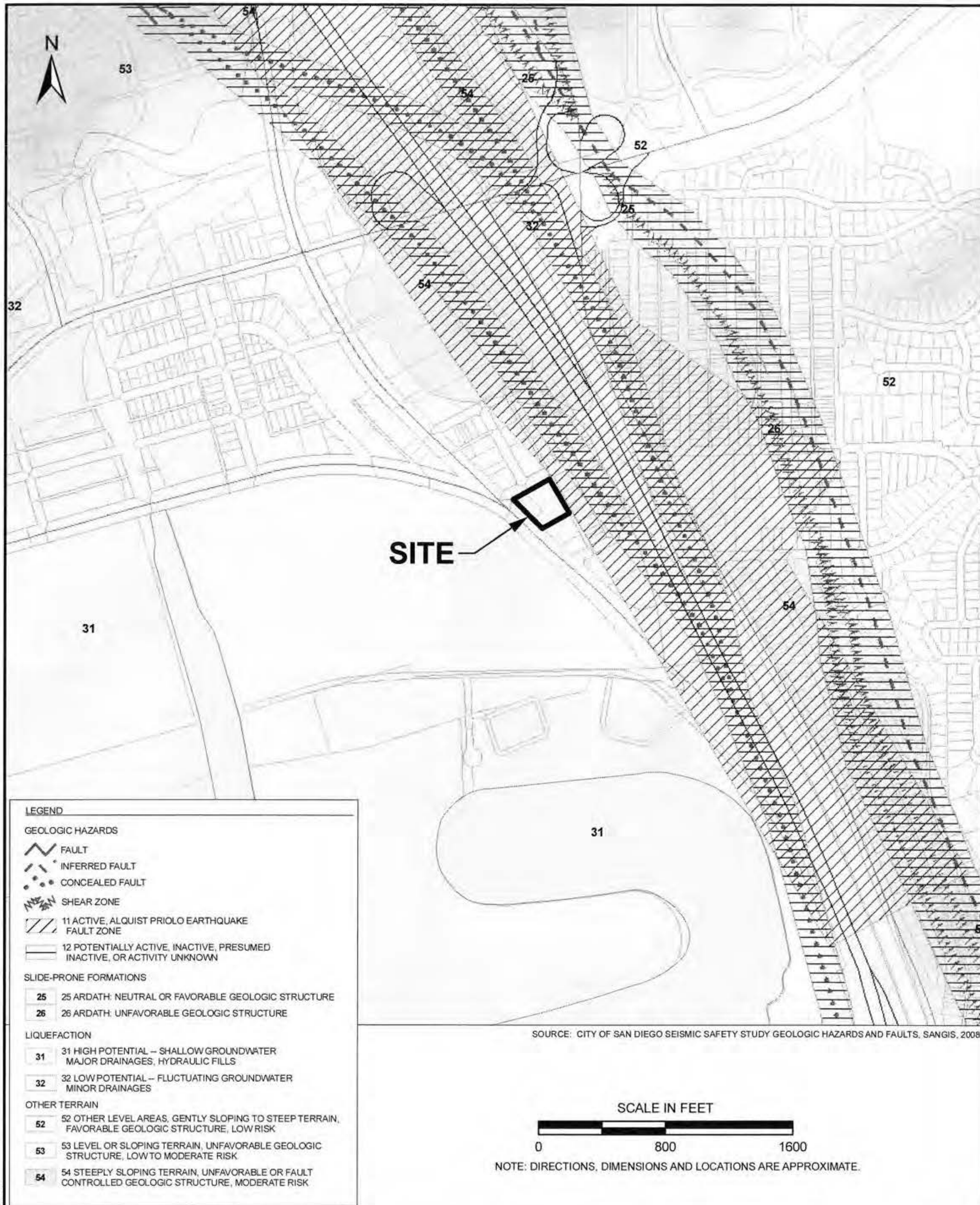
SOURCE: JENNINGS, C.W., AND BRYANT, W.A., 2010, FAULT ACTIVITY MAP OF CALIFORNIA, CALIFORNIA GEOLOGICAL SURVEY.

SCALE IN MILES

0 30 60

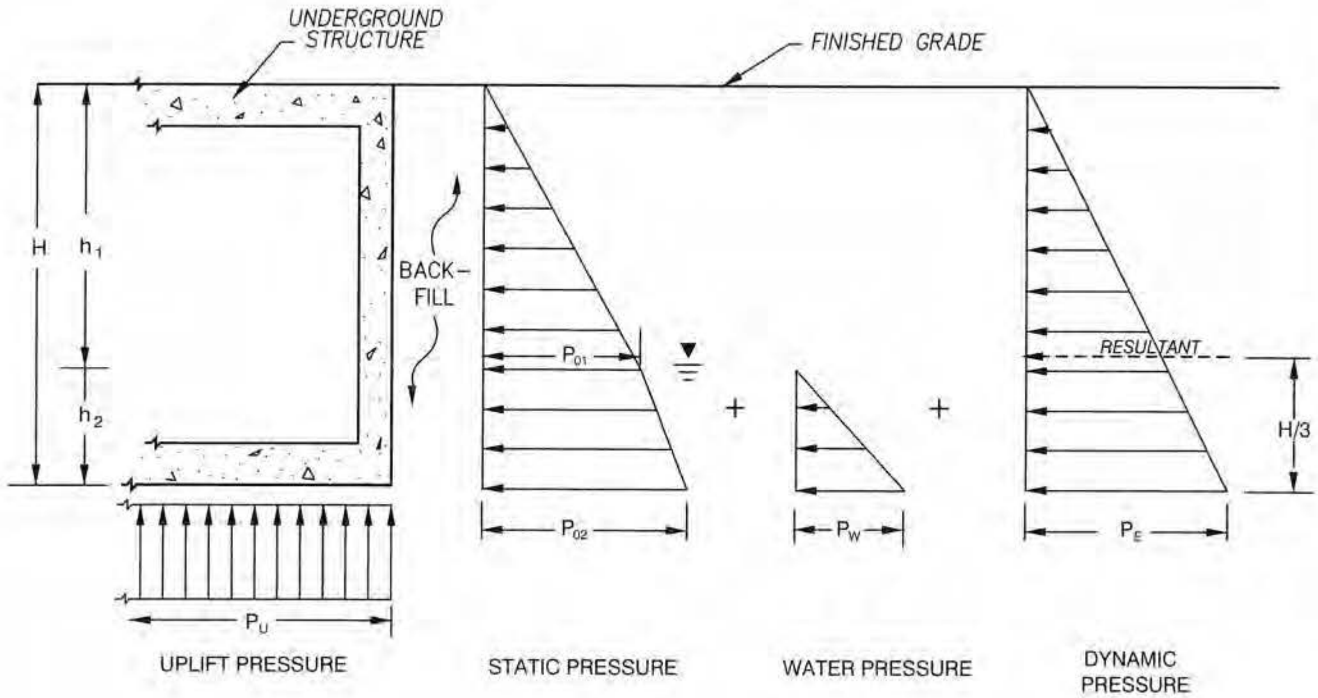
NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

| Ninyo & Moore | | FAULT LOCATIONS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 6 |
| 108107001 | 3/16 | | |




| *Ninyo & Moore* | | GEOLOGIC HAZARDS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 7 |
| 108107001 | 3/16 | | |

Z:\108107001_GH_93.mxd AOB



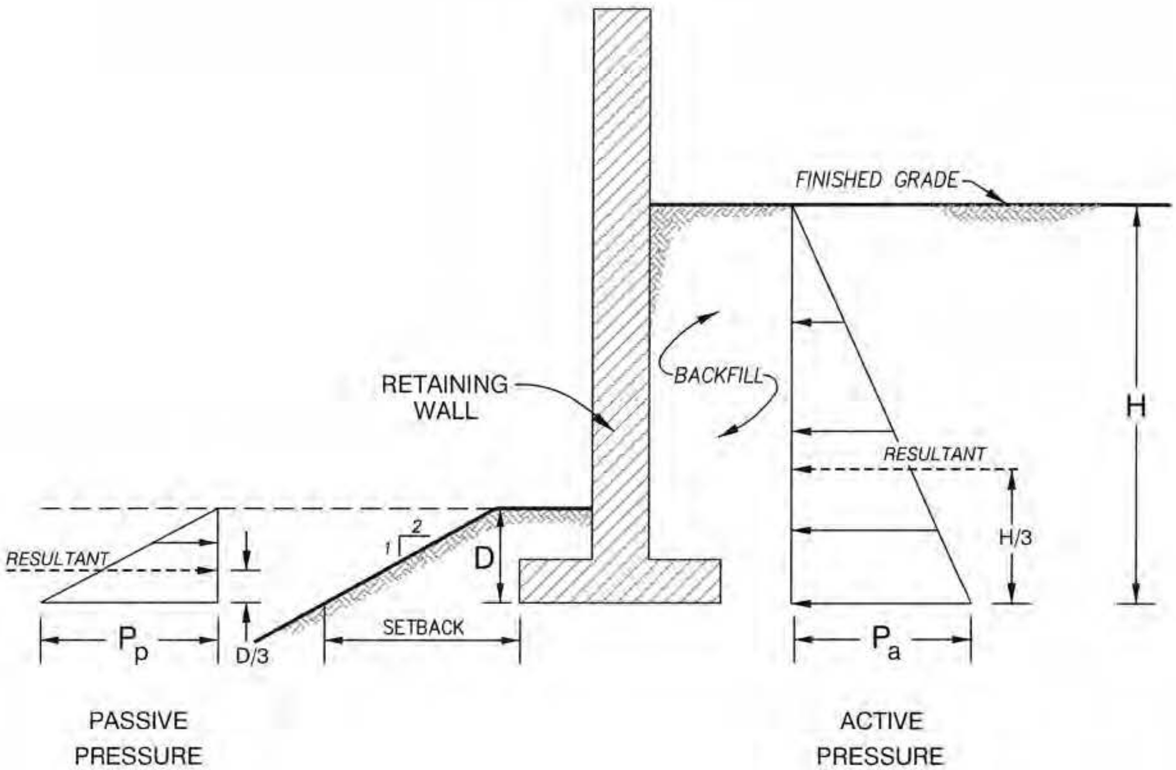

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, $P_{01}$ AND $P_{02}$
   $P_{01} = 60\ h_1$ psf
   $P_{02} = 60\ h_1 + 29\ h_2$ psf
2. WATER PRESSURE, $P_w$
   $P_w = 62.4\ h_2$ psf
3. DYNAMIC LATERAL EARTH PRESSURE, $P_E$, IS BASED ON A PEAK GROUND ACCELERATION, OF 0.57 g
   $P_E = 20\ H$ psf
4. $P_E$ IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
5. UPLIFT PRESSURE, $P_u$
   $P_u = 62.4\ h_2$ psf
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H, $h_1$ AND $h_2$ ARE IN FEET
8. ▼ GROUNDWATER TABLE

*NOT TO SCALE*

8 108107001 d-us.dwg

| Ninyo & Moore | | LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 8 |
| 108107001 | 3/16 | | |




NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET (H IS LESS THAN 6 FEET)
6. SETBACK SHOULD BE IN ACCORDANCE WITH THE CBC (2013)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

| Lateral Earth Pressure | Equivalent Fluid Pressure ($lb/ft^2/ft$)[1] |
|---|---|
| $P_a$ | Level Backfill with Granular Soils [2] |
| | 40 H |
| $P_p$ | Level Ground |
| | 300 D |

NOT TO SCALE

9 108107001 d-yrw.dwg

Ninyo & Moore

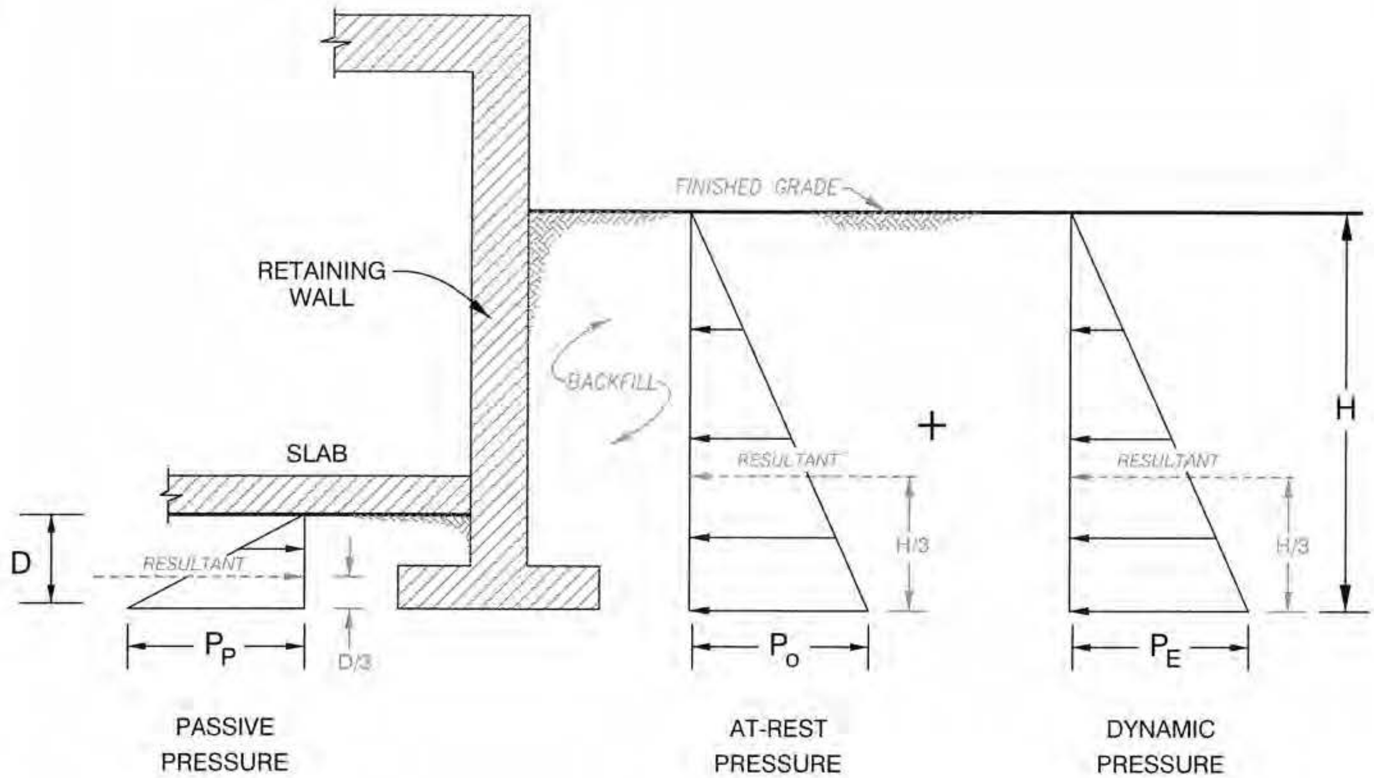
| PROJECT NO. | DATE |
|---|---|
| 108107001 | 3/16 |

**LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS**

4345 MISSION BAY DRIVE
SAN DIEGO, CALIFORNIA

FIGURE

9




NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A PEAK GROUND ACCELERATION OF 0.57g
5. $P_E$ IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET

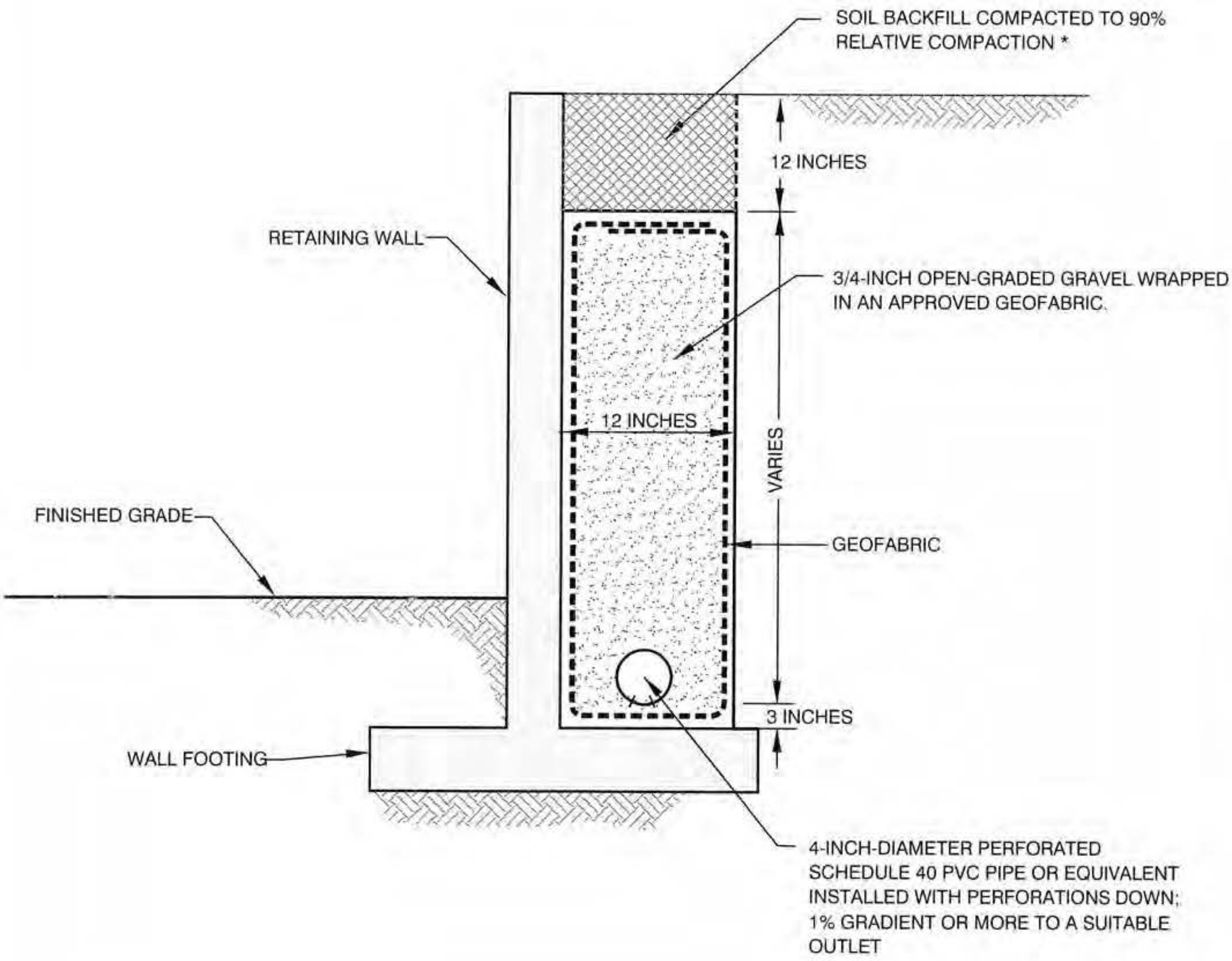
RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

| Lateral Earth Pressure | Equivalent Fluid Pressure ($lb/ft^2/ft$) [1] |
|---|---|
| $P_O$ | Level Backfill with Granular Soils [2] |
| | 60 H |
| $P_E$ | 20 H |
| $P_P$ | Level Ground |
| | 300 D |

*NOT TO SCALE*

10 108107001 d-rrw.dwg

| Ninyo & Moore | | LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 10 |
| 108107001 | 3/16 | | |




*BASED ON ASTM D1557

*NOT TO SCALE*

11 108107001 d-rw.dwg

NOTE: AS AN ALTERNATIVE, AN APPROVED GEOCOMPOSITE DRAIN SYSTEM MAY BE USED.

| Ninyo & Moore | | RETAINING WALL DRAINAGE DETAIL | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | 11 |
| 108107001 | 3/16 | | |

## APPENDIX A

## BORING AND CPT LOGS

**Field Procedure for the Collection of Disturbed Samples**

Disturbed soil samples were obtained in the field using the following methods.

**Bulk Samples**

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

**The SPT Sampler**

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

**Field Procedure for the Collection of Relatively Undisturbed Samples**

Relatively undisturbed soil samples were obtained in the field using a Modified Split-Barrel Drive sampler. The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer of the drill rig in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

# BORING LOG EXPLANATION SHEET

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | Bulk sample. |
| | | | | | | | | Modified split-barrel drive sampler. |
| | | | | | | | | 2-inch inner diameter split-barrel drive sampler. |
| | | | | | | | | No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler. |
| | | | | | | | | Sample retained by others. |
| 5 | | | | | | | | Standard Penetration Test (SPT). |
| | | | | | | | | No recovery with a SPT. |
| | | | XX/XX | | | | | Shelby tube sample. Distance pushed in inches/length of sample recovered in inches. |
| | | | | | | | | No recovery with Shelby tube sampler. |
| | | | | | | | | Continuous Push Sample. |
| 10 | | | | | | | | Seepage. |
| | | | | | | | | Groundwater encountered during drilling. |
| | | | | | | | | Groundwater measured after drilling. |
| | | | | | | | SM | <u>MAJOR MATERIAL TYPE (SOIL)</u>:<br>Solid line denotes unit change. |
| | | | | | | | CL | Dashed line denotes material change. |
| 15 | | | | | | | | Attitudes: Strike/Dip<br>b: Bedding<br>c: Contact<br>j: Joint<br>f: Fracture<br>F: Fault<br>cs: Clay Seam<br>s: Shear<br>bss: Basal Slide Surface<br>sf: Shear Fracture<br>sz: Shear Zone<br>sbs: Shear Bedding Surface |
| | | | | | | | | The total depth line is a solid line that is drawn at the bottom of the boring. |
| 20 | | | | | | | | |

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| BORING LOG | | |
|---|---|---|
| Explanation of Boring Log Symbols | | |
| PROJECT NO. | DATE | FIGURE |

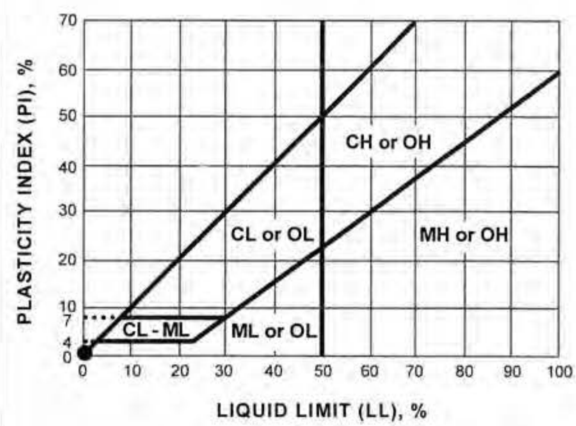
## SOIL CLASSIFICATION CHART PER ASTM D 2488

| PRIMARY DIVISIONS | | | SECONDARY DIVISIONS: GROUP SYMBOL | SECONDARY DIVISIONS: GROUP NAME |
|---|---|---|---|---|
| COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve | GRAVEL more than 50% of coarse fraction retained on No. 4 sieve | CLEAN GRAVEL less than 5% fines | GW | well-graded GRAVEL |
| | | | GP | poorly graded GRAVEL |
| | | GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines | GW-GM | well-graded GRAVEL with silt |
| | | | GP-GM | poorly graded GRAVEL with silt |
| | | | GW-GC | well-graded GRAVEL with clay |
| | | | GP-GC | poorly graded GRAVEL with clay |
| | | GRAVEL with FINES more than 12% fines | GM | silty GRAVEL |
| | | | GC | clayey GRAVEL |
| | | | GC-GM | silty, clayey GRAVEL |
| | SAND 50% or more of coarse fraction passes No. 4 sieve | CLEAN SAND less than 5% fines | SW | well-graded SAND |
| | | | SP | poorly graded SAND |
| | | SAND with DUAL CLASSIFICATIONS 5% to 12% fines | SW-SM | well-graded SAND with silt |
| | | | SP-SM | poorly graded SAND with silt |
| | | | SW-SC | well-graded SAND with clay |
| | | | SP-SC | poorly graded SAND with clay |
| | | SAND with FINES more than 12% fines | SM | silty SAND |
| | | | SC | clayey SAND |
| | | | SC-SM | silty, clayey SAND |
| FINE-GRAINED SOILS 50% or more passes No. 200 sieve | SILT and CLAY liquid limit less than 50% | INORGANIC | CL | lean CLAY |
| | | | ML | SILT |
| | | | CL-ML | silty CLAY |
| | | ORGANIC | OL (PI > 4) | organic CLAY |
| | | | OL (PI < 4) | organic SILT |
| | SILT and CLAY liquid limit 50% or more | INORGANIC | CH | fat CLAY |
| | | | MH | elastic SILT |
| | | ORGANIC | OH (plots on or above "A"-line) | organic CLAY |
| | | | OH (plots below "A"-line) | organic SILT |
| | Highly Organic Soils | | PT | Peat |

## GRAIN SIZE

| DESCRIPTION | | SIEVE SIZE | GRAIN SIZE | APPROXIMATE SIZE |
|---|---|---|---|---|
| Boulders | | > 12" | > 12" | Larger than basketball-sized |
| Cobbles | | 3 - 12" | 3 - 12" | Fist-sized to basketball-sized |
| Gravel | Coarse | 3/4 - 3" | 3/4 - 3" | Thumb-sized to fist-sized |
| | Fine | #4 - 3/4" | 0.19 - 0.75" | Pea-sized to thumb-sized |
| Sand | Coarse | #10 - #4 | 0.079 - 0.19" | Rock-salt-sized to pea-sized |
| | Medium | #40 - #10 | 0.017 - 0.079" | Sugar-sized to rock-salt-sized |
| | Fine | #200 - #40 | 0.0029 - 0.017" | Flour-sized to sugar-sized |
| Fines | | Passing #200 | < 0.0029" | Flour-sized and smaller |

## PLASTICITY CHART




## APPARENT DENSITY - COARSE-GRAINED SOIL

| APPARENT DENSITY | SPOOLING CABLE OR CATHEAD: SPT (blows/foot) | SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot) | AUTOMATIC TRIP HAMMER: SPT (blows/foot) | AUTOMATIC TRIP HAMMER: MODIFIED SPLIT BARREL (blows/foot) |
|---|---|---|---|---|
| Very Loose | ≤ 4 | ≤ 8 | ≤ 3 | ≤ 5 |
| Loose | 5 - 10 | 9 - 21 | 4 - 7 | 6 - 14 |
| Medium Dense | 11 - 30 | 22 - 63 | 8 - 20 | 15 - 42 |
| Dense | 31 - 50 | 64 - 105 | 21 - 33 | 43 - 70 |
| Very Dense | > 50 | > 105 | > 33 | > 70 |

## CONSISTENCY - FINE-GRAINED SOIL

| CONSISTENCY | SPOOLING CABLE OR CATHEAD: SPT (blows/foot) | SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot) | AUTOMATIC TRIP HAMMER: SPT (blows/foot) | AUTOMATIC TRIP HAMMER: MODIFIED SPLIT BARREL (blows/foot) |
|---|---|---|---|---|
| Very Soft | < 2 | < 3 | < 1 | < 2 |
| Soft | 2 - 4 | 3 - 5 | 1 - 3 | 2 - 3 |
| Firm | 5 - 8 | 6 - 10 | 4 - 5 | 4 - 6 |
| Stiff | 9 - 15 | 11 - 20 | 6 - 10 | 7 - 13 |
| Very Stiff | 16 - 30 | 21 - 39 | 11 - 20 | 14 - 26 |
| Hard | > 30 | > 39 | > 20 | > 26 |

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# USCS METHOD OF SOIL CLASSIFICATION

**Explanation of USCS Method of Soil Classification**

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| | | |

DATE DRILLED 2/02/16 BORING NO. B-1

GROUND ELEVATION 20.5' ± (MSL) SHEET 1 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Ingersol A-300) (Scott's)

DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY JMM

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | ASPHALT CONCRETE: Approximately 6-1/2 inches thick (4 inches overlay over 2-1/2 inches old asphalt concrete). |
| | | | | | | | SC | FILL: Brown, moist, medium dense, clayey fine SAND. |
| | | | 70 | 19.9 | 108.3 | | SM | OLD PARALIC DEPOSITS: Light brown, moist, dense, silty fine SAND. |
| | | | | | | | SC | Dark brown, moist, dense, clayey fine SAND; scattered sand-filled high-angle fractures. |
| 10 | | | 36 | | | | | Mottled brown and reddish brown; scattered manganese nodules. |
| | | | | | | | SM | Dark yellowish brown, moist to wet, medium dense, silty fine SAND. |
| | | | 42 | 14.9 | 116.8 | | | |
| | | | | | | | CL | Mottled brown and reddish brown mottled, moist to wet, very stiff, sandy CLAY; scattered manganese nodules. |
| | | | | ▽ | | | | |
| 20 | | | 25 | | | | | Wet. |
| | | | | | | | CH | Dark gray and brown, wet, very stiff, silty CLAY; high plasticity. |
| | | | 30 | | | | | Mottled reddish brown and brown; high plasticity; trace fine sand; scattered manganese nodules. |
| 30 | | | 25 | | | | | |
| | | | | | | | SP | Light brown, wet, very dense, poorly graded fine SAND; cohesionless. |
| | | | 82/9" | | | | | Scattered angular gravel up to approximately 1 inch in diameter. |
| | | | | | | | | @ 37': Grout added to boring. |
| 40 | | | | | | | | |

Ninyo & Moore

**BORING LOG**

4345 MISSION BAY DRIVE
SAN DIEGO, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 108107001 | 2/16 | A-1 |

DATE DRILLED 2/02/16 BORING NO. B-1

GROUND ELEVATION 20.5' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Ingersol A-300) (Scott's)

DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY JMM

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 40 | | ◤ | 26 | | | | CL | OLD PARALIC DEPOSITS: (Continued) Light brown, wet, very stiff, sandy CLAY; trace rounded gravel up to approximately 1-1/2 inches in diameter. |
| | | ◤ | 50/6" | | | | | |
| 50 | | ◤ | 81 | | | | SP-SM | Light brown, wet, very dense, poorly graded fine SAND with silt; micaceous; cohesionless. |
| | | | | | | | SM | Brown, wet, very dense, silty fine SAND; micaceous. |
| | | ◤ | 53 | | | | | |
| 60 | | ◤ | 32 | | | | ML | Brown, wet, hard, clayey SILT; micaceous; scattered yellowish brown mottling. |
| | | | | | | | SM | Brown, wet, very dense, silty fine SAND; micaceous; scattered shell fragments. |
| | | ◤ | 50/6" | | | | | |
| | | | | | | | | Reddish brown; trace rounded gravel up to approximately 1 inch in diameter. |
| 70 | | ◤ | 79/11" | | | | CL | Olive, wet, hard, silty CLAY; scattered caliche. |
| | | | | | | | ML | Light olive, wet, very dense, fine sandy SILT; scattered fine laminations. |
| | | | | | | | | Brown. |
| | | ◤ | 42 | | | | CL | Brown, wet, hard, silty CLAY; high plasticity; scattered manganese nodules. |
| | | | | | | | | Total Depth = 76.5 feet. Groundwater encountered at approximately 19.5 feet during drilling. Backfilled with approximately 25 cubic feet of bentonite grout and patched with concrete shortly after drilling on 2/02/16. |
| 80 | | | | | | | | |

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**BORING LOG**

4345 MISSION BAY DRIVE
SAN DIEGO, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 108107001 | 2/16 | A-2 |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. |
|---|---|---|---|---|---|---|---|

DATE DRILLED 2/02/16 BORING NO. B-1

GROUND ELEVATION 20.5' ± (MSL) SHEET 3 OF 3

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (Ingersol A-300) (Scott's)

DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY JMM

**DESCRIPTION/INTERPRETATION**

80

Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretation of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

90

100

110

120

Ninyo & Moore

**BORING LOG**

4345 MISSION BAY DRIVE
SAN DIEGO, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 108107001 | 2/16 | A-3 |

| DEPTH (feet) | SAMPLES: Bulk | SAMPLES: Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | |
| | | | | | | | SM |
| | | | | | | | CL |
| | | | | | | | SC |
| 10 | | | | | | | |
| 20 | | | | | | | |
| 30 | | | | | | | |
| 40 | | | | | | | |

DATE DRILLED 2/02/16 BORING NO. B-2

GROUND ELEVATION 19' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING Manual

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY JMM

**DESCRIPTION/INTERPRETATION**

ASPHALT CONCRETE:
Approximately 5-1/2 inches thick (4 inches overlay over 1-1/2-inch old asphalt concrete).
FILL:
Brown, moist, medium dense, silty fine to medium SAND.
Brown, moist, stiff, sandy CLAY.
Brown, moist, medium dense, clayey fine to medium SAND.
Total Depth = 3.9 feet.
Groundwater was not encountered during drilling.
Backfilled and patched with concrete shortly after drilling on 2/02/16.

Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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**BORING LOG**

4345 MISSION BAY DRIVE
SAN DIEGO, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 108107001 | 2/16 | A-4 |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SM |
| | | | | | | | CL |
| 10 | | | | | | | |
| 20 | | | | | | | |
| 30 | | | | | | | |
| 40 | | | | | | | |

DATE DRILLED 2/02/16 BORING NO. B-3

GROUND ELEVATION 19' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING Manual

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY JMM

**DESCRIPTION/INTERPRETATION**

FILL:
Dark brown, moist, medium dense, silty fine to medium SAND; scattered grass and roots up to approximately 1/4 inch in diameter.
Brown.
Few gravel up to approximately 1/4-inch in diameter.
Brown, moist, stiff, sandy CLAY; roots up to approximately 1/4-inch in diameter.
Total Depth = 3.5 feet.
Groundwater was not encountered during drilling.
Backfilled and patched with concrete shortly after drilling on 2/02/16.

Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

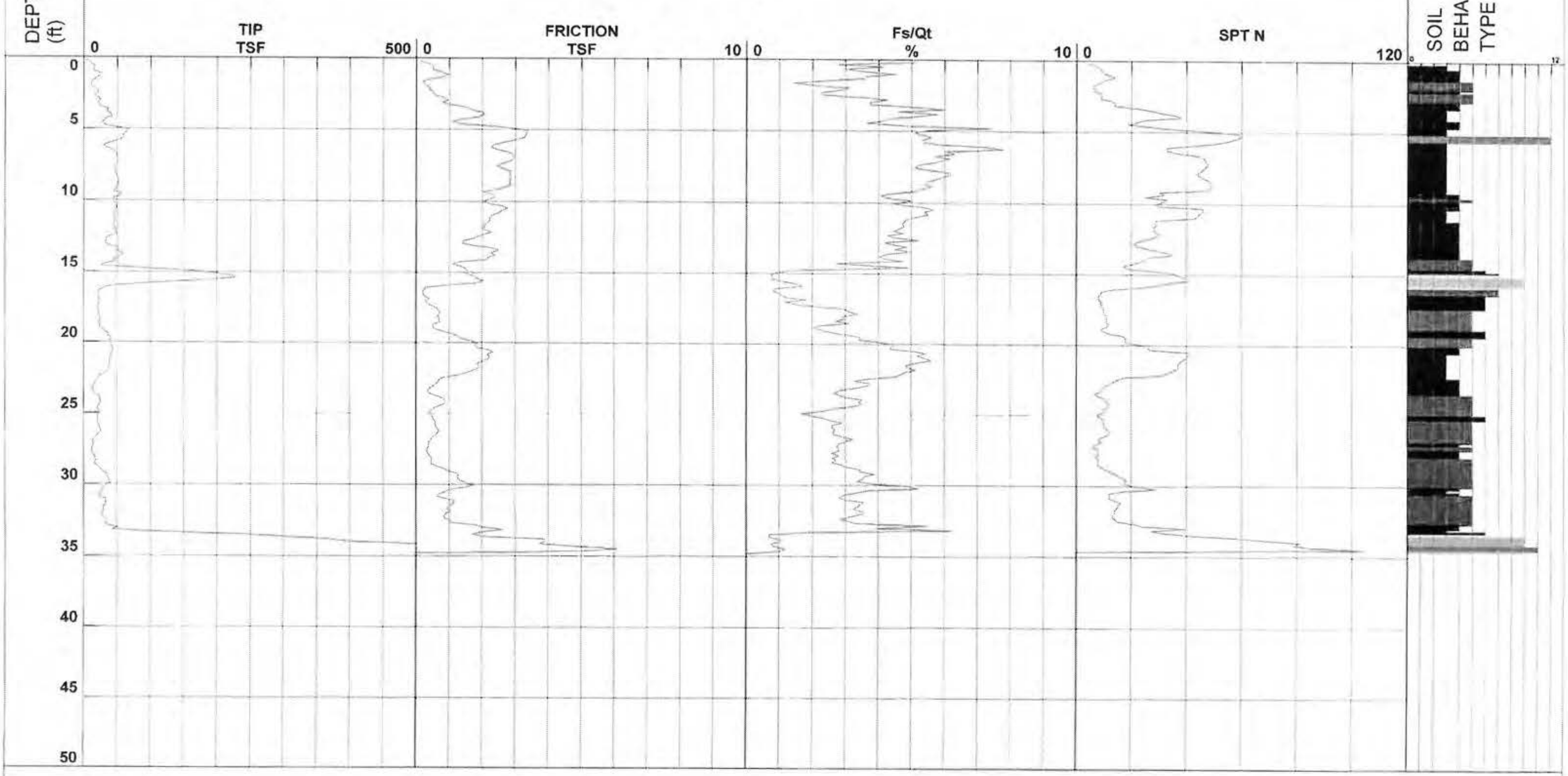
The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

# Ninyo & Moore

| | | | | | |
|---|---|---|---|---|---|
| Project | 4345 Mission Bay Drive | Operator | DG-RC | Filename | SDF(093).cpt |
| Job Number | 108107001 | Cone Number | DDG1333 | GPS | |
| Hole Number | CPT-01 | Date and Time | 2/8/2016 1:03:00 PM | Maximum Depth | 34.94 ft |
| EST GW Depth During Test | >34.94 ft | | | | |

Net Area Ratio .8

## CPT DATA




1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

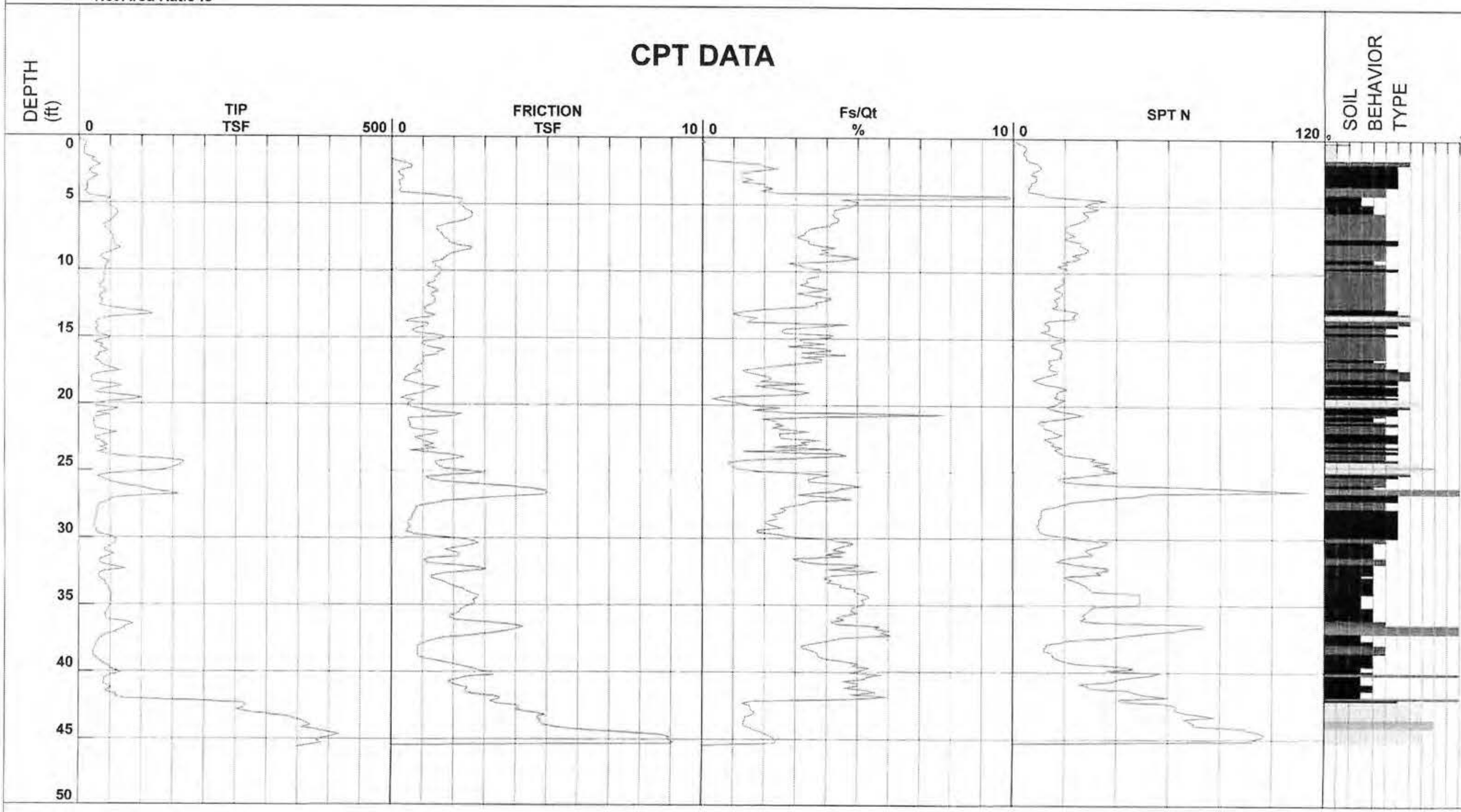
S*Soil behavior type and SPT based on data from UBC-1983

# Ninyo & Moore

| | | | | | |
|---|---|---|---|---|---|
| Project | 4345 Mission Bay Drive | Operator | DG-RC | Filename | SDF(094).cpt |
| Job Number | 108107001 | Cone Number | DDG1333 | GPS | |
| Hole Number | CPT-02 | Date and Time | 2/8/2016 2:14:20 PM | Maximum Depth | 45.60 ft |
| EST GW Depth During Test | >45.60 ft | | | | |

Net Area Ratio .8

## CPT DATA




■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

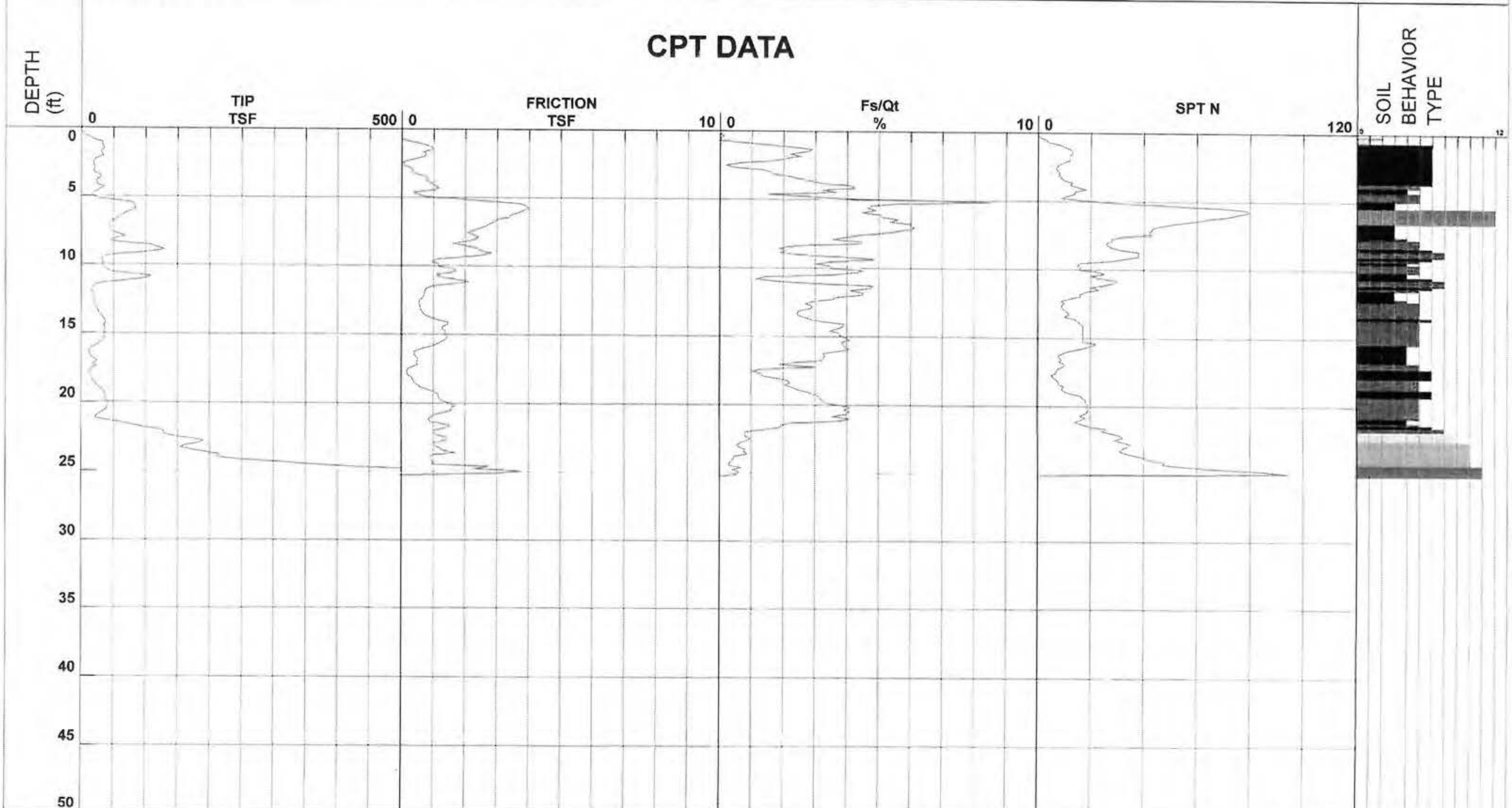
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

## Ninyo & Moore

| | | | | | |
|---|---|---|---|---|---|
| Project | 4345 Mission Bay Drive | Operator | DG-RC | Filename | SDF(095).cpt |
| Job Number | 108107001 | Cone Number | DDG1333 | GPS | |
| Hole Number | CPT-03 | Date and Time | 2/8/2016 3:14:01 PM | Maximum Depth | 25.43 ft |
| EST GW Depth During Test | >25.43 ft | | | | |

Net Area Ratio .8




| | | | |
|---|---|---|---|
| 1 - sensitive fine grained | 4 - silty clay to clay | 7 - silty sand to sandy silt | 10 - gravelly sand to sand |
| 2 - organic material | 5 - clayey silt to silty clay | 8 - sand to silty sand | 11 - very stiff fine grained (*) |
| 3 - clay | 6 - sandy silt to clayey silt | 9 - sand | 12 - sand to clayey sand (*) |

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

## APPENDIX B

## GEOTECHNICAL LABORATORY TESTING

**Classification**

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

**In-Place Moisture and Density Tests**

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

**Gradation Analysis**

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-1 through B-3. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

**Atterberg Limits**

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The test results and classifications are shown on Figure B-4.

**Direct Shear Test**

A direct shear test was performed on relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-5.

**Expansion Index Test**

The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of the test are presented on Figure B-6.

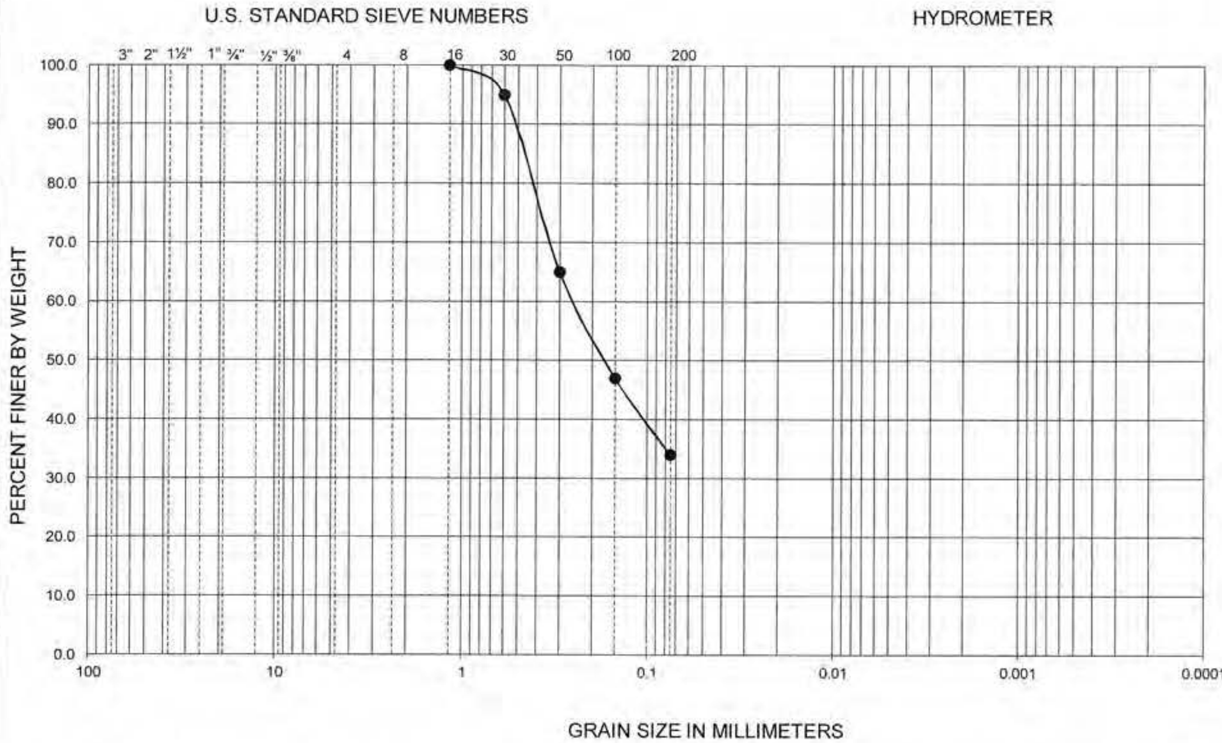
**Soil Corrosivity Tests**

Soil pH and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-7.

**R-Value**

The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. The sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figures B-8 and B-9.

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY |

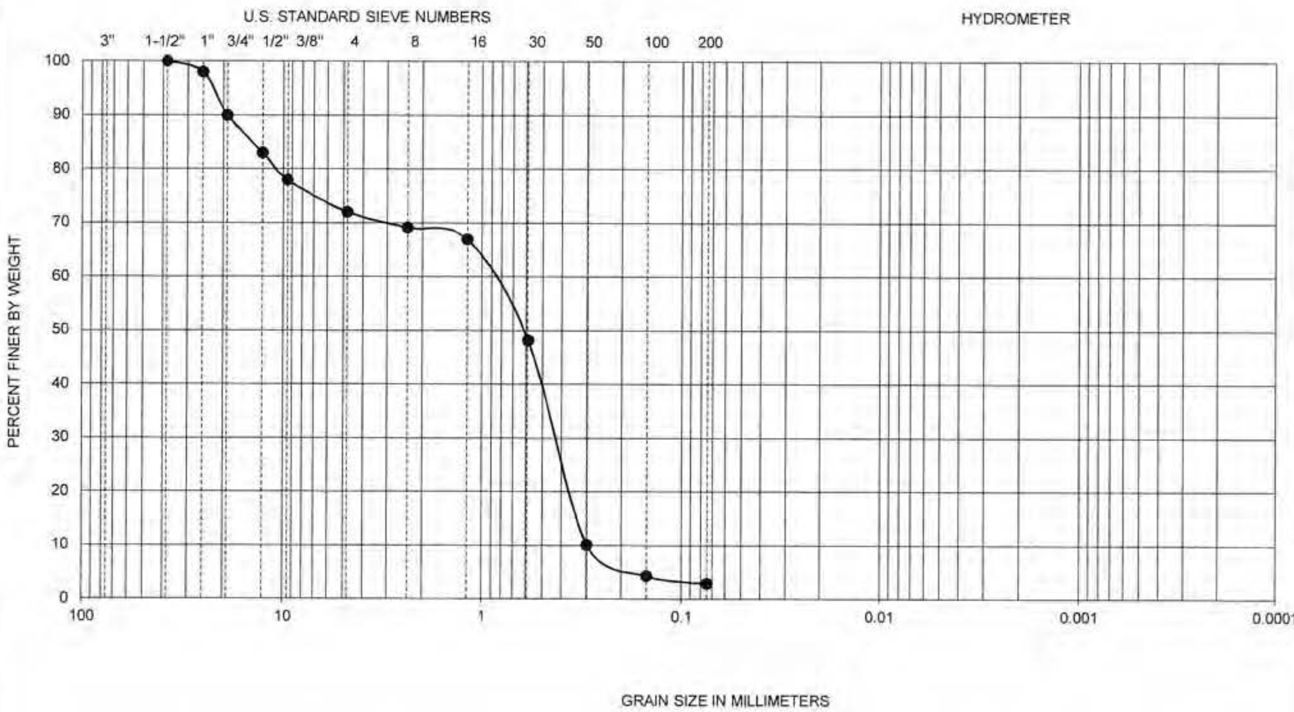



| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 15.0-16.5 | -- | -- | -- | -- | -- | -- | -- | -- | 34 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

| Ninyo & Moore | | GRADATION TEST RESULTS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | B-1 |
| 108107001 | 3/16 | | |

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |

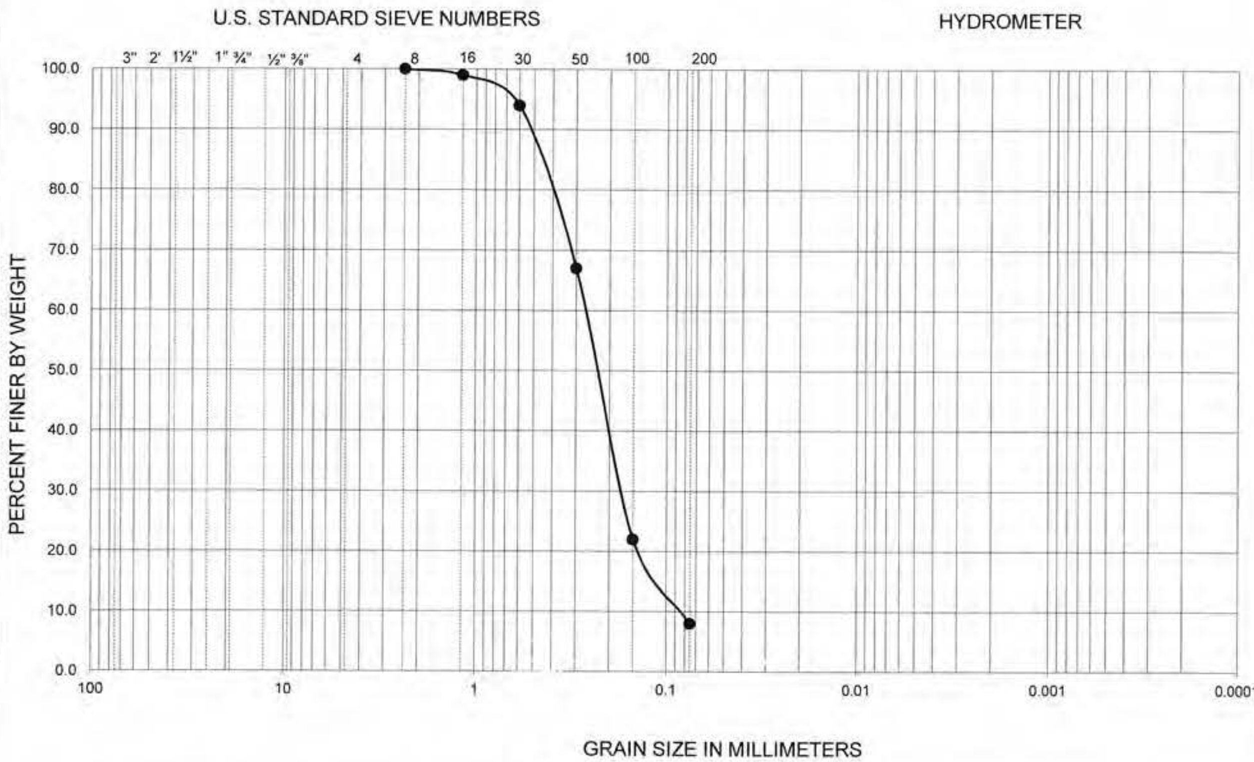



| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 35.0-36.3 | -- | -- | -- | 0.30 | 0.42 | 0.85 | 2.8 | 0.7 | 3 | SP |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

| *Ninyo & Moore* | | GRADATION TEST RESULTS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | B-2 |
| 108107001 | 3/16 | | |

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY |



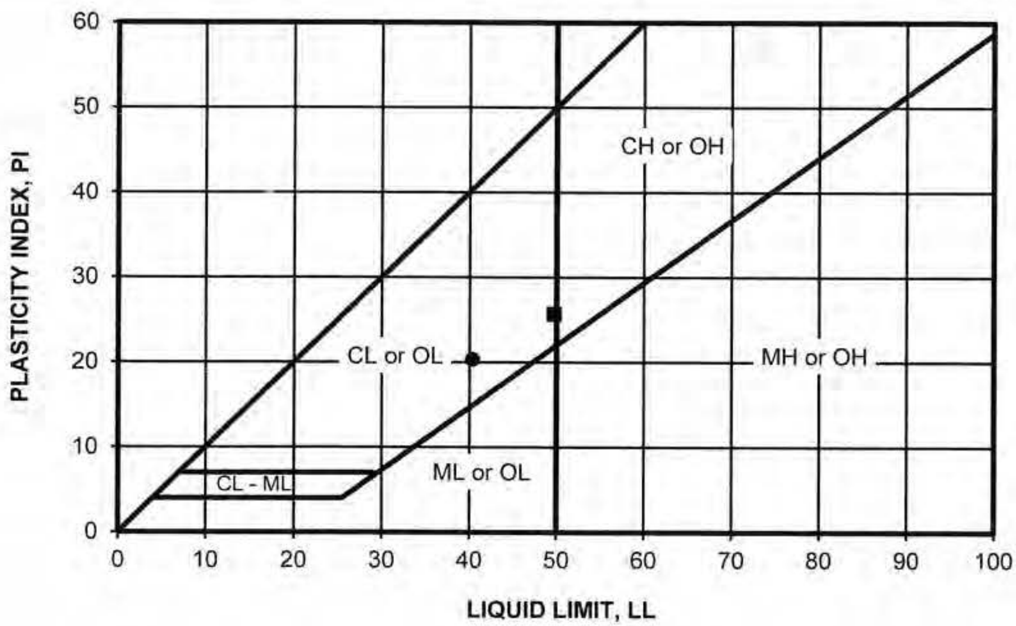

| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (%) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 50.0-51.5 | -- | -- | -- | 0.09 | 0.18 | 0.27 | 3.0 | 1.3 | 8 | SP-SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

| Ninyo & Moore | | GRADATION TEST RESULTS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | B-3 |
| 108107001 | 3/16 | | |

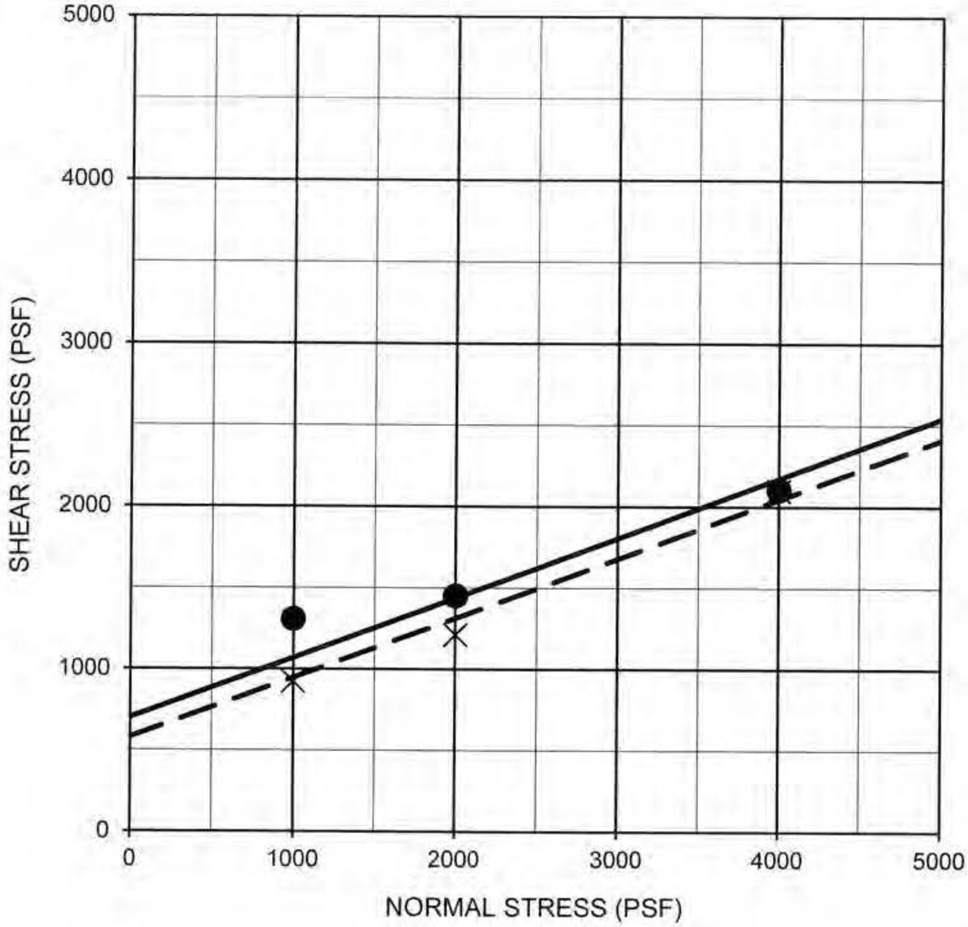
108107001_SIEVE B-1 @ 50.0-51.5.xls

| SYMBOL | LOCATION | DEPTH (FT) | LIQUID LIMIT, LL | PLASTIC LIMIT, PL | PLASTICITY INDEX, PI | USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve) | USCS (Entire Sample) |
|---|---|---|---|---|---|---|---|
| ● | B-1 | 20.0-21.5 | 40 | 20 | 20 | CL | CL |
| ■ | B-1 | 30.0-31.5 | 50 | 24 | 26 | CH | CH |




PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

| Ninyo & Moore | | ATTERBERG LIMITS TEST RESULTS | FIGURE |
|---|---|---|---|
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | B-4 |
| 108107001 | 3/16 | | |




| Description | Symbol | Sample Location | Depth (ft) | Shear Strength | Cohesion, c (psf) | Friction Angle, $\phi$ (degrees) | Soil Type |
|---|---|---|---|---|---|---|---|
| Clayey and Silty SAND | —●— | B-1 | 5.0-6.5 | Peak | 700 | 20 | SM+SC |
| Clayey and Silty SAND | - - X - - | B-1 | 5.0-6.5 | Ultimate | 580 | 20 | SM+SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore

## DIRECT SHEAR TEST RESULTS

| PROJECT NO. | DATE | | FIGURE |
|---|---|---|---|
| 108107001 | 3/16 | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | B-5 |

| SAMPLE LOCATION | SAMPLE DEPTH (FT) | INITIAL MOISTURE (%) | COMPACTED DRY DENSITY (PCF) | FINAL MOISTURE (%) | VOLUMETRIC SWELL (IN) | EXPANSION INDEX | POTENTIAL EXPANSION |
|---|---|---|---|---|---|---|---|
| B-1 | 0.5-5.0 | 11.5 | 105.2 | 22.5 | 0.038 | 39 | Low |

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☒ ASTM D 4829

Ninyo & Moore

## EXPANSION INDEX TEST RESULTS

FIGURE

**B-6**

| PROJECT NO. | DATE |
|---|---|
| 108107001 | 3/16 |

4345 MISSION BAY DRIVE
SAN DIEGO, CALIFORNIA

| SAMPLE LOCATION | SAMPLE DEPTH (FT) | pH [1] | RESISTIVITY [1] (Ohm-cm) | SULFATE CONTENT [2] | | CHLORIDE CONTENT [3] (ppm) |
|---|---|---|---|---|---|---|
| | | | | (ppm) | (%) | |
| B-2 | 0.5-3.9 | 7.5 | 1,400 | 60 | 0.006 | 190 |

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

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## CORROSIVITY TEST RESULTS

| PROJECT NO. | DATE | | FIGURE |
|---|---|---|---|
| 108107001 | 3/16 | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | B-7 |

| SAMPLE LOCATION | SAMPLE DEPTH (FT) | SOIL TYPE | R-VALUE |
| --- | --- | --- | --- |
| B-3 | 1.5-3.5 | Sandy CLAY (CL) | LESS THAN 5 |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

| Ninyo & Moore | | R-VALUE TEST RESULTS | FIGURE |
| --- | --- | --- | --- |
| PROJECT NO. | DATE | 4345 MISSION BAY DRIVE<br>SAN DIEGO, CALIFORNIA | B-8 |
| 108107001 | 3/16 | | |

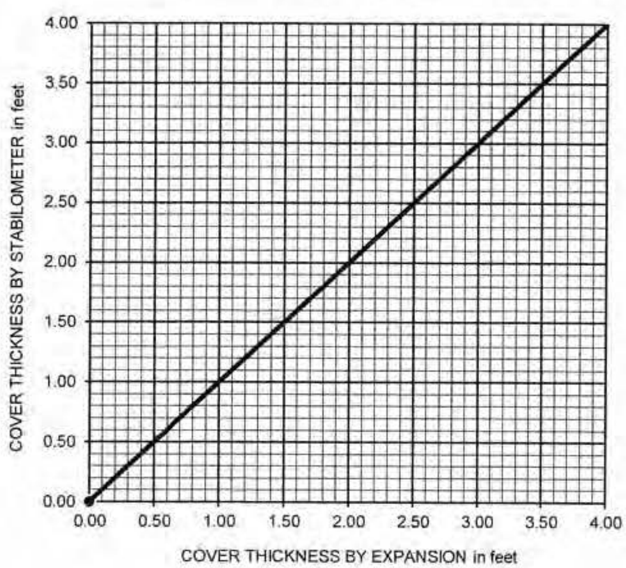
Ninyo & Moore

# R-VALUE TEST RESULTS

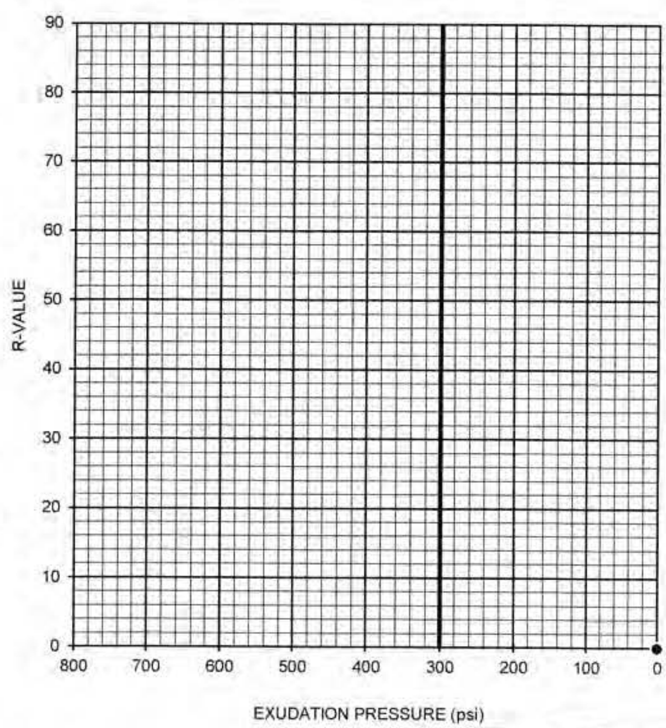
| | | | |
|---|---|---|---|
| **PROJECT NAME:** | 4345 MISSION BAY DRIVE, SAN DIEGO, CA | **PROJECT NUMBER** | 108107001 |
| **SAMPLE DESCRIPTION:** | Sandy CLAY (CL) | **DATE SAMPLED:** | 2/2/2016 |
| **SAMPLE LOCATION:** | B-3 @ 1.5-3.5 feet | **TECHNICIAN:** | APT |

| **TEST SPECIMEN** | a | b | c |
|---|---|---|---|
| MOISTURE AT COMPACTION % | 0.0 | 0.0 | 0.0 |
| HEIGHT OF SAMPLE, Inches | 0.00 | 0.00 | 0.00 |
| DRY DENSITY, pcf | #DIV/0! | #DIV/0! | #DIV/0! |
| COMPACTOR AIR PRESSURE, psi | 0 | 0 | 0 |
| EXUDATION PRESSURE, psi | 0 | 0 | 0 |
| EXPANSION, Inches x 10exp-4 | 0 | 0 | 0 |
| STABILITY Ph 2,000 lbs (160 psi) | 0 | 0 | 0 |
| TURNS DISPLACEMENT | 0.00 | 0.00 | 0.00 |
| R-VALUE UNCORRECTED | #DIV/0! | #DIV/0! | #DIV/0! |
| R-VALUE CORRECTED | 0 | 0 | 0 |
| R-VALUE BY EXUDATION | | **LESS THAN 5** | |
| **DESIGN CALCULATION DATA** | a | b | c |
| GRAVEL EQUIVALENT NEEDED ft. | 1.60 | 1.60 | 1.60 |
| TRAFFIC INDEX | | 5.0 | |
| STABILOMETER THICKNESS, ft. | #VALUE! | #VALUE! | #VALUE! |
| EXPANSION PRESSURE THICKNESS, ft. | 0.00 | 0.00 | 0.00 |

EXPANSION PRESSURE CHART




EXUDATION PRESSURE CHART




| | |
|---|---|
| R-VALUE BY EXPANSION: | N/A |
| R-VALUE BY EXUDATION: | LESS THAN 5 |
| EQUILIBRIUM R-VALUE: | Less than 5 |

**FIGURE B-9**

# APPENDIX C

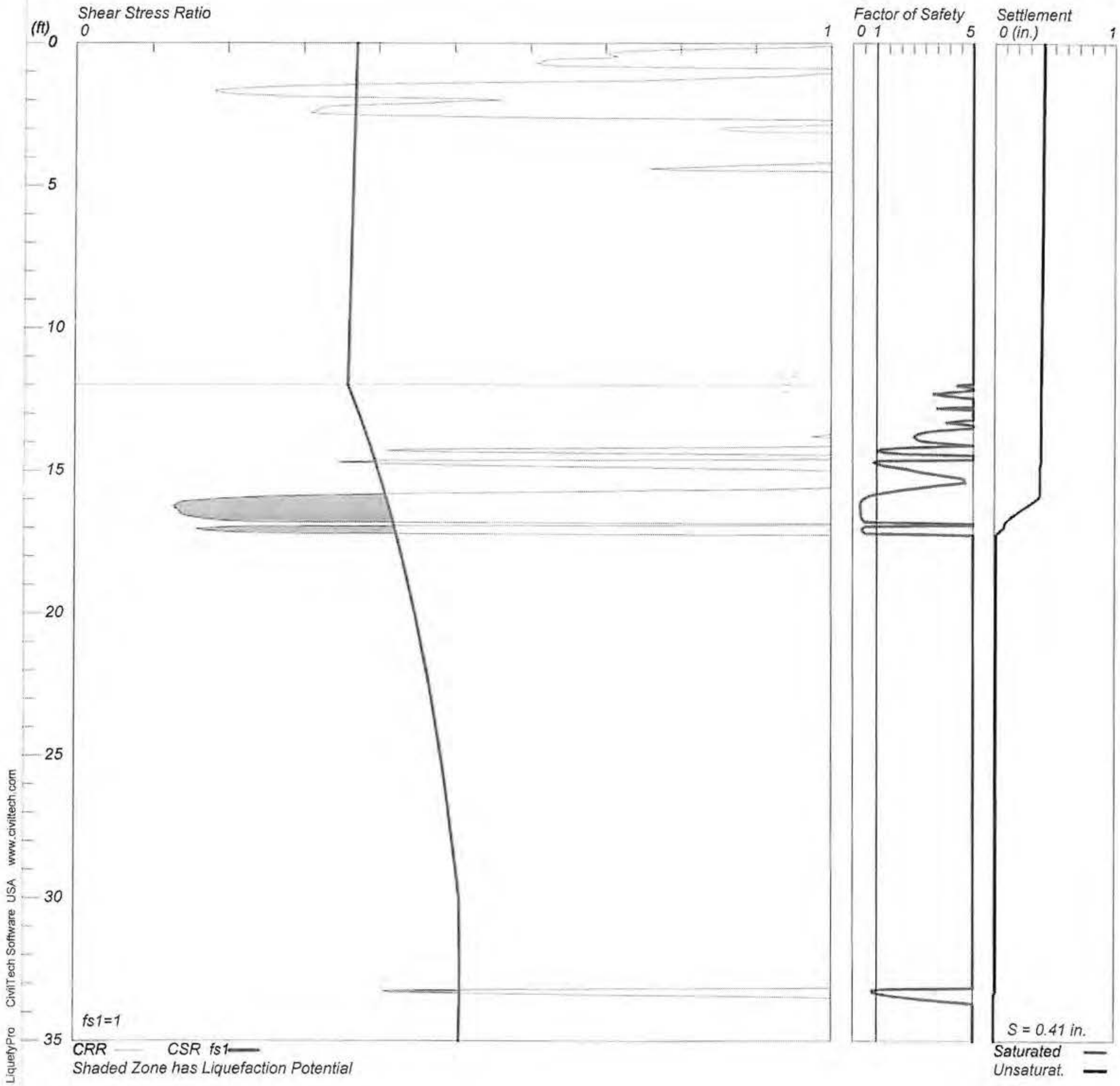
# LIQUEFACTION ANALYSIS

# LIQUEFACTION ANALYSIS

## 4345 Mission Bay Drive

*Hole No.=CPT-1 Water Depth=12 ft Surface Elev.=20.5*

*Magnitude=6.6*
*Acceleration=0.57g*

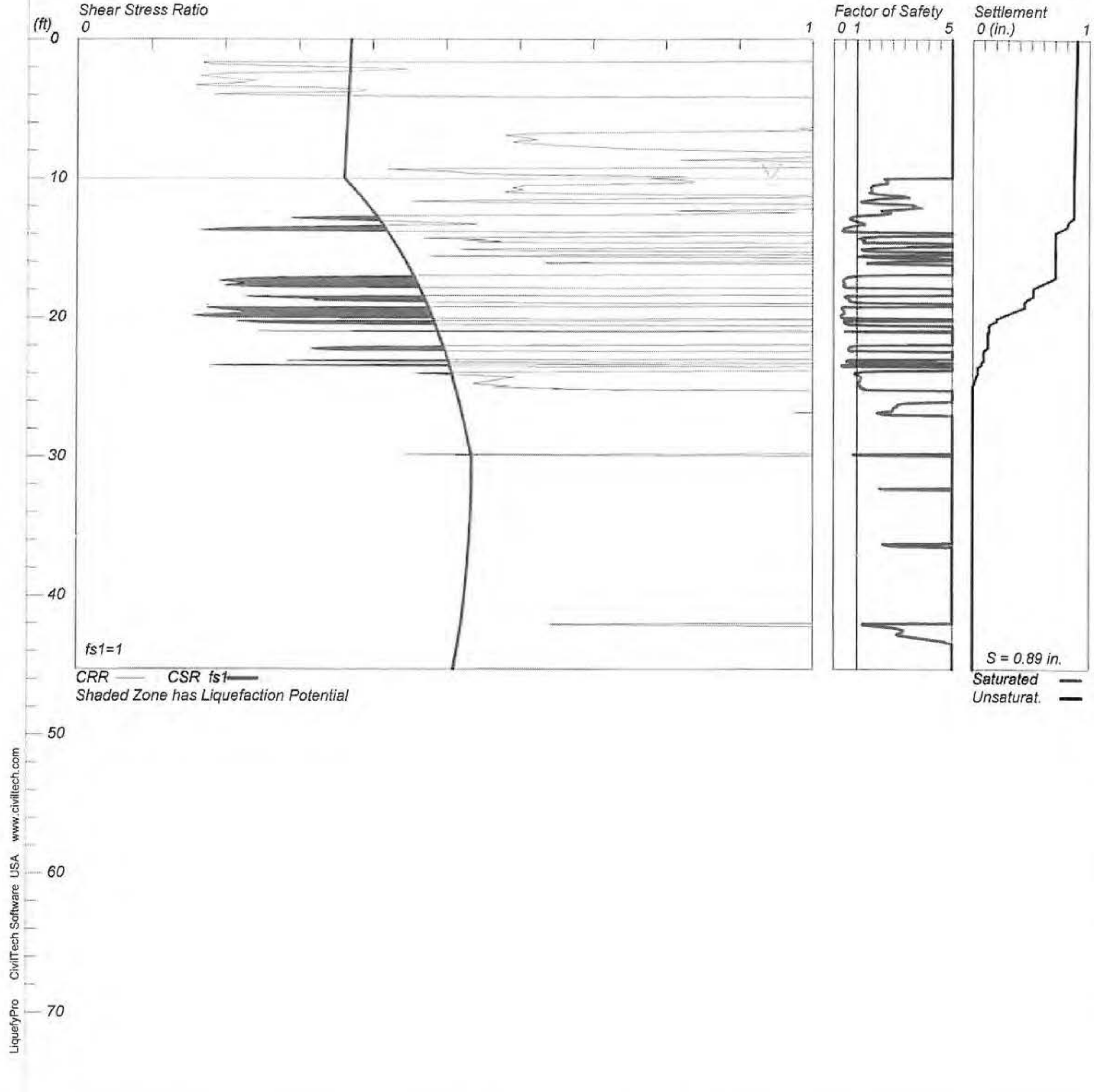


# LIQUEFACTION ANALYSIS

## 4345 Mission Bay Drive

*Hole No.=CPT-2 Water Depth=10 ft Surface Elev.=18*

*Magnitude=6.6*
*Acceleration=0.57g*



# LIQUEFACTION ANALYSIS

## 4345 Mission Bay Drive

*Hole No.=CPT-3   Water Depth=10 ft   Surface Elev.=18*

***Magnitude=6.6***
***Acceleration=0.57g***