Attachment 1

GEOTECHNICAL CRITIQUE
PALISADES LANDMARK CONDOMINIUM PROJECT DRAFT EIR

ENV-2000-2696-EIR; SCH 2002051086; January 2003
(re Vesting Tentative Tract #52928)
E.D. Michael
March 31, 2003

1.0 INTRODUCTION

This subject of this critique is the Christopher H. Joseph & Associates, Inc. January, 2003 Draft Environmental Impact Report, hereinafter "DEIR," of the Palisades Landmark Condominium Project, hereinafter the "PLC Project," City of Los Angeles Tentative Tract #52928. It is specifically limited to a consideration of the geotechnical aspects of that project as it refers to modifications in the area of the Revello Drive landslide. That landslide, which was initiated in 1965, is one of a number that in aggregate cover about half of the slopes below Castellammare Mesa which is located in the western area of the Pacific Palisades, City of Los Angeles. 26

The DEIR has been prepared for the City of Los Angeles Planning Department which apparently is acting as the lead agency consistent with the basic requirements of the California Environmental Quality Act (CEQA). As is well established, CEQA invites public comments and generally provides for as much public participation as possible. Nevertheless, communications between the public and the lead agencies commonly are less efficient than they could be.

A case in point concerns receipt of the DEIR for this critique was on March 25, 2003 for delivery on April 2. Consequently, its scope is limited to a brief examination of the site, research on certain immediately available references, and the DEIR volumes themselves. No time is available for review of various references upon which, in part, the geotechnical reports for the projects are based. In general, the principal geotechnical investigator for the PLC Project, the J. Byer Group, Inc. (JBG) refers to numerous earlier geotechnical reports of the local area and presents some data from those reports. This critique accepts those data at face value. Nevertheless, they necessarily are taken out of context. The conclusions contained herein therefore are qualified to that extent. 27

1.1 REVIEWED DOCUMENTS

Geotechnical references in the DEIR relevant to this critique are contained the DEIR Appendix I and include the following:

[1] Irvine, Jon A., John W. Byer, and Robert I. Zweigler, 2000, Geologic and soils engineering exploration, proposed landslide repair, and multi unit condominium and town home buildings, Tentative Tract 52928, 17331-17333 Tramonto Drive, Pacific Palisades, California: The J. Byer Group, Inc. consultant rpt., Project Number 18457-I, August 16.

[2] Irvine, Jon A., and Robert I. Zweigler, 2000, Addendum geologic and soils engineering exploration report, proposed landslide repair, and multi-unit condominium and town home buildings, Tentative Tract 52928, 17331-17333 Tramonto Drive, Pacific Palisades, California: The J. Byer Group, Inc. consultant rpt., JB 18457-I to Palisades Landmark LLC, November 29. 28

[3] Irvine, Jon A., and Robert I. Zweigler, 2001, Addendum geologic and soils engineering exploration report #2, proposed landslide repair, and multi-unit condominium and

town home buildings, Tentative Tract 52928, 17331-17333 Tramonto Drive, Pacific Palisades, California: The J. Byer Group, Inc. consultant rpt., JB 18457-I, to Palisades Landmark LLC, June 29.

[4] Irvine, Jon A., and Robert I. Zweigler, 2001, Addendum geologic and soils engineering exploration report #3, proposed landslide repair, and multi-unit condominium and town home buildings, Tentative Tract 52928, Lot 1 (condominiums), 17331-17333 Tramonto Drive, Pacific Palisades, California: The J. Byer Group, Inc. consultant rpt., JB 18457-I, to Palisades Landmark LLC, August 28.

[5] Irvine, Jon A., and Robert I. Zweigler, 2001, Addendum geologic and soils engineering exploration report #4, proposed landslide repair, and multi-unit condominium and town home buildings, Tentative Tract 52928, Lot 1 (condominiums), 17331-17333 Tramonto Drive, Pacific Palisades, California: The J. Byer Group, Inc. consultant rpt., JB 18457-I, to Palisades Landmark LLC, October 2.

[6] Irvine, Jon A., and Robert I. Zweigler, 2001, Addendum geologic and soils engineering exploration report #4 (*sic*), proposed landslide repair, and multi-unit condominium and town home buildings, Tentative Tract 52928, Lot 1 (condominiums), 17331-17333 Tramonto Drive, Pacific Palisades, California: The J. Byer Group, Inc. consultant rpt., JB 18457-I, to Palisades Landmark LLC, December 12.

28

1.2 PUBLISHED REFERENCES

References that are relevant in terms of the limited scope of this critique include the following.

Bruington, A.E., 1971, Hydrology Manual: Hydraulic Div., L.A. County Flood Control District, December.

Campbell, Russel H., 1975, Soil slips, debris flows, and rainstorms in the Santa Monica Mountains and vicinity, southern California: U.S. Geol. Survey Prof. Paper 851.

Dibblee, Thomas W., Jr., 1992, Geologic map of the Topanga and Canoga Park (South ½) quadrangles, Los Angeles County, California: Dibblee Geological Foundation Map #DF-35.

Hoots, H. W., 1934, Geology of the eastern part of the Santa Monica Mountains, Los Angeles County, California: U.S. Geol. Survey Prof. Paper 165C

Hunt, Roy E., 1986, Geotechnical engineering analysis and evaluation: McGraw-Hill Book Co., NY, etc., 729 pp.

Lambe, T. William, and Robert v. Whitman, 1979, Soil Mechanics, SI Version: John Wiley & Sons, Inc., NY, 553 pp.

McGill, John T., 1989, Geologic maps of the Pacific Palisades area, City of Los Angeles, California: U.S. Geol. Survey Misc. Investigation Series Map I-1828.

Michael, E.D., 2002, Reducing the mudflow risk: AEG NEWS, Program with Abstract, 2002 Annual Meeting, v. 45, p. 77, July 28

Rutledge, Philip, and James P. Gould, 1959, Final report, Pacific Palisades landslide study: Moran, Proctor, Mueser & Rutledge consult. rpt. for State of Calif. Dept. Pub. Works, July.

2.0 PLC PROJECT DESCRIPTION

The PLC Project involves the removal of two condominium structures of the original Ocean Woods Estate development and the construction of four new ones according to three of development alternatives of 50, 61, and 102 units. Each of these alternatives includes development in the western part of the property where two of the structures would be located in an area that presently is affected by the active Revello Drive landslide. The primary focus of this critique is the issue of the feasibility of the repair of that landslide. 29

2.1 PLC PROJECT DEVELOPMENT AND THE CEQA PROCESS

The fundamental purpose of the CEQA process is to predict the kind and extent of environmental impacts of a particular development other than single-family residences and certain other categorically exempt projects. Since such impacts can vary, alternatives to the development of a particular property commonly are considered. Intrinsic to such consideration is a sort of balancing between the levels of impact, the costs to achieve it, and the developed value of the property. In the case of the PLC Project, this is especially a problem because it involves remediation of the Revello Drive landslide. 30

2.1.1 Conceptual Character of the EIR Process

It is important to understand that the EIR process considers developments only conceptually. Although there may well be actual grading plans the developer is considering, such plans are not normally included in the DEIR, nor, generally, are detailed grading plans necessary for purposes of environmental review. This is because in the most cases, whatever grading is required has been considered by the developer at least in broad terms and found to be economically feasible. However, problems during actual construction arise that cannot be foreseen at the conceptual stage. The seriousness of such problems varies directly with the magnitude of the development and the extent to which some remedial work is required. 31

The DEIR describes a development plan presumably devised by the PLC Project geotechnical consultant, JBG. This plan describes in general terms, the extensive remedial work necessary to eliminate the risk presented by the Revello Drive landslide. Fundamentally, it postulates: [i] three lines of soldier piles along tract boundaries adjacent to the landslide mass; [ii] removal of landslide debris within those lines of soldier piles; [iii] grading the surface exposed below the debris to receive fill compacted so as to be suitable, generally, to bear foundation loads of normal wall footings; [iv] importation of the fill and its compaction.

Less clear is the relationship of this remedial work to the G.H. Palmer (GHP) Project immediately south of the westernmost 240 feet of the PLC Project. The GHP Project has received approval for a 21 unit

Revello Drive landslide at its toe. Consequently, massive excavation will be necessary not only to eliminate the slide debris but also to remove much underlying bedrock in order to provide automobile parking space. Specific plans or other indications of how the PLC and GHP projects are to interact during construction are not addressed in the DEIR or its appendices. 31

2.1.2 50-Unit Alternative

32

Consistent with CEQA requirements, the DEIR considers several alternatives for development. Among these, a "... 50-Unit Planned Unit Development (PUD) Alternative, (Alternative C)..." has been selected as "... environmentally superior ..." (DEIR, p. 292).

2.1.2.1 Hauling in Support of Proposed Grading

Grading for Alternative C would require 30,000 cubic yards (cy) of cut, 5,000 cy of fill, the export of 100,000 cy, and the import of 75,000 cy of fill for landslide repair..." (DEIR, p. 280). These data presumably mean first that 100,000 cy would be excavated including, 30,000 cy for structural cuts to make room for various buildings and 70,000 cy to remove landslide debris, and exported to some staging area. Second, 75,000 cy of this excavated material, probably having been reworked at some staging area, would be imported, 5,000 cy of which would be used for local structural fill(s) and 70,000 cy to replace the volume of the excavated landslide debris. This would leave a balance of 25,000 cy off site.

It is asserted that the grading would require exportation of 3,500 cy per day and importation of 2,500 cy per day. Furthermore, the hauling would occur during the 7-hour period from 9AM to 5PM on weekdays and would require transport probably along Pacific Coast Highway and the Santa Monica Freeway to one or more of several landfills. Finally, the hauling is to be done with 10-wheel dump trucks (DEIR, p. 219) capable of carrying 14 cubic yards (DEIR, p. 219, footnote 15).

2.1.2.2 General Plan of Stabilization - Revello Landslide Area

Reference [1, pp. 18 - 19] indicates that stabilization of Revello Drive landslide debris within the area of the PLC Project requires the installation of at least the northern (upslope) line of soldier piles and the western line as well prior to excavation of the landslide debris. It appears that the southern line would be of less concern because of a deep depression there due to secondary landsliding within the main mass largely or entirely within the GHP property. Eventually, the lower line of piles would be required "... to support the future compacted fill along the downslope property line." However, that lower line is the northern boundary of the GHP development that will have retaining walls "... which will be the full height of the slide" [*ibid.*].

The fill that is to replace the landslide debris is to be installed within the upper and lower lines of soldier piles at some depth below the existing slide surface. Figure 1 indicates relationships of the PLC Project to the Revello Drive landslide. The area of the debris mass within the PLC boundaries is roughly 40,000 square feet (sf) based upon the small-scale geologic map included in the DEIR documents (DEIR, Fig. IV.D-1; DEIR App I. [1]). The side contacts of the debris mass have been found by subsurface exploration to be nearly vertical [2, p. 2, Item 3]. Since the postulated volume of debris is 70,000 cubic yards (cy), i.e., 1,890,000 cubic feet, the average depth of the slide debris must be about 47 feet. However, "... removal depths could be up to 60 feet..." [1, p. 20].

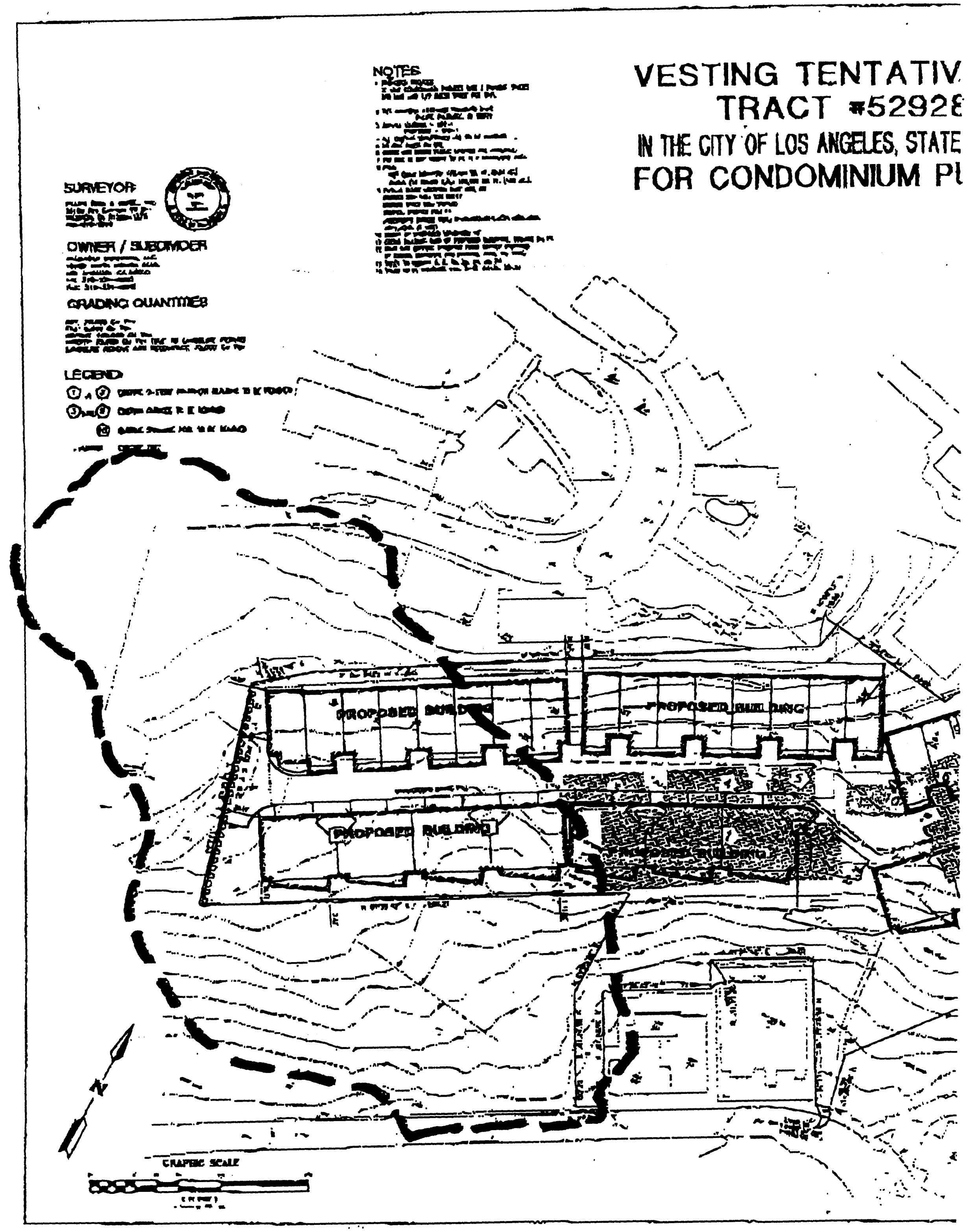

Figure 1. Revello Drive Landslide in Relation to the PLC Project.
The dashed line is the contact of the Revello Drive landslide from DEIR Figure IV.D-1. It is superimposed over DEIR Figure III-1. The slide movement is to the southeast.

subsurface drainage is to be controlled through the use of "chimney" drains, and at least three continuous drains located beneath replacement fill transverse to the slope. The lateral rains are to be constructed of ¾-inch crushed gavel along the pilings where they penetrate bedrock at the base of the landslide. The chimney drains are to be masses of ¾-inch crushed gravel in the spaces between the piles [3, attached untitled diagram]. 32

2.1.2.3 Surface Drainage

Drainage from Alternative C will be directed partly to Tramonto Dive and partly to Castellammare Drive. Peak flows generated from runoff are estimated through use of a computer program (DEIR, App. F). Based upon this program, it is asserted that runoffs from the completed project will be only slightly different from those that existed prior to development of the Revello Drive landslide. In particular, peak flows to Castellammare Drive would be 13.0 cubic feet per second (cfs) from the completed Alternative C development compared to 13.4 cfs prior to landsliding (DEIR, p. 161, Figs. IV.E-1; IV-E.2). As part of the plan for controlling peak flow, a catch basin is to be located on Castellammare Drive. 33

3.0 GEOLOGICAL CONTEXT

The geologic characteristics of the Pacific Palisades generally, and in particular that of the area of the PLC Project, while not directly related to the manner in which it will modify the local area, nevertheless, has some relevance when considering overall environmental impact. In a word, parts of Pacific Palisades, including that of the PLC Project, are especially prone to landsliding.

Hoots (1934) was the first to map the geology of the Pacific Palisades area in significant detail as part of his study of the eastern Santa Monica Mountains. He recognized the major geological formations of the area, much of its geomorphic character, and many of the faults. However, the primary purpose of his work was an evaluation of economic potential with special emphasis on the occurrence of structures that might be petroleum reservoirs. He was either unaware of landslides, or did not consider them relevant. He mapped as bedrock many areas in Pacific Palisades now known to be underlain by landslide debris. 34

The surge in property development beginning in the latter half of the 1940s had two important geological aspects in terms of landsliding. First, building was undertaken in hillside areas without proper consideration for potential or existing problems of slope instability, and this began to result in major property losses. Second, increased residential development produced a net increase in ground water that has initiated landslides in some instances and reactivated masses of pre-historic landslide debris in others.

Such conditions soon became especially apparent in the seaward-facing slopes and adjacent canyons of the Pacific Palisades. Partly as a result of this and also a question of improvements along the Roosevelt Highway, now Pacific Coast Highway, public concern led to the first comprehensive study of landsliding in the Palisades as well as elsewhere along the shores of Santa Monica Bay by Rutledge and Gould (1959). They recognized many landslides previously unknown, but they did not consider the slope now underlain by the Revello Drive landslide as one, even though the topography then suggested it (*op. cit.*, Pl. L-10).

John T. McGill of the U.S. Geological Survey began his work on landslides in the Pacific Palisades about the same time as Rutledge and Gould began theirs. He produced a number of maps beginning with his 1959 preliminary map of landslides. That was supplemented with two others of increasing detail and culminated in a final map (McGill, 1989) which is unquestionably the best source of data for the Pacific Palisades to date not only for landslides, but geologic interpretation in general.

34

McGill's work, which carefully distinguishes pre-historic and historic landslides, leaves no doubt that current landsliding in the Pacific Palisades is generally a result of ground water recharge due to the direct infiltration of: [i] rain, [ii] residential irrigation, and [iii] local artificially concentrated surface runoff. Knowledge not only of how ground water occurs, *i.e.*, where it is located and how it gets there, is necessary for slope stability analysis, because its presence can have a significant effect on the static forces operating in a slope. Through the principle of effective stress, ground water reduces the weight of earth materials at particular elevations in the slope and consequently the forces they otherwise would exert at such locations. Furthermore, as a result of groundwater movement, a seepage force is created that also can affect stability. There are no studies of ground water in the Pacific Palisades that could be used as one basis for predicting slope stability.

4.0 SUMMARY GEOTECHNICAL ANALYSIS - PROPOSED PLC PROJECT

The following analysis of the of the reviewed DEIR documents summarizes the more important geotechnical aspects of the PLC Project. Generally, in such an analysis of DEIR documents, it is very important to distinguish project geotechnical feasibility from related environmental impacts. In all instances, the issue is whether the technical problems are of such a character that actual development would result in impacts significantly different from those the documents describe. Feasibility, or lack thereof, are technical matters either of which may have an important impact. However, the significance of the impact is an administrative matter.

35

4.1 SLOPE STABILITY

Slope stability analysis as routinely presented in geotechnical engineering reports is essentially a study in statics, *i.e*, the branch of mechanics that deals with bodies at rest and hence in equilibrium, meaning that the sum of the forces is zero. Generally, such an analysis does not go beyond two dimensions. Rather, it is directed to one or more "critical" surfaces of failure shown in cross-section as a sort of worst-case scenario. The goal is the calculation of the ratio of forces tending to resist gravitational movement to forces tending to cause gravitational movement. That ratio is called the "safety factor." Even in the case of the seismic force, which is dynamic, the time-honored pseudo-static model for analyzing slope stability substitutes a static force for the seismic effect.

36

A technique for considering the dynamic effects of seismic activity on slopes called "Newmark displacement analysis" now is being considered by public agencies as a building code requirement. If Newmark analysis is adopted before building permits are issued for the PLC Project, an entirely different approach to the analysis of pile-supported slopes may be necessary.

Estimates of safety factors by JBG are based the REAME program [1, Calc. Sheets 1 - 37]. Forces exerted on pilings that consequently affect safety factors have been calculated using something called the Mononobe-Okabe method [1, Calc. Sheets 38, 39, 41, 42]. Since these sheets are not accompanied by at least free-body force diagrams upon which such programs must be based, the validity of their use cannot be determined. Although it is reasonable to assume that the programs produce correct results, there is no basis for an assumption that JBG has applied variables that correctly reflect field conditions. In computer parlance, this is referred to as the GIGO principle: garbage in - garbage out. 36

4.1.1 Surficial Stability

A similar concern to be evaluated in geotechnical documents submitted in support of the application for the building permit in the City of Los Angeles is that of "surficial stability," *i.e.*, the stability of surficial materials in slopes. It is standard practice to analyze this problem in terms of "infinite slope analysis" an example of which is given in Appendix I of the DEIR [1. Calc. Sheet 40]. The primary objection to such an analysis is the common use of a cohesion that is too high. In the case at hand, a value of 400 pounds per square (psf) for surficial materials at a depth of 3 feet is utilized, although no evidence is presented justifying such a high value. Possibly, JBG has assumed that the results of shear tests of "slide plane" material or "future compacted fill" [1, Calc. Sheets 4 and 5] are representative of the cohesive strength of the natural local surficial materials. Nevertheless, no basis for this is presented. In fact, the standard "shear-box" test commonly used in soils engineering laboratories is incapable of producing accurate results at normal loads less than about 1000 psf. Therefore, the linearity of the shear stress - normal stress envelope below that level of stress is merely assumed. 37

Authorities recognize that cohesion should be much lower than the intercept value derived from the standard shear-box test. As a matter of fact, Campbell (1975, p.19, footnote) indicates that cohesion in such analyses should be zero. Geotechnical engineers commonly understand that unless a high cohesion is utilized in infinite slope analysis, a safety factor significantly less than 1.5 almost invariably is the result. This is about as politically incorrect as the geotechnical engineer can get when dealing with public agencies. Beyond this, as recently discussed by Michael (2002), the standard laboratory shear test in which friction angle and cohesion are determined by the addition of load does not reflect field conditions where failure results from the reduction of load through the principle of effective stress. This implies different and lower real values for friction angle and cohesion.

4.1.2 Soldier Pile Mechanism

The steps to be taken in developing the part of the PLC Project affected by the Revello Drive landslide include: [i] drilling the borings and installing cast-in-place piles; [ii] excavating debris in the PLC property temporarily leaving the piles free-standing; [iii] installation of chimney drains between piles as compacted replacement fill is added. 38

4.1.2.1 Bridging

Soldier piles work by the bridging effect that occurs, hopefully, when the retained material begins to be forced between them. Fundamentally, bridging causes the material to become denser, thus increasing its frictional strength. Whether bridging will occur between the proposed pilings that are to support landslide debris depends upon the mechanical characteristics of the materials as well as the pile spacing which in this case is

10 feet on centers [1, p. 18], and initially ". . assumed (to be) fixed at 10 feet into bedrock below the slide debris..." [*op. cit.*, p. 30].

4.1.2.2 Stress on Free-standing Pile

The design loads that the landslide debris will exert on pilings 1 - 30 prior to installation of the replacement fill will range from 145 to 175 kips (1 kip = 1,000 pounds) [1, p. 20; 2, p. 7]. An embedment depth of "... 20 feet into bedrock below the 1½: 1 setback plane..." [3, Item 1, Item 11, p. 2] apparently is meant to apply to pilings 31-40 which will support bedrock [2, Item 11, p. 7]. However, based upon Cross-sections A-A, B-B, and C-C [2], all of which pass through the landslide debris, it appears that pilings 1 - 30 will be about 60 feet deep and extend a revised 20 feet into bedrock as well. A fair model of the geometry these data appear to represent is shown in Figure 2.

39

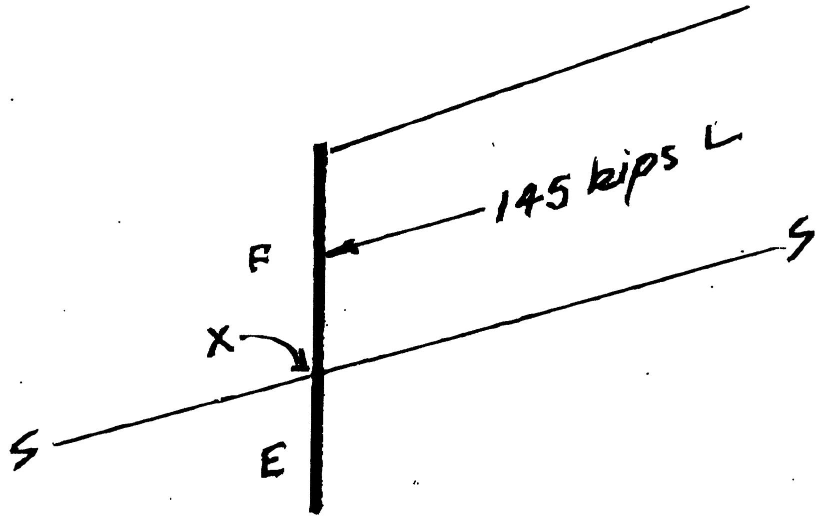

Figure 2. Pile-supported Slope Cross-section after Excavation of Slide Debris. - Dimension F is in the range of 40 - 60 feet. Dimension E is 20 feet. S-S represents the slide surface and X the point of fixity. The load, L, 145 kips, is due to the debris when there is saturation. It is exerted through the centroid of the pile column above S-S rather than at an elevation 1/3 of the column height above the slide surface, because the force is evenly distributed along the pile length and is not a function of increased pressure with depth as in the case for active or passive pressure calculations.

4.2.3 Hydrogeologic Aspect

It has been the practice for many years in the field of geotechnical engineering to routinely record the occurrence of ground water in exploratory borings and to assume from such observations the manner in which ground water will occur in the future. In certain cases, such an assumption may be valid, but in most it is not. It is a matter of common knowledge that the occurrence of ground water is in part a function of rainfall and in hillside areas of southern California at least, especially important. Nevertheless, the exigencies of property development are generally such that a protracted study of groundwater occurrence is seldom undertaken, and that is true in the case of the PLC Project.

40

A proper analysis of the slope in which the PLC Project is to be located would be based upon a record of ground-water levels over a period of years sufficient to determine, through the construction of ground-water contour maps how ground water actually occurs with respect to time. It was the lack of such knowledge that resulted in the extensive damage from the Revello Drive landslide. Scattered observations of seepage in borings in the local area over many years is certainly not adequate for use in slope stability analysis there.

City officials, in recognizing this problem, first requested clarification regarding "... highest acceptable ground water levels..." and how such levels were to be verified. In response, JBG stated that grading should not commence until the summer when "... it can be demonstrated that ground water is not present above the lower slide plane..." and that the "... water level can be demonstrated by logging the shoring pile excavations..." [2, Item 5, p. 4; 6, Item 1, p. 2]. Thereafter, in recognizing that it "... "may not be possible to de-water the off-site properties..." JBG indicated that calculation of the safety factor would produce a value greater than the required standard of 1.5 even if ground water rose to the top of the pilings [3, Item 6, p. 4]. Apparently, this assumes a resisting force due to the presence of the replacement fill.

40

4.3 HYDROLOGY

The hydrologic analysis presented in the DEIR is based upon a computer program which apparently solves some form of the rational method for calculating peak flows. Presumably, this is the Los Angeles County Flood Control District (LACFCD) capital storm hydrology method set out by Bruington (1971). It appears that the LACFCD's "K" rainfall zone and its related rainfall intensities for various storm frequencies have been accepted as controlling (DEIR, App. F). If that is the case, the LACFCD runoff coefficient curves probably have been used. As in the case of slope stability analysis, a computer program has been utilized, but the underlying rationale is not presented.

41

5.0 CONCLUSIONS

The geotechnical environmental impacts of the PLC Project during its development are to a great extent temporary, but very significant. The impact of hauling has been greatly under estimated, and in terms of safety it is inadvisable to attempt slope stabilization as currently planned until additional data are developed..

42

5.1 HAULING IMPACT

The estimated hauling period of 120 days (DEIR, p. 219) is far too low. The assumption that there is available a 10-wheel truck with a 14-cy capacity (*ibid*, footnote 15) is incorrect according to earth-moving contractors with whom I have consulted. They unequivocally assert that there is no such thing as 10-wheel truck with a 14 cy capacity. Furthermore, it appears that in estimating the hauling period for the PLC Project no consideration is given to "break-out" which is the increase in volume that occurs when relatively dense earth materials are excavated.

43

Hauling contractors commonly employ a break-out factor of 1.2 - 1.3 for bedrock materials. For landslide debris, a fairer break-out factor would be perhaps 1.15. Assuming a factor of 1.25 for the 30,000 cy of cut material, that export volume would be would be 37,500 cy. Similarly, allowing for a factor of 1.15 for the 70,000 cy of landslide debris, that export volume would be 80,500 cy. Consequently, the total export volume would be 118,000 cy. Import of 5,000 cy for structural fills, and 70,000 cy for debris-replacement fill would not require a break-out factor because those materials would be reworked loose material. Therefore, the total volume of export and import would be 193,500 cy. Furthermore, the project is such that the export and import operations could not be done simultaneously.

43

It is inconceivable that the massive self-loading scraper could be used to move earth materials for the PLC Project because of the residential character of the local area. All hauling will require the use of 10-wheel trucks that have a capacity of about 7.5 cy. This means that 25,800 trips would be required, a "trip," being the travel required to move from the staging area to the site and return. The different rates at which materials would be imported and exported are difficult to estimate because of the number of trucks the contractor could employ, the haul distance, and the loading and unloading operations which require different times for export loading and import dumping. However, assuming a favorable staging area for storage and blending on Los Liones Drive in the vacant area behind Fire Station 23, an average trip probably would be in the range of 10 to 15 minutes, for an efficient operator. If hauling had to be along thoroughfares such as Pacific Coast Highway or a freeway, additional time would be required to cover the load to prevent dust loss during transit.

Assuming then an average trip time of 12.5 minutes, the total haul time required would be 322,500 minutes, or 5,375 hours. Finally, assuming a 35-hour work-week for hauling, and a 50-week work-year, hauling for the PLC Project would require 3.07 years. Considering unforeseen conditions due to such conditions as breakdown or adverse weather, it is reasonable to expect considerably more than 3 years to accomplish the hauling. Even an overly optimistic 5-minute trip time would require 1.23 years.

5.2 SLOPE REMEDIATION IMPACT

Slope stability analyses presented in support of the DEIR are unsatisfactory for two reasons. First, it is virtually certain that the line of soldier piles, and particularly the northern line, will fail when the existing adjacent landslide debris is removed. This is because the depth of embedment in bedrock below the slide mass of 20 feet is too shallow. Second, the use of a design fill cohesive strength of 400 psf does not appear to be justified. Third, the effective stress that may act on the retained landslide debris has not be properly evaluated.

44

5.2.1 Questionable Pile Resisting Force

A serious question to be examined is whether the free-standing plies along the northern PLC Project boundary will stand during the period when the landslide debris has been removed and replacement fill not yet installed. The over-turning moment represented by Figure 1 needs to be considered. The initial question in this regard seems to be whether resistance offered to the buried 20-foot pile section due to the strength of the bedrock is greater than over-turning moment due to the 145- kip per foot of landslide debris force acting at the centroid of the exposed pile section.

45

The period during which JBG assumes the free-standing condition would exist is unreasonably short. It is proposed that excavation should "... not commence until the summer and it can be demonstrated that ground water is not present ..." [2, p. 4]. However, it is well established that highest ground water levels lag as much as several months after recharging rains and may be as late as August. More to the point, however, if the grading is to take as much as three years, it is virtually certain that temporary slopes within the grading area, and particularly the temporary free-standing pile-supported slopes, will have to exist during through at least two and perhaps three storm seasons. Moreover, the JBG stability analyses fail to take into account seepage force which may add significantly to the 145-kip static load that is utilized, and no data on the bridging capability of the landslide debris. 45

5.2.2 Questionable Value for Cohesion

It appears that a certain amount of guesswork has been employed to arrive at a design cohesion of 400 pounds per square foot (psf) for the proposed compacted fill that will replace the landslide debris as shown along sections A, B, and C [1, Cal. Sheets 18 - 25]. Such a value is necessarily a matter of judgment for which the geotechnical engineer rather than the engineering geologist, but some evidence to support such judgment should be presented. Generally, the strength of the compacted fill will be dependent upon the manner in which it its blended and the resulting equivalent soil group. The question is: can the excavated landslide debris be blended to a texture which, when properly compacted will have a cohesive strength of 400 psf? None of the reviewed documents addresses this issue.

Presumably, the 400-psf value is based upon Shear Test Diagram #5 [1, App. I]. However, the assumption that a single test from a bulk sample of slide debris can be representative of that 70,000 cy strains the imagination. On the other hand, the few data presented in the logs of borings [*ibid.*] indicate that the granular materials locally derived as landslide debris or reworked colluvium or fill are of the Unified Soil Classification soil group ML, *i.e.*, " inorganic silts and very fine ands, rock flour, silty or clayey fine sands with slight plasticity" (Lambe and Whitman, 1979, p. 35). According to Hunt (1986, p. 212, Table 5.3) typical cohesion for compacted materials of this type range between 190 and 460 psf when saturated. As previously noted, the linearity of the typical shear test envelope as routinely conducted in soils engineering laboratories [1, App. 1, Shear Test Diagram #5] is highly questionable at lower normal loads. 46

Based upon the geological data available, it seems fair to say that insufficient work has been done to support engineering judgment that the debris of the Revello Drive landslide is suitable in terms of compacted strength to estimate slope stability as calculated [1, Calculation Sheets 18 - 25].

5.2.3 Questionable Effective Stress Analysis

The principle of effective stress is fundamental in the practice of geotechnical engineering. Briefly, effective stress is the reduced stress subsurface earth materials exert in the presence of water. This reduction is due to the fact the water causes the materials to "weigh" less. Essentially two mechanisms are involved. In one, which is considered in the stability analysis performed for the PLC Project thus far, the loss in weight is due to buoyancy. In the other, the weight is effectively reduced when hydrostatic pressure works against an impermeable surface such as the base of a mass of landslide debris. In this case, the mechanism is much like that of a hydraulic jack so that a force is applied which reduces the weight of the debris and hence the coefficient of friction. 47

The JBG analyses fail to take into account the possibility that, as a result of periodically especially high recharge in Castellammare Mesa, a high piezometric head acting at the base of the retained mass of Revello Drive debris could cause a pressure much greater than that due to pore pressure from simple saturation of the debris mass above the slide surface. In fact, it is well established that with sufficiently high head developed in this manner, frictional resistance can be entirely eliminated. 47

5.3 FLOODING

Regardless of the validity of the manner in which the software program used to model runoff in the PLC Project [DEIR, App. F], the data are based strictly upon the assumption of vertical rainfall. However, it is well established that local rains commonly are wind-driven and fall at some angle less than vertical. In such cases, the presence of building walls and other impermeable vertical surfaces has the effect of increasing the effective catchment area. The PLC Project has a number of such vertical surfaces, and the peak flows to be expected along Castellammare Drive under especially intense wind-drive rains will be greater than those currently calculated. 48

There are no data from which the direction or angle of rain approach can be estimated. In cases where it appears vertical surfaces may be a factor in rain catchment, it is appropriate to apply some factor to increase the calculated peak flows.

E. D. Michael

E.D. Michael

3/31/03