4.9 SOILS AND GEOLOGY

The information and analysis presented in this section of the Draft EIR is based on the "Conceptual Grading Plan Review Report – Condominium Project, TTM 16882," prepared by Neblett & Associates, Inc., and the "Preliminary Geotechnical Engineering Exploration and Analysis for the Proposed Aerie Dock Replacement," prepared by Leighton and Associates, Inc. A "third party" review of both technical studies was conducted by Goffman, McCormick & Urban, LLC (GMU). The findings and recommendations of these most recent studies as well as prior studies referenced in those reports are presented below.

4.9.1 Existing Conditions

Agricultural Soils

The subject property is located in a residential area of Corona del Mar. Based on the National Cooperative Soil Survey for Orange County, the soils on the site encompasses an area classified as "beaches" and Myford sandy loam (2 to 9 percent slopes). The site and adjacent areas are designated as "Urban and Built-up Land" and "Other Land" on the Orange County Important Farmland Map. No Prime Farmland, Farmland of State or Local Importance, or Unique Farmland occurs within or in the vicinity of the site. Further, neither the site nor the adjacent areas are designated as prime, unique or important farmlands by the State Resources Agency or by the Newport Beach General Plan.

Geologic Setting/Conditions

The geologic units underlying the subject property and environs include artificial fill (afu), marine and non-marine terrace deposits (Qt), and bedrock units assigned to the upper-middle Miocene Monterey Formation (Tm). These units are described below.

Artificial Fill (afu)

Local fill is primarily derived from the underlying terrace deposits and contain broken shell fragments of marine fossils. The fill soils encountered in previous borings and tests are generally comprised of reddish brown silty sands and extend to maximum depths on the order to 13 feet.

Terrace Deposits (Qt)

Isolated, older marine terrace deposits with frequent mollusk shells and shell fragments indicative of a near shore depositional environmental cap the wave-cut terrace. These older marine terrace deposits are approximately 80,000 to 120,000 years old and are slightly moist to moist, medium dense to dense and have occasional secondary carbonate mineralization in fractures. Non-marine terrace deposits that capped the marine terrace represent continental deposits that have accumulated since uplift of the terrace bench and contain sediments ranging in age from upper Pleistocene to Recent age. The depth of terrace deposits encountered in the prior borings ranges from approximately 26 to 29 feet below the existing grades.

Monterey Formation (Tm)

The bedrock consists of sandstones and shales, which are assigned to the upper-middle Miocene Monterey Formation. Most of the site is underlain by locally hard to very hard with occasional (i.e., less than five percent) fissile sandstone, siltstone interbeds and siltstone inclusions. The bedrock onsite is largely west striking, moderate to steeply northeast dipping with localized moderate southwest dip.

Faulting and Seismicity

The site is located in the Corona del Mar area of the City, which is near the intersection of the Southwestern Block and the Central Block of the Los Angeles Basin. The Southwestern Block is the westerly seaward portion of the Los Angeles Basin, which includes Palos Verdes Peninsula and Long Beach, and is bounded on the east by the Newport-Inglewood Fault Zone (NIFZ). The Central Block extends easterly from the NIFZ to the Whittier Fault (WFZ). The main structural features in the area are the NIFZ and the WFZ. The landward part of the NIFZ is a northwesterly-trending zone that extends from Beverly Hills on the north to Newport Bay on the south, where it continues offshore to the south; however, it eventually returns ashore again near La Jolla, where it is expressed by the Rose Canyon Fault. The main trace of the NIFZ is approximately 1.7 miles offshore to the south-southwest, and has documented surface or near-surface rupture within the past 11,000 years. It is, therefore, "active" according to the State of California. The WFZ extends from the Chino/Corona area in the south to the Whittier Narrows area in the north. Historical earthquakes have occurred on both faults with the 1933, 6.l4M Long Beach Earthquake on the NIFZ[1] and the 1987 6.1M Whittier Narrows Earthquake on the WFZ. The NIFZ within the project environs is not included on the State-published Alquist-Priolo Special Studies zonation map.

The subject property is located within a seismically active area. Based on a literature review, photo interpretation and a site-specific fault investigation conducted by Neblett & Associates, Inc., in 2003, two faults were identified on the subject property, consisting of sheared bedrock zones. One small fault, which had been previously mapped in 1994, was located southerly adjacent to the 201-205 Carnation Avenue apartment building. The second (northerly) fault, which was very well exposed and had been mapped, was identified as a buried fault trace beneath the single-family residence at 207 Carnation Avenue. Trenching undertaken to evaluate the faults indicated that the rock within the trenches have not been displaced for at least the last 80,000 to 120,000 years. Based on the findings of the 2003 fault investigation, both faults were classified as "inactive." According to CDMG Special Publications 42, "active" faults are defined as those faults that have displaced during the last 11,000 years (i.e., Holocene age). Therefore, the faults identified on the site are not considered "active."

Although a literature review conducted for the preliminary geologic/geotechnical investigation indicated that a fault was mapped on the site, site mapping, aerial photo analysis, fault trenching, and age dating conducted for the proposed project concluded that no active faults are present on the subject property. There are no known local or regional active earthquake faults on or in close proximity to the site, and the site is not within an Alquist-Priolo Zone. The Newport-Inglewood Fault is located approximately 1.7 miles to the west of and off-shore from the site, the Whittier-Elsinore Fault is located approximately 25 miles to the northeast, and the San Andreas Fault is located more than 50 miles to the northeast. Although episodes on those faults could cause ground shaking at the project site, it is highly unlikely that the site would experience surface rupture. Even though the project site and surrounding areas could be subject to strong ground movements, adherence to current building standards of the City of Newport Beach would reduce ground movement hazards to a less than significant level.

Liquefaction

Liquefaction is the loss of strength of cohesionless soils when the pore water pressure in the soil becomes equal to the confining pressure. Liquefaction generally occurs as a "quicksand" type of ground failure caused by strong groundshaking. The primary factors influencing liquefaction potential include groundwater, soil type, relative density of the sandy soils, confining pressure, and the intensity and duration of groundshaking. The majority of the liquefaction hazards are associated with uncompacted, saturated or nearly saturated, non-cohesive sandy and silty soils. Based on the field mapping and subsurface exploration conducted for the proposed project, the artificial fill and terrace materials occurring

[1] The actual epicenter of the 1933 Long Beach Earthquake was located offshore in the Huntington Beach, Newport Beach area.

on the subject property are very shallow, unsaturated, fine-to-coarse-grained, silty sand with abundant gravels, cobbles and boulders.

Tsunami and Seiche

The subject property is located at the coastal margin of the Pacific Ocean, at the southern end of Newport Beach, within the Newport Harbor area. While this area is protected by jetty emplacement at the harbor mouth, long water waves generated by offshore mechanisms such as tectonic displacement present a potential for tsunamis. Recent tsunamis include the 1957 tsunami, which originated from the Aleutians and the 1964 tsunami, which originated from the Gulf of Alaska. These events resulted in recorded maximum wave heights of 0.9 feet and 1.8 feet, respectively in Newport Bay.

Seiche is defined as a standing wave oscillation effect generated in a closed or semi-closed body of water caused by wind, tidal current, and earthquake. Seiche potential is highest in large, deep, steep-sided reservoirs or water bodies. Newport Bay lacks significant potential for damaging seiche because it is very shallow.

Groundwater

Subsurface water was not observed during the field investigation.

4.9.2 Significance Criteria

Implementation of the proposed project would result in a significant adverse environmental impact if any of the following occurs as a result of project implementation.

- Loss or elimination of "prime" agricultural lands as designated by the State of California and/or County of Orange and such designated soils are capable of sustained, viable agricultural production.
- Ground shaking and/or secondary seismic effects (i.e., liquefaction, slope failure, etc.) could cause substantial structural damage and/or an unmitigated risk to human safety, even after implementation of the recommended geotechnical measures, required local and State seismic design parameters, and common engineering practices for seismic hazard abatement.
- Adverse soil conditions such as compressible, expansive, or corrosive soils are not mitigated and present a damage hazard to occupied structures or infrastructure facilities.

4.9.3 Standard Conditions

SC 4.9-1 All activities associated with the implementation of the proposed residential development shall comply with the City's Excavation and Grading Ordinance.

SC 4.9-2 The project shall comply with all applicable City and 2007 California Building Code requirements.

SC 4.9-3 The property owner(s) shall execute and record a waiver of future shoreline protection for the project prior to the issuance of a building permit. Said waiver shall be subject to the review and approval of the City Attorney.

SC 4.9-4 Accessory structures shall be relocated or removed if threatened by coastal erosion. Accessory structures shall not be expanded and routine maintenance of accessory structures is permitted.

4.9.4 Potential Impacts

4.9.4.1 Short-Term Construction Impacts

Although the terraced deposits on the site extend approximately 16 to 19 feet below existing grades, they can be excavated using conventional earthmoving equipment. However, the underlying bedrock consists of sandstones and shales. The sandstone is generally dense and massive, and includes hard and more resistant sandstone dikes. Generally, rock masses displaying seismic shear wave velocities of up to approximately 5,500 feet per second (fps) are considered economically rippable using conventional mechanical grading equipment. Rock masses displaying seismic shear wave velocities ranging from 5,500 to 7,000 fps are considered marginally rippable. Rock masses with seismic shear wave velocities greater than about 7,000 fps may require special excavation techniques. The shallow seismic profile velocities for the adjacent property at 2494 Ocean Avenue ranged from 4,000 to 8,350 fps; similar velocities can be anticipated for the subject property. Although the majority of the bedrock at the site is considered rippable; however, localized areas (i.e., those with seismic profile velocities greater than 7,000 fps) may require special excavation techniques.

4.9.4.2 Long-Term Operational Impacts

Agricultural Soils

Development of the subject property with Aerie residential structures as proposed will not result in the conversion of any designated prime agricultural soils or otherwise significant farmland. The site is located within a developed and urbanized area of the City of Newport Beach. As previously indicated, the project site and surrounding area are designated as "Urban and Built Up Land." Therefore, project implementation will not result in any impacts to agricultural soils or important farmland. No significant impacts are anticipated and no mitigation measures are required.

Faulting and Seismicity

Surface Rupture and Strong Ground Motion

Based on the site-specific fault investigation conducted for the proposed project, fault activity levels have not displaced terrace deposits for at least 80,000 to 120,000 years before present. According to Special Publication 42 prepared by the California Division of Mines and Geology (CDMG), "active faults are defined as those faults that have displaced during the last 11,000 years (i.e., Holocene age). Therefore, the faults identified on the subject property are not considered "active" and it is unlikely that the subject site will experience fault-related surface rupture. Nonetheless, the subject property may experience ground motion as a result of regional seismic activity

As indicated in Section 4.9.1, the subject property is located in the seismically active southern California region; several active faults are responsible for generating moderate to strong earthquakes throughout the region. Table 4.9-1 identifies the active regional faults that are capable of generating seismic ground shaking in the region. The maximum magnitude for each of the faults is also presented in that table.

Table 4.9-1
Regional Active Fault Parameters

Fault Name	Approx. Distance (km)	Source Type (A/B/C)	Max. Magnitude (Mw)	Slip Rate (mm/yr)	Fault Type[1]
Newport-Inglewood (Offshore)	2.8	B	6.9	1.50	SS
Newport-Inglewood (LA Basin)	4.2	B	6.9	1.00	SS
Palos Verdes	22.3	B	7.1	3.00	SS
Chino-Central Avenue	33.7	B	6.7	1.00	DS
Elsinore-Whittier	34.8	B	6.8	2.50	SS
Elsinore-Glen Ivy	36.1	B	5.8	5.00	SS
Coronado Bank	37.0	B	7.4	3.00	SS
San Jose	49.0	B	6.5	0.50	DS
Elsinore-Temecula	49.3	B	6.8	5.00	SS
Sierra Madre (Central)	59.6	B	7.0	3.00	DS
Cucamonga	60.2	A	7.0	5.00	DS
Raymond	62.9	B	6.5	0.50	DS
Verdugo	64.5	B	6.7	0.50	DS
Clamshell-Sawpit	65.4	B	6.5	1.00	DS
Hollywood	66.5	B	6.5	1.00	DS
Rose Canyon	66.9	B	6.9	1.50	SS
Santa Monica	72.8	B	6.6	1.00	DS
San Jacinto-San Bernardino	74.2	B	6.7	12.00	SS
San Jacinto-San Jacinto Valley	75.4	B	6.9	12.00	SS
Malibu Coast	77.4	B	6.7	0.30	DS
Elsinore-Julian	83.8	A	7.1	5.00	SS
San Andreas (Southern)	84.6	A	7.4	24.00	SS
Sierra Madre (San Fernando)	84.8	B	6.7	2.00	DS
San Andreas (1857 Rupture)	85.5	A	7.8	34.0	SS
Anacapa-Dume	87.0	B	7.3	3.00	DS
Cleghorn	88.0	B	6.5	3.00	SS
San Gabriel	88.2	B	7.0	1.00	SS
San Jacinto-Anza	90.4	A	7.2	12.00	SS
North Frontal Fault Zone (West)	96.1	B	7.0	1.00	DS
Santa Susana	98.9	B	6.6	5.00	DS

[1]SS – strike-slip; DS – dip-slip; BT – blind thrust.

SOURCE: Neblett & Associates, Inc. (March 28, 2003)

As indicated above, the nearest Type A fault is the Cucamonga Fault, which is located approximately 60.2 miles from the site. This fault is capable of generating a 7.0 magnitude earthquake. The nearest Type B fault is the offshore Newport-Inglewood fault (2.8 km from the subject property), which is capable of generating a maximum magnitude of 6.9. In addition, peak ground acceleration values were also calculated for the proposed project. Those values should be utilized for the design and construction of the residential structures. These values represent ground motions that, at a minimum, have a 10 percent probability of being exceeded in 50 years. The estimated mean peak ground acceleration at the site is 0.345g. As indicated above, the preliminary geologic/geotechnical investigation report identifies the appropriate CBC seismic coefficients for structural design. Implementation of the recommendations prescribed in the preliminary geologic/geotechnical investigation, Conceptual Grading Plan Review Report, and compliance with CBC structure design parameters will ensure that potential impacts associated with ground shaking associated with a seismic event on one of the causative faults are reduced to an acceptable level (i.e., minimize loss of life and/or property).

Secondary Seismic Effects

Liquefaction

According to the conceptual grading plan prepared for the proposed project, excavation necessary to implement the proposed project will extend to an ultimate elevation of approximately 30 feet NAVD88 in order to accommodate the subterranean levels of the proposed structure. The proposed excavation will effectively remove the artificial fill and terrace materials and will expose bedrock throughout the excavation. The removal of these materials, combined with the lack of subsurface water, effectively eliminates the potential for liquefaction to occur. Therefore, no significant impacts are anticipated and no mitigation measures are required.

Compressible, Expansive and Corrosive Soils

The project site and the surrounding area are not known to be located within an unstable geologic area and, therefore, are not expected to be exposed to or create on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse hazards. A representative soil sample was tested for expansion potential in accordance with Table 18-1-B, which concluded that existing site soils have a "very low" potential for expansion and, therefore, are not a significant issue given on-site soil conditions. A final geotechnical analysis will be completed as part of the final building permit review process, and strict adherence to the design recommendations are mandatory with building permit issuance. As required in SC 4.9-2, the project must comply with the applicable design parameters prescribed in the 2007 CBC as well as those required by the City of Newport Beach.

Soluble sulfate and corrosivity testing on representative samples of the on-site soils conducted for the project indicate a negligible sulfate concentration; however, because the project is located in a coastal environment, the potential for severe sulfate exposure to concrete exists. As a result, the type of concrete utilized should be consistent with the requirements of the 2007 CBC and City of Newport Beach.

Bluff Erosion

The major lithologic bedrock unit currently exposed and which will be exposed post-construction along the bluff face is generally moderately-to-well-cemented petroliferous sandstone cut by frequent sandstone dikes. The dikes are hard and more resistant to weathering than the host sandstone. This bedrock bluff is resistant to weathering and degradation, based on the aerial photo analysis conducted for the project, which establishes that the bluff has maintained its existing configuration for at least the past 80 years. No open fractures or adverse bedding planes were observed during the geologic investigation that could jeopardize the integrity and stability of the bluff (Neblett & Associates, Inc., 2008).

The results of the diamond core samples (refer to Appendix A of the Conceptual Grading Plan Review Report) indicate that rock quality designation (RQD) averages approximately 80 percent, which indicates low fracture index and a relatively high quality of rock. Excavations for and construction of planned subterranean levels, which will remove existing fill soils as well as a majority of the terrace deposits capping the bedrock and daylighting on the bluff face, will leave a trapezoidal (i.e., pillar) section of intact rock as part of the exposed bluff face to approximately Elevation 52.8 NAVD. With the removal of these materials, the bluff face will be less vulnerable to bluff erosion. Considering the both the lithologic bedrock unit exposed and the rock quality, the remaining trapezoidal section of intact rock will have sufficient strength to remain in place during the economic life of the structure (i.e., 75 years). Furthermore, the Coastal Hazard Study prepared by

GeoSoils, Inc., concluded that the proposed improvements will neither create nor contribute significantly to erosion, geologic instability, or destruction of the site or adjacent area.

The proposed grading plan indicates that excavation will daylight on the bluff face at approximately 52.8 NAVD, resulting in the removal of existing fill soils as well as a majority of the terrace deposits capping the bedrock and daylighting on the bluff face. The removal of these materials as well as the incorporation of site drainage measures recommended by the geotechnical consultant in the conceptual Grading Plan Review Report will also further reduce the potential for future bluff erosion. Based on the analysis conducted for the proposed project, bluff erosion is not considered a factor in design over the life of the structure.

Slope Stability

A slope stability analysis was included in the Conceptual Grading Plan Review Report prepared by Neblett & Associates, Inc. The excavation slope was analyzed by calculating the factors of safety for a circular-type failure and near the toe and along the base of excavation. The results of the slope stability analyses are summarized in Table 4.9-2.

Table 4.9-2
Results of Stability Analyses

Cross-Section	Method of Analysis	Computed Minimum Static Factor of Safety	File Reference
Typical	Circular-type failure near toe of excavation	1.93	416-1
	Circular-type failure through base of excavation	3.63	416-2
SOURCE: Neblett & Associates, Inc. (September 2008)			

The computed factor of safety for the temporary excavation under static conditions is greater than the minimum required 1.25. Therefore, based on the results of the stability analyses, the project geotechnical consultant concluded that the temporary excavation with soldier pile shoring system is acceptable, provided the recommendations prescribed in the Conceptual Grading Plan Review Report are implemented during construction, including temporary shoring during excavation and construction of the deeper excavations, tie-back anchors or internal bracing, etc. In addition, the structural design would also include provisions to accommodate basement wall water-proofing, drain installation, etc.

Earthquake-Induced Landsliding/Rocksliding

The slopes descending from the proposed development expose very resistant sandstone of the Monterey formation. Literature reviews, site mapping, aerial photo analysis, and subsurface exploration conducted for the project during the preparation of the Conceptual Grading Plan Review Report (Neblett & Associates, Inc., 2008) revealed that landslides do not exist on or adjacent to the subject property. The lack of landslide features indicates that the area has been relatively stable in the recent geologic past (i.e., Holocene) and has not been subject to earthquake-induced large-scale landsliding. Therefore, the potential for earthquake-induced landsliding is considered low.

Tsunamis and Seiches

A Coastal Hazard Study was prepared by GeoSoils, Inc., (October 2006), which includes an analysis of wave run-up, including that generated from a tsunami. The potential surface gravity waves that arrive at the subject property are small (i.e., less than 1.0 foot) waves and boat wakes, both of which are dampened by the moored vessels and dock systems that are located in front of and adjacent to the site. The maximum possible waves that can be generated at the site are those from ocean swells propagating down the entrance channel. As revealed in that report, the analysis was conservative and based on the open ocean wave height instead of the expected lower tsunami wave height inside Newport Bay. A 1.5-foot high wave was used as the basis for the analysis with a water level of +8.0 feet NAVD 88, which represents an approximately 100-year recurrence interval oceanographic conditions. Based on that analysis, the study concluded that there is no potential hazard from surface gravity waves or boat wakes to the proposed development.

Tsunami are waves generated by submarine earthquakes, landslides, or volcanic action. The maximum tsunami runup in the Newport Harbor area is less than two meters in height. Any wave, including a tsunami, that approaches the site in Corona del Mar will be refracted, modified, and reduced in height by the Newport jetties. Based on the same methodology that was used to estimate the surface gravity wave and boat wakes, the 6 foot high tsunami would yield a runup to elevation +16.2 feet NAVD 88 (i.e., six feet runup + 10 feet NAVD 88 water elevation). The basement elevation of the proposed structure is proposed to be approximately 30 feet NAVD88, with the lowermost exposed face of the structure daylighting on the slope at approximately 52.8 feet NAVD88. In addition, the dock access/emergency exit is located at elevation 40.5 feet NAVD88 and would also be located above the potential tsunami/wave runup limits discussed above. The tsunami, like the design extreme wave/wake, will not reach the proposed improvements. The analysis is conservative because the open ocean tsunami wave height was used instead of the expected lower tsunami wave height inside Newport Bay. Due to the infrequent nature and the relatively low 500-year recurrence interval tsunami wave height, combined with the elevation of the proposed improvements, the site is reasonably safe from tsunami hazards. Therefore, no significant impacts are anticipated during the 75-year economic life of the proposed project and no mitigation measures are required. Further, considering the proposed finish pad elevation, the potential for seiche effects to the project site is considered remote due to the shallow depth of Newport Harbor; no significant impacts are anticipated and no mitigation measures are required.

Coastal Erosion

Because the proposed project includes the replacement of the existing dock and landing facilities, an engineering study (Coastal Engineering Assessment for the "Aerie" Dock Project) was prepared by Noble Consultants, Inc. (May 9, 2008) to evaluate the potential effects of high winds and sand transport associated with these facilities. The findings and recommendations of this study are summarized below.

<u>Wave Conditions and Potential Impacts</u>

Wind stations derived from measurements at Long Beach Airport and San Clemente Island were analyzed to define typical and extreme wind conditions for the prediction of wind waves at the project site. Based on the data from the Long Beach Airport, approximately 25 percent of the time, the wind blows from the WNW-NNW sector at an average speed of approximately six knots. In addition, the one-hour average wind speed from this sector never exceeded 40 knots. Winds from the SSE-S sector have a relatively low probability of occurrence (i.e., less than 10 percent) and would typically blow at about six knots but would not exceed 27 knots. Wind data from San Clemente Island indicated that the WNW-NNW wind section would also blow at approximately the same speeds as shown for the Long Beach Airport. Winds from the SSE-S sector typically blow at about four knots; however, extreme winds from this sector could blow above 56 knots, significantly higher than this wind probability at Long Beach

Airport. Extreme wind speeds and fetches for the project site were calculated SSE-S sector (refer to Table 4.9-3) based on the data available at both Long Beach Airport and San Clemente Island.

Table 4.9-3
Selected Wind Conditions for Wind Wave Predictions

Condition	Direction	Speed	Fetch
Typical	WNW-NNW	6 Knots (3 m/sec)	Newport Bay, 4,300 Feet (1.3 km)
	SSE-S	6 Knots (3 m/sec)	Pacific Ocean, 60 miles (110 km)
Extreme	WNW-NNW	36 knots (19 m/sec)	Newport Bay, 4,300 Feet (1.3 km)
	SSE-S	48 knots (24 m/sec)	Pacific Ocean, 60 miles (110 km)

SOURCE: Noble Consultants, Inc. (May 9, 2008)

Based on the wind conditions identified in Table 4.9-3, wind wave conditions at the project site have been estimated and are summarized in Table 4.9-4. Based on that information, it can be concluded that wind-induced wave conditions at the project site would be typically mild. For about 65 percent of the time, there would be no wind waves. For the remainder of the time, significant wave heights would be 0.5 foot or less. On less frequent occasions, wind-induced significant wave heights would be higher than one foot and up to 2.5 feet, as indicated in Table 4.9-4.

Table 4.9-4
Wind Wave Conditions at the Project Site Resulting from
Typical and Extreme WNW-NNW and SSE-S Winds

Condition	Direction	Significant Wave Height (Feet)	Wave Period (sec)	Frequency of Occurrence
Typical	WNW-NNW	0.13	<1,0	25% of the time
	SSE-S	0.5	1.7	10% of the time
Extreme	WNW-NNW	1.3	1.5	Less frequent[1]
	SSE-S	2.5	9 to 10	Less frequent[1]

[1]A detailed wave hindcast, beyond the scope of the study prepared for the project, would be required to determine the frequency of occurrence (or return period) of this event.

SOURCE: Noble Consultants, Inc. (May 9, 2008)

Typical and extreme swell conditions for offshore Newport Beach were also calculated and presented in the Noble study. The results of this analysis are presented in Table 4.9-5. Based on that information, it can be concluded that wave conditions at the project site would, in general, be mild for approximately 65 percent of the time with either no wind waves or waves of negligible relevance at the project site. For about 25 percent of the time, winds from the WNW-NNW would generate a short 0.13-foot significant height, less than 1-second period wind wave; and for 10 percent of the time, the offshore SSE-S sea breeze would generate a 0.5-foot significant height, 1.7 period wind wave. On less frequent occasions, WNW-NNW winds within Newport Bay could generate 1.3 foot significant height, 1.5-second period wind waves. Similarly less frequent local storms from the SSE-S could generate 2.5-foot significant height, 9 – 10-second significant wind waves at the project site.

The particular orientation of the Newport Beach jetties and the presence of the Santa Catalina and San Clemente Islands prevent the predominant swell conditions, which approach the Southern California Bight from the W-NW sector for approximately 86 percent of the time, from affecting the site. With a frequency of occurrence of less than two percent, typical SSE-SW, 12 to 16-second swell would reach the project site with a significant height of 0.5 foot. On less frequent occasions, extreme SSE-SSW swell generated by distant storms could reach the project site with significant heights of approximately 1.5 feet and periods in the 12 to 14-second range. Table 4.9-5 summarizes the wave conditions at the project site resulting from typical and extreme SSE-SSW swell conditions offshore.

Based on the dock plan proposed for the project wave conditions from the WNW-NNW will approach moored vessels at the proposed facility approximately from the beam, whereas wave conditions from the SSE-SSW would be entering through the entrance channel and approach the moored vessels from the bow (head seas). Under typical WNW-NNW wave, and SSE-SSW wave and swell conditions, wave heights would be below the recommended one-foot limit,[2] regardless of the recurrence intervals recommended for wave conditions in small craft harbors.

Table 4.9-5
Wave Conditions at the Project Site Resulting from
Typical and Extreme SSE-SSW Swell Conditions Offshore

Condition	Offshore	Project Site	Frequency of Occurrence
Typical	Hs = 5 Feet T = 12 – 16 Seconds From SSE-SSW	Hs = 0.5 Feet T = 12 – 16 Seconds Parallel to entrance channel	Less than 2%
Extreme	H = 15 Feet T = 12 – 14 Seconds From SSE-SSW	H = 1.5 Feet T = 12 – 14 Seconds Parallel to entrance channel	Less Frequent[1]

Hs = significant wave height; T = period.

[1]A detailed wave hindcast, beyond the scope of the study conducted for the project, would be required to determine the frequency of occurrence (or return period) of this event.

SOURCE: Noble Consultants, Inc. (May 9, 2008)

The project site is exposed to impinging waves from either wind-generated period waves in the bay or ocean swells that will propagate through the entrance channel. For about 65 percent of the time, there would be no wind waves. For the remainder of the time, significant wave heights would be 0.5 foot or less. On less frequent occasions, wind-induced significant wave heights would be higher than one foot and up to 2.5 feet. Extreme SSE-SSW swell generated by distant storms could reach the project site with significant heights of approximately 1.5 feet and periods in the 12 to 14-second range. Because of the orientation of the harbor entrance channel, the study concluded that the site will be more exposed to storm waves generated associated with passage of winter pre-frontal storm winds and southern hemisphere swell that typically occurs in the summary months. As a result, the design of the proposed dock should be based on the extreme wave conditions where the structures will be most susceptible to damage from wave-induced forces and motion.

[2]Mercer, A.G., Isaacson, M. and Mulcahy, M. (198). "Design Wave Climate in Small Craft Harbours," 18th Conference on Coastal Engineering, Cape Town, South Africa.

The Noble Consultants study concluded that from a wave climate perspective, the proposed docking facility is feasible in a wide range of conditions. However, extreme wind waves from the SSE-SSW are expected to exceed the recommended maximum wave heights and, therefore, damage to the moored vessels and/or docking facilities may occur. In these less frequent conditions, vessels should be moved and sheltered in a less exposed location.

Sediment Processes and Flow Patterns

In the coastal/harbor zone, sediment typically moves in accordance with the impinging wave direction. Thus, sediment movement in the Newport Harbor entrance area depends strongly on the two distinguished wave patterns, winter north or northwest swells and southerly swells, that are typically observed in the region. The north and northwest swells occurring in the winter months have a deep water approach direction of between 275° and 285° toward Southern California. Sediment movement along the Newport Beach shoreline would, therefore, be toward the southeast (i.e., toward Newport Harbor). At The Wedge, the beach adjacent to the harbor entrance area, sands are partially pushed through the riprap jetty as well as moved around the jetty. The transported sands deposit in the harbor entrance adjacent to the jetty area during the winter months.

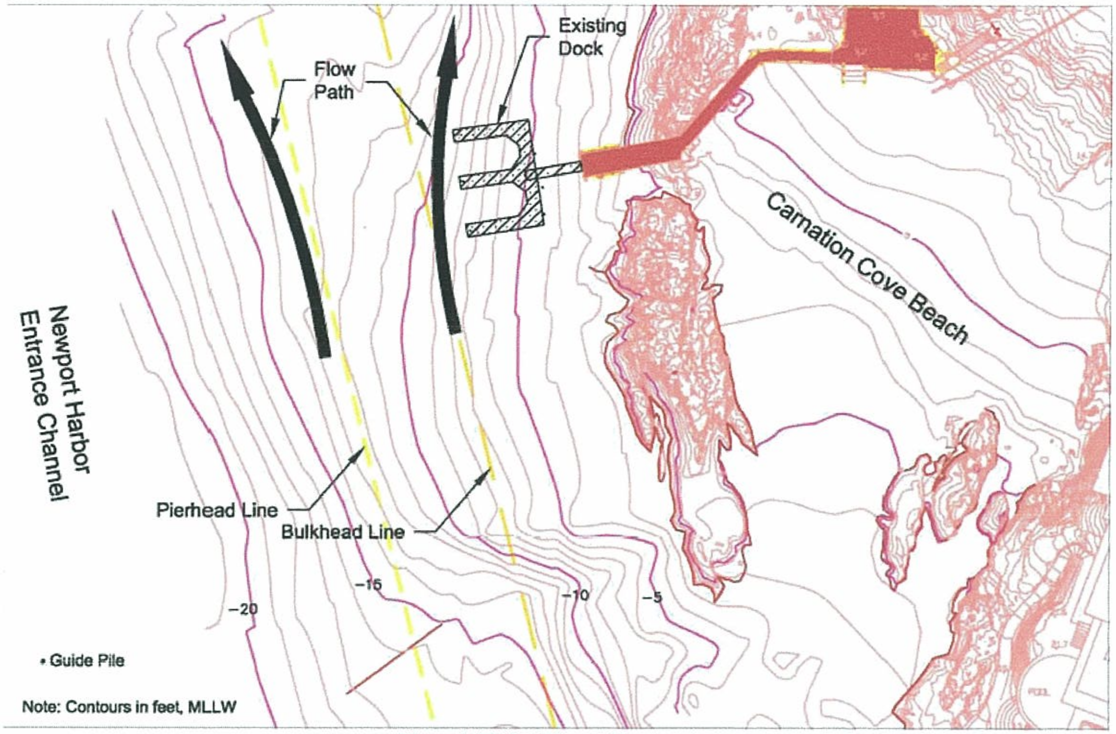
Based on information provided by the City of Newport Beach (Chris Miller, Harbor Resource Division), an annual total of approximately 5,000 cubic yards of sands are transported by waves into cove beaches in the area, resulting in a need for dredging from some dock facilities in order to maintain an adequate depth for boat berthing. The vast majority of sand depositing in the cove areas is coastal littoral sediment transported through the entrance channel. Sediment discharged either from the Upper Newport Bay or storm drains in the adjacent area would be fine silt, which is not beach-quality material.

The project site's waterfront area is characterized by various rock outcrops that form a cove beach, which appears to be stable because little change has occurred over the years based on a review of aerial photographs between 2001 and 2006. The bottom gradient where the proposed replacement dock will be constructed is approximately 9:1 (horizontal to vertical). A patch of sand along the channel side of the site's rock outcrop that is parallel to the navigational channel was observed at the time the study was conducted by Noble Consultants, Inc. The patch of sand, which is located in the depth shallower than 5 feet at the MLLW line, appears to be stable.

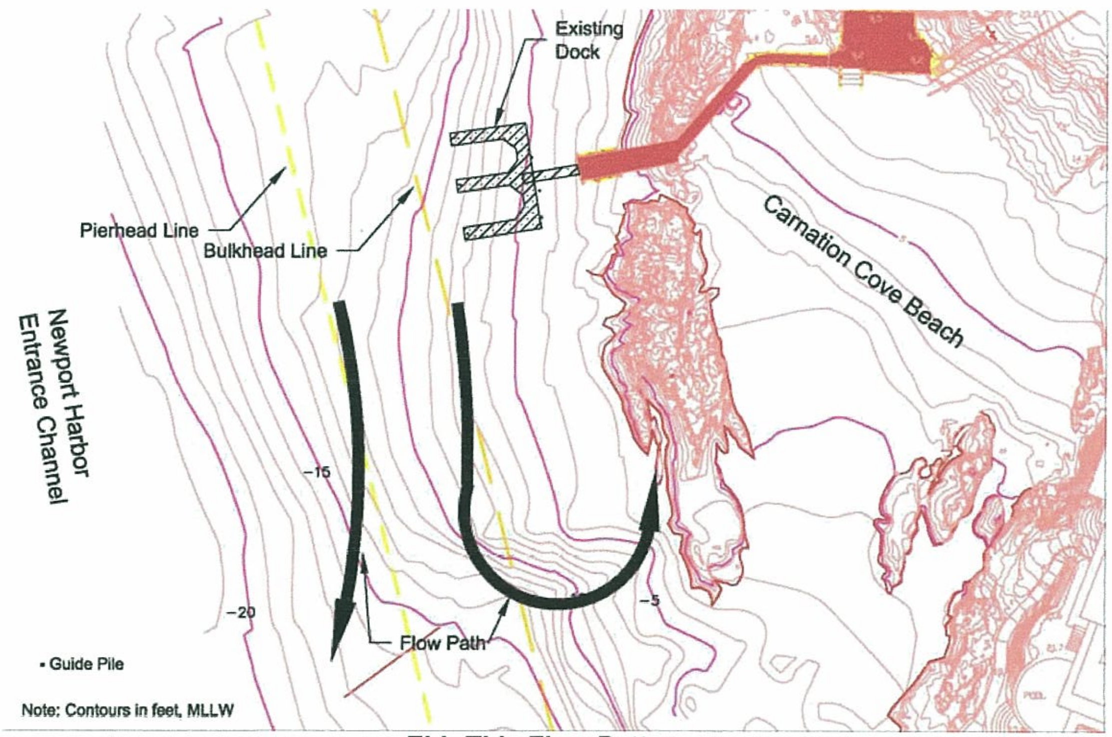
Exhibit 4.9-1 illustrates the flow patterns that characterize the channel in the vicinity of the project during the flood/ebb tide cycle. The flood tide water flows somewhat parallel to the depth contours at the site and splits either into Carnation Cove or along the main navigational channel. These two flow fields would eventually converge and continue toward the upper bay. During the ebb tide, the reverse flow patterns were observed, except for an eddy zone located 100 feet oceanward from the existing pier. The occurrence of this eddy zone may be attributed to the abrupt deepening of water depth, which not only slows down the flow rate but also alters the flow direction.

Based on the findings presented in the coastal engineering assessment prepared for the project, sediment deposited along the east side of the entrance channel at Newport Harbor is due to the uniqueness of sequential sediment transport patters that are typically observed in the harbor entrance area. Coastal alongshore drifted sands are transported either through the east jetty or via the entrance channel during the winter months and moved further into the bay by southerly swells primarily occurring in the following summer season.

Sand-quality sediment movement within the project region is typically in the along-channel direction from the harbor entrance to the inner bay. A stable bayshore condition is observed at the project site. Regular sedimentation observed at China Reef located in the updrift area is primarily due to the groin-like outcrop feature that entraps the along-channel transported sediment.



Flood-Tide Flow Patterns



Ebb-Tide Flow Patterns

Exhibit 4.9-1
Flood-Tide and Ebb-Tide Flow Patterns

SOURCE: Noble Consultants

With a small percentage (approximately six percent) of the along-channel blockage area resulting from the proposed new dock facility, the potential impact to this unique sediment movement process in the entrance channel is insignificant, although localized sand deposit resulting from the presence of the proposed guide piles within the sand-moving path may occur. In addition, the project is located in the downdrift direction of the neighboring China Reef, the project's potential impact on sedimentation at the updrift location such as China Reef is inconsequential. No significant impacts to sand transport resulting from project implement are anticipated and no mitigation measures are required.

4.9.5 Mitigation Measures

Impact 4.9-1 ***Although the site is suitable for the proposed development, construction of the proposed residential structure may be affected by the existing geologic and geotechnical engineering factors, including regional seismicity, bedrock, corrosive soils, erosion, etc.***

MM 4.9-1a Project implementation shall adhere to the engineering recommendations for site grading and foundation design and construction presented in the Conceptual Grading Plan Review Report prepared by Nebeltt & Associates, Inc., and subsequent detailed geotechnical engineering analyses.

MM 4.9-1b Accessory structures shall be relocated or removed if threatened by coastal erosion. Accessory structures shall not be expanded and routine maintenance of accessory structures is permitted.

Impact 4.9-2 ***The site (i.e., proposed dock) will be exposed to storm waves generated associated with passage of winter pre-frontal storm winds and southern hemisphere swell that typically occurs in the summary months. Extreme wind waves from the SSE-SSW are expected to exceed the recommended maximum wave heights, which may result in damage to the moored vessels and/or docking facilities.***

MM 4.9-2a During periods when boats would be exposed to excessive wave-induced motions, boats should be sheltered at mooring can locations that are available inside Newport Harbor to avoid damage.

MM 4.9-2b The dock design shall be based on the extreme wave conditions identified in the coastal engineering study (Noble Consultants, Inc., 2008).

4.9.6 Level of Significance After Mitigation

Implementation of the standard conditions prescribed in Section 4.9.3 and proposed mitigation measures will ensure that potential soils and geologic and related wave-induced impacts identified in Section 4.9.4 will be reduced to a less than significant level. No significant unavoidable adverse impacts will remain.