ALAN KROPP
& ASSOCIATES, INC.

January 9, 2007
2392-11A, L-27716

Mr. Mark Kelley
Miller Brown Dannis
71 Stevenson Street, 19th Floor
San Francisco, CA 94105

RE: Phase *2* Geotechnical and Geologic Hazards Study
Riverside Elementary School 1300 Amador Street San
Pablo, California

Dear Mr. Kelley:

This report presents the results of our Phase 2 Geotechnical and Geologic Hazards Study for Riverside Elementary School in San Pablo, California. We provided our services in general accordance with our proposal dated June 12,2006. The location of Riverside Elementary School is indicated on the Vicinity Map, Figure 1.

INTRODUCTION

In May 2005, we conducted a Phase I Geotechnical and Geologic Hazards Study for Riverside Elementary School. The purpose of our Phase I study was to explore subsurface conditions at one location and develop preliminary conclusions regarding the potential for significant geologic hazards and/or geotechnical concerns at the site. Based on a review of available data and a single 50-foot-deep exploratory boring, we concluded that earthquake-induced soil liquefaction, lateral spreading and/or landsliding could potentially present a serious hazard to occupied structures.

In a subsequent meeting with a representative of your firm and Mr. William Savidge, District Engineering Officer for the West Contra Costa Unified School District (District), we were asked to prepare a proposal to conduct supplemental Phase 2 investigations to further characterize and evaluate these concerns. As outlined in our June 12, 2006 proposal, the scope of the Phase 2 services described herein included:

- Advancing six. cone penetration test (CPT) probes and collecting samples;
- Performing geotechnical laboratory testing;
- Evaluating potential earthquake-induced hazards;
- Consulting with the District; and
- Preparing this report.

We emphasize that this report is not intended as a design-level study and that additional subsurface explorations would be needed prior to our providing recommendations for the mitigation of geologic hazards and/or geotechnical concerns.

INVESTIGATION

Cone Penetration Testing

On October 25 and 26, 2006, we explored subsurface conditions by advancing six CPT probes to depths between 48 and 70 feet at the approximate locations shown on the Site Plan, Figure 2. Gregg In Situ, Inc. (Gregg) of Martinez, California performed the CPT probes under the observation of Ms. Dona Mann, C.E., Project Engineer for Alan Kropp & Associates.

The CPT method involves pushing a 2-inch-diameter conical probe into the ground using a hydraulic ram system. The CPT probe is equipped with sensors that produce a continuous record of tip resistance, sleeve friction and pore pressure as the cone is advanced. During cone advancement real-time data obtained from the cone sensors are displayed on a computer monitor together with preliminary interpretive logs of soil type. During cone advancement Ms. Mann monitored the real-time cone data, selected depths at which subsurface samples would later be obtained and determined the depths to which the probes would extend.

Groundwater levels were evaluated indirectly in the CPTs by conducting intermittent pore pressure measurements. Pore pressures were recorded until the excess pore pressure caused by the penetration of the cone was observed to have dissipated, allowing a stabilized groundwater pressure measurement to be made. This pressure measurement was then used to indirectly evaluate groundwater depth. Groundwater levels were also read directly with a water meter within the probe holes after the probing was complete and a 1'A-inch-diameter, piston-type sampler was used to obtain relatively undisturbed soil samples at selected depths. Following the groundwater measurements and subsequent soil sampling activities, the holes were grouted closed in accordance with Contra Costa County permit regulations.

Gregg's interpretive logs of the CPTs are attached in Appendix A. Also included in Appendix A is information obtained from Gregg describing their CPT equipment and the correlative methods they used in developing their interpretive logs. The attached CPT logs prepared by Gregg In Situ, Inc. represent their interpretation of the subsurface conditions at the approximate CPT locations indicated on the Site Plan (Figure 2) on the particular date designated on the logs. We note that the soil behavior type (SET) reported on the logs presented in Appendix A has not been corrected to account for overburden stress. The CPT locations and elevations indicated on the attached materials were determined by measuring from existing fences and buildings shown on the project plans and should be considered approximate.

Laboratory Analyses

Following the CPT operations, the samples obtained were transported to our laboratory for subsequent analyses. The following geotechnical laboratory tests were performed:

- Water content per ASTM Test Designation D-2216,
- Atterberg Limits per ASTM Test Designation D-4318,
- Percent passing the No. 200 sieve per ASTM Test Designation D-l 140.

SITE CONDITIONS

Surface Conditions

The school property is situated on gently sloping terrain near the base of the East Ba> hills. In general, surface elevations at the site (USGS, 1980) vary from about 130 feet above sea level at the north end of the school property to about 120 feet at the south end. Wildcat Creek flows within an approximately 20- to 25-foot-deep incised channel that borders the eastern and southern sides of the property. Adjacent creek banks along the eastern and southern site margins slope downward at inclinations which are locally steeper than 1:1 (horizontal to vertical).

The approximate relationship between the existing school buildings and the top of the Wildcat Creek channel is shown on the Site Plan, Figure 2. As can be seen on Figure 2, several of the existing school buildings (identified as Buildings B, D and E) are in close proximity to the top of the creek banks.

In the site vicinity, the flow of Wildcat Creek trends from northeast to southwest. We understand that a building that previously existed between Buildings D and E was destroyed by flooding that occurred as a result of plugging of the culvert beneath Amador Street and Highway 80 (southwest of the site). During this and our Phase 1 study, the creek banks adjacent to the site were inaccessible to us due to physical constraints that included fences, dense vegetation and steep terrain.

Soil Conditions

Based on the interpreted soil behavior types reported by Gregg, the CPTs encountered mostly clay and silt mixtures with inter-bedded layers of sand mixtures. These results are generally consistent with the results of our previous test boring. Above about 20 feet, the soils encountered generally consisted of loose to medium-dense sands and stiff clays. Below about 20 feet, the soils encountered included soft to firm clay with some silt mixtures and inter-bedded layers of stiff clays and medium-dense sands.

Our laboratory test program was primarily directed at an evaluation of liquefaction and related hazards, which are most often correlated to moisture content, Liquid Limit, Plasticity Index, and grain size. The tests were performed on samples of fine-grained material judged to have a low plasticity and therefore a possible susceptibility to liquefaction. The results of our laboratory analyses are summarized in the following table:

Table 1. Laboratory Test Results Summary

CPT#	Sample Depth (feet)	Moisture Content (%)	Liquid Limit (LL)	Plasticity Index (PI)	Percent Passing #200 Sieve
3	25.5	23	29	13	56
3	32.5	27	31	13	60
3	38	29	32	16	68
3	53.5	25	32	14	45
4	25	28	34	18	75
5	20.5	21	35	17	56
5	26.5	25	35	17	70
5	45.5	25	33	17	51
6	21	27	32	14	79
6	26	23	34	17	52
6	32.5	25	37	19	69

Groundwater Conditions

The pore pressure dissipation records from the CPTs indicate the following groundwater depths (in feet below the ground surface):

Table 2. Groundwater Depths from CPT Dissipation Records

CPT#	**Groundwater Depth (feet below ground surface)**
1	**not recorded**
2	**14.1**
3	**21.7**
4	**22.7**
5	**22.3**
6	**20.2**

Direct groundwater measurements made from within the CPT holes are presented below.

Table 3. Groundwater Depths from Direct Measurements

CPT#	Groundwater Depth (feet below ground surface)	Time of Measurement
1	not measured	—
2	33.5	Immediately after drilling
3	24.5	Immediately after drilling
4	23.8	4 hours after drilling
5	23.0	3.5 hours after drilling
6	18.0	3 hours after drilling

We note that the borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions and that fluctuations in the groundwater level likely occur due to changes in seasons, variations in rainfall and other factors.

GEOTECHNICAL ANALYSES

Liquefaction Susceptibility

As an initial step, we evaluated whether the CPTs encountered soils that would be considered susceptible to liquefaction. In CPT-based analyses, soil types (e.g. sand, silt, clay) are interpreted based on measured values of cone tip resistance, sleeve friction and pore pressure. Under the procedures outlined by Robertson and Wride (1998), "soil behavior types" are obtained using a logarithmic plot of normalized friction ratio versus normalized cone resistance. On this plot, the primary boundaries between soil types can be approximated as concentric circles, the radius of which is termed the soil behavior type index, or (more commonly) I_c. In CPT-based liquefaction analyses, 1) soils with an I_c of less than 2.6 are considered predominantly granular in nature and susceptible to liquefaction, and 2) soils with an I_c greater than or equal to 2,6 are classified as too "clay-rich'" to liquefy.

Because the relationship between *lc* and soil type is approximate, it is generally recommended (see Youd and Idriss, 2001) that soils having an l_c greater than or equal to 2.4 be sampled and tested to verify the soil type and evaluate whether the soil is susceptible to liquefaction using other criteria. We utilized the interim recommendations presented in the recent paper by Seed, et al. (2003) to evaluate liquefaction susceptibility of samples of fine-grained soils obtained during our investigation. These interim recommendations correlate liquefaction susceptibility of fine-grained soils to their Plasticity Index (PI), Liquid Limit (LL) and natural water content (w_c) as follows:

- Soils plotting in Zone A (a PI of 12 or less and a LL of 37 or less) with a w_c greater than 80 percent of the LL are considered susceptible to liquefaction;
- Soils plotting in Zone B (a PI of 20 or less, a LL of 37 or less and not within Zone A) with aw_c greater or equal to 85 percent of the LL are considered possibly susceptible to liquefaction; and
- Soils plotting in Zone C (not within Zones A or B) are considered not susceptible to liquefaction.

The interim recommendations presented by Seed, et al. (2003) are intended to apply to: (1) soils having a fines content of at least 20 percent if their PI is greater than 12; or (2) soils having a fines content of at least 35 percent if their PI is less than 12.

Because Ic is not known at the time of our field investigation, we selected to obtain samples in areas we judged to be of possible concern with respect to liquefaction of fine-grained soils. We sampled silty and clayey soils with low cone tip resistance and low sleeve friction (i.e., weak, fine-grained soils with low plasticity). Of the samples obtained, we performed lab tests on the soils we judged could fall into either Zone A or Zone B as discussed above. Of the 11 samples tested, we found none that plotted in Zone A (soils susceptible to liquefaction). All 11 samples tested plotted in Zone B; however, only 3 of the 11 samples had natural water contents high enough to indicate possible liquefaction susceptibility. In addition, our liquefaction analyses calculate I_c values greater than 3.0 at the depths of our tested samples.

Because we found the soil in the areas that we judged to be of possible concern with respect to liquefaction of fine-grained material to be moderately plastic, have low water contents with respect to liquid limits and have high Ic values, we have relied primarily on the "$Ic \geq 2.6$" criteria and have assumed the clay-rich soils at the site will not experience excessive cyclic strength loss or liquefaction type behavior.

Liquefaction Potential

Where potentially liquefiable soils are found to be present, additional steps are needed to evaluate whether groundshaking at the site during a large earthquake will be strong enough to cause liquefaction. Factors that are considered in evaluating whether a susceptible soil will liquefy include pore water pressure, in-place density and the magnitude of groundshaking anticipated for the site.

We evaluated the liquefaction potential at the site using the CPT data and methodology outlined in the summary paper by Youd and Idriss (2001). This method involves assessing the seismic demand on a soil layer, expressed in terms of the cyclic stress ratio (CSR), and comparing this value to the capacity of the soil to resist liquefaction, expressed in terms of the cyclic resistance ratio (CRR). The factor of safety against liquefaction is determined by dividing the CRR by the CSR. Soils having a factor of safety less than or equal to 1.0 are considered liquefiable.

To account for fluctuations in groundwater levels due to variations in rainfall, temperature and other factors, we used a groundwater depth between 11 and 17 feet (below the ground surface) in our liquefaction analyses. Estimates of in-place density were obtained directly from the interpretive CPT data provided by Gregg. Levels of groundshaking used in our analyses were based on an earthquake rnoment magnitude (Mw) of 7.1 with a peak ground acceleration (corresponding to a 10 percent chance exccedence in 50 years level of hazard) of 0.703g. We note that these values were obtained from published data (California Geological Survey, 2003) and not from a site-specific probabilistic seismic hazard:; assessment.

The results of our analyses found that groundshaking at the site during a large earthquake on the Hayward fault would be large enough to induce liquefaction within multiple layers of susceptible soils. As shown on Figures 3 and 4, all of the CPTs, with the exception of CPT-1, encountered a liquefiable layer which varied in thickness between depths of about 11 and 21 feet below the ground surface which could be laterally continuous across the site. In addition, there were other thin layers which were found to liquefy at various depths but do not appear to be continuous across the site. In general, the liquefiable materials encountered consisted of medium-dense silt and sand mixtures.

Interpretive Subsurface Profiles

Figures 3 and 4 present interpretive subsurface profiles (cross sections) we developed based on: (1) our detailed review of the CPTs and geotechnical laboratory test data, and (2) the results of our liquefaction susceptibility analysis. The cross sections indicate the soil types, the groundwater level used in our analyses and layers (in red) that our analyses indicate are potentially liquefiable.

We recognize that considerable variation may exist between individual CPTs, as would be anticipated for a depositional environment. The cross sections we developed are intended to be a reasonable interpretation of the subsurface conditions based on the available data, and not as either a "best-case" or "worst-case" condition. We also note that the groundwater level assumed in the analyses played a critical role in determining what soils would liquefy. For example in CPTs 4 and 5, if the ground water level rose to 12 feet below the ground surface, the medium-dense sand layer encountered at about that depth would also liquefy.

Liquefaction Effects

Surface manifestations of liquefaction include settlement, bearing capacity failure, sand boils, and lateral spreading. Any of these effects, if severe enough, have the potential to cause damage to structures and other site improvements. In our judgment, lateral spreading is the most serious concern from a hazard perspective. The results of our evaluations are summarized below.

Five out of the six CPTs encountered a potentially liquefiable layer which varied in thickness between depths of about 11 and 21 feet. This layer could be laterally continuous across the site and it appears to us possible and perhaps likely that this liquefiable layer would provide a natural plane of weakness over which lateral movements toward the 25-foot-deep creek channel could occur. On this basis, we judge that the site has a relatively high potential for lateral spreading and/or earthquake-induced landsliding toward this open face. This type of failure could be progressive and affect an area extending perhaps several hundreds of feet back from the creek channel.

We also used the CPT data and the methods outlined in Zhang, Robertson, Brachman (2002) to estimate the magnitude of liquefaction-induced settlement at each of the CPT locations. As shown below on Table 4, we calculate between about 0.3 inches and 2.5 inches of dynamic, compressional settlement could occur within the potentially liquefiable layers as a result of a large seismic event.

Table 4, Liquefaction-Induced Settlemement

CPT#	Total Settlement (inches) (Zhang, Robertson, Bradunaa, 2002).
1	0.29
2	0.33
3	0.72
4	0.63
5	0.49
6	2.47

We note that if water levels were to rise above the groundwater level used in our analyses, settlements in CPTs 4 and 5 could increase to 0.85 inches and 0.91 inches, respectively. The amount of differential settlement that may occur over a specified distance (for example, between adjacent rows of building foundations) cannot be calculated directly using the existing data and available methods. However it is not uncommon to assume that differential settlements between adjacent foundations could be on the order of one-half the total settlement.

Based on the available data, it also appears to us that there is the potential that soil could erupt from liquefied layers where they intersect the free face of the creek channel and/or through ground fissures caused by lateral spreading. By either mechanism, the loss of soil has the potential to significantly increase the magnitude of total and differential surface settlements.

Slope **Stability**

The soils encountered at the site below about 15 to 20 feet were generally found to be weak and potentially susceptible to slope failure under seismic loading. We used the computer program XSTABL to check creek bank stability under static and seismic conditions. Two sections were analyzed as shown on Figures 5 and 6. Initially, the simplified section on Figure 5 was analyzed with the generalized slope geometry and subsurface conditions shown. Secondly, Section C-C' was analyzed using more detailed slope geometry and subsurface data.

We conducted a total stress analysis using undrained shear strengths (S_u). The strengths used in our analyses were obtained by converting the CPT tip resistance values to an equivalent standard penetration test (SPT) blow count. By using recognized SPT correlations and engineering judgment, we estimated S_u values for each layer. The S_u values used in our analyses are presented on Figures 5 and 6.

The parameters used in our slope stability analyses are generally summarized below:

- A 25-foot-deep creek channel with 1:1 side slopes;
- A liquefiable layer (1 to 4 feet thick) between 14 and 20 feet deep with a seismic undrained shear strength of 56 percent of the estimated static S_u value;
- A 15-foot-decp groundwater level within the slope and corresponding groundwater levels within the creek channel varying from 0 to 7 feet above the channel bottom;
- No building surcharge loads;
- A seismic horizontal coefficient equal to 0.35g (approximately half of the PGA).

The critical failure surfaces and associated factors of safety (F.S.) are presented on Figures 5 and 6. Statically, the sections analyzed appear to be stable with factors of safety ranging between 1.3 and 2.0. Seismically, the sections analyzed appear to be unstable with factors of safety ranging between 0.5 and 0.6, As shown on Figures 5 and 6, the failure surfaces all fail through the soft to firm clay encountered below about 20 feet at the site.

The weak soils encountered below about 20 feet generally control the slope stability at the site. It is also notable that the calculated seismic factors of safety are less than one regardless of whether or not there is a liquefiable layer within the slope.

CONCLUSIONS

General

The results of our slope stability analyses generally indicate that the site is stable under static (i.e. non-earthquake) conditions. We are not aware of any indications that previous landslide-type slope movements have affected any of the school buildings since they were constructed. In the absence of significant creek bank erosion or extreme hydrologic events (such as massive flooding) it is our opinion that the overall risk of landslide movements affecting the school buildings under static, non-earthquake conditions can be considered generally low.

Based on the Phase 2 investigation results, we judge that seismically-induced landsliding presents a serious hazard to Buildings B, D and E, and that seismically-induced liquefaction and lateral spreading presents a serious hazard for all of the buildings at Riverside Elementary School. It is virtually certain that the existing permanent school buildings were not designed to accommodate the ground deformations and failures that could potentially occur by either of these two mechanisms. The potential consequences of such movements on the existing permanent school buildings are not well understood at this time but could be severe.

Previously-identified concerns associated with differential settlement and the seismic criteria used to design Building E were also re-evaluated as part of this study. In general, we view these concerns as secondary in comparison to the potentially more serious hazards associated with seismically-induced landsliding and lateral spreading.

We emphasize here that evaluating the ability of the school buildings to tolerate earthquake-induced ground deformations and earthquake shaking is outside the scope of this geotechnical study; these structural evaluations would best be performed by a licensed Structural Engineer.

The results of our geotechnical evaluations are discussed further in the sections that follow.

Seismically-Induced Landsliding

The results of our evaluations indicate that strong earthquake shaking could cause creek bank failures that would extend into areas currently occupied by school buildings. The circular surfaces labeled C and D on Figures 5 and 6 represent the surfaces for which the computer program calculates the lowest factor of safety under seismic conditions. These "critical" failure surfaces extend more than 100 feet back from the top of the creek banks into areas that are currently occupied by school Buildings B, D and E. The calculated factors of safety associated with these potential failure modes are significantly less than one (0.5 and 0.6), which is indicative of seismic instability. A rotational failure occurring on one of these surfaces toward the creek banks would result in both vertical and lateral movement of the ground surface within the failed area.

Significant rotation failures commonly assume the form of what is recognized as a landslide, with a sharply-defined headscarp and lateral margins surrounding a downward-dropped and distorted mass of soil comprising the landslide body. As previously noted, the site is susceptible to seismically-induced landsliding due to the weak clayey soils that underlie the site at depth and that this risk exists regardless of whether or not soil liquefaction occurs.

Liquefaction and Lateral Spreading

The results of our evaluations indicate that strong earthquake shaking could result in liquefaction-induced lateral spreading that would extend into areas currently occupied by school buildings. With the exception of CPT-1, all of the CRTs encountered a layer of liquefiable soil which varied in thickness between depths of about 11 and 21 feet. Therefore, based on the available data, it appears that this layer may be continuous across much of the site and could potentially provide a preferred plane of weakness over which significant movements of non-liquefied ground toward the creek could occur. This general phenomenon is known as lateral spreading.

We note that this potentially-continuous liquefiable layer was present in CPT-2, which was drilled in the school playground approximately 300 to 400 feet away from the creek banks. Lateral spreading commonly occurs as progressive failure, with blocks of un-liquefied soil sliding toward a steep free-face (such as a creek bank). Lateral spreads can occur over large horizontal distances. Characteristic patterns of ground deformation include ground fissures or tension cracks perpendicular to the direction of movement, with intervening blocks that have subsided, translated, rotatated, or in some cases disintegrated.

Liquefaction-Induced Settlement

The results of our evaluations indicate that strong earthquake shaking could result in liquefaction-induced differential settlement of the ground surface beneath the school buildings. The results of our Phase 2 analyses indicate that up to about 2 1/2 inches of total compressional settlement could occur. It is not uncommon in conventional practice to assume that differential settlements between adjacent foundations could be half this amount (1 1/4 inches). As previously noted, greater settlements would be anticipated if liquefied soil is ejected through sand boils or fissures. Although a potentially significant concern, these estimated deformations are small in relationship to the large-scale movements that could potentially occur as a result of seismically-induced landsliding or lateral spreading.

UBC/CBC Soil Profile Type

Based on the available data, we judge that the use of an Sc (Very Dense Soil and Soft Rock) Soil Profile which was used during the design of the Measure M improvements cannot be justified. AnSD (Stiff) Soil Profile can be considered reasonable from a geotechnical standpoint where liquefiable soil layers are not laterally continuous or if ground improvement is performed to mitigate liquefaction effects.

LIMITATIONS

This report has been prepared for the exclusive use of the District and their attorneys for specific application to Riverside Elementary School in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after a period of one year without being reviewed by this office.