Appendix B
Scoping Meeting

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CALIFORNIA ENVIRONMENTAL QUALITY ACT (CEQA)
NOTICE OF PREPARATION AND
PUBLIC SCOPING MEETING

The City of Diamond Bar will be the Lead Agency and will prepare an environmental impact report (EIR) for the proposed project identified below and will hold a scoping meeting for the stakeholders and the public. We need to know your views as to the scope and content of the upcoming EIR.

Project Title: Diamond Bar Boulevard and Brea Canyon Road (Site D) Specific Plan EIR

Project Location: Southeast corner of Brea Canyon Road and Diamond Bar Boulevard, City of Diamond Bar, California, Los Angeles County Assessor's Parcel Nos. 8714002900, 8714002903, and 8714045001.

Project Description: The proposed project consists of the adoption of a General Plan Amendment No. 2007-01 to change the land use designations from Public Facility and Commercial to Specific Plan with a corresponding Specific Plan No. 2007-01 to change the zoning districts from Low Medium Residential (R-1 7,500), Low Density Residential (R-1 10,000) and Community Commercial (C-2) to Specific Plan (SP) for an approximately 30-acre site owned by the Walnut Valley Unified School District, the City of Diamond Bar, and the Los Angeles County Flood Control District. Also proposed is the approval of a tentative subdivision map establishing separate residential and commercial parcels, an internal circulation system and common open space areas. The proposed project would allow the construction, habitation and occupancy of 202 dwelling units and 154,000 gross leaseable square feet of commercial use.

Project Applicant: Walnut Valley Unified School District and City of Diamond Bar

Lead: Agency City of Diamond Bar, Community Development Department
21825 Copley Drive, Diamond Bar, CA 91765

Contact: Person Nancy Fong, AICP, Community Development Director - (909) 839-7030

Address of where copy of Notice of Preparation and Initial Study is available: A complete copy of the Initial Study and Notice of Preparation prepared for the proposed project may be reviewed at City of Diamond Bar Community Development Department, 21825 Copley Dr., Diamond Bar, CA 91765; between the hours of 7:30 a.m. and 5:30 p.m. Monday through Thursday and 7:30 a.m. to 4:30 p.m. on Friday. In addition, the Initial Study and Notice of Preparation may be viewed at the City's website at http://www.ci.diamond-bar.ca.us and go to the Community Development Services page.

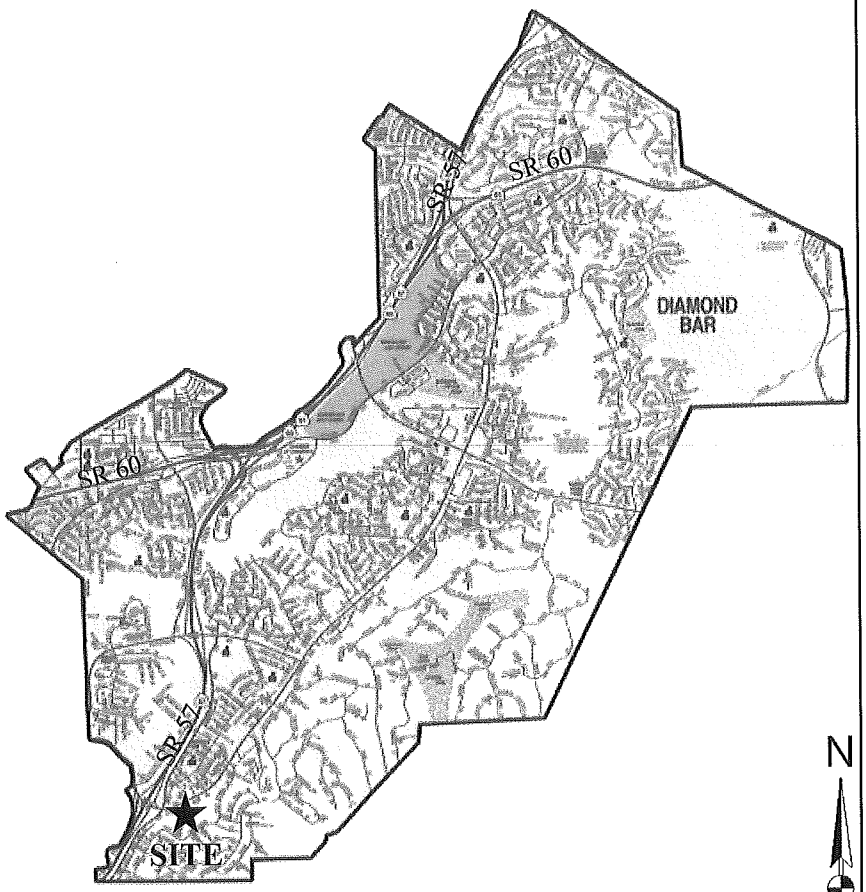
Scoping Meeting: The City of Diamond Bar will hold the following scoping meeting for stakeholders and the public:

- Thursday, February 21, 2007, 6:30 p.m. to 8:30 p.m. at South Coast AirQuality Management District, 21825 Copley Drive, Room CC-6, Diamond Bar, California 91765

Send Responses to: Nancy Fong, AICP, Community Development Director, City of Diamond Bar, 21825 Copley Drive, Diamond Bar, CA 91765-4178. Email: nancy.fong@ci.diamond-bar.ca.us Telephone: (909) 839-7030. Fax: (909) 861-3117. Due to the time limits mandated by State law, your response must be sent at the earliest possible date but ***not later than Wednesday, March 5, 2008, at 5:00 p.m. (30-day review period)*** Please send your response to the City of Diamond Bar at the address shown above. We will need the name for a contact person in your **agency**.

CITY OF DIAMOND BAR
INCORPORATED 1989

Diamond Bar Blvd / Brea Canyon Rd (Site D) Specific Plan Project Area



Vicinity Map

Project Map

City of Diamond Bar

Scoping Meeting - Site D

February 21, 2008

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1. What kind of buffer will be provided between existing residential and the commercial site?
2. How large of a setback will be between existing homes and the commercial site?
3. What sort of landscaping will be provided between existing homes and commercial site and who will be responsible for maintenance?
4. Where will the delivery trucks access the commercial site and will noise be a factor?
5. Will proposed residential block existing residential views?
6. Question regarding who is taking names and notes for the Scoping meeting and is the meeting being recorded?
7. When was or when will the traffic studies take place?
8. How large is the setback from Diamond Bar Boulevard to the residential site?
9. Would like to see a park or a large landscaped area at the corner of Diamond Bar Boulevard and Brea Canyon as it is the entrance to the City.
10. Would rather have the site be planned as all residential - there are enough strip centers in City already.
11. Diamond Bar Boulevard cannot handle anymore traffic.
12. Commercial site is too large.
13. Lower the density of the residential and reduce commercial FAR.
14. Would rather have park in place of commercial site.
15. Has the property been re-zoned yet?
16. What are the performance standards and setbacks?
17. Brea Canyon is currently a parking lot - what will happen with the extra traffic?
18. Is there an underground stream on the property and how would that affect development?
19. From Cold Springs and Ambushers streets, what will existing homes see?
20. Would like to see senior citizen housing on site as it is needed in the City.
21. Who is included on the list for notification?
22. The project will create traffic issues, would like to see less homes, lower densities and more parks
23. Are there other WVUSD properties on Brea Canyon?

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Appendix C

Preliminary Geotechnical Investigation

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KFM GEOSCIENCE

Geotechnical Engineering ● Engineering Geology

Preliminary Geotechnical Engineering Report

Site D – Mass Grading
Walnut Valley Unified School District
Diamond Bar, California

Prepared for:

TRG Land Inc.
898 Production Place
Newport Beach, California 92663

Prepared by:

KFM GeoScience
1360 Valley Vista Drive
Diamond Bar, California 91765

April 24, 2009
(Supersedes January 15, 2008 Report)
Project No. TRG 07-02E

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KFM
GEOSCIENCE

Project No. TRG 07-02E
April 24, 2009
(supersedes January 15, 2008 report)

Mr. Mark Rogers
TRG Land Inc.
898 Production Place
Newport Beach, California, 92663

Subject: **Preliminary Geotechnical Engineering Report**
Site D – Mass Grading
Walnut Valley Unified School District
Diamond Bar, California

Dear Mr. Rogers:

KFM GeoScience (KFMg) is pleased to submit the results of our geotechnical investigation at the subject site. The purpose of this study was to evaluate the surface and subsurface conditions and provide recommendations for the design and construction of the mass graded pad for the proposed mixed-use development. The results of the geotechnical field explorations and laboratory tests are presented herein.

The recommendations provided within are based on our current understanding of the proposed project and the 60-scale grading plans prepared by Penco Engineering, Inc., dated November 14, 2007 showing 3 large, mass graded pads. Once the project building layout is developed and the design is complete, KFMg should review the plans and specifications to evaluate if these preliminary geotechnical design recommendations remain appropriate for the proposed construction and have been incorporated as intended.

We appreciate the opportunity to provide our professional services on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
KFM GeoScience

Edward Sabins, C.E.G.
Senior Engineering Geologist

CERTIFIED ENGINEERING GEOLOGIST
EDWARD H. SABINS
No. EG 1571
Exp. 8/31/09
STATE OF CALIFORNIA

Peter Skopek, Ph.D., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
PETER SKOPEK
GE 2635
EXP. 06/30/09
GEOTECHNICAL
STATE OF CALIFORNIA

Filename: WVUSD Site D RPT 4-24-09 (supersedes 3-1-08)

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Appendix A – Logs of Exploratory Investigations
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1. INTRODUCTION

Presented herein are the results of KFM GeoScience's (KFMg) preliminary geotechnical engineering investigation for the proposed mass graded pads to accommodate a future mixed-use commercial and residential development of an approximately 30-acre undeveloped parcel located southeast of the intersection of Brea Canyon Road and Diamond Bar Boulevard in the City of Diamond Bar, California. The location of the subject project is shown on Figure 1. The site plan showing the proposed layout and grading for the 3 large pads and intervening slopes and walls is used as the base map for the Site Geologic Map, presented as Figure 2.

The purpose of this study was to evaluate the subsurface conditions at the site and to provide recommendations for the proposed site grading and generalized preliminary recommendations for the future commercial and residential construction. This report summarizes the data collected and presents the findings, conclusions, and recommendations. This study was performed in accordance with KFMg's proposal dated June 15, 2007.

This report is a stand-alone document and supersedes any previous reports and recommendations provided for this site.

2. SCOPE OF SERVICES

KFMg's scope of services for this project consisted of the following tasks:

- Review of readily available background data, including in-house geotechnical data, geotechnical literature, geologic maps, aerial photographs, and seismic hazard maps relevant to the subject site.

- Review of previous geotechnical investigations of the site obtained from the City of Diamond Bar.

- A site reconnaissance to observe the general surficial site conditions and to select test pit and boring locations.

- A subsurface field investigation performed in 2 phases:

    - An initial investigation consisting of backhoe excavating, logging, and sampling of 7 test pits to depths of up to approximately 15 feet below the existing grade, along with brushing and grading several existing roads for drill rig access.
    - A bucket auger drilling program consisting of advancing 4 borings.

- Collection of soil samples from the borings for transport to a geotechnical laboratory for further visual classification and testing.

- Laboratory testing on selected soil samples to evaluate geotechnical engineering properties of the on-site soils and bedrock materials.

- Preparation of a geologic map and 4 representative geologic cross-sections through proposed graded slopes summarizing the interpreted geologic conditions.

- Engineering evaluation of the geotechnical data collected to develop geotechnical recommendations for the design and construction of the proposed site development, including the following items:

  - Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
  - Evaluation of potential geologic hazards, including landsliding and seismic settlement potential and recommendations for appropriate mitigation measures.
  - Evaluation of general groundwater conditions and potential impact on the proposed design and construction.
  - Preparation of geologic cross-sections for slope stability analyses of key slopes.
  - Performance of static and pseudostatic slope stability analyses for existing and design conditions.
  - Evaluation of global stability of proposed mid-slope Loffel walls.
  - Provision of general evaluation of project feasibility.
  - Determination of seismic design parameters in general accordance with Chapter 16 of the 2007 California Building Code.
  - Provision of recommendations for slope design and site grading.
  - Provision of geotechnical recommendations for design of building and appurtenance foundations.
  - Provision of drainage and subdrainage recommendations.
  - Evaluation of suitability of on-site soils for backfill.
  - Evaluation of the corrosion potential of the on-site materials.
  - Preliminary recommendations for pavement design.

- Preparation of this report, including reference maps and graphics, summarizing the data collected and presenting the findings, conclusions, and geotechnical recommendations for the design and grading of the proposed project.

3. SITE BACKGROUND REVIEW

3.1. Records Review – City of Diamond Bar

KFMg obtained 2 soils and geologic reports pertaining to adjacent Tract 29053 and Tract 32974 prepared by S.E. Medall & Associates (Medall) dated October 4, 1976 and June 6, 1977 from the City of Diamond Bar Public Works. The obtained reports are fully referenced in Section 13 of this report. Tracts 29053 and 32974 are located adjacent to southeastern sloping portions of Site D. No pertinent geotechnical reports for the adjacent development within Tract 34160 located to the southwest of Site D were obtained. This is apparently due to misplacement of the files

following the transfer of records from Los Angeles County to the City of Diamond Bar, when the City was incorporated in 1989.

The grading of Tract 32974 was reported to have taken place between February 14, 1977 and May 23, 1977. According to the Medall reports, Puente Formation bedrock consisting of tan to dark gray shales with interbedded tan and gray sandstone was exposed in the backcuts. Bedding in the excavations was measured dipping 9 to 25 degrees toward the northwest (Medall, 1976).

Based on Medall (1977), several large buttresses and keyways ranging from 15 to 190 feet wide were constructed to support unfavorably oriented bedding within Tract 29053 and Tract 32974. A large keyway, 60 to 190 feet wide by 3 feet deep, was constructed adjacent to the southeast property line of Site D. The reported maximum depth of fill place in the buttress fill slope above the keyway is approximately 85 feet. The reported limits of these adjacent fill keys are shown on the Cross-sections in Figure 4. The Medall reports also indicate that subdrains were placed in the backcut of all keyways and within canyon fills.

3.2. Aerial Photo Review

KFMg's project geologist conducted a review of the historic stereographic aerial photographs covering the period from January 1953 through July 1995, in order to look for evidence of past slope instability and to evaluate if any major grading or man-made activities have occurred at the subject site. Prior to 1965, there is little development in the immediate vicinity of Site D. In the 1953 set of photos it appears that the southwest portion of the site may have been plowed for agriculture purposes. By May 1965, the cut slope for construction of Diamond Bar Boulevard has been excavated along the north side of the site and the residential development to the southwest (Tract 34160) is under construction. Prior to May 1965, Brea Canyon Wash was realigned into a concrete-lined channel. The blueline stream course that traverses the site appears to have been realigned to the northeast in order to avoid the new housing development (Tract 34160). In addition, a portion of a ridge on the northeast portion of the site appears to have been leveled by grading operations. An access road onto Site D from Diamond Bar Boulevard has been graded prior to 1965. No other significant changes are noted until October 1975 when several new bulldozer roads appear on the site and brush clearing is evident.

The residential development south and east of the site is being graded by March 1978 at which time fill slopes estimated to be 75 to 125 feet high are placed in natural ravines that formerly emptied onto Site D. Surficial erosion of these new slopes is visible on the 1978 photos, with a large area of eroded material having accumulated in the central portions of the Site D, shown as artificial fill (af) over alluvium (Qal) on Figure 2, Site Geologic Map. The access road from the south from Posada Drive has been graded by January 1981. The natural slopes east of Posada Drive and below the residential development appear to have been disced to reduce brush fire hazards in both 1990 and July 1995.

In most of the reviewed sets of photos, a subtle bowl-shaped depression is visible on the southwest trending slope above Brea Canyon wash that may represent an eroded old landslide scarp. The interpreted limits of this possible old landslide are shown in Figure 2. No other

geomorphic evidence of past slope instability on the slopes on or adjacent to the site was observed on the air photos or during the field investigation.

3.3. Previous Investigations

A previous preliminary geotechnical investigation was conducted by Bing Yen & Associate, Inc. (BYA) in 2002. Test pit logs from the BYA investigation are presented in Appendix B. ATC Associates conducted a Phase I Environmental Site Assessment (ESA) at Site D in March 2002. The ESA report generated from the site assessment and document review was dated April 17, 2002 and concluded that there is a low likelihood for significant environmental impairment to the site due to current or past land usage or from surrounding properties. ATC developed this conclusion based on a reconnaissance site visit, interviews with persons familiar with the site, review of available environmental databases and related agency file information for the site and surrounding properties, review of aerial photos, and review of geologic and hydrogeologic information.

In 2004, Vicente Geotechnical Services (VGS) evaluated the geotechnical constraints for Site D, based on the previous studies. The VGS evaluation contained preliminary slope stability analyses of several of the natural slopes at the site.

4. PROPOSED DEVELOPMENT AND SITE DESCRIPTION

The site is characterized as a relatively flat plateau with natural and graded slopes facing north and southeast. The site is bounded by the channelized Brea Canyon Wash along the northwest portion of the property, descending cut slopes to Diamond Bar Boulevard to the north, and residential developments in the south and east. Pasado Drive, at the southwest extremity of the site, provides a point of entry to the site.

The site ranges in elevation from approximately 815 feet along the southeast boundary to about 670 feet near the western corner of the site. The natural slopes below the residential development to the south and adjacent to Brea Canyon Wash are inclined from approximately 2.5:1 to 4.5:1 (horizontal:vertical). During the construction of Diamond Bar Boulevard portions of the northern boundary of the site adjacent to the street were cut to approximately 1.5(H):1(V). Also, an ascending 15- to 20-foot high cut slope inclined at approximately 1.5(H):1(V) is present immediately adjacent to the housing development along the southern boundary of the site.

The majority of the site is covered with annual grasses, brush, and small trees. However, a few areas of dense brush and trees are located in the central and northern portions of the site. The site generally drains toward the west to southwest toward Brea Canyon Wash. Currently, the site receives rainfall and irrigation runoff from the terrace drains present on the adjacent man-made slopes of the residential development to the southeast.

Based on the provided 60-scale civil engineering plans dated November 14, 2007 prepared by Penco Engineering Inc. (Penco), the proposed development will consist of 3 mass-graded pads including one proposed commercial pad with an area of about 10 acres and 2 proposed

residential pads ranging in area from about 4 to 6 acres, as shown on Figure 2. The commercial pad is located in the northwest portion of the site adjacent to both Brea Canyon Road and Diamond Bar Boulevard with site access provided from Brea Canyon Road. The two residential pads are located in the east and southeast portions of the site downslope from the existing residential developments. Access to the residential pads will be provided by a backbone road extending through the site from the intersection of Cherrydale Drive and Diamond Bar Boulevard. A secondary right turn in and right turn out access to the commercial pad is planned from Brea Canyon Road at the southwest corner of the site.

The pads will be developed by balanced cut and fill grading with cuts ranging from less than approximately 5 feet to about 40 feet high and fills ranging from approximately 5 to 40 feet thick. The proposed fill slopes will range in height from a few feet to approximately 60 feet downslope from the upper residential pad to Diamond Bar Boulevard. All fill slopes are planned for gradients of 2(H):1(V) or shallower. Variable height Loffel walls, ranging from several feet to about 23 feet high, are proposed near the mid-slope of the 2(H):1(V) fill slopes between each of the pads.

5. FIELD EXPLORATIONS AND LABORATORY TESTING

5.1. Field Explorations

The subsurface exploration was conducted in 2 phases and consisted initially of backhoe test pits followed by bucket auger exploration mobilizations conducted between September 24 and September 27, 2007. During the initial investigation, KFMg supervised and logged 7 test pits. Test pits TP-3 through TP-6 were located to further evaluate the extent and depth of undocumented fill over alluvial deposits identified in the central portion of the site, as shown on Figure 2. Test pits TP-1, TP-2, and TP-7 were excavated to observe the bedrock conditions near the proposed cut/fill transitions and proposed toe of graded slopes. The backhoe was also used to clear and recondition several old access roads for subsequent bucket auger drill rig access. The test pits were observed by a KFMg engineering geologist, who logged the exposed test pit walls and obtained soil samples. Bulk samples were collected in plastic bags at major changes in materials. During the test pit logging, the description of the material type, color, moisture, grain size, density/consistency, and other pertinent geologic characteristics were recorded. Logs of Test Pits TP-1 through TP-7 are presented in Appendix A. Test pit logs generated by BYA (2002) and reviewed for this report are presented in Appendix B.

The bucket-auger drilling was performed from September 25 through September 27, 2007, and included advancing 4 borings with either an Earthdrill rig or an EZ Bore 120 rig. Bucket auger borings B-1 through B-4 were each located on or near the north-facing natural slopes to investigate the stratigraphy and geologic structure of the area. These borings ranged in depth from approximately 34 to 50 feet. All of these borings were downhole logged and observed by a KFMg engineering geologist, who logged the drill cuttings and collected soil samples. Samples were obtained in each hole by driving a thick-walled ring sampler using the weight of the Kelly Bar (various weights as recorded on the logs) falling 12 inches. Drive samples were generally taken at 10 foot depth intervals. For each sample, blowcounts were recorded for each 6-inch

interval of the 12-inch drive. In addition to the drive samples, bulk samples of prevailing materials were collected in plastic bags. Also, samples of observed potentially critical thin, soft clay layers were collected during downhole logging. Following the drilling completion a KFMg geologist entered each boring to log the exposed geology in detail. During the downhole logging, we recorded the description of the material type, color, moisture, grain size, density/consistency, and other pertinent geologic characteristics. The logs of the bucket auger borings are presented in Appendix A.

5.2. Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings in order to aid in the soil classification and to evaluate the engineering properties of the foundation soils. The following tests were performed:

- In-situ moisture content and dry density;
- Grain size analysis;
- Atterberg Limits;
- Shear strength of in-situ and remolded samples;
- Expansion Index;
- Water soluble sulphates content; and
- Maximum density.

Testing was performed in general accordance with applicable ASTM standards and California Test Methods. The moisture content and density data are presented on the boring logs in Appendix A. The remaining laboratory test results are presented in Appendix C. Details of the laboratory testing program are also included in Appendix C.

6. SITE AND SUBSURFACE CONDITIONS

The site is located in the Puente Hills within the northwest portion of the Peninsular Ranges Physiographic Province. The Peninsular Ranges Physiographic Province extends from the San Gabriel Mountains south to the tip of Baja California (Norris and Webb, 1990). The Peninsular Range Physiographic Province is characterized by northwest-trending mountains and hills separated by alluvial filled valleys. Regional faults within Peninsular Ranges Province, such as the nearby Whittier Fault, are also dominantly oriented towards the northwest. Based on geologic mapping by the USGS (1964) and Dibblee (2001), used as the basemap for Figure 3, the site is underlain by marine sedimentary bedrock consisting of sandstone and siltstone of the Puente Formation. The bedrock units have been tilted and locally folded as the Puente Hills were being uplifted and deformed. The USGS (1964) and Dibblee (2001) mapping indicates that the bedding in the Puente Formation exposed in the area generally dips shallowly towards the northwest producing apparent out-of-slope dips along the southeastern slopes on the site. No landslides are mapped at the site by USGS (1964), Tan (1988), or Dibblee (2001). Alluvial deposits are also mapped in Brea Canyon wash and underlying the blueline stream as shown on Figure 3.

Based on the results of our field exploration, Site D is predominantly underlain by Puente Formation bedrock which is locally overlain by alluvial deposits, relatively thin colluvium/topsoil, and an area of slope wash deposited from erosion of the adjacent fill slopes. The Puente Formation bedrock was encountered in all of the exploratory bucket auger borings and in 3 of the 7 test pits excavated. Geologic data collected at the site are summarized on the Figure 2, Site Geologic Map. Interpretation of the available geologic data shown on Figure 2 and presented in the boring logs by KFMg, presented in Appendix A, were used to develop models of the subsurface geologic conditions underlying Site D. These interpreted geologic models are presented as Figure 4, Cross-sections A-A' through D-D'.

Generalized descriptions of the encountered units are provided in the sections below. More complete descriptions of the soil conditions encountered during the field exploration are presented on the test pit and boring logs in Appendix A.

6.1. Undocumented Fill (af)

Undocumented fill located on site is associated with surficial erosion of the adjacent rough graded tract during heavy rain in early 1978 based on the aerial photo review. Undocumented fill encountered in KFMg TP-3, TP-4, and TP-5 and in BYA TP-3 is approximately 4 to 6 feet thick and is isolated within the central portions of the site. The artificial fill consists predominantly of silty to sandy clays with abundant siltstone bedrock clasts from the La Vida Member of the Puente Formation. The silty and sandy clay materials were observed to be yellow brown to dark brown, dry to damp, and soft to stiff. Drive samples in test pit TP-5 were advanced with moderate effort in the undocumented fill, with standard penetration blowcounts of about 11 blows per foot. In addition to the mapped undocumented fill, randomly located spoil piles of unknown origin are present in the central portion of the site.

6.2. Colluvium/Topsoil (Qcol)

Colluvium within the subject site consists of a relatively thin mantle of soil above the on-site bedrock materials and is generally located along the hillsides within the site. Colluvial deposits encountered in test pits TP-1 and TP-2 were observed to be approximately 1 to 3-feet in thickness. Where observed these materials consisted of yellow to dark brown silty to sandy clays that were generally soft to firm, porous and damp to moist. Bedrock fragments were commonly encountered in the colluvial soils. As encountered in BYA test pits TP-3 and TP-6, the colluvium is moderately to highly expansive with reported Expansion Indices ranging from 84 to 93, respectively.

6.3. Alluvium (Qal)

Alluvium within the subject site is generally located in the center portion of the site and adjacent to the Brea Canyon Channel. The alluvium generally consists of dark brown to yellowish orange brown silty to sandy clays. Alluvial materials were observed in test pits TP-3 through TP-6 and were observed to be generally fine-grained, moist, firm to stiff and porous. Alluvial soils were observed to total depth of all the above-mentioned test pits which extended 10 to 12 feet below

the undocumented fill. The total thickness of the alluvial deposits was not determined during this investigation.

6.4. Puente Formation – La Vida Member (Tplv)

Sedimentary bedrock of the Puente Formation, La Vida Member (Tplv) underlies the entire subject site. It generally consists of siltstone and fine-grained sandstone with medium-grained sandstone interbeds as observed during our investigation. Bedrock, as observed in the excavations, is highly weathered near the surface and at contact with other units, with the degree of weathering decreasing with depth. Slightly to moderately weathered bedrock is generally orange to yellow brown and moderately fractured. Unweathered bedrock materials were generally olive gray and hard to extremely hard at depth. Below depths of 30 to 35 feet the drilling became generally difficult.

The stratigraphy of the siltstone and sandstone unit consists of predominantly massive sandstone, silty fine sandstone beds interlayered with siltstone beds ranging from 1 to 3 feet thick as described in detail on the logs of borings B-1 through B-4, presented in Appendix A. The interbedded relationship between the sandstone and siltstone layers often exhibit grading or fining-upwards sequence.

Also observed and sampled were occasional weak or soft claystone layers consisting of olive to grayish brown clay as encountered in borings BA-1 and BA-2. No significant shearing of the claystone or siltstone beds was observed in any of the borings. The weak and soft claystone layers are depicted on the Cross-section A-A' through D-D', Figure 4.

6.5. Geologic Structure

The geologic structure of the site is characterized by a generally western dipping homocline dipping away from the northwest flank of the Puente Hills. Bedding in the Puente Formation as observed in the borings and test pits generally striking toward the northeast and typically dips 4 to 19 degrees towards the northwest, producing apparent shallow out-of-slope dips underlying most of the slopes. Dips towards the southwest were less common. No significant faults have been recognized at the subject site.

6.6. Groundwater

Regional groundwater was not encountered within the bedrock units during this investigation. A seep or apparent perched groundwater condition was encountered in bucket auger boring B-1 at a depth of 37 feet below ground surface elevation. This perched groundwater condition was observed to be resting on a cemented bed. No other perched groundwater conditions were encounter within the subsurface soil or bedrock units during this investigation. No seeps were observed on-site during the field investigation. Minor subsurface seepage may occur on site, particularly during or after periods of heavy rainfall. Also, the depth to groundwater may fluctuate depending on the rainfall and possible groundwater recharge or pumping activities within the subject site vicinity.

7. ENGINEERING SEISMOLOGY

7.1. General Seismic Setting

The Southern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are generally capable of generating ground shaking of engineering significance to the proposed construction. The project area is located in the general proximity of several active and potentially active faults. Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years).

Review of pertinent geologic maps of the area, including CDMG (1981) and Jennings (1994) indicate that no known major active or potentially active faults are mapped within 2.2 miles of the site. The closest mapped active faults to the site are the Whittier Fault, located about 2.2 miles southwest, the Elsinore Fault, located approximately 16 miles southeast of the site, the Cucamonga Fault located approximately 18 miles to the northeast, the San Andreas Fault located about 29 miles northeast, and the south-central segment of the San Jacinto Fault located 32 miles east of the site.

Notable damaging earthquakes in the project region include the 1994 magnitude M6.7 Northridge earthquake and the 1987 magnitude M5.9 Whittier Narrows earthquake, both attributed to blind thrust faults (low angle faults that are not expressed at the ground surface); the 1971 magnitude M6.4 San Fernando earthquake which occurred on the San Fernando Fault (the easternmost fault of the Sierra Madre system); the 1933 magnitude M6.3 Long Beach earthquake on the Newport Inglewood fault; and the 1857 magnitude M7.9 Fort Tejon earthquake on the south central segment of the San Andreas Fault.

7.2. Seismic Hazards

The engineering seismology study for the subject site included reviewing local and regional faulting maps and the review of historical earthquake data. Specifically, the following engineering seismology issues were addressed:

Surface Fault Rupture: Official Maps of Earthquake Fault Zones were reviewed to evaluate the location of the project site relative to active fault zones. Earthquake Fault Zones (known as Special Studies Zones prior to 1994) have been established in accordance with the Alquist-Priolo Special Studies Zones Act enacted in 1972. The Act directs the State Geologist to delineate the regulatory zones that encompass surface traces of active faults that have a potential for future surface fault rupture. The purpose of the Alquist-Priolo Act is to regulate development near active faults in order to mitigate the hazard of surface fault rupture.

The site is not located within a designated Alquist-Priolo Earthquake Fault Zone for fault surface rupture hazard. The surface traces of any active or potentially active faults are not known to pass directly through or project towards the site. Therefore, the potential for surface rupture due to

faulting occurring beneath the site during the design life of the proposed development is considered low.

Seismic Hazard Zones: Maps of seismic hazard zones are issued by the California Geological Survey (formerly California Department of Conservation, Division of Mines and Geology) in accordance with the Seismic Hazards Mapping Act enacted in April 1997. The intent of the Seismic Hazards Mapping Act is to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in developing compliance requirements to protect the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

Based on the Seismic Hazard Zone map for the Yorba Linda 7.5-minute Quadrangle (2005), the majority of the proposed development is underlain by dense bedrock and is not located within an area identified by the State of California as subject to the hazard of liquefaction. However, the west portion of the site adjacent to Brea Canyon wash and east of Brea Canyon Road is identified as being potentially liquefiable due to the presence of Quaternary alluvium (Qal) and potentially shallow groundwater. Several slope areas potentially susceptible to earthquake-induced landslide are identified on the map. The slope stability of he existing and proposed graded slopes within the project site is analyzed and discussed in detail in Section 8 of this report.

An evaluation of the liquefaction potential and dynamic settlement potential is discussed in following section of this report.

7.3. Liquefaction and Dynamic Settlement Potential

Liquefaction is a phenomenon wherein a saturated or near-saturated mass of soil looses a large portion of its shear resistance when subject to monotonic, cyclic, or shock loading, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear strength. Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays is not adversely affected by ground shaking.

Dynamic settlement occurs due to soil densification during cyclic or shock loading, typically due to earthquake shaking, and can occur in dry sands or as a consequence of liquefaction in saturated sands.

Due to the presence of dense bedrock and the absence of groundwater within the depth of liquefaction significance, the potential for liquefaction and its adverse effects, including dynamic settlement, impacting most of the site is considered negligible. There is potential for liquefaction in the area of Brea Canyon wash. However, subsurface investigation in that area was not part of the scope of this study. The subsurface condition of Brea Canyon wash should be investigated when the plans to enclose and cover the flood control channel have been developed.

8. SLOPE STABILITY EVALUATION

Slope stability of representative cross-sections through the main manufactured slopes of the proposed development were performed to investigate the stability of the existing and proposed grades and to establish the extent of grading mitigation. The following sections provide the rationale, basis, and results of the analyses.

8.1. Shear Strength of On-Site Materials

Strength parameters selected for the slope stability analyses considered the results of laboratory testing performed during this investigation as well as observed condition of material types along the interpreted potential failure surfaces. The determination of the shear strength parameters along the weak clay layers encountered in the borings was based on residual shear strength obtained from direct shear tests of remolded weak clay layer materials. Additional direct shear tests were performed on undisturbed bedrock materials and anticipated remolded fill materials as presented in Appendix C.

In general, the selection of the appropriate shear strength parameters was based on the following premises:

- For static analyses, shear strengths near the lower bound limits of the appropriate collected laboratory data were considered.

- To verify the selected residual shear strength parameters, Atterberg Limits of pertinent weak clay materials were compared to published correlations (e.g., Dewoolkar & Huzjak, 2005; Mesri & Shahien, 2003; Stark & Eid, 1994; Brandon et al, 1991; Mesri & Cepeda-Diaz, 1986; Cancelli, 1977; Voight, 1973). Based on the range of Liquid Limits and Plasticity Indices between approximately 64 and 80 percent and 38 and 51 percent, respectively, of the weakened clay materials, the associated residual friction angle is correlated between 9.5 and 13.5 degrees with average value around 12 degrees.

- For pseudo-static analyses, shear strength parameters were increased by 20 percent which resulted in increase of shear strength parameters closer to the average of the appropriate collected data (Ishihara and Nagao, 1983).

The following table summarizes the shear strength parameters utilized in the analyses and includes description of the basis for the shear strength parameter determination.

Table 1
Summary of Shear Strength Parameters used in Slope Stability Analyses

Material	Static analyses	Basis of shear strength
Weak Clay Layers	c = 150 psf $\phi = 13^o$	Based on residual shear strength of remolded hand samples and published correlations
Puente Formation - along bedding strength	c = 250 psf $\phi = 20^o$	Based on strength between fully softened and residual shear strength of high plastic (LL > 60) Puente Formation materials
- cross-bedding strength	c = 400 psf $\phi = 30^o$	Based on strength between peak and fully softened strength of low plastic (LL ≈ 40) Puente Formation materials
Alluvium	c = 200 psf $\phi = 30^o$	Based on typical shear strength of alluvium (Qyf) reported in the Yorba Linda Quadrangle (CDMG, 1998b)
New Fill	c = 100 psf $\phi = 30^o$	Based on strength between fully softened and ultimate shear strength of remolded fill
Off-Site Fill	c = 750 psf $\phi = 26^o$	Based on Medall (1976) for adjacent Tract 32974

8.2. Slope Stability Analyses

The Factors of Safety of the critical slopes were evaluated using the computer program SLOPE/W (Geo-Slope Office, 2007) using Spencer's limit equilibrium method. Both static and pseudo-static analyses were performed on the 4 cross-sections representative of the site conditions. On cross-sections A-A' and B-B' weak clay layers have been conservatively extrapolated in the stratigraphy based on the downhole logging of borings. In general, stability of the existing slopes was analyzed on failure planes that included the identified weak clay layers and potential failure planes along weaker bedding through the Puente Formation bedrock. As appropriate, if entry or exit configurations were unknown, a search for the critical configuration was undertaken.

Static Slope Stability Acceptance: In the static analyses the calculated Factor of Safety is required to be at least 1.5 for the slope to be considered to possess adequate static stability. For temporary conditions during construction Factor of Safety of 1.25 is required.

Seismic Slope Stability Acceptance: A screening evaluation of the seismic slope stability of the adversely oriented bedding was performed in accordance with the Section 11.2 of the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California" (Southern California Earthquake Center, 2002). Based on this approach, the slope is required to perform adequately under the effects of a Design Basis Earthquake (DBE). The DBE is defined as an earthquake event having a 10% probability of exceedance during a 50-year time period.

In order to evaluate the performance of the slope under DBE event, first a screening analysis is performed. The screening analysis consists of pseudo-static analysis utilizing an earthquake coefficient based on seismic response specific to the site and on the acceptable magnitude of seismically induced slope movements. Based on the recommended procedures, it is required that the pseudo-static Factor of Safety is greater than 1 in order for the slope to pass the screening analysis and to be considered seismically stable. If the Factor of Safety is less than 1, then deformation analysis is required.

Based on the Seismic Hazard Zone Report 010 – Seismic Hazard Zone Report For the Yorba Linda 7.5-minute Quadrangle, Orange County, California (CDMG, 1998b, revised 2005) the DBE Peak Ground Acceleration (PGA, also called Maximum Horizontal Acceleration, MHA) for soft rock conditions is 0.46g. Based on the de-aggregation analysis as presented in the referenced Seismic Hazard Zone Report 010, the predominant earthquake magnitude (or mode magnitude, M_w) and the associated modal distance are M_w6.8 and 2 km, respectively. For the screening analysis, based on these de-aggregated parameters, a horizontal seismic coefficient (k_{eq}) of 0.239 was estimated for 5 cm threshold displacement.

The results of all critical slope stability analyses are summarized in the Table 2 below. The graphical outputs of the critical slope stability analyses are presented in Appendix D.

Table 2
Summary of Slope Stability Analyses

A-A'			
Condition	**Factor of Safety** Static / Pseudo-static (k_{eq} = 0.239)	**Analysis description**	Appendix D page
Upper slope (Slope A)			
Existing	1.78 / 0.79	- stability along upper weak clay layer	D-1a/D-1b
	1.70 / 0.80	- stability along lower weak clay layer	D-2a/D-2b
	2.43 / 1.15	- stability along bedding	D-3a/D-3b
Conclusion: the existing slopes are grossly stable under static conditions but are potentially susceptible to excessive deformations during DBE earthquake event.			
Current Design	1.43 / 0.74	- stability along upper weak clay layer	D-4a/D-4b
	1.63 / 0.81	- stability along lower weak clay layer	D-5a/D-5b
Conclusion: the currently proposed slopes are not adequately stable on the upper weak clay layer under static condition and overall are potentially susceptible to excessive deformations during DBE earthquake event; consequently, further mitigation is recommended.			
Recommended Remediation (shear key and buttress with select import material)	1.72 / 1.04	- stability along upper weak clay layer after construction of shear key and buttress using select import fill material (c=200 psf, ø = 36°)	D-6a/D-6b
	1.94 / 1.14	- stability along lower weak clay layer after construction of shear key and buttress using select import fill material (c=200 psf, ø = 36°)	D-7a/D-7b
Conclusion: the recommended remediated slopes are grossly stable under both static and pseudo-static condition.			
Temporary (during shear key and buttress construction)	1.30 (static)	- temporary stability along upper weak clay layer	D-8
	0.93 (static)	- temporary stability along lower weak clay layer INADEQUATE	D-9
	1.25 (static)	- for slot cuts 150 feet wide	
Conclusion: the temporary cuts are adequately stable provided the construction is performed in slot cut sequence A-C-B, with slot cuts 150 feet wide.			
Middle slope w/Loffel Wall			
Design	1.58 / 1.03	- stability of fill slope and Loffel wall	D-10a/D-10b
Conclusion: the middle slope supported by Loffel wall are adequately stable			

B-B'			
Condition	**Factor of Safety** Static / Pseudo-static (k_{eq} = 0.239)	**Analysis description**	Appendix D page
Upper slope (Slope B)			
Existing	1.42 / 0.68	- stability along weak clay layer	D-11a/D-11b
	2.05 / 1.04	- stability along bedding	D-12a/D-12b
Conclusion: the existing slopes are grossly stable under static conditions but are potentially susceptible to excessive deformations during DBE earthquake event.			
Current Design	1.50 / 0.77	- stability along weak clay layer	D-13a/D-13b
	1.97 / 1.07	- stability along weak bedding	D-14a/D-14b
Conclusion: the currently proposed slopes are not adequately stable along the weak clay layer under static condition and overall are potentially susceptible to excessive deformations during DBE earthquake event; consequently, further mitigation is recommended.			
Recommended Remediation (shear key and buttress with select import material)	1.85 / 1.00	- stability along weak clay bed after construction of 35 feet wide shear key and 18 feet high buttress using select import fill material	D-15a/D-15b
Conclusion: the recommended remediated slopes are grossly stable under both static and pseudo-static condition.			
Temporary	1.18 (static)	- temporary stability along weak clay layer INADEQUATE	D-16
	1.25 (static)	- for slot cuts 135 feet wide	
Conclusion: the temporary cuts are adequately stable provided the construction is performed in slot cut sequence A-C-B, with slot cuts 135 feet wide.			
Lower slope w/Loffel Wall			
Existing	2.18 / 1.21	- stability along weak clay layer	D-1a/D-17b
Recommended Remediation (stepped shear key and Loffel wall)	2.15 / 1.00	- stability is adequate for fill slope supported by stepped shear key intercepting the weak clay layer and a Loffel wall	D-18a/D-18b
Conclusion: the existing and remediated proposed slopes are adequately stable.			

C-C’			
Condition	**Factor of Safety** Static / Pseudo-static (k_{eq} = 0.239)	**Analysis description**	Appendix D page
Upper slope			
Existing	2.17 / 1.09	- stability along weak bedding	D-19a/D-19b
Current Design	1.95 / 1.05	- stability along weak bedding	D-20a/D-20b
Conclusion: the existing and currently proposed slopes are adequately stable.			
Lower slope			
Existing	2.17 / 0.988	- stability along weak bedding	D-21a/D-21b
Conclusion: the existing slopes are grossly stable under static conditions but are potentially susceptible to excessive deformations during DBE earthquake event.			
Recommended Remediation (stepped shear key and Loffel wall)	Design governed by the adjacent section on B-B’		
Conclusion: the remediated proposed slope is adequately stable.			

D-D'			
Condition	**Factor of Safety** Static / Pseudo-static (k_{eq} = 0.239)	**Analysis description**	Appendix D page
Lower slope along Diamond Bar Boulevard			
Existing	1.95 / 0.95	- stability along weak clay layer	D-22a/D-22b
	2.71 / 1.32	- stability along bedding	D-23a/D-23b
Conclusion: the existing slopes are grossly stable under static conditions but are potentially susceptible along the weak clay layer to excessive deformations during DBE earthquake event.			
Current Design	1.48 / 0.84	- stability of proposed sliver fill over natural	D-24a/D-24b
Conclusion: the currently proposed slopes are not adequately stable along the weak clay layer under static condition and overall are potentially susceptible to excessive deformations during DBE earthquake event; consequently, further mitigation is recommended.			
Recommended Remediation	1.64 / 1.09	- stability of graded slope incorporating 20-foot wide key at toe	D-25a/D-25b
Conclusion: the remediated proposed slope is adequately stable.			

The analysis of the results of the presented in the table above offers the following observations and conclusions:

- **Static Slope Stability**: The static Factors of Safety for the existing slope conditions at the site are generally close to or greater than 1.5. The lowest calculated Factor of Safety for the existing condition is 1.42 along a weak clay layer within Slope B. However, the proposed grading of the upper slopes at Slope A and Slope B to 2(H):1(V) generally creates a reduction in the static Factors of Safety below 1.5 along these slopes. Further, the additional loads imposed by the proposed Loffel walls at the toe of Section B-B' and C-C' result Factors of Safety below 1.5. Therefore, remediation measures including shear keys to intercept the critical mapped weak clay layers, buttress fills at the toe of Slopes A and B, and buttress fill shear strength improvements are required to achieve the code minimum static Factors of Safety for the site slopes. The remediation measures for each slope are described in Site Preparation and Earthwork, Section 9.3.

- **Seismic Slope Stability**: Using the site-specific seismic coefficient k_{eq} of 0.239, pseudo-static Factors of Safety of less than 1 were computed for many of the existing and proposed graded slopes at the site. The high seismic coefficient is due to the close proximity of the design seismic source, the Whittier Fault, to the site. The remediation measures described in section "Site Preparation and Earthwork" of this report, incorporate sufficient elements to achieve adequate pseudo-static stability with Factors of Safety greater than 1 and

consequently the site passed the screening analysis and the design sections are considered stable.

9. DESIGN RECOMMENDATIONS

9.1. General

Based on the results of the field explorations and engineering analyses, it is KFMg's opinion that the proposed mass grading construction is feasible from a geotechnical standpoint, provided that the recommendations contained in this report are incorporated into the design plans and implemented during construction.

The following considerations are reflected in the design recommendations:

- Two of the 3 proposed cut slopes along the southeastern margin of the site (labeled Slope A, Slope B, and Slope C on Figure 2) will expose Puente Formation bedrock exhibiting adversely oriented bedding and weak clay layers. As discussed in Section 8, Slopes A and B in the existing condition generally exhibit static Factors of Safety of about 1.5 but do not meet current code requirements for seismic stability. However, when graded to 2(H):1(V) the static Factor of Safety falls below 1.5 and shear keys with buttress fills are required to achieve the minimum Factor of Safety for static and seismic conditions. The recommendations for the key configurations, buttress fills, and for the associated subdrain system are described in the "Site Preparation and Earthwork" and "Subdrainage Provisions" sections of this report.

- The central portion of Site D (identified as af/Qal on Figure 2) is underlain by slope wash, undocumented fill, and soft alluvium to the depth explored of 16 feet. These materials are considered compressible and unsuitable for support of foundations or additional fills and should be removed to firm bedrock, grubbed of unsuitable materials, and be replaced with a properly compacted fill in accordance with the recommendations in the "Site Preparation And Earthwork" section of this report.

- The mid-slope segmental retaining walls, identified as "Loffel walls" on Figures 2 and 5, are generally shown founded above variable height, 2(H):1(V) inclination cut slopes and supporting variable height, 2(H):1(V) inclination fill slopes. Due to the presence of unfavorably oriented bedding within the bedrock, it is recommended that cut portions of the slopes be overexcavated to intercept the critical bedding, be keyed a minimum of 4 feet below the adjacent pad grades, and be replaced with engineered fill in accordance with the recommendations in the "Site Preparation And Earthwork" section of this report.

- Due to the presence of potentially medium to high expansive soils, as reported by BYA (2002) and Medall (1977) in the site vicinity, the proposed commercial and residential structures should be supported on structurally designed floor slab systems incorporating thickened edges and post-tensioned reinforcing ribs established in properly moisture-

conditioned and placed engineered fill. The pads will need to be maintained, re-certified every 6-months and immediately before installation of the building slabs for moisture content and compaction. In order to confirm the design of foundations and slabs, additional testing of Expansion Index and water-soluble Sulphates should be performed following completion of the rough graded pads. Preliminary design recommendations are provided in the "Foundations and Concrete Slabs-on-Grade" section of this report.

- The alluvium present underlying Brea Canyon Wash and the L.A. County flood control channel is identified by the California Geological Survey as having potential for liquefaction and liquefaction-induced settlement. Because of the presence of the flood control channel, which will be enclosed and covered, it is anticipated that the area overlying the alluvium will only be used for paved parking. Future geotechnical investigation in this area should address potential liquefaction and seismic-induced settlement.

The design recommendations presented below are based on KFMg's current understanding of the project. Once the project configuration is finalized and the design is complete, KFMg should review the plans and specifications to evaluate if the geotechnical design recommendations have been incorporated as intended.

9.2. California Building Code Seismic Design Parameters

Geotechnical parameters for design to resist seismic forces in accordance with the 2007 California Building Code, which, in turn, is based on the 2006 International Building Code, are presented in Table 3 below. The values below were obtained from the USGS Ground Motion Parameter Calculator (version 5.0.9) using latitude and longitude coordinates near the center of the site.

Table 3
2007 California Building Code Seismic Design Parameters

2007 CBC Seismic Design Parameter	Parameter
Site Class	D
Maximum Considered Earthquake Spectral Response Acceleration (for Site Class B):	
• Short Period, Ss (0.2 seconds) • 1 Second Period, S_1	197% of gravity 74% of gravity
Site Class Modification Coefficients (to obtain S_{DS} and S_{D1} for Site Class D)	
• F_a	1.0
• F_v	1.5

9.3. Site Preparation and Earthwork

Prior to any grading activities, the surface of areas to be graded should be cleared of any existing structures, terrace drains, vegetation, trash, and debris. Site grading is not anticipated to encounter unusual excavation difficulties and it is anticipated that conventional excavation and grading equipment can be used.

The following bullet points provide specific recommendations for remedial grading of key areas of the site.

- The densely vegetated area in the central portion of Site D (identified as af/Qal on Figures 2 and 5) is underlain by slope wash, undocumented fill, and soft alluvium to the explored depth of 16 feet. These materials are considered compressible and unsuitable for support of foundations or additional fills and should be overexcavated to firm bedrock, grubbed of unsuitable materials, and be replaced with a properly compacted fill following the recommendations in this section of the report. Following overexcavation of the unsuitable material, a subdrain should be installed in the low point of the excavated drainage course.

- The proposed graded Slopes A and B (Figures 2 and 5) will expose weak clay beds based on borings B-1 and B-2 and will require toe shear keys and buttress fills to achieve the required static and pseudo-static Factors of Safety. Based on the analyses, a toe key 35 feet wide and 18 feet deep below the adjacent level pad will be required for Slope A, as shown on Figure 5. Slope B will require a toe key 35 feet wide and about 12 feet deep below the adjacent level pad as shown on Figure 5. Because of the high site seismic coefficient, buttress fills extending about 10 feet above the current toe for Slope A and 18 feet above the current toe for Slope B will also be required. The materials used to construct the Slope A and B buttress fills will need to achieve higher strength (c = 200 psf, ø = 36°) than the general on-site fill tested during this investigation. The proposed key locations at the toe of Slopes A and B are approximately shown on Figure 5 – Remedial Grading Plan. However, the actual extent of the removals as well as the location of the critical weak clay beds should be determined during the grading activities by the Project Geologist. The slopes should be constructed with compacted fill benched into competent bedrock as schematically shown on Figure 6 – Fill Slope Grading and Benching Recommendations.

- The proposed 1(H):1(V) backcuts needed to construct the toe keys and buttress fills for Slopes A and B result in temporary stability conditions that fall below the Code required Factor of Safety of 1.25. To mitigate these conditions the toe keys will have to be excavated and backfilled following slot cut sequencing. For Slope A, the approximately 460 feet long toe key should be constructed in 3, 150-foot long slot sequenced A-C-B. For Slope B, the approximately 380 feet long toe key should be constructed in 3, 135-foot long slot sequenced A-C-B. Each slot should be backfilled above critical weak beds as determined during grading prior to initiating the subsequent slot.

- The proposed graded Slope C (Figures 2 and 5) will not expose weak clay beds based on boring B-3. Based on the slope stability analyses the proposed slope will be stable without any remedial grading measures

- The proposed 50 feet high fill slope located south of the intersection of Diamond Bar Boulevard and Crooked Creek Drive, depicted on cross-section D-D', Figure 4, produces a sliver fill over the natural slope. Based on the stability analyses presented in Section 8.2, a toe key 20 feet wide and 10 feet deep is required to achieve the required static and seismic stability for the proposed fill slope. The fill slope above the key should be benched into competent material as schematically shown on Figure 6 – Fill Slope Grading and Benching Recommendations.

- The areas of the segmental retaining walls, identified as "Loffel walls" on Figures 2 and 5, are located above 2:1 inclination cut slopes and supporting variable height, 2(H):1(V) fill slopes. Due to the presence of unfavorably oriented bedding and weak clay beds within the bedrock, it is recommended that cut portions of the slopes be overexcavated and keyed a minimum of 4 feet below the adjacent pad grades, and be replaced with engineered fill. The approximate key dimensions are shown on Figure 5. Additional subgrade preparation may be required by the manufacturer of the selected segmental wall system.

- All fill slopes steeper than 5(H):1(V) should be keyed into competent soil as indicated on Figure 6. Benching into competent material should continue as the fill progresses. All benching should be performed under the observation of the Geotechnical Engineer of Record.

- Lightly loaded ancillary structures (e.g., site walls, trash enclosures, retaining walls) outside the fill areas and therefore located on native materials should be overexcavated to a depth of at least 1 foot below the bottom of the proposed footing or to competent soils, whichever is deeper. The excavation should extend a horizontal distance of at least 2 feet beyond the outside perimeter of the structure.

- Pavement/Flatwork areas should be overexcavated to a depth of at least 1 foot below the bottom of the pavement section/flatwork, or to competent subgrade, whichever is deeper.

- Disturbed soils at structural and non-structural areas will likely occur after demolition of existing site improvements and during construction activities. These soils should be overexcavated and recompacted to the total depth of the disturbed material.

The subgrade soils exposed during excavation should be scarified to a depth of 6 inches, moisture-conditioned to at least 120 percent of optimum moisture content, and compacted to at least 90 percent of the maximum dry density, as evaluated by the latest version of ASTM D1557.

Fill should be placed in horizontal lifts not more than 8 inches in loose uncompacted thickness. In order to adequately compact the face of reconstructed fill slopes, it is strongly recommended to overbuild the slopes by 1 to 2 feet and to cut the slope back to the final configuration. If this

method is not practical, the contractor must be prepared to skillfully compact the outer slope edge and face to meet the compaction requirements. The edge of the constructed slope should be placed slightly elevated and not be allowed to roll off. All fill should be moisture-conditioned to at least 120 percent of the optimum moisture content and compacted to at least 90 percent of the maximum dry density (ASTM D1557) for fills with vertical thickness of less than 45 feet and to at least 95 percent of maximum dry density for fills at depths below 45 feet and for the upper 1 foot below the pavement section. The moisture condition of the placed fill should be checked frequently and maintained or re-established as necessary during all phases of the placement of the fill.

It is likely that oversize rocks derived from cemented layers within the Puente Formation will be encountered during the grading activities, placement of the fill, and subgrade preparation. Any oversize particles, i.e., sizes greater than 6 inches, should be disposed off site or buried in the fill according to the table below. The on-site soils may be re-used as compacted fill provided they are free of organics, deleterious materials, and debris. All fill material should comply with the oversize rock recommendations provided in Table 4 below.

Table 4
Geotechnical Recommendation
Placement of Oversize Rocks

Depth below grade (feet)	Allowable Oversize Placed in Fill (inch)	
	Building Pad	Open Fields and Pavement Areas
5 to 10	6	6
10 to 20	36	48
deeper than 20	48	60

In the event that fill soils are imported to meet the grading plan requirements, such fill soils should be sampled, tested, and approved by the Geotechnical Engineer of Record prior to arrival on-site. In general, any soils imported to the site for use as fill should be predominantly granular and have an Expansion Index less than 20. Additional recommendations for site grading are provided in the "General Grading Recommendations" section of this report.

Based on the available data, the compaction of the on-site materials to a percentage of the maximum dry density (ASTM D1557) will result in the approximate volume changes indicated in Table 5 below:

Table 5
Volume Changes during Fill Placement

Material Type & Relative Compaction (%)	Volume Change
Alluvium/Undocumented Fill (90%)	Decrease 2-7 % (shrinkage)
Bedrock (90%)	Increase 0-5 % (bulking)

It must be noted that the on-site materials vary significantly and therefore the above volume change estimates may be taken only as a general guideline.

9.4. Mitigation of Effects of Expansive Soils

The on-site soils are considered to possess a predominantly medium expansion potential. Our experience indicates that a significant reduction of post-construction expansion potential can be achieved by fill placement at moisture contents near 120 percent of optimum moisture content (ASTM D1557).

Although there is no known economically feasible method of entirely preventing movement of expansive soils, current state-of-the-art practice in the southern California area dictates implementation of foundation designs consisting of stiff mat foundations (e.g. post-tensioned slabs or heavily reinforced mats) which distribute differential movements over larger areas. In addition, deepened perimeter foundation embedment which acts as a moisture cut-off and limits moisture migration under the structure and increases confining stresses are beneficial to minimize the effects of expansive soils. In-grading moisture treatment and post-grading moisture maintenance of subgrade soils within the building pads are also considered critical factors in reducing post-construction movements.

Reasonable mitigation of expansive soil effects at this project site incorporating a combination of the above methods is considered feasible from a geotechnical standpoint. However, it must be noted that soil movements cannot be entirely precluded when building on expansive soils.

Implementation of all of the following procedures is recommended to mitigate the effects of expansive soils.

- Stiffening of slab/foundation systems using post-tensioning.
- Use of deepened thickened edges to provide a moisture cut-off and confinement.
- Conditioning of fill soils by moisture addition and re-compaction.
- Maintaining moisture content of pads and re-certification of pads undeveloped longer than 6-months.
- The recommendations provided in this report assume medium expansion potential of the foundation subgrade. Each pad should be tested before finalization and if the Expansion Index exceeds 90, lot-specific foundation and slab design should be provided.

The potential for expansion movements depends critically on maintenance of constant and uniform moisture distribution within the expansive materials. KFMg has carried out numerous investigations of structural distress where the inferred mechanism of movement has been heave resulting from swelling due to post-construction landscape watering in combination with inadequate surface drainage. Consequently, judicious use of irrigation in close proximity to foundations and proper design of surface and subsurface drainage are considered critical to the adequate performance of foundations and slabs constructed on this site.

9.5. Import Soils

The majority of near-surface on-site soils are not considered suitable for backfill behind retaining walls and for any applications which call for near-free draining material. Consequently it should be anticipated that all segmental retaining wall backfill materials will likely have to be imported. Specific benchmark properties parameters for retaining wall backfill should be proposed by the Manufacturer of the selected segmental wall system and be approved by the Geotechnical Engineer of Record.

Specific benchmark properties parameters for import fill soils for the buttress fills are provided in the "Site Preparation And Earthwork" section of this report. Alternatively, on-site soils reinforced with man-made fibers, e.g., "Geofibers," may be used to achieve the required strength properties for soils to be used in the buttress fills for Slopes A and B. Additionally, use of soil cement in strategic zones within the buttresses may be considered to mitigate the need for select import fill. This firm can provide upon request specific recommendations for such alternatives as the site planning progresses.

9.6. Subddrainage Provisions

Toe key drains should be installed at the heel of the toe keys. The toe key drains should consist of a 4- to 8-inch-diameter perforated ABS or PVC Schedule 40 drain pipe, or approved equivalent. The diameter of the pipe will depend on the length of the perforated section and the distance to the outlet pipe. The maximum length of the perforated drain pipe between discharge outlets should not exceed 200 feet, 400 feet, and 500 feet for 4-, 6- and 8-inch diameter pipe, respectively. Multiple overlapping perforated-solid pipes may be considered to avoid using 8-inch diameter pipe. The drainage pipe should be sloped at least 1 percent towards the outlet pipe and be surrounded by 4 cubic feet per foot of the Class II Permeable Material (Caltrans Standard Specifications - Section 68), or by of ¾-inch crushed rock (Standard Specification for Public Works Construction - Section 200-1.2) wrapped in suitable non-woven filter fabric, e.g., Mirafi 140NL or approved equivalent.

Additional bench backdrains should be installed at vertical intervals of no more than 30 feet and at selected locations at the interface of bedrock and native soils or if seepage is encountered, as determined during construction by the Geotechnical Consultant of Record. The bench backdrains should consist of a 4-inch-diameter perforated ABS or PVC Schedule 40 drain pipe, or approved equivalent. The maximum length of the drain pipe between discharge outlets should not exceed 200 feet. The drainage pipe should be sloped at least 1 percent towards the outlet

pipe and be surrounded by 3 cubic feet per foot of the Class II Permeable Material (Caltrans Standard Specifications - Section 68), or by of ¾-inch crushed rock (Standard Specification for Public Works Construction - Section 200-1.2) wrapped in suitable non-woven filter fabric, e.g., Mirafi 140NL or approved equivalent. The outlet pipe should be sloped at least 2 percent towards suitable outlet locations.

Perforations in all drain pipes should have a maximum diameter of 0.25 inches or 3/8 inches for Class 2 Permeable or ¾-inch crushed rock drain material, respectively, spaced staggered 3 inches on center, and be arranged in 2 rows at a radial spacing of approximately 120 degrees. The axis of the included angle between the perforation rows should be positioned downward to form a flowline. The drain pipe should discharge through a solid pipe, sloped at least 2 percent, to appropriate outlets.

9.7. Foundations and Concrete Slabs-on-Grade

9.7.1. Building Foundations

The foundations for the proposed commercial and residential development may be supported on spread footings established in compacted fill. Recommendations for the design and construction of shallow foundations are presented below.

Grading recommendations have been provided to mitigate, albeit not eliminate, the effects of expansive soils, however, even with these measures and proper site drainage, the potential for some expansive soils-related ground movement still exists. A proper, structurally designed thickened edge post-tensioned slab with interior stiffening beams is considered an appropriate system of support for the proposed construction. The thickened edge around the perimeter of the structure should be extended at least 30 inches below the lowest adjacent grade and should be continuous to act as a moisture migration barrier to assist in minimizing of long term changes in the moisture content of the foundation soils beneath the footprint of the buildings.

Although the recommended post-tensioned floor slabs do not eliminate the potential for slab deflection, they provide for a strengthened, more rigid slab which tolerates differential movement with less cracking than a conventionally reinforced slab on grade. As a minimum, post-tensioned slabs should be designed in accordance with Post-Tension Institute design philosophy for post-tension slabs on expansive soils as outlined in the 2001 CBC, Volume 2, Chapter 18, Division III. The recommended geotechnical parameters for post-tension slab design for soils with Expansion Index of less then 90 are provided in the following table.

Table 6
Geotechnical Design Parameters
Post-tension Slab-on-grade

	Edge Lift	**Center Lift**
Assumed Thornthwaite Index	+ 30	- 20
Edge moisture variation distance	5.4 feet	5.6 feet
Estimated differential swell	1.5 in	3.8 in
Minimum embedment of thickened edge	30 inches	
Allowable bearing pressure - may be increased by 33% for seismic loading conditions	2500 psf	
Coefficient of friction (contingent on meeting underlayment requirements below)	0.4 for "best" and "better" slab underlayment system described in Table 7 0.6 for "standard" slab underlayment system described in Table 7	

The post-tension slab must be designed by a Structural Engineer to properly address the design and construction issues. Special care needs to be taken during construction to ensure that the perimeter thickened edges as well as stiffening interior ribs do not impede the transfer of stresses into the slab during post-tensioning.

To reduce potential for vapor transmission through the slabs, it is recommended that the concrete has a thickness of at least 5 inches, water cement ratio of 0.45 or less, and a slump of 4 inches or less. Table 7 provides alternatives for control of vapor transmission through concrete floor slab support placed on a properly prepared subgrade.

Table 7
Geotechnical Recommendation
Alternatives for Mitigation of Vapor Migration through Concrete Slab

Objective	Recommendation
"Best" protection against vapor intrusion	• Concrete floor slab-on-grade placed directly on a plastic membrane 10 mils in thickness[1] (ACI 302.1R-96). • The membrane should be placed on at least 2 inches of dry silty sand[2]. • The dry silty sand should be separated from the underlying capillary break layer by non-woven geotextile, Mirafi 140N or equivalent. • The geotextile should be placed on at least 4 inches of ¾-inch crushed rock[3] or clean gravel[4] to act as a capillary break.
"Better" protection against vapor intrusion	• Concrete floor slab-on-grade placed directly on a plastic membrane 10 mils in thickness[1] (ACI 302.1R-96). • The membrane should be placed on at least 2 inches of silty sand[2].
Standard protection against vapor intrusion	• 2 inches of dry silty sand[2]; • placed over plastic membrane 8 mils in thickness. • The membrane should be placed on place at least 2 inches of silty sand[4].

[1] If additional protection is desired, the plastic membrane may be replaced with a 10-mil thick moisture vapor retarder that meets the requirements of ASTM E 1745 Class C (for example, Stego Wrap or similar).
[2] The silty sand should have a gradation between approximately 15 and 25 percent passing the No. 200 sieve and a plasticity index (PI) of less than 4.
[3] The ¾-inch crushed rock should conform to Section 200-1.2 of the latest edition of the Standard Specifications for Public Works Construction (Greenbook).
[4] The gravel should contain less than 10 percent of material passing the No. 4 sieve and less than 3 percent passing the No. 200 sieve.

All underslab materials should be adequately compacted prior to the placement of concrete. The materials should be slightly moist and not be wetted or saturated prior to the placement of concrete. Care should be taken during placement of the concrete to prevent displacement of the underslab materials. The concrete slab should be allowed to cure properly prior to placing vinyl, hardwood, or other moisture-sensitive floor covering.

9.7.2. Foundations for Appurtenant Structures

Shallow foundations for appurtenant structures, e.g., trash enclosures, perimeter walls, retaining walls (less than 9 feet in height), should be designed using the geotechnical design parameters presented in Table 8. Footings should be designed and reinforced in accordance with the recommendations of the structural engineer and should conform to the 2007 California Building Code.

Table 8
Geotechnical Design Parameters
Continuous and Isolated Spread Footing Foundations

Foundation Dimensions	• At least 18 inches in width • At least 18 inches below the lowest adjacent grade
Net Allowable Bearing Capacity	• 2,500 pounds per square foot (psf) • The allowable bearing value may be increased by one-third for transient live loads from wind or seismicity.
Estimated Settlement	• Approximately 1-inch total settlement • Approximately 0.5-inch differential settlement over 20 feet • Approximately 0.75-inch differential settlement over the structure length
Allowable Coefficient of Friction	• 0.40
Allowable Lateral Passive Resistance	• 220 pounds per cubic foot equivalent fluid pressure (pcf EFP)

The total allowable lateral resistance can be taken as the sum of the friction resistance and passive resistance. The passive resistance values may be increased by one-third when considering wind or seismic loading.

Footings should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the requirements of the 2007 California Building Code.

9.7.3. Concrete Slabs-On-Grade

The recommendations provided in the "Site Preparation and Earthwork" section of this report and in this section are intended to provide a firm bearing subgrade to help reduce the occurrence of cracks and fissures in concrete and to reduce their horizontal separation and vertical offset. However, it should be understood that the concrete slabs may still crack due to structural design or detailing, curing, construction execution or during its use even when these recommendations are implemented. Therefore, in order to minimize the cracking of the concrete, the reinforcement, placement of control joints, concrete mix, and curing specifications should be designed by the Structural Engineer and Concrete Specialist.

9.7.3.1. Post-Tensioned Floor Slabs

Design parameters for post-tensioned floor slabs are provided in "Building Foundations" section of this report. Good performance of post-tensioned slabs is dependant on both the proper design and proper installation. It is therefore recommended that a responsible Structural Engineer provide the design drawings, construction specifications.

The Structural Engineer of Record should be present during the post-tensioning of the slabs to verify that appropriate procedures described in the project specifications are followed to facilitate a proper transfer of stresses into the slab. As discussed above, for slabs with thickened edges and interior reinforcing ribs the stress transfer may be impeded, if incorrect procedures are followed.

In order to provide a benchmark for evaluating the performance of the post-tensioned system and to evaluate the quality of installation, it is recommended that a floor level survey of the slab surface be taken within 1 week of the post-tensioning of the slab.

9.7.3.2. Conventionally Reinforced Floor Slabs

For design of concrete slabs, presumably for appurtenant structures, a modulus of subgrade reaction (k) of 140 pounds per cubic inch may be used. Floor slabs should be designed and reinforced in accordance with the Structural Engineer's recommendations. The minimum reinforcement to reduce separation and offset of concrete cracks should consist of No. 4 reinforcing bars spaced at 16 inches on-center, each way, placed in the middle one-third of the section. Reinforcement should be properly placed and supported on "chairs." Welded wire mesh reinforcement is not recommended.

9.7.3.3. Exterior Slabs

Exterior slabs should be placed on subgrade prepared in accordance with the recommendations provided in the "Site Preparation" section of this report. As a minimum for exterior walkways, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 4 reinforcing bars placed longitudinally at 16 inches on center. Wide exterior slabs should be reinforced with at least No. 4 reinforcing bars placed 16 inches on center, each way. The reinforcement should be extended through the control joints to reduce the potential for differential movement. To mitigate the adverse effects of expansive soils, the exterior slabs should be designed with thickened edges at least 12 inches deep to provide a moisture cut-off to stabilize the moisture content distribution under the slab.

As discussed above, the provided recommendations are intended to reduce the occurrence of the cracks but not to eliminate them. Therefore, in order to minimize the cracking of the concrete, the reinforcement, placement of control joints, concrete mix, and curing specifications should be designed by the Structural Engineer and Concrete Specialist.

9.7.4. Foundation Observations

To evaluate the presence of satisfactory foundation subgrade materials at design elevations, footing excavations should be observed to be clean of loosened soil and debris before placing steel or concrete and probed for soft areas. If soft or loose soils or other unsatisfactory materials are encountered, such materials should be removed and replaced with compacted fill prior to pouring the footing.

During construction, the Geotechnical Engineer of Record should specifically test and accept the slab subgrade for each lot. The testing should consist of verification of compaction, moisture content, and of sampling and testing of near-surface soils for expansion and corrosion potential. If Expansion Index greater than 90 is determined, a lot-specific foundation design should be provided. Since at the time when concrete is placed at the construction site the presence of the Geotechnical Engineer of Record representative may not be continuous, it is the construction manager's responsibility to request the inspection and testing from the Geotechnical Engineer of Record

9.8. Retaining Walls

It is understood that 2 extensive segmental retaining walls ranging from about 10 feet high to 23 feet high are proposed within slopes separating the 3 pads, as shown on Figure 2. Although not shown on the plans, it is assumed that other relatively short CMU retaining walls less than approximately 6 feet high will likely be incorporated into the future site development plans. The following sections provide the respective retaining wall recommendations.

9.8.1.1. MSE Retaining Wall Design

The following recommendations are provided for mechanically supported earth (MSE) segmental retaining wall (SRW) identified as "Loffel walls" on Figures 2 and 5. MSE walls are proposed for walls up to about 23 feet high supporting 2(H):1(V) fill slopes. Considering that many different MSE wall systems are available, the actual design of the wall is typically provided by the specialty MSE wall contractor, and the design is reviewed and accepted by the Geotechnical Engineer of Record.

The MSE wall designer should consider the following geotechnical parameters and recommendations in the design:

- The spacing and type of reinforcing geogrids, the MSE wall facing, and the internal stability of the wall should be evaluated and designed by a specialty contractor/supplier.

- MSE walls may be supported on a subgrade prepared in accordance with the "Site Preparation and Earthwork" section of this report.

- Allowable bearing capacity for MSE wall founded on competent native bedrock or engineered fill and keyed in a minimum of 3 feet below adjacent grade is 3,000 pounds per square foot (psf) or 2,000 psf, respectively; this value may be increased by 10 percent for each foot of embedment up to 6000 psf. The allowable bearing value may be increased by one-third for transient live loads from wind or seismicity. It is noted, that an adverse descending slope may exist in front of some of the MSE walls; consequently, additional reductions in bearing capacity may be appropriate. This office should carry out review of the MSE wall configurations and provide appropriate recommendations, once the final configuration of the slopes above and below the MSE walls is known.

- If an open face wall system (i.e., free draining) is used, only select very low or low expansive soils may be used within 7 feet of the wall facing. If closed face wall system is selected, in addition to the select material zone behind the facing of the wall, a drainage zone behind the facing of the wall as generally outlined in the "Retaining Wall Backfill and Drainage" section of this report must be installed. If suitable, alternatively a 12-inch wide zone of a ¾-inch crushed rock or Caltrans Class II Permeable Material separated from the backfill material by a non-woven geotextile (e.g., Mirafi 160N or approved equivalent) may be used.

- The shear strength of the general backfill material behind the zones described in the previous paragraph should be taken as cohesion c = 100 psf and friction angle $\phi = 30^o$, and unit weight of 120 pcf.

- The shear strength of the underlying bedrock material should be taken as cohesion c = 400 psf and friction angle $\phi = 30^o$, and unit weight of 120 pcf.

- The internal MSE wall design for the seismic condition should be based on horizontal seismic coefficient of 0.31. The global MSE wall design should be based on horizontal seismic coefficient of 0.239.

- As a preliminary guideline the following table provides a minimum distance to the critical global slip surface behind the toe of the wall. This distance is shorter than the necessary length of the geogrid reinforcement since the critical slip surface typically intersects several layers of the geogrids at the heel of the wall. For preliminary evaluations, it is anticipated that the geogrids will be at least 7 feet longer then the indicated distance

Table 9
Geotechnical Design Parameters
Geotechnical Design Parameters for Retaining Walls

MSE Wall Location	Commercial Pad (Section A-A')	Backbone Road (Section B-B')
Representative MSE Wall Height (feet)	8 -13	18 - 23
Typical minimum distance from the wall toe to critical global slip surface (feet)	18	18

- The target Factor of Safety of 1.5 and 1 should be used for global stability assessment for static and pseudostatic conditions, respectively. The internal stability of a MSE wall should be evaluated based on the manufacturer's guidelines. The Geotechnical Engineer of Record should provide a final review and approval of the MSE wall design.

9.8.1.2. CMU Retaining Wall Design

The following recommendations are provided for CMU retaining walls less than 6 feet in height. Such retaining walls may be supported on spread footings constructed in accordance with the "Site Preparation and Earthwork" and "Foundations for Appurtenant Structure" sections of this

report. The lateral loading and soil resistance should be calculated based on the recommendations presented in Table 10. Design values are provided for both the active and at-rest conditions for level ground behind the top of the retaining wall and assume that a drainage system will be installed behind the wall, so that external water pressure will not develop. If a drainage system is not installed, the wall should be designed to resist also the hydrostatic pressure.

Table 10
Geotechnical Design Parameters
Lateral Pressures for CMU Retaining Walls

	Level Ground Condition	**2(H) : 1(V) Backslope**
Lateral ***at-rest*** pressure (psf)	64 z + 0.53 Q	111 z + 0.92 Q
Lateral ***active*** pressure (psf)	43 z + 0.36 Q	75 z + 0.63 Q
Allowable lateral ***passive*** resistance	220 pcf EFP for level grade at the toe	

where: z.... Depth below the grade behind the wall (ft)
Q.... Uniform surcharge load (psf) within a 1(H):1(V) plane drawn upward from the heel of the wall footing

Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. Walls supporting on-site materials that are free to rotate at least 0.04 radians (deflection at the top of the wall of at least 0.04 x H) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) located within a 1(H):1(V) plane drawn upward from the heel of the wall footing should be added to the lateral earth pressures.

9.8.1.3. Seismic Lateral Earth Pressure

In addition to the above-provided lateral earth pressures, retaining walls higher than 6 feet, should be designed to support a seismic active pressure. The recommended seismic active pressure distribution on the wall is an inverted triangular with the maximum pressure equal to 19H psf and 50H psf for level grade and 2(H):1(V) backslope, respectively, where H is the wall height in feet.

9.8.1.4. Retaining Wall Backfill and Drainage

An approved import material will likely be used for the retaining wall backfill. Suitable material should have a Sand Equivalent of about 30, an Expansion Index of less than 20, and fines content (passing #200 sieve) of less than 15 percent. The suitability of the on-site and/or import material for retaining wall backfill should be verified at the time of construction. This select backfill should extend at least 2.5 feet behind the back side of the wall.

If the surrounding native materials are granular and relatively permeable, the granular backfill may be densified by water jetting. However, this condition is not anticipated at this site. Otherwise the backfill should be moisture-conditioned to at least optimum moisture content and compacted in loose horizontal lifts not more than 8 inches in uncompacted thickness to at least 90 percent of the maximum dry density as evaluated by the latest version of ASTM D1557. The backfill should be capped with a concrete swale/slab or with at least 12 inches of relatively impervious clayey material and sloped to prevent ponding of water.

Retaining walls should be constructed to limit potential for hydrostatic pressure built-up behind the wall. If irrigation or precipitation infiltration is expected, adequate drainage is essential to provide a free-drained backfill condition to limit hydrostatic buildup behind the wall. In order to control efflorescence and/or staining on the air side of the wall due to percolation of the water through the wall, the wall should be appropriately waterproofed. Adequate drainage and waterproofing behind the wall may be provided by a backdrain consisting of geosynthetic drainage composite such as TerraDrain, MiraDrain, or approved equivalent, properly placed against the entire backside of the wall. The drainage composite should be connected to a 4-inch-diameter perforated ABS or PVC Schedule 40 drain pipe, or approved equivalent. The drain pipe should be sloped at least 2 percent and surrounded by 1 cubic foot per foot of the Class II Permeable Material (Caltrans Standard Specifications - Section 68), or by of ¾-inch crushed rock (Standard Specification for Public Works Construction ("Greenbook") - Section 200-1.2) wrapped in suitable non-woven filter fabric, e.g., Mirafi 140NL or approved equivalent. Perforations in the drain pipe should have a maximum diameter of 0.25 inches or ⅜ inches for Class 2 Permeable or ¾-inch crushed rock drain material, respectively, spaced 3 inches on center, and be arranged in 2 rows at a radial spacing of approximately 120 degrees. The axis of the included angle between the perforation rows should be positioned downward to form a flowline. The drain pipe should discharge through a solid pipe to appropriate outlets, such as the storm drain system or through the wall. The maximum length of the drain pipe between discharge outlets should not exceed 200 feet. Alternatively, weep holes through the wall, at least 3-inches in diameter, spaced no more than 10 feet apart may be considered.

9.9. Temporary and Trench Excavations

All trench excavations should be performed in accordance with CalOSHA regulations. The on-site soils may be considered a Type C soil, as defined the current CalOSHA soil classification. Sloughing/raveling of exposed soil intervals should be anticipated. All applicable excavation safety requirements and regulations, including CalOSHA requirements, should be met.

Unsurcharged excavations: Temporary short-term (generally less than 5 days) unsurcharged excavations shallower than 4 feet may be excavated with vertical sides. Sides of temporary, unsurcharged, excavation deeper than 4 feet should be sloped back at an inclination of 1.5(H):1(V) or flatter. Where space for sloped sides is not available, shoring will be necessary. This office can provide appropriate shoring recommendations, once the excavation layout of specific utilities is known.

Surcharge setback recommendations: Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1(H):1.5(V), but no closer than 4 feet. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. KFMg should be advised of such heavy vehicle loadings so that specific setback requirements can be established for the used equipment. Alternatively, a shoring system may be designed to allow reduction in the setback distance.

Personnel from KFMg should observe the excavation progress so that appropriate modifications to the excavation design may be recommended, if necessary due to encountered conditions differing from the design assumptions.

9.10. Appurtenant Structures within Influence of Fill Slopes

Due to the presence of expansive high plastic clayey soils, the outer 5 to 10 feet of fill slopes will be susceptible to gradual long-term slope movements. Although these slope movements cannot be eliminated because gravity cannot be controlled, it is important to design the structures located within the zone of influence of these slopes to either resist or accommodate such movements. These movements generally diminish after 10 to 15 years while the majority of the movements occur within the initial 5 to 10 years. The cumulative magnitude of such movements can be up to several inches. This phenomena can manifest itself as downslope translation and rotation of rear yard walls and corner pilasters, separation between the top-of-slope walls and abutting walls, flatwork cracking and separations, separations between the flatwork and pool coping, tilting of swimming pools, pulling away of patio slabs from the building, separations and cracking of sideyard walls, etc. Although the list of potentially adverse near-slope effects is rather long, a proper design can successfully accommodate such movements with no effect on the use of the facilities. It is our experience that however predictable and normal the movements are, the homeowners tend to respond negatively to unsightly features, which may create a strained situation between the homeowner and the developer. The following list provides guidance for installation and operation of appurtenances located within 30 feet or within a distance equivalent to ⅓ of the fill slopes height, whichever is greater, to the top of the slope:

- Off-the-shelf designs for backyard improvements should not be utilized. Each element should be specifically designed with full incorporation of the design considerations described herein.
- Top-of-slope walls should be offset at least 5 feet from the top of the slope.
- The fill slope should be vegetated with drought-resistant, deep-rooted vegetation and judiciously irrigated.
- To reduce the movements of top-of-slope walls, the walls should be supported on a grade beam embedded at least 18 inches below the adjacent subgrade and founded on 8-foot deep 16-inch diameter pilasters spaced 8 to 12 feet. If movements on the order of 2 inches can be tolerated, the top-of-slope wall may be founded on continuous footing at least 30 inches deep.

- The wall adjacent to the top-of-slope walls, i.e., sideyard walls, should not be connected to the top-of-slope wall to allow for as-needed movement and rotation of the top-of-slope wall without creating an unsightly separation.
- Large continuous flatwork areas in the backyards are not recommended as they will likely crack and the cracks will shift, offset, and separate and become unsightly. The patios should be broken up into smaller areas separated by landscaping that readily accommodates lateral soil movements.
- Whenever possible, abutting of flatwork and other appurtenances, e.g., patio-wall, pilaster-planter wall, should be eliminated and the elements should be separated by landscaping.
- Pool coping should be designed to accommodate/mask lateral soil movements.
- The homeowners should be notified of the likelihood of top-of-slope movements and mandate to retain qualified professionals for design of any lot improvements.

This office should be contacted if any foundation questions regarding the design of the appurtenances arise.

9.11. Asphalt Concrete Pavement

9.11.1. Subgrade Preparation

The pavement subgrade should be moisture-conditioned, scarified, and compacted just prior to placement of the base course to 95 percent of maximum dry density as described in "Site Preparation and Earthwork" section of this report. Positive drainage of the pavement and pavement subgrade areas should be provided since moisture infiltration into the subgrade may decrease the life of pavements. Curbing located adjacent to paved areas should be founded in the compacted subgrade soils, not the aggregate base, in order to provide a cutoff to reduce water infiltration from adjacent irrigated parkways into the base course under the pavement.

9.11.2. Pavement Design

The required pavement surface and base thicknesses will depend on the expected wheel loads and volume of traffic (TI, Traffic Index). Assuming that the pavement subgrade will consist of the on-site or comparable soils compacted as recommended, pavement structural sections provided in Table 11 may be used for design.

Table 11
Geotechnical Recommendation
Asphalt Concrete Pavement Structural Sections

Typical Traffic Use	Design Traffic Index	AC / AB (inches)	Full AC (inches)
Parking / driveways	3	3.0 / 4.5	3.5
Light duty	4.5	3.5 / 5.0	5.5
Backbone Road / Fire lanes	6	4.0 / 8.5	8.0

The pavement structural sections were established using the design criteria of the State of California, Department of Transportation, an assumed R-value of 25, and the assumed Traffic Indices as indicated. Given the extensive proposed grading and the likelihood that intermixing of on-site soils will occur, confirmatory R-value tests on the subgrade soils during grading should be performed to verify the recommended design sections.

The base course should meet the specifications for Class II Aggregate Base as defined in Section 26 of the State of California, Department of Transportation, Standard Specifications, current edition. Alternatively, the base course could meet the specifications for untreated base materials as defined in Section 200-2 of the current edition of the Standard Specifications for Public Works Construction ("Greenbook"). The base course should be compacted to at least 95 percent of the maximum dry density (ASTM D1557).

9.11.3. Pavement Construction Observation

The preparation of the pavement subgrade and the placement of the pavement section should be observed by KFMg personnel. Careful observation is recommended to evaluate that the pavement subgrade is uniformly compacted and the recommended pavement and base course thicknesses are achieved, and that good construction procedures are used.

9.12. Drainage Control

The intent of this section is to provide general information regarding the control of surface water. The control of surface water is essential to the satisfactory performance of any building construction and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the structure, even during periods of heavy rainfall. The following recommendations are considered minimal.

- Ponding and areas of low flow gradients should be avoided.
- Paved surfaces should be provided with a gradient of at least 1 percent sloping away from improvements.
- Unpaved areas, e.g., lawn, should be provided with a drainage gradient of at least 2 percent away from structures.
- Bare soil, e.g., planters, within 5 feet of the structure should be sloped away from the improvement at a gradient of 5 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.

- Planters should not be located adjacent to the structure. If planters are to be located adjacent to the structure, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive infiltration. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

9.13. Soil Corrosion

The corrosion potential of the on-site materials to buried steel and concrete was evaluated. Laboratory testing was performed on representative soil samples to evaluate pH, minimum resistivity, and soluble sulfate content. Table 12 presents the results of the corrosivity testing. General recommendations to address the corrosion potential of the on-site soils are provided below. If additional recommendations are desired, it is recommended that a corrosion specialist be consulted.

Table 12
Corrosivity Test Results

Boring	**B-2, SK-2**
Depth (feet)	13-17
Soluble Sulfate Content	0.0062
Sulphate Exposure	Negligible

The corrosion potential of the on-site soils should be verified during construction for each encountered soil type. Imported fill materials should be tested to confirm that their corrosion potential is not more severe than assumed for the project.

9.13.1. Reinforced Concrete

Laboratory tests indicate that the potential of sulfate attack on concrete in contact with the on-site soils is “negligible” based on 2001 California Building Code Table 19-A-4. However, due to the presence of high plastic clays and experience in the area, concrete mix with Type V cement and maximum water/cement ration of 0.45 should be used unless additional corrosion testing of building pad subgrade soils are performed following grading.

10. GENERAL SITE GRADING RECOMMENDATIONS

The intent of this section is to provide general information regarding the site grading. Site grading operations should conform with applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction.

The grading contractor is responsible for notifying governmental agencies, as required, and a representative of KFMg at the start of site cleanup, at the initiation of grading, and any time that grading operations are resumed after an interruption. Each step of the grading should be accepted in a specific area by a representative of KFMg, and where required, should be approved by the applicable governmental agencies prior to proceeding with subsequent work.

The following site grading recommendations should be regarded as minimal. The site grading recommendations should be incorporated into the project plans and specifications.

1. Prior to grading, existing vegetation, trash, surface structures and debris should be removed and disposed off-site at a legal dumpsite. Any existing utility lines, or other subsurface structures, which are not to be utilized should be removed, destroyed, or abandoned in compliance with current governmental regulations.

2. Subsequent to cleanup operations, and prior to initial grading, a reasonable search should be made for subsurface obstructions and/or possible loose fill or detrimental soil types. This search should be conducted by the contractor, with advice from and under the observation of a representative of KFMg.

3. Prior to the placement of fill or foundations within the building area, the site should be prepared in accordance with the recommendations presented in the Site Preparation section of this report. All undocumented fill or disturbed soils and colluvium within the building areas should be removed and processed as recommended by the representative of KFMg.

4. Special attention should be given to compaction along the outer edges of the fill slopes as the fill is brought up. It is recommended that fill slopes be overbuilt by 1-2 feet and trimmed back to expose a uniform compacted core. If this method is not practical, the contractor must be prepared to skillfully compact the outer slope edge and face to meet the compaction requirements. It may be possible to backroll the fill slopes every 3 vertical feet with a short shank sheepsfoot roller or suitable alternate as the fill slope is constructed. Additional rolling and trimming may be required at the completion of the slope construction. The edge of the constructed slope should be placed slightly elevated and not be allowed to roll off. Compaction and processing should be such that 90 percent of maximum density (ASTM D1557) at the slope face is achieved. It should be noted that even slopes which are properly compacted and maintained are not totally immune to local surficial slumping or erosion and may require periodic maintenance.

5. The exposed subgrade and/or excavation bottom should be observed and approved by a representative of KFMg for conformance with the intent of the recommendations presented in this report and prior to any further processing or fill placement. It should be understood that the actual encountered conditions may warrant grading and/or subgrade preparation beyond the extent recommended and/or anticipated in this report.

6. Where building construction will not follow immediately after grading, the finished pad grade should be re-certified every 6 months and immediately before construction. A consideration may be given to placement of a minimum of 6 inch protective soil cover to help maintain pad moisture.

7. On-site inorganic granular soils that are free of debris or contamination are considered suitable for placement as compacted fill. Any rock or other soil fragments greater than 3 inches in size should not be used within 5 feet of the foundation subgrade.

8. Observation and field tests should be performed during grading by a representative of KFMg in order to assist the contractor in obtaining the proper moisture content and required degree of compaction. Wherever, in the opinion of a representative of KFMg, an unsatisfactory condition is being created in any area, whether by cutting or filling, then the work should not proceed in that area until the condition has been corrected.

11. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

11.1. Plans and Specifications

The design plans and specifications should be reviewed and approved by KFMg prior to bidding and construction, as the geotechnical recommendations may need to be re-evaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report have been incorporated into the project plans and specifications as intended.

11.2. Construction Monitoring

Site preparation, removal of landslides and unsuitable soils, locating of backdrains, assessment of imported fill materials, fill placement, foundation installation, and other site grading operations should be observed and tested. The soils exposed during the construction may differ from that encountered in the test borings. Continuous observation by a representative of KFMg during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where appropriate.

12. LIMITATIONS

KFMg has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

The recommendations and opinions expressed in this report are based on KFMg's review of background documents and on the limited information obtained from field explorations and the associated laboratory testing. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Consequently conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of removal of unsuitable soil and the associated additional effort required to mitigate them. Uncertainties relative to subsurface conditions can always be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request.

Site conditions, including groundwater level, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of 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 KFMg has no control.

KFMg's recommendations for this site are, to a high degree, dependant upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for KFMg to observe grading operations and foundation excavations for the proposed construction. If parties other than KFMg are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the Geotechnical Engineer of Record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

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. KFMg should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. Reliance by others on the data presented herein or for purposes other than those stated in the text is authorized only if so permitted in writing by KFMg. It should be understood that such an authorization may incur additional expenses and charges.

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Diamond Bar, CA 91765
Phone (949) 860-5096

Project Name:
WVUSD Site D - City of Diamond Bar, CA

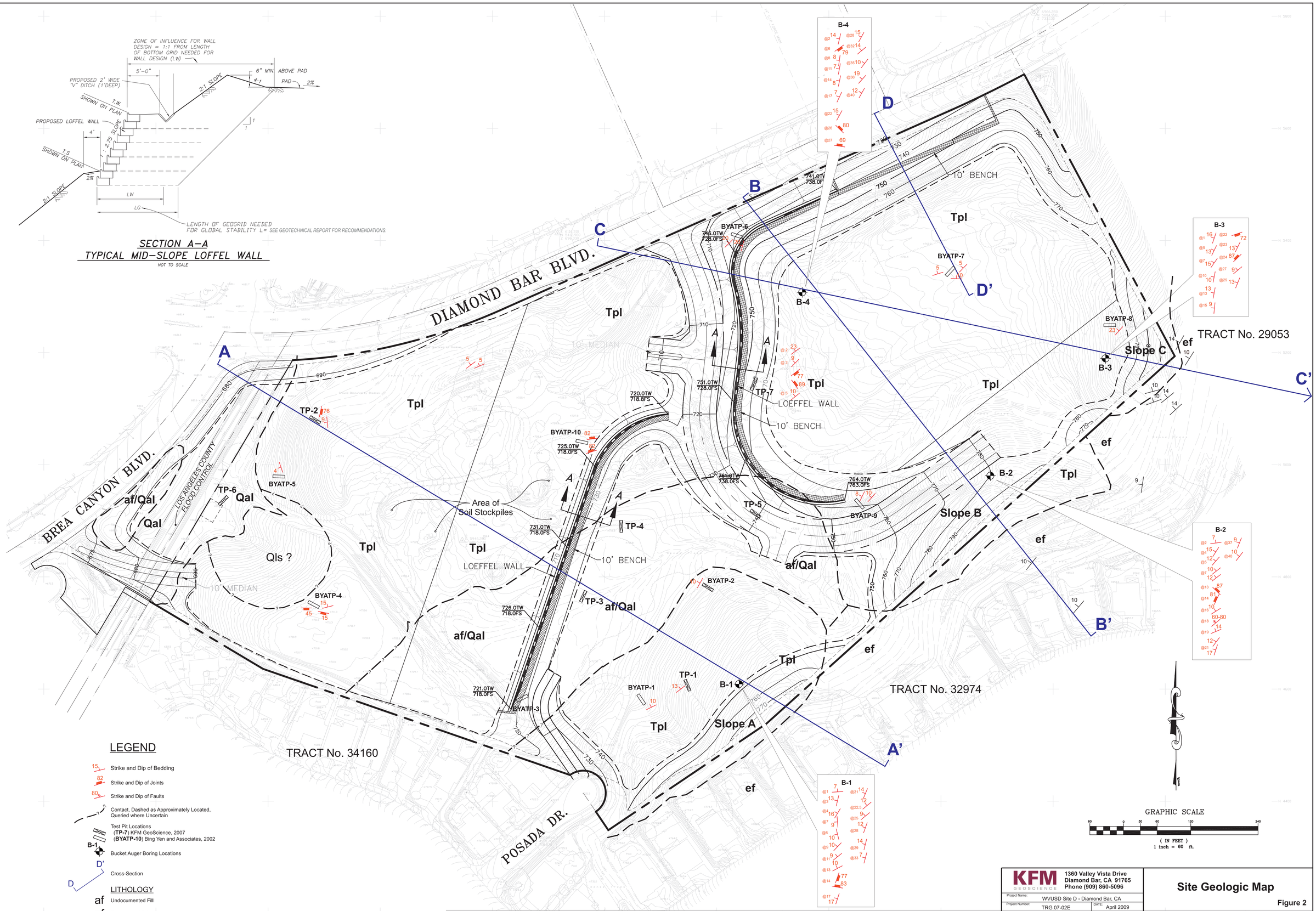
Project Number:
TRG 07-02E

DATE:
November 2007

Site Location Map

Figure 1

SECTION A–A
TYPICAL MID–SLOPE LOFFEL WALL
NOT TO SCALE



LEGEND

- Strike and Dip of Bedding
- Strike and Dip of Joints
- Strike and Dip of Faults
- Contact, Dashed as Approximately Located, Queried where Uncertain
- Test Pit Locations (**TP-7**) KFM GeoScience, 2007 (**BYATP-10**) Bing Yen and Associates, 2002
- B-1 Bucket Auger Boring Locations
- D' D Cross-Section

LITHOLOGY

af	Undocumented Fill
ef	Documented Fill
Qls	Landslide
Qal	Alluvium
Tpl	Puente Formation (La Vida Member)

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

NO	DATE	REVISION	APP	DATE
	11/14/07	ADDED WALL ELEVATIONS		

PROJECT:
REVISION:
DESIGNED BY O.N. DATE 11/05/07
CHECK BY T.B. REVISED BY
PROJECT MANAGER

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Planning
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Construction Management
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Project Name: WVUSD Site D - Diamond Bar, CA
Project Number: TRG 07-02E DATE April 2009

Site Geologic Map

Figure 2

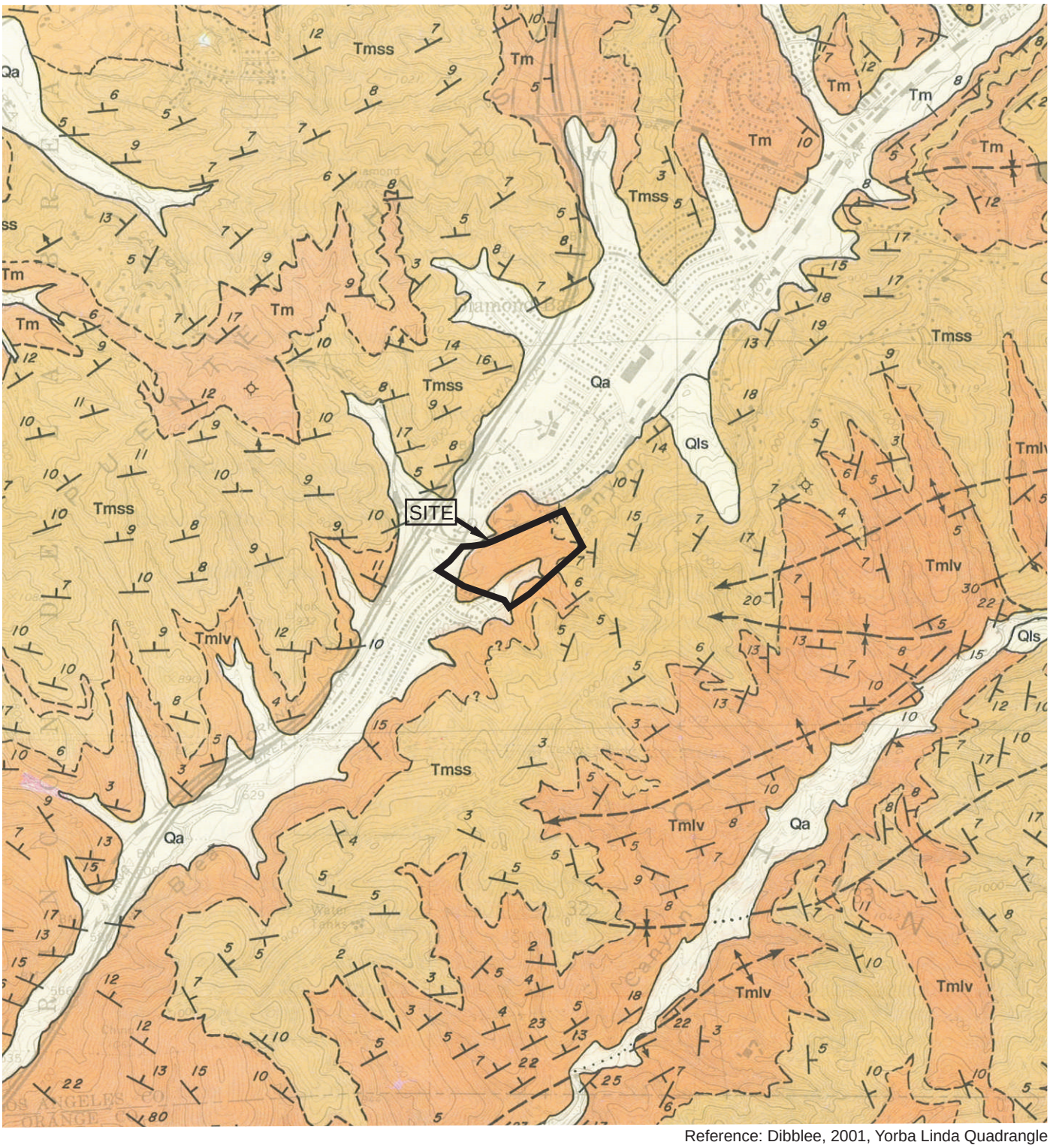
WALNUT VALLEY UNIFIED SCHOOL DISTRICT

DIAMOND BAR BOULEVARD AND BREA CANYON ROAD
DIAMOND BAR, CALIFORNIA

CITY OF DIAMOND BAR

SHEET 1 OF 1

REFERENCE NUMBER 1776.01



Reference: Dibblee, 2001, Yorba Linda Quadrangle

Scale 1:24000

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WVUSD Site D - Diamond Bar, CA

Project Number:
TRG 07-02E

DATE:
January 2008

Regional Geologic Map

Figure 3

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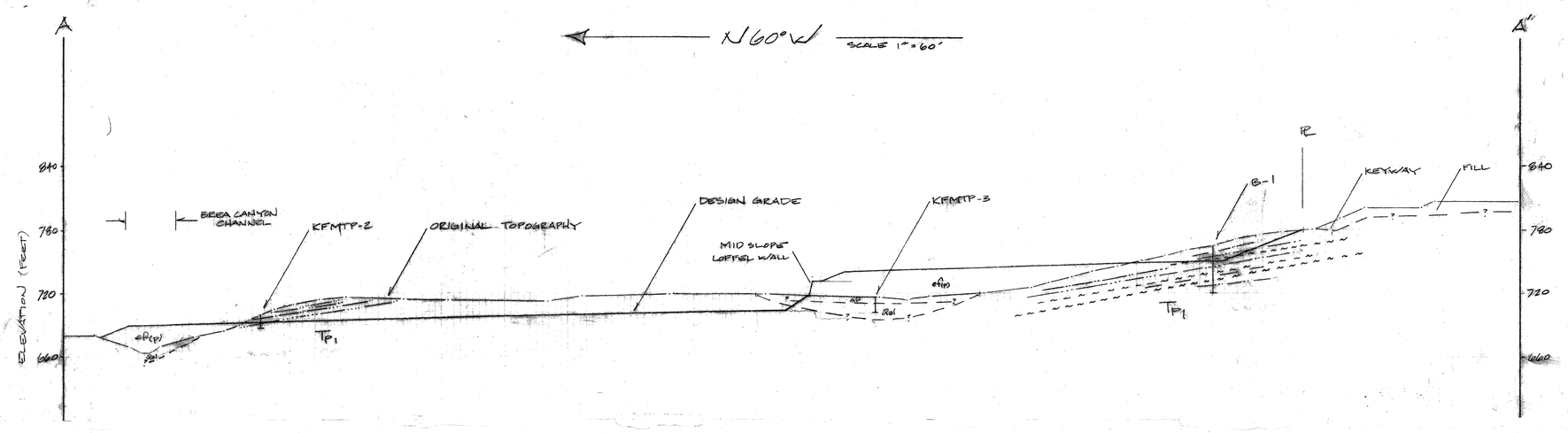

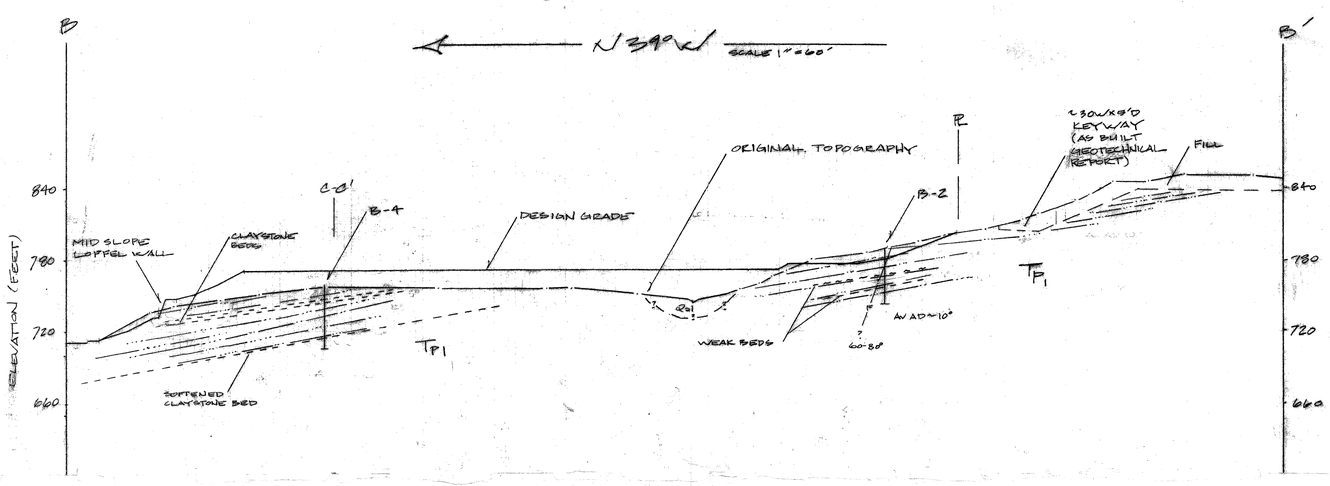

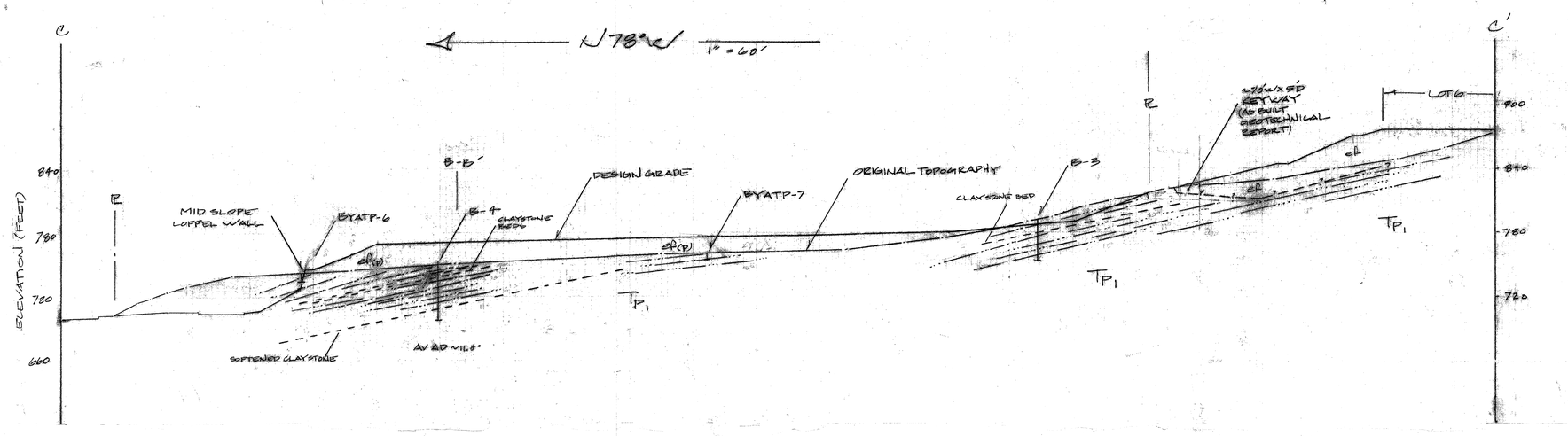

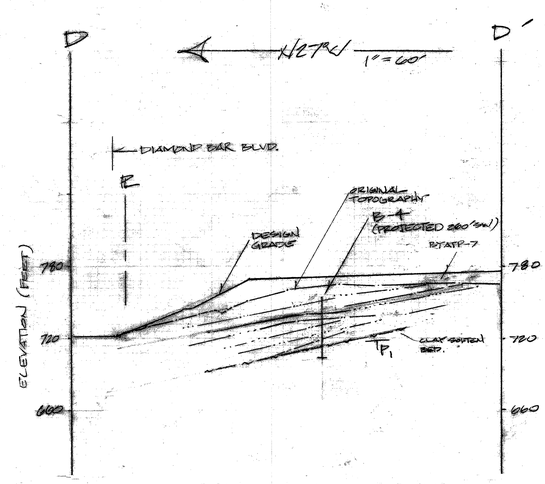

LEGEND

See Figure 2 for Cross-Section Locations

ef(p) - Engineered fill proposed
ef - Engineered fill
af - undocumented fill
Tpl - Puente Formation (La Vida Member)

- Sandstone and siltsone bedding

- Weak clay layer

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

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Cross-Sections A-A through D-D'

Figure 4