IV. ENVIRONMENTAL IMPACT ANALYSIS
D. GEOLOGY AND SOILS

The following section is a summary of two geotechnical reports conducted for the proposed project site. The two reports include the Report of Geotechnical Investigation Proposed High-Rise Condominium Building Development, prepared by MACTEC, dated August 26, 2003 and the Technical Transmittal Support of Environmental Impact Report Proposed High-Rise Condominium Building Development, prepared by MACTEC, dated September 20, 2004. A copy of these reports can be found in Appendix C.

ENVIRONMENTAL SETTING

The project site consists of a 0.57-acre parcel, which was previously occupied by buildings that have since been demolished. The project site is currently vacant. Underground utility lines and old obstructions may still exist beneath the project site.

Geologic Conditions

The project site is located in the northwestern portion of the Coastal Plain of the Los Angeles Groundwater Basin within the physiographic feature known as the Sawtelle Plain. On a regional scale, the Sawtelle Plain is a Pleistocene-age alluvial fan that was developed by alluvial dissection of the surrounding Santa Monica Plain, the surface of which now lies below the general elevation of the surrounding Santa Monica and Ocean Park Plains. The Sawtelle Plain slopes gently toward the southeast and is generally mantled by a thin veneer of younger alluvium (Dibblee, 1991 and Department of Water Resources, 1961).

In the immediate project vicinity, the exposed geologic materials consist of Pleistocene-age alluvial deposits (older alluvial-fan deposits). Locally, the Pleistocene alluvial surface has been incised by stream activity from drainage of the fan surface, resulting in low-lying drainages located along the southern edge of the older alluvial deposits. Specifically, the project site is located at the western edge of a broad low-lying drainage, one of the several drainages that have been incised into the southern edge of the upper surface of the Pleistocene fan. The drainage is in-filled with a thin veneer of Holocene-age (younger) alluvial deposits.

The geologic materials underlying the project site consist of an interbedded sequence of well-graded sand, silty-sand, silt, silty-clay, and sandy-clay with some gravel to a depth of 100 feet beneath the existing ground surface (bgs). These soils form a complex multiple-aquifer system at the project site, which is comprised of at least seven distinct soil layers (refer to Figure IV.E-1, Groundwater Elevations and Direction of Groundwater Flow). The coarse-grained soils form aquifers and the finegrained deposits form aquitards. The upper 45 to 50 feet of materials encountered in the explorations are likely to be of Holocene age and the lower portion of the sequence is considered to be Pleistocene age.

Soil Conditions

Three of the five exploration borings at the project site were drilled in 1965 and 1972 and the other two borings were drilled more recently. The soil conditions beneath the project site would not significantly change over time. Therefore, the results of the prior exploration borings drilled at the project site are still applicable to the currently proposed project.

Existing fill soils, about 2.5 feet thick, were encountered in one of the two recent borings. The existing fill soils, which consist of clay are not uniformly well compacted and contain some debris. Deeper and/or poorer quality fill could occur between the boring locations and elsewhere at the project site, particularly at the location of the existing buildings that were demolished.

Based on the current and prior exploration borings drilled to depths of about 100 feet below the existing grade, the soils underlying the project site consist primarily of silt, clay, silty sand, and sands with considerable amounts of gravel and some cobbles in the sandy deposits. The upper natural soils are medium stiff or medium dense and become stiffer and denser with an increase in depth. The soils at and below the excavation level are generally stiff or dense.

Water was encountered at depths as shallow as 23 feet below the existing grade in the previous borings and at a depth of 18 feet in the recent borings. However, based on the official data from the California Geological Survey (CGS), the historical ground-water level at the project site is at a depth of about 23 feet below the existing grade (refer to Section IV.E. Hydrology for more details).

Based on soil samples obtained at the project site, the soil is classified as corrosive to ferrous metals, aggressive to copper, and not corrosive to portland cement concrete.[1]

Liquefaction

Liquefaction is a process by which water-saturated unconsolidated sediments lose their strength due to increased pore pressure during or after an earthquake. Liquefaction potential is greatest where the groundwater level is shallow, and loose, fine sands occur in the shallow areas. Liquefaction potential

[1] *Based on the soil corrosivity study performed by M.J. Schiff & Associates, Inc., Consulting Corrosion Engineers (report can be found in Appendix C).*

decreases with increasing grain size and clay and gravel content, and increases as the depth to groundwater decreases.

The Californian Geological Survey (CGS) has been performing a comprehensive and thorough evaluation of the liquefaction potential of the soils throughout California, including the area where the project site is located. The evaluation of the liquefaction potential is based on the geologic and soils conditions within each geologic quadrangle and on the depth to groundwater. The geologic and soils conditions are determined by exploration borings performed by CGS and by geotechnical engineering firms. The depth to groundwater is determined by water monitoring wells that were installed throughout California.

Based on the information provided above, CGS determines the zones underlain by liquefiable soils and the zones underlain by non-liquefiable soils for each geologic quadrangle in California. The zones are identified by a light color for the zones underlain by unliquefiable soils and by a dark color for zones underlain by liquefiable soils.

The project site is part of the Beverly Hills Quadrangle and its location is identified in Figure IV.D-1, Liquefaction Map. Based on the liquefaction map presented in Figure IV.D-1, the site is not within a liquefiable zone. In addition, based on past experience at the site and at nearby sites, the soils underlying the site are not subject to liquefaction. Accordingly, the potential for liquefaction of the soils underlying the Site is considered to be very low.

Seismic Conditions

The entire Southern California area is considered a seismically active region. The region has numerous active, potentially active, and inactive faults. Active faults are defined as a fault that has had a surface displacement within Holocene times (about the last 11,000 years). A potentially active fault is a fault that has demonstrated surface displacement of Quaternary age deposits (within the last 1.6 million years).

There are no known faults on the proposed project site. As shown in Figure IV.D-2, based upon the "Maps of known Active Fault, Near Source Zones in California and Adjacent Portions of Nevada," dated February 1988 (part of the 1997 Uniform Building Code (UBC)). Based on a review of the local soil and geologic conditions, the project site may be classified as Soil Profile Type SD, as specified in the 1997 UBC. The project site is located within UBC Seismic Zone 4 and is near the Santa Monica fault, which has been determined to be a Type B seismic source by the California Division of Mines and Geology. The site is located at a distance of less than 2 kilometers from the Santa Monica fault according to Map M-32 (ICBO, 1998). The Santa Monica fault is the western segment of the Santa Monica-Hollywood fault zone which trends east-west from the Santa Monica coastline on the west to the Hollywood area on the east. In the Santa Monica area, the Santa Monica fault splays into two segments, the North Branch and the South Branch. Several investigators have indicated that the fault is active, based on geomorphic evidence and fault trenching studies. Recent studies indicate that the Santa Monica fault does not extend east of the northerly extension of the Newport-Inglewood fault zone or the West Beverly Hills Lineament of Dolan and Sieh (1992). The Santa Monica fault has not been zoned as active under the Alquist-Priolo Earthquake fault Zone Act because of the absence of well-defined fault traces. However, the Santa Monica fault is still considered active by the State Geologist.

ENVIRONMENTAL IMPACTS

Thresholds of Significance

Appendix G of the CEQA Guidelines indicates that a project could have a potentially significant geology and soils impact if it were to cause one or more of the following conditions:

- Expose people or structures to potential substantial adverse effects, including the risk of loss, injury, or death involving:
- Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map or based on other substantial evidence of a known fault?
- Strong seismic ground shaking?
- Seismic-related ground failure, including liquefaction?
- Result in substantial soil erosion or the loss of topsoil?
- Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse?
- Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial risks to life or property?

Based on the City of Los Angeles CEQA Thresholds Guide, the proposed project would also result in a significant geotechnical impact if it exceeds the following threshold. Where applicable, the thresholds from the City's CEQA Thresholds Guide are provided in the EIR because they address potential environmental impacts that are not entirely addressed by Appendix G of the CEQA Guidelines.

Figure IV.D-1, Liquefaction Map

Figure IV.D-2, Regional Faults and Seismicity

- A project would normally have a significant geologic hazard impact if it would cause or accelerate geologic hazards which would result in substantial damage to structures or infrastructure, or expose people to substantial risk of injury.

Project Impacts

Grading and Construction Considerations

Foundations

Based on the results of the geotechnical investigation, the geologic materials underlying the project site consist of an interbedded sequence of well-graded sand, silty-sand, silt, silty-clay, and sandy-clay with some gravel to a depth of 100 feet beneath the existing ground surface (bgs). A corrosion study performed as part of the previous investigation by Corrosion and Cathodic Protection Engineering Services indicates that the soils are corrosive to ferrous metals and aggressive to copper. The soils were found to be non-detrimental to portland cement concrete.[2] Expansive soils could have a significant impact on the proposed development. However, the significant impacts resulting from expansive or corrosive soils can be completely mitigated, as discussed in the mitigation measures below.

The proposed building may be supported on spread footings established in the stiff and dense undisturbed natural soils. In addition, individual footings or a combination of individual and combined or continuous footings may be used. As an alternative to spread footings, the proposed building may be supported on a mat-type foundation at the excavation level. Furthermore, where the necessary space is available for sloped excavation during the construction of the subterranean parking structure, temporary unsurcharged embankments may be sloped back without shoring. Shoring will be used where sloped excavations are not possible.

The actual foundation design requirements will be determined during the design phase of the project. The proposed building shall be designed so that footing will be supported in firm natural soils or properly compacted fill or so that drilled or driven piles extend into dense natural soils, in conformance with the City of Los Angeles Building Code.

Grading and Excavation

The construction of the three level subterranean parking structure would require the excavation and removal of approximately 38,600 square feet of soil. Total excavation would be approximately 35 to

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[2] *Based on the soil corrosivity study performed by M.J. Schiff & Associates, Inc., Consulting Corrosion Engineers (report can be found in Appendix C).*

40 feet. The natural soils at and below the planned level of excavation are generally stiff or dense. For this reason, no significant difficulties due to the soil conditions are anticipated in excavating the project site.

In addition, the planned excavation would extend below the groundwater level, and dewatering prior to excavation would be necessary to permit excavation for the basement level. Please refer to Section IV.E. Hydrology for a detailed discussion pertaining to groundwater. Therefore, with implementation of the mitigation measures listed below geology impacts would be less than significant.

Erosion

The project site is currently vacant and unpaved. Localized erosions during construction could occur during periods of heavy precipitation. This condition will be mitigated during the design phase of the proposed project, as discussed in the mitigation measures.

Seismic Hazards

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear walls and reinforcements. The proposed construction would be consistent with all applicable provisions of the city of Los Angeles Building Code, as well as the seismic design criteria contained within the Uniform Building Code. Additional precautions may be taken to protect personal property and reduce the chance of injury including strapping water heaters and securing furniture. It is likely that the project site will be shaken by future earthquakes produced in southern California. However, with the incorporation of mitigation measures listed below, these impacts would be less than significant.

Liquefaction

As discussed previously, the project site is not within a liquefiable zone. In addition, based on past experience at the site and at nearby sites, the soils underlying the site are not subject to liquefaction. Accordingly, the potential for liquefaction of the soils underlying the site is considered to be very low. Therefore, liquefaction impacts would be less than significant.

CUMULATIVE IMPACTS

Development of the proposed project in conjunction with the related projects listed in Section II.B would result in further development of the Westwood area in the City of Los Angeles. Geotechnical hazards are site-specific and there is little, if any, cumulative relationship between development of the proposed project and the related projects. Therefore, cumulative geology and soils impacts would be less than significant.

MITIGATION MEASURES

The following mitigation measures are required to reduce geology and soils impacts to less than significant levels:

Erosion

1. Drainage collection devices shall be designed in conformance with the City of Los Angeles grading and building codes to ensure that all runoff will be collected and transferred to the proper collection devices. The applicant shall provide analysis of the drainage volume created by the proposed project. All design of drainage flow, collection, and discharge shall be in conformance with current city codes and subject to approval by the City of Los Angeles. On-site grading shall be performed in accordance with City codes to ensure that the erosion of graded areas will not occur. All areas of construction shall be fine-graded to direct runoff to the street or to the nearest available storm drain. No runoff within the property boundaries shall be allowed to flow uncontrolled.

Foundations

Bearing Values

2. Spread footings, if used for support of the proposed building, carried at least 1 foot into the undisturbed stiff or dense natural soils at the planned excavation level, and at least 3 feet below the lowest adjacent grade or floor level can be designed to impose a net dead-plus-live load pressure of up to 8,000 pounds per square foot. The footing excavations shall be deepened as necessary to extend into undisturbed natural soils.

3. The mat-type foundation supporting the entire building established in the undisturbed natural soils shall be designed to impose a net dead-plus-live load soil pressure of up to 5,000 pounds per square foot. The mat shall be sufficiently reinforced and thickened to distribute the imposed loads uniformly across the mat.

4. Footings for minor structures (including auxiliary retaining walls, free-standing walls, and elevator pit walls) that are structurally separate from the proposed building shall be designed to impose a net dead-plus-live load pressure of 1,500 pounds per square foot, at a depth of 2 feet below the adjacent grade. Such footings shall be established in either properly compacted fill and/or undisturbed natural soils.

5. The recommended bearing values are net values, and the weight of concrete in the footings and mat shall be taken as 50 pounds per cubic foot; the weight of soil backfill shall be neglected when determining the downward loads from the structure. A one-third increase in the above bearing values shall be used when considering wind or seismic loads.

6. While the actual bearing value of the compacted fill will depend on the materials used and the compaction methods employed, the quoted bearing value will be applicable if acceptable soils are used and are compacted as recommended. The bearing value of the compacted fill shall be confirmed during grading.

Settlement

7. As the degree of settlement of the proposed condominium building will depend on the foundation loads imposed, settlement analyses shall be performed when the foundation load information is available. The results of the settlement analyses will be used to confirm or modify the foundation design recommendations presented in this report.

Modulus of Subgrade Reaction

8. To assist in the structural analyses of the mat foundation, a modulus of subgrade reaction (k) of 50 pounds per cubic inch shall be used for the soils underlying the mat foundation. This value was estimated from available data and published empirical relationships.

Lateral Loads

9. Lateral loads shall be resisted by soil friction and passive resistance against the footings or the mat foundation. A coefficient of friction of 0.5 shall be used between spread footings, the floor slab, the mat and the supporting soils. The passive resistance of the undisturbed natural soils against footings or the mat shall be assumed to be 300 pounds per cubic foot. A one third increase in the passive value shall be used for wind or seismic loads. The passive resistance of the soils and the frictional resistance between the floor slab, footings or the mat and the supporting soils shall be combined without reduction in determining the total lateral resistance.

Ultimate Values

10. The recommended bearing and lateral load design values for the proposed building are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values shall be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	3.0
Passive Pressure	1.75
Coefficient of Friction	1.25

In no event, however, shall foundation sizes be less than those required for dead-plus-live loads when using the working stress design values.

Foundation Observation

11. To verify the presence of satisfactory soils at design elevations, all footing or mat excavations shall be observed by personnel of MACTEC. Footings or mat shall be deepened as necessary to reach satisfactory supporting soils. Where foundation excavations are deeper than 4 feet, the sides of the excavations shall be sloped back at 1:1 or shored for safety.

12. Backfill around and over foundations and utility trench backfill within the building area shall be mechanically compacted; flooding shall not be permitted.

13. Inspection of the foundation excavations shall also be required by the appropriate reviewing governmental agencies. The contractor shall be familiar with the inspection requirements of the reviewing agencies.

Shoring

14. Where there is not sufficient space for sloped embankments, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied back with earth anchors. Some difficulty shall be encountered in the drilling of the soldier piles and the anchors because of shallow ground water and caving of the sandy and gravelly deposits. Special techniques and measures will be necessary in some areas to permit the proper installation of the soldier piles and the tie back anchors. In addition, if there is not sufficient space to install the tie back anchors to the desired lengths on any side of the excavation, the soldier piles of the shoring system shall be internally braced.

Lateral Pressures

15. For design of cantilevered shoring, a triangular distribution of lateral earth pressure shall be used. It shall be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. Where retained soils are partially sloped at 1:1 above the shoring, it shall be assumed that the soils will exert lateral pressures equal to 60 pounds per cubic foot.

16. For the design of tied-back or braced shoring, a trapezoidal distribution of earth pressure shall be used. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in Appendix C (Geotechnical Reports) with the maximum pressure equal to 22H in pounds per square foot, where H is the height of the shoring in feet. Where a combination of sloped embankment and shoring is used, the pressure would be greater and must be determined for each combination. However, where the required soils are sloped at 1:1 above the shoring, it shall be assumed that the soils will exert lateral pressure equal to 44H pounds per cubic foot.

17. The upper 10 feet of shoring adjacent to the streets and vehicular traffic areas shall be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge shall be neglected. Furthermore, the shoring system shall be designed to support the lateral surcharge pressures imposed by concrete trucks, cranes, and other heavy construction equipment placed near the shoring system.

Design of Soldier Piles

18. For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation shall be assumed to be 500 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 5,000 pounds per square foot. To develop the full lateral value, provisions shall be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations shall be a lean-mix concrete. However, the concrete used in that portion of the soldier pile, which is below the planned excavated level, shall be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

19. The frictional resistance between the soldier piles and the retained earth shall be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier piles and the retained earth shall be taken as 0.3. In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete,

the soldier piles below the excavated level shall be used to resist downward loads. For resisting the downward loads, the frictional resistance between the concrete soldier piles and the soils below the excavated level shall be taken equal to 400 pounds per square foot.

Lagging

20. Continuous lagging will be required between the soldier piles. The soldier piles and anchors shall be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. The lagging shall be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot.

Anchor Design

21. Tie back friction anchors shall be used to resist lateral loads. For design purposes, it shall be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. The anchors shall extend at least 30 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

22. The capacities of anchors shall be determined by testing of the initial anchors as outlined in a following section. For design purposes, it is estimated that drilled friction anchors will develop an average friction value of 500 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors needs to be considered due to group action.

Anchor Installation

23. The anchors shall be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes shall be anticipated and provisions made to minimize such caving. The anchors shall be filled with concrete placed by pumping from the tip out, and the concrete shall extend from the tip of the anchor to the active wedge. To minimize chances of caving, M the portion of the anchor shaft within the active wedge shall be backfilled with sand before testing the anchor. This portion of the shaft shall be filled tightly and flushed with the face of the excavation. The sand backfill shall contain a small amount of cement to allow the sand to be placed by pumping.

Anchor Testing

24. A MACTECT representative shall select at least two of the initial anchors for a 24 hour 200% test, and five additional anchors for quick 200% tests. The purpose of the 200% test is to verify the friction value assumed in the design. The anchors shall be tested to develop twice the assumed friction value. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length shall be increased until satisfactory test results are obtained. The total deflection during the 24-hour 200% test shall not exceed 12 inches during loading; the anchor deflection shall not exceed 0.75 inches during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than 0.5 inches, and the movement over the previous 4 hours has been less than 0.1 inches, the test shall be terminated.

25. For the quick 200% tests, the 200% test load shall be maintained for 30 minutes. The total deflection of the anchor during the 200% quick test shall not exceed 12 inches; the deflection after the 200% test load has been applied shall not exceed 0.25 inches during the 30-minute period. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length shall be increased until satisfactory test results are obtained.

26. All of the production anchors shall be pretested to at least 150% of the design load; the total deflection during the tests shall not exceed 12 inches. The rate of creep under the 150% test shall not exceed 0.1 inches over a 15-minute period for the anchor to be approved for the design loading.

27. After a satisfactory test, each production anchor shall be locked off at the design load. The locked off load shall be verified by rechecking the load in the anchor. If the locked off load varies by more than 10% from the design load, the load shall be reset until the anchor is locked off within 10% of the design load.

28. The installation of the anchors and the testing of the completed anchors shall be observed by MACTEC.

Internal Bracing

29. Raker bracing shall be used to internally brace the soldier piles. If used, raker bracing could be supported laterally by temporary concrete footing (deadmen) or by the permanent interior footings. For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 4,500 pounds per square foot shall be used, provided the shallowest point of the footing is at least

1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers shall be tightly wedged against the footings and/or shoring system.

Deflection

30. It is estimated that deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing shall be provided to minimize settlement of the utilities in the adjacent streets. To reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

Monitoring

31. The performance of the shoring system shall be monitored. The monitoring shall consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. The precise monitoring program shall be when the shoring system design is finalized.

32. The adjacent existing structures shall be surveyed for horizontal and vertical locations. Also, a careful survey of existing cracks and offsets in any adjacent structure shall be performed and recorded and photographic records made.

Walls Below Grade

Lateral Pressures

33. For design of cantilevered retaining walls, where the surface of the backfill is level, it shall be assumed that the soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. The subterranean walls shall be designed to resist a trapezoidal distribution of lateral earth pressure. The lateral earth pressure on the permanent subterranean walls will be similar to that recommended for design of temporary shoring except that the maximum lateral pressure will be 24H in pounds per square foot, where H is the height of the walls in feet. The recommended earth pressure assumes that a subdrain system will be installed below the floor slab of the lower subterranean level and behind the subterranean walls, so that external hydrostatic pressure will not be developed, if the ground water rises to the historical depth of 25 feet below the existing grade.

34. In addition, to the recommended earth pressure, the upper 10 feet of walls adjacent to streets and vehicular traffic areas shall be designed to resist a uniform lateral pressure of 120 pounds per square foot, acting as a result of an assumed 350 pounds per square foot surcharge behind the walls due to normal traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge shall be neglected.

Waterproofing

35. As discussed in section IV.E. Hydrology, a subdrain system shall be installed so that external water pressure will not be developed against the basement walls. In addition, walls below grade shall be waterproofed.

Backfill

36. Required soil backfill shall be mechanically compacted, in layers not more than 8 inches thick, to at least 90% of the maximum density obtainable by the ASTM Designation D1557 91 method of compaction. The backfill shall be sufficiently impermeable when compacted to restrict the inflow of surface water. The placement of the upper on-site clay soils as a backfill behind walls below grade shall be avoided. Some settlement of the deep backfill shall be allowed for in planning sidewalks and utility connections.

Paving

37. To provide support for paving, the subgrade soils shall be prepared as recommended below in Grading. Compaction of the subgrade to at least 90%, including trench backfills, will be important for paving support. The preparation of the parking area subgrade shall be done immediately prior to the placement of the base course. Proper drainage of the paved areas shall be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

38. To provide information for paving design, stabilometer (R-value) test was performed on a sample of the upper soils. The results of the test, which indicated an R-value of 34, are presented in Appendix C (Geotechnical Reports), Figures A-5.1 through A-5.3.

Asphaltic Paving

39. The required asphaltic paving and base course thickness will depend on the anticipated wheel loads and volume of traffic. The recommended paving sections for a range of Traffic Indices are presented below. The paving sections were determined using the Caltrans design method. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved and that proper construction procedures are used.

Assumed Traffic Index	Asphaltic Paving (Inches)	Base Course (Inches)
4½ (Automobile Parking)	3	4
5½ (Driveways with Light Truck Traffic)	3	7
6½ (Roadways with Heavy Truck Traffic)	4	7

Portland Cement Concrete Paving

40. Portland cement concrete paving sections were determined in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for a range of Traffic Indices are presented below. It is assumed that the portland cement concrete will have a compressive strength of at least 3,000 pounds per square inch. The concrete paving shall be properly reinforced. In addition, dowels are recommended at joints in the paving to reduce possible offsets.

Assumed Traffic Index	Paving Section (Inches)	Base Course (Inches)
4½ (Automobile Parking)	6	2
5½ (Driveways with Light Truck Traffic)	6½	2
6½ (Driveways with Heavy Truck Traffic)	7	2

Base Course

41. The base course for both asphaltic and concrete paving shall meet the specifications for Class 2 Aggregate Base as defined in Section 26 of the latest edition of the State of California, Department of Transportation, Standard Specifications. Alternatively, the base course could meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction. The base course shall be compacted to at least 95%.

Grading

42. To provide support for the footings of minor structures, and for at-grade concrete walks and slabs adjacent to the building and for paving, all existing fill and disturbed natural soils shall be excavated and replaced with properly compacted fill, and all required additional fill shall be properly compacted. The footings of the minor structures shall be established in properly compacted fill and/or undisturbed natural soils.

43. Where excavations for minor footings are deeper than 4 feet, the sides of the excavations shall be sloped back at 1:1 (horizontal to vertical) or shored for safety. All footing and utility trench backfills shall be mechanically compacted; flooding shall not be permitted.

Site Preparation and Moisture Conditioning

44. After excavating as recommended, the exposed soils in areas to receive additional fill shall be inspected and any disturbed deposits shall be excavated. The moisture content of the soils shall be determined, and the soils shall be slowly and uniformly moistened (or dried) as necessary to bring the soils to a uniformly moist condition. The moisture content of the cohesive soils and compacted fill shall be brought to between 2% and 4% over optimum moisture content to a depth of 6 inches. The moisture content of any non-expansive materials shall be brought to within 2% of optimum moisture content. The moisture content of the subgrade shall be checked and approved prior to placing the required fill.

Subgrade Preparation

45. After moistening as required, the exposed soils shall be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-91 method of compaction. The upper cohesive soils have a high moisture content, and it will be necessary to allow the surface to dry out prior to compacting. As an alternative, a layer of coarse crushed rock about 12 inches thick could be placed over the exposed soils to provide a base for the compaction of the required fill. A geotextile fabric could be placed to help stabilize the subgrade soils and reduce the amount of gravel required. Where grading is interrupted by rain, fill operations shall not be resumed until the moisture content and dry density of the placed fill are satisfactory.

Compaction

46. After compacting the exposed soils, or after placing the gravel layer, the required fill shall be placed in horizontal lifts not more than 8 inches thick and compacted to at least 90%. Relatively non expansive soils shall be compacted at a moisture content varying no more than 2% below or above optimum moisture content. It is recommended that the moisture content of the on site cohesive soils at the time of compaction be brought to between 2% and 4% over optimum moisture content.

Material for Fill

47. The on site soils, less debris or organic materials within any existing fill soils, shall be used in the required fills. Any on site clay soils shall not be used as backfill behind any walls below grade. All required imported fill shall consist of relatively non expansive soils. The Expansion Index of the selected relatively non-expansive material shall be less than 35. Any import material shall contain sufficient fines (binder material) so as to provide a compacted fill that will be relatively impermeable and will be stable in shallow trenches.

Field Observation

48. The reworking of the upper soils and the compaction of all required fill shall be observed and tested by a qualified geotechnical expert. This representative shall have at least the following duties:

- Observe the clearing and grubbing operations to assure that all unsuitable materials have been properly removed.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade, observe subgrade scarification, and delineate areas requiring overexcavation.
- Perform visual observation to evaluate the suitability of on site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Perform field density and compaction testing to determine the percentage of compaction achieved during fill placement.
- Observe and probe foundation bearing materials to confirm that suitable bearing materials are present at the design grades.

LEVEL OF SIGNIFICANCE AFTER MITIGATION

After incorporation of the mitigation measures listed above, impacts related to geology and soils would be less than significant.