N00236.000427
ALAMEDA POINT
SSIC NO. 5090.3

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/ GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RACII-02-0190
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION, AND GEOTECHNICAL AND SEISMIC EVALUATIONS AT INSTALLATION RESTORATION SITE 1 ALAMEDA POINT ALAMEDA, CALIFORNIA

Comments by:	Responses by:
Department of Toxic Substances Control (DTSC) 700 Heinz Avenue, Suite 200 Berkeley, CA 94710-2721	Foster Wheeler Environmental Corporation 1940 E. Deere Avenue, Suite 200 Santa Ana, CA 92705

Specific Comments on Draft OEW/GC Report by DTSC

Comment 1. Site 1, being a Solid Waste Management Unit (SWMU), is subject to the corrective action requirements of RCRA Subpart S. Therefore, management of this unit must conform to RCRA, either directly or as ARARs. Please revise Section 1.5 (Regulatory Framework), particularly subsection 1.5.5, as appropriate.	**Response 1.** Site 1 was identified through a prior RCRA Facility Assessment (RFA) as a SWMU and this SWMU was included in the facility's RCRA permit. However, the permit did not contain a specific process or schedule for completing environmental investigation, remediation, and/or closure under the proposed RCRA Subpart S regulations. The Navy understands the Subpart S process was a proposed regulation that served as a means of policy or guidance for addressing SWMUs during the infancy of RCRA Corrective Action, but it was never codified into an actual regulation. Since it was assumed that there is no hazardous waste placed at Site 1, design of Site 1 landfill closure will follow the requirements of Title 27 CCR, which provide guidelines for Class II (designated waste) and Class III (non-hazardous solid waste) landfills. Title 22, which governs seismic and precipitation design standards for hazardous waste landfills (Class I), was not determined to be applicable for Site 1, and therefore, there was no reference to Title 22 in this report. However, the proposed seismic design of Site 1 landfill closure satisfies Title 22 requirements specifically pertaining to MCE or seismic design. These requirements under Title 22 are generally more conservative. Since Site 2 may need to follow Title 22 requirements due to the nature of waste disposed, MCE and seismic design requirements under Title 22 were followed for Site 1 for the purpose of consistency. In order to address DTSC comment, Section 1.5.5.1 (page 1-14) cites Title 22 as it applies to seismic requirements.

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Comment 2. Section 1.5.5 concerns the applicable regulations and criteria for geotechnical and seismic design. There is no comparable section for ordnance and explosive waste (OEW). Please provide one for OEW management.	**Response 2.** Section 1.5.6 has been added to address applicable regulations for OEW management has been incorporated into the report.
Comment 3. The OEW work described in this document includes not only the OEW characterization, as suggested by the report title, but also demilitarization and disposal. Please consider modifying the report title to reflect the full scope of work involved.	**Response 3.** The Report mainly addresses the OEW characterization activities and provides adequate details of the demilitarization and disposal of recovered OEW. The title adequately addresses these activities. Text has been added to Executive Summary (page ES-1) and Section 3.1 (page 3-2) to clarify that OEW encountered during the characterization was treated as investigated-derived waste.
Comment 4. It is unclear if Section 1.1.3 (Previous Investigation) contains all investigations conducted to date for Site 1. This has caused some confusion. For example, ♦ Page 1-3, paragraph 4 discusses a radiological survey conducted in September 1995 while paragraph 4 on the same page references a 1998 radiological survey. Does this mean that there had been two separate radiological surveys done on site? ♦ Page 1-3, paragraph 3 discusses a 1995 soil sampling. Dopes this mean that the 1995 study is the only chemical investigation done to date at the subject site? ♦ Page 1-4, paragraph 2 states that a RI was conducted by TtEMI and references documents dated 1999 as well as 2001. It is unclear that except hydrological and geotechnical data what other information is available through these two documents. For the purpose of this document, it is the Navy's choice to make Section 1.1.3 a summary of *all* or only *selected* investigations done to date. Please make it clear.	**Response 4.** Section 1.1.3 (page 1-3) contained selected investigations conducted to date at IR Site 1. This section has been amended to list all investigations as follows: • Two preliminary surveys were completed at IR Site 1 during 1995 and 1996. A comprehensive radiological survey, which was planned as a removal action, was conducted in 1998 and 1999. • Soil and groundwater chemical investigations were conducted at various locations on IR Site 1 in 1985, 1990, 1991, 1994, 1995, 1996, 1998 and 1999. • Two documents prepared by TtEMI in 1999 and 2001 contain various information including geology, hydrogeology, background chemical concentrations, sample collection, soil and groundwater analyses, ARARs, and human health and ecological risk assessments.

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Comment 5. Page 1-1, paragraph 2 states "Alameda Point is rectangular in shape, approximately 2 miles long east-to-west and 1 mile wide from north-to-south." However, a number of figures (e.g. Figures 1-1 through 1-3) show that Site 1 is actually longer in the north-to-south direction than east-to-west direction. Please revise as necessary.	**Response 5.** Alameda Point encompasses the entire (former) Alameda Naval Air Station, including the housing area as shown in Figure 1-1. This area is approximately 2 miles long, east-to-west and one mile long north-to-south. Figures 1-2 and 1-3 show only IR Site 1, which is a small part of Alameda Point. The location of IR Site 1 in Alameda Point is also shown on Figure 1-1.
PART 2: ORDNANCE AND EXPLOSIVE WASTE Please refer to the memorandum prepared by Mr. James Austreng. **PART 3: GEOTECHNICAL AND SEISMIC EVALUATION** Please refer to the memorandum prepared by Mr. Ram Ramanujam.	
General Comments by Mr. James C. Austreng, P.E.	

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Comment 1. On page 1-3 of the report, the Navy states – "During a 1998 radiological survey of IR (Installation Restorations) Site 1 (which found low-level radiation), a number of live 20 mm high explosive projectiles were discovered." Because of this discovery, I have raised questions in the following comments and conclusion regarding the adequacy of the proposed surface clearance. (Ref: page 1-4, the City of Alameda intends to convert IR Site 1 into a golf course after the DON (Department of the Navy) turns over the site. An OEW investigation and removal of OEW, if encountered **on the surface**, must occur prior to property transfer to the city. (Emphasis added) In addition, I have concerns about the absence of details pertaining to uncertainties and what risk management activities will be established.	**Response 1.** The September 1998 radiological survey of IR Site 1 was suspended when live ordnance was encountered. Due to the discovery of ordnance, the radiological survey was suspended for approximately 8 acres of IR Site 1 in and around the former pistol range. An emergency removal action was then conducted by SSPORTS personnel, which consisted of a surface sweep of the 8 acres. A total of 335, 20-mm high explosive projectiles were recovered, after which the radiological survey was resumed and subsequently completed. During the Ordnance and Explosives Waste Characterization conducted by FWENC, Unexploded Ordnance (UXO) technicians conducted a surface characterization of IR Site 1 in its entirety to verify that no UXO/OEW were missed during the previous sweeps. Because a landfill cap will be placed on IR Site 1, a surface characterization of the site to verify that no OEW exists on the ground surface is considered adequate for the planned use of the land. Therefore, the surface characterization conducted by FWENC is determined to be adequate for the planned transfer of the land to the City of Alameda. Although the OEW characterization of Site 1 verified the absence of OEW on the surface, there are still some uncertainties as to types of OE material buried in the landfill and pistol range sections of the site. When the Final Feasibility Study (FS) is promulgated, information concerning appropriate land use controls for the Site will be provided as part of the CERCLA process, that is, development of the Proposed Plan and Record of Decision. This discussion has been added to Section 3.6 (page 3-5). **(continued)**

Response 1. (continued)

A risk assessment was completed prior to beginning activities on the site.

Based on the results of earlier surveys and an Emergency Removal Action (ERA), the 20 mm HEI round with a single-action point detonating fuze was identified as the Most Probable Munition (MPM). The risks associated with the MPM were considered when developing the project Health and Safety and Work Plans, and the Standard Operating Procedures for OE/OEW. They include:

- Maximum Fragment Throw Range: 320 feet.
- Maximum Credible Event: detonation of 165 grains (.37 oz) of explosives and 20 grains of incendiary mixture in a single round (representative for 20 mm HEI rounds of the M563A3/A4 variety
- Methods of initiation: actions that would function a fired, single-action point detonating fuze with arming and firing features similar to the M503A3 nose fuze (typical for 20m HEI rounds) – striking, dropping, rough handling and static electricity.
- Transportation and storage
- The probability of occurrence and possible quantities of the MPM that could be encountered (results of earlier ERA divided by the total acreage of the project).
- Barricades, Personnel Protective Equipment, Exclusion Zones

Section 1.1.3.3 (page 1-4) has been modified to include the above discussion.

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Specific Comments by Mr. James C. Austreng, P. E.	
Comment 1. A Department of Defense Explosives Safety (DDESB) approved Explosive Safety Submission was not included in the document. It is DTSC's practice that an ESS is incorporated into the investigative report and DDESB's concurrence is obtained prior to initiating the response action. Furthermore, based on the intended land use, I do not believe limiting the OEW response action to surface is consistent with Department of Defense (DoD) Explosive Safety Standard 6055.9. Ordnance removal should be to depth or at a minimum of four feet below the deepest planned excavation whichever is greatest.	**Response 1.** Department of Defense Ammunition and Explosives Safety Standards (DoD 6055.9-STD) do not require the submission of an Explosive Safety Submission for characterization of a site to determine if the property is/is not contaminated with OEW. Confirmation regarding this interpretation was received via e-mail from the Navy Ordnance Safety & Security Activity (NOSSA) reiterating that an ESS is only required for a comprehensive ordnance removal action on land "known or suspected" of ordnance contamination, and that it is not required before or after an intrusive investigation. SSPORTS conducted an emergency removal action in 1999 that consisted of a sweep of the site to locate other possible ordnance. No further ordnance were encountered as a result of the SSPORTS surface sweep. NOSSA also confirmed that an ESS is not required before or after an intrusive investigation that confirms no ordnance contamination exists or that incidentally removes all known or suspected ordnance contamination. Based on these considerations, an Explosive Safety Submission was not prepared for the OEW characterization work. A landfill cap at least 4 feet thick will be placed at IR Site 1 before a golf course is constructed on the site. The depth of the cover will comply with the DoD default removal depth for interim planning for sites planned for surface recreation. Future site activities will include removal of the baseball backstop and an earthen berm 10 to 15 feet high located behind firing lines within the former pistol range. There are no excavation activities planned during construction of the landfill cap and golf course according to the current configuration, which will involve placement of fill only. Therefore, it is determined that the OEW surface removal complies with the DoD requirements.

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Comment 2. Page 1-3 also states "The rounds [the live 20 mm high-explosive rounds referenced above] were taken to an area between the runways and were detonated." There is no text, which discusses actions taken to ensure the detonation was complete and no unexploded items were thrown, or kicked out. Furthermore, there is no reference that such detonation constitutes treatment of a hazardous waste subject to compliance with California Code of Regulations (CCR) Section 66264.600, Miscellaneous Units. Questions remain whether this action was authorized by the DTSC.	**Response 2.** The 20 mm high explosive rounds recovered during the 1998 SSPORTS ordnance removal response had all been fired and their fuzes were considered armed. They could not be safely transported in that condition so they were disposed of by open detonation as a part of an Emergency Removal Action (ERA). The ERA met the criteria for emergency disposal authorized by the Code of Federal Regulations (CFR) 40, section 270.1 (c) (3) to mitigate "an immediate threat to human health, public safety, property or the environment from the known or suspected presence of military munition." There is no information available that specifically discusses the actions that were taken to ensure the detonation was complete and no unexploded items were thrown or kicked out. However, the Standard Operating Procedures (SOPs) used by SSPORTS personnel performing the open detonation during the ERA required a visual inspection of the detonation site by the Demolition Operations Supervisor 5 minutes after the detonation. The SOPs also required the demolition site be clean and free of trash prior to securing the operation. It is believed that these procedures were followed and no unexploded items were found. Emergency Removal Actions are not subject to CCR 66264.600 and do not require the approval of the DTSC.

Comment 3. The Report also fails to address subsequent finds of OEW and the need to comply with CCR 66264.600. Such compliance includes the analysis of potential impacts from treatment of hazardous waste as well as the assessment of treatment alternatives. Open detonation cannot be presumed to be the preferred treatment method. Contained detonation methods must be included in the analysis.	**Response 3.** The Miscellaneous Units requirements defined in CCR 66264.600 were not considered applicable to OEW characterization activities conducted on IR Site 1 because there were no plans for routine hazardous waste transfer, treatment, storage or disposal. Section 1.2.1 of the Final Focused Remedial Investigation Work Plan (FWENC, 2002) reiterated that if OEW were encountered and characterized by FWENC UXO Technicians as not safe to move/transport, then Travis Air Force Base (AFB) Explosive Ordnance Disposal (EOD) personnel would respond and determine the ultimate treatment alternative for the OEW. Several treatment alternatives (including contained detonation) for encountered OEW were considered during development of the Site 1 Work Plan. Open detonation, however, was not considered the preferred treatment method of treating OEW. Implementation of this alternative would have been determined by Travis AFB EOD personnel in response to OEW that was unsafe to ship and presented a threat to human health or the environment. The alternative was developed in accordance with the requirements of Emergency Removal Action (ERA) and included engineering controls to contain/control the open detonation activities. Section 3.0 (page 3-1) has been revised to summarize treatment alternatives for encountered OEW and the reason for selecting open detonation during the 1998 SSPORTS ordnance removal response.
Comment 4. While Abid Loan, P. E. signed the report, it is not clear if this individual is a California licensed professional. Pursuant to DTSC's practices and the California Business Code, such reports should be signed and stamped by a licensed California professional.	**Response 4.** Abid Loan, P. E., is a California licensed professional engineer. The Professional Engineering Act requires that only final documents be stamped and sealed. When this document is promulgated in its Final format, it will be signed and stamped in accordance with the Professional Engineering Act..

Comment 5. Conclusion: Based on the reported discoveries of live ordnance, it is my opinion that limiting the level of ordnance detection and removal to surface clearance is insufficient and does not adequately support the intended land use (golf course). Further information, which includes an assessment and protection of potential users coming in contact with any remaining ordnance needs to be provided. In addition, a Covenant with DTSC that "runs the land", needs to be developed and approved by DTSC prior to transfer.	**Response 5.** The DoD 6055.9-STD assessment depth for land planned for surface recreation is 4 feet. A 4-foot landfill cap is planned for installation at IR-Site 1 prior to the construction of the planned golf course. Additional fill will be placed over the landfill cap as a part of the golf course construction. The surface characterization of the site was completed to ensure that there was no surface OEW contamination present prior to the 4-foot cap installation. This layer of fill complies with the DoD minimum requirements for planned use of the site. Many types of wastes were buried in the IR Site 1 disposal area. Institutional/engineering controls are planned for the site prior to its transfer. These measures will include warning the potential users of the hazards and limiting excavation to less than 4 feet, which is the minimum thickness of the assumed landfill cap.

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Comments from Ram Ramanujam, P.E.

Comment 1. Section 5.0, Conclusions and Recommendations: The calculated permanent slope deformations at the site range from 2 to 19 feet. And, the liquefaction- induced settlements are estimated to be up to 12 inches. These slope-deformations and settlement values may not be restricted within the site and may extend beyond the site boundary. The proposed remedial measures should also consider the seismic impact on the site due to the outside boundary effects.	**Response 1.** The slope deformations and settlements are not restricted within the site and may extend beyond the site boundary. Slope deformations along the Oakland Inner Harbor shoreline and east of the site eastern boundary are expected to be comparable to results reported for cross sections G-G' and H-H' due to similarity of geometry and subsurface conditions (see Figures 4-6, 4-7c, 4-7g and 4-7h). Remedial measures proposed will address the potential slope deformations and settlements within the site boundary. The main objective of these measures will be to prevent release of waste from the site into San Francisco Bay during seismic activities. The objective of these measures will not necessarily be to preserve the golf course or adjacent areas from seismic effects. Therefore, the focus of the proposed remedial measures will be to control release of waste into San Francisco Bay and to address the geotechnical and seismic hazards identified at Site 1. It should also be noted that all references in the report to permanent slope deformations ranging from "2 to 19 feet" have been revised to "5 to 23 feet" as a result of using more conservative shear strength properties for fill and Young Bay Mud materials in revised pseudo-static slope stability analyses, which took into account the effect of material strength reduction during earthquake due to strain softening and liquefaction. The reduced strength properties were equal to the average of pre-earthquake and post-earthquake properties. This increase was based on new yield acceleration (Ky) values in the range of 0.02 to 0.09 as presented in revised Table 4-14. Liquefaction-induced settlements are estimated to be up to 12 inches.
Comment 2. Table 4-1: The table should include the surface elevation at which the Cone Penetrometer test (CPT) probes were taken.	**Response 2.** Surface elevations for CPT locations are listed in Table 4-2 and have been added to Table 4-1 for completeness.

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Comment 3. Table 4-6: The long term stability analyses require effective stress parameters such as c' and Ø' for various subsurface materials. The summary of material design parameters Table 4-6 should include effective stress parameters c' and Ø'.	**Response 3.** Table 4-6 lists effective stress (CD shear strength) parameters such as c' and Ø' for generalized soil strata I, IIB, and III. These properties have been added to Table 4-6a (revised table number) for soil strata IIA and IV for completeness.
Comment 4. Figure 4-17: Please provide a detailed reference to the publication in the reference Section 6.0.	**Response 4.** Detailed reference has been provided in the References section (6.0) under the subheading References from HAI Report (page 6-4). References from HAI Report have been combined with the main reference list (pages 6-1 to 6-2) for clarification.
Comment 5. Appendix A: All the identification of the subsurface soil profiles in the Geological cross sections should be the same as the soils profile provided in the Figures 4-7a through 4-7h (for consistency).	**Response 5.** The identification of the subsurface soil profiles in the geological cross sections in Appendix A are different from the soils profile provided in Figures 4-7a through 4-7h. The geological cross sections in Appendix A were reproduced based on information provided in a previous report (TtEMI, 1999), whereas soil profiles in Figures 4-7a through 4-7h are based on the information provided in this report (FWENC 2002). A direct correlation (same identification) cannot be made between the two sets of soil profiles because the geological cross section descriptions in Appendix A include a Bay Sediments Layer, which does not correspond to either the Young Bay Mud or Merritt Sand directly. The location of the water table was not directly measured during soil borings but was inferred from Cone Penetration Tests (see Section 4.4.2). Figures 4-7a through 4-7h have been revised to show the water table.

Comment 6. Appendix G: Some of the laboratory test results indicate very high moisture content values. Refer to Boring Nos. B8 and B10, the moisture content values are greater than the liquid limit. The Report should include some discussion on the high moisture content values of the samples.	**Response 6.** The laboratory test results presented in Appendix G (under Borehole No. B8 and B10) indicate very high moisture content values greater than the liquid limit. These values were primarily found in soft harbor sediments classified under Generalized Stratum IIA (Table 4-6). These soils are described as normally consolidated (Nc) to slightly under consolidated with average measured in-situ moisture content of 61% and liquid limit (LL) of 55%. A discussion regarding the high moisture content values of the soft harbor sediments has been added to Section 4.4.1 (page 4-16) under the Stratum II subheading.
Comment 7. Appendix K: - Liquefaction induced settlements should include back calculations for review. - All figures should include subsurface layer identification using Unified Soil Classification System (USCS).	**Response 7.** The empirical method for estimating liquefaction-induced settlements is described in Sections 4.6.6 and 4.6.7 (pages 4-31 to 4-36) and in Appendix L. A sample calculation for estimating liquefaction-induced settlements has been added and discussed in Appendix L. All figures include soil type descriptions for subsurface layers encountered based on I_c (CPT soil behavior type index) plots. Unified Soil Classification System (USCS) identifications have been included for each subsurface layer in interpreted subsurface soil profiles in Figures 4-7a through 4-7h of the report based on boring logs and laboratory testing results plotted on the CPT tip resistance and friction ratio profiles. The CPT results presented in Figures L-1 to L-14 may only be used to identify soil types for different soil layers based on the I_c profile.

Comment 8. Appendix L:	**Response 8.**
- The Appendix should include a summary of conclusions of the stability analyses.	A paragraph summarizing conclusions of the slope stability analysis results has been added in Section 4.6.8 (pages 4-38 and 4-39). Also, Table 4-14, Summary of Slope Stability Analysis Results, was reproduced and added in Appendix M.
- For selected stability analyses cover material is used. The Report should include the thickness of the cover materials and the justification of shear strength properties used for the stability analyses.	A cover thickness of 4 feet was assumed in the stability analyses. Engineering properties of cover materials are unknown at this time, however, a friction angle of $Ø = 34°$ and cohesion of $c = 200$ psf as shear strength properties were used in the stability analyses. These values are typically used for compacted cover material composed of (medium dense) Silty Sand (SM) to Clayey Sand (SC). A table from the NAVFAC DM-7.2 manual showing typical properties of compacted soils has been added in Appendix M and discussed in Section 4.6.8 (page 4-37). The text has been revised to include the above discussion. Static stability of the site perimeter slopes were analyzed using conservative undrained shear strength properties representing soils in slightly under consolidated to normally consolidated undrained (CU) condition. These analyses model static stability of the site for initially consolidated or partially consolidated materials. Shearing from additional surface loading is applied without sufficient time for dissipation of excess pore water pressures (namely, short to medium term conditions).
- The report should include long term static stability analyses using effective shear strength properties (c' and Ø') of various subsurface materials.	Long-term static stability analyses for a few critical cross sections were performed using CD shear strength properties (c' and Ø') provided in Table 4.6a. Long term stability analyses simulated conditions where the materials had enough time to dissipate excess pore water pressure. In general, these analyses resulted in higher factors of safety compared to analyses performed using CU shear strength properties. . The results have been added in Section 4.6.8 (pages 4-38 and 4-39) and Appendix M. **(continued)**

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- The Report should include backup calculations for the permanent slope deformations.	**(continued)** Newmark procedure and backup calculations for the permanent slope deformations have been added in Appendix M for completeness.
Comment 9. I have not visited to observe the existing conditions. Next time, I will join you for the site visit. I will be available to attend any project meeting to resolve the technical issues identified in this memorandum. In the meantime, if you need any clarification on this memorandum, please contact me at (916) 255-6662.	**Response 9.** Comment noted.
Comments by: Lea Loizos, Staff Scientist, Arc Ecology (Environment, Economy, Society & Peace)	
Comment 1. Have the Work Plans and subsequent reports been reviewed by the U.S. Army Engineering and Support Center, Huntsville, Ordnance and Explosives Team?	**Response 1.** Work plans and subsequent reports were not reviewed by the Army Corps of Engineers because the Corps does not have jurisdiction over activities performed at the former Naval Air Station. The Naval Ordnance Safety and Security Activity (NOSSA) was consulted during Work Plan and Explosives Safety Remediation Plan (ESRP) development. NOSSA reviewed the pre-draft version of the Work Plan and the draft-Final version of the ESRP.

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Comment 2. Please explain why the Schonstedt GA-52 CX was used to conduct the surface OEW characterization rather than the MK 26, which is the standard issue magnetometer used by U.S. Explosive Ordnance Disposal Units to locate ordnance. Was the same magnetometer used in the test pits and boreholes?	**Response 2..** The Schonstedt GA-52 CX was used to conduct the surface OEW characterization rather than the MK 26 because it has the better maneuverability. The Foerster Ferex® 4.021/MK 26 is versatile, supersensitive search instrument with a sole purpose of locating ferromagnetic items buried in the ground or underwater at depths to of up to 6 meters. In use, the sensor probe is held stationary and is moved in parallel lines over the area to be searched in lanes 1 meter apart. It weighs nearly 14 pounds and requires 2 hands to operate. The Schonstedt GA-52CX is over 10 pounds lighter than the MK 26, requires only one hand to operate and can detect large, subterranean ferromagnetic items at depths approaching 3 meters. It is swept side-to-side in front of operators as they proceed down search lanes during a surface characterization of an area. This technique helps personnel to concentrate on the ground in front of the probe as they walk. It is the instrument preferred by FWENC UXO technicians for conducting surface characterizations and was also used for OEW avoidance procedures during test pit excavations. The Shonstedt MG 220 magnetic locator was used for OEW avoidance procedures in boreholes. This discussion has been added to Section 3.1 (page 3-2).
Comment 3. It is unclear in the report where exactly the OEW was located. For example, was all OEW found during the surface sweep or was some recovered while digging the test pits? A figure showing the location (and depth, where appropriate) of all OEW would be helpful.	**Response 3.** All of the OE scrap (inert 20 mm Target Practice rounds) was found on the ground adjacent to the old pistol range during the surface sweep. Figure 3-3 has been included in the report to show the location of the recovered scrap.
Comment 4. Is the removal of OEW considered complete or will further action be taken to ensure that all OE has been removed? Why was surface OE the only concern?	**Response 4.** The removal of OEW is considered complete. A 4-feet thick landfill cap, which is the default depth assessed by the DoD 6055.9-STD, will be constructed on the site. Therefore, the only concern regarding any OEW was that it could have been encountered on the surface during construction activities.

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Comments by: The Environmental Protection Agency 75 Hawthorne Street San Francisco, CA 94105-3901	Responses by: Foster Wheeler Environmental Corporation 1940 E. Deere Avenue, Suite 200 Santa Ana, CA 92705
General Comments by the Environmental Protection Agency	
Comment 1. The seismic slope stability analyses were first conducted with the existing soil covers and materials (such as fill) and then with an assumed cover. Cross-section figures 4-7d through 4-7h do not indicate that a cover system was assumed. Thus, no details are provided about the assumed cover. Section 4.6.8 indicates that a proposed landfill cover was assumed in the analyses; however, it is not clear what assumptions about the landfill cover were used in the evaluation of slope stability.	**Response 1.** The seismic slope stability analyses were first conducted with the existing soil covers and materials (such as fill) and then with an assumed cover. This approach was adopted to evaluate both the existing conditions and the effects of the proposed landfill cap. Cross-section figures 4-7d through 4-7h do not include an assumed cover system because these figures are used to present interpreted subsurface soil profiles under existing conditions (without the proposed/assumed cover). These figures do not represent the cross sections used for analyses after the assumed soil cover is placed. The details regarding the assumed cover are included in Appendix M. These include soil type, geometry, and material properties. The statement that indicates that a proposed landfill cover was assumed in the analyses, has been deleted from Section 4.6.8. Details regarding the assumed cover are provided in Appendix M. Section 4.6.8 (page 4-37) to references Appendix M for information regarding the assumed landfill cover (soil type, geometry, and material properties).

Comment 2. The title of the report (Ordnance and Explosives Waste/Geotechnical Characterization Report) would seem to indicate that a complete OEW characterization, including subsurface work, is covered in the report. Even though a number of prior ordnance and explosives (OE)/OEW characterization and removal actions have been conducted in the area of concern, a complete removal to depth has not been accomplished. The documentation from these prior actions has indicated that there is confirmed and anecdotal information which indicates that ordnance items described as "inert" were placed in the landfill in former periods. It is possible that some live ordnance may be present in the landfill cells that are located in the site. Any activities conducted in the future which involve intrusion into the landfill wastes should consider the possible presence of live ordnance of an undetermined size and quantity as a potential hazard. Engineering and institutional controls should be established to ensure that all concerned are advised of this risk	**Response 2.** The explanation for the use of term "characterization' related to OEW is provided in the Executive Summary. The area of concern (as described by SSPORTS in their UXO Site Investigation Final Summary Report) at IR Site 1 is referred to as the former pistol range. Information exists that indicates that this area was excavated to a depth of 8 feet during the construction of the former pistol range. Barrels of fired 20 mm projectiles from the gun re-work facility were placed in the excavation as part of the backfill and were also used as concrete aggregate for the range foundations. A surface sweep conducted in 1998 recovered 335 20-mm High Explosive rounds from a small area near the small arms range backstop. These rounds were disposed of by open detonation. The subsequent surface characterization completed by FWENC UXO personnel verified the surface of Site 1 did not contain any live OEW. Removal to depth is not necessary because a minimum of 4 feet thick landfill cap will be placed at IR Site 1 before the golf course construction. The DoD 6055.9-STD's default depth for interim planning of land proposed for surface recreation is 4 feet. Additional fill will also be placed over the landfill cap as part of the golf course construction. Any future excavation, other than fill activity during the golf course construction, will require consideration of the possible presence of live ordnance. There are still some uncertainties as to types of OE items buried in the 7 disposal areas at Site 1. Therefore, engineering and institutional controls will be established to mitigate potential risks associated with intrusive activities such as excavations deeper than 4 feet after the golf course is constructed. This discussion has been added to Section 3.6 (page 3-5).

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Specific Comments by the Environmental Protection Agency

Comment 1. Executive Summary, Page ES-2: The text in the second paragraph states that "all cross sections analyzed were determined to be stable, with static factors of safety greater than 1." However, the executive summary does not make it clear that the static factor of safety required by the State of California is 1.5; therefore, the landfill, as is, does not meet the State of California requirements. Furthermore, when the final cover was included in the analysis, the static factors of safety decreased. Also, the executive summary does not include the information that post-earthquake static factors of safety calculated were less than 1 in most of the cross sections analyzed (Section 4.6.9 of the Report). For clarity and completeness, please revise the executive summary to include the information that all but two cross sections analyzed had static factors of safety less than 1.5 and most post-earthquake factors of safety were less than 1.	**Response 1.** A definition of the factor of safety has been added to the Executive Summary, which defines this factor as the ratio of resisting (stabilizing) forces to the driving forces trying to displace the slope. Based on this definition, the slopes are physically stable. Guidelines for the stability analyses are provided in the California Code of Regulations (CCR) Title 27. However, no specific value for the static factor of safety is provided. The current state of practice in California is to require a minimum static factor of safety of 1.5, therefore, based on this requirement, the landfill, as is, does not meet the minimum factor of safety in most areas. Text has been added to clarify that the landfill does not meet the current standard of practice in California. Furthermore, the text has been modified in the Executive Summary (page ES-2) to indicate that static factors of safety decreased after the final cover was included in the analysis. The statement in Section 4.6.9 (page 4-41) of the Report regarding the post-earthquake static factors of safety has been revised to read, "Post-earthquake static factors of safety calculated were **greater** than 1 for all cross sections except two sections. Permanent slope deformations calculated ranged from 5 to 23 feet." This statement is based on stability analysis results presented in Table 4-14. The table was modified to include increased seismic permanent displacement values calculated based on revised, more conservative strength properties equal to the average of pre-earthquake and post-earthquake properties. Therefore, the Executive Summary (page ES-2) has also been revised to indicate that all but two cross sections analyzed had static factors of safety less than 1.5 and most post-earthquake factors of safety were greater than 1. Furthermore, Sections 4.6.8 and 4.6.9 have been revised to explain limitations of post-earthquake slope stability analysis results when pseudo-static stability and Newmark deformation analyses indicate large seismic induced slope deformations. This discussion has been added to the Executive Summary (pages ES-2 and ES-3).

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Comment 2. Section 1.1.1, Site Description, Page 1-1: IR Site 1 is described as relatively flat; however, it appears that slopes exist where the site borders San Francisco Bay. Since the report includes analysis of slope stability, this section should include a description of the existing slopes. For clarity, please revise this section to describe the slopes at IR Site 1.	**Response 2.** IR Site 1 is relatively flat. The presence of slopes along the San Francisco Bay, which are mostly underwater, has been addressed in Section 4.4.1 Subsurface Soil Conditions (pages 4-15 to 4-17), which references Figures 4-7a to 4-7h, and in Section 4.6.8 Seismic Slope Stability. Discussion was added in Section 1.1.1 (page 1-1) indicating the presence of shoreline slopes and rip rap placed along the shoreline.
Comment 3. Figure 1-2: The shaded area and the short-dash/long-dash line that outlines the shaded area are not included in the legend, so it is unclear what these symbols represent. Please include the shading and the short-dash/long-dash line in the legend.	**Response 3**. An arrow with a note shown in Figure 1-2 designates this shaded area as a former disposal area. The short-dash/long-dash line designating the border of the former disposal area has been included in the legend.
Comment 4. Section 1.3.2, Geotechnical and Seismic Evaluation, Page 1-6: The sentence at the top of this page states that static and dynamic stability of perimeter dikes were evaluated; however, perimeter dikes are not described or discussed elsewhere in the report. Please clarify whether perimeter dikes are present at the site and, if so, include a discussion of the evaluation of static and dynamic stability of perimeter dikes in the report. Also, please include these features on a figure.	**Response 4.** Only static and dynamic stability of perimeter slopes, and not dikes, was evaluated. Therefore, the sentence (page 1-7) has been revised to indicate that "static and dynamic stability of perimeter slopes with and without placement of new soil cover fill (landfill cap) along the shoreline." An earthen berm (dike) 10 to 15 feet high is located adjacent to the shoreline near the former pistol range area (middle western boundary of IR Site 1). The stability of this earthen berm was not considered critical and no stability analysis was performed. Additional text describing this earthen berm was added to Section 1.1.1 (page 1-2) The location of the former pistol range is shown in Figure 1-3.
Comment 5. Section 3.4.1, Page 3-3: The text states that "UXO technicians checked the test pit with the magnetometer after each lift and hand-excavated all detected metals," but does not describe the metal objects that were found. Please add a brief description of the metal objects that were hand-excavated by the unexploded ordnance (UXO) technicians.	**Response 5.** Findings of test pit explorations are described in Section 4.2.2, "Test Pit Exploration Logs." A summary table (Table 4-5) and test pit logs (Appendix I) also include this information. For completeness, a reference to Section 4.2.2 has been added in Section 3.4.1 (page 3-4).

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Comment 6. Section 4.3.2.2, Young Bay Mud, Page 4-11 : This section gives the shear-wave velocity of the Young Bay Mud (400 to 650 feet per second) and states that the shear-wave velocity can provide an indication of the density and firmness of soils and rocks. However, the Report does not explain what 400 to 650 feet per second implies regarding the density and firmness of Young Bay Mud. For clarity, please revise this and subsequent sections to explain what the shear-wave velocity implies in terms of density and firmness of the various stratigraphic layers.	**Response 6.** Text has been modified in Section 4.3.2.2 (page 4-11) and subsequent sections (4.3.2.3 to 4.3.2.7) to explain what the shear-wave velocity implies in terms of density and firmness of the various stratigraphic layers.

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Comment	Response
Comment 7. Section 4.5.1, Bearing Capacity, Page 4-17: This section concludes that "bearing capacity failures from placement of a landfill cap are not an issue at this time," however, this statement is not supported by the information presented in the report. First, this section assumes a landfill cap approximately 4 feet thick in order to evaluate the bearing capacity of the exiting fill material; however, in section 4.5.3, the report indicates that the minimum landfill cap thickness has not yet been determined. In addition, in order to maintain drainage, additional fill must be placed. Assuming minimum 3% slopes are maintained, 15 feet or more of additional fill may be required in some areas (depending upon the design). It appears that the applied pressure of additional fill plus cap thickness could be five times greater than the assumed average unit weight of 125 pounds per square foot (psf). Second, the allowable bearing pressures to SP and SM soils (1,500 to 4,500 psf) are cited; however, these values do not take the presence of waste material into account. The report states that the impact of the presence of waste materials is accounted for in CPT and SPT test data, but it is not clear how this data was used in evaluation the bearing capacity of fill soils. Please revise this section to include a more realistic assumption for the applied pressure of the landfill cap and an evaluation of the allowable bearing pressure of the fill material that takes into account the presence of waste.	**Response 7.** Bearing capacity failures from placement of a landfill cap are not an issue at this time because the proposed landfill cap (approximately 4 feet thick) applies a uniform load of approximately 500 psf, which is significantly less than the allowable bearing capacity of 1,500 to 4,500 psf for SP and SM soils. SP and SM soils make up most of the existing fill material as mentioned in Section 4.5.1. Construction of the golf course will require additional fill on top of the landfill cap to create rolling hills and fairways. This fill will be spread out over the entire site with gradual changes in thickness, yielding relatively uniform loads. Under this simplified conservative evaluation, soil foundations are not expected to heave under uniform loads. Therefore, without this mechanism (potential for heaving), bearing capacity failures cannot occur regardless of soil shear strength. Also, this approach assumes cap as a surcharge load only and doesn't take into account its shear strength. Section 4.5.1 assumes a landfill cap approximately 4 feet thick in order to evaluate the bearing capacity of the existing fill material. This thickness was selected based on anticipated design considerations. Section 4.5.3 indicates that the minimum landfill cap thickness has not yet been determined because the landfill cap design has not yet been initiated. While some areas may require more fill (15 feet or more in thickness) than others as part of the golf course design, the changes will be gradual and spread out over the entire site. The average unit weight of fill material for the landfill cap was assumed to be 125 pounds per cubic foot (pcf) and not 125 psf as noted in Section 4.5.1 (Page 4-18). This has been corrected. The applied pressure of additional fill plus cap thickness will exceed the assumed 500 psf applied by the landfill cap only. However, the additional fill will be spread out over the entire site. Therefore, bearing capacity failure is not considered a concern. Localized bearing capacity failures may be an issue during construction but can be controlled by temporarily storing fill materials away from the shoreline slopes. **(continued)**

	Response 7 (continued) The allowable bearing pressures for SP and SM soils (1500 to 4500 psf) was cited since the existing cover consisted mainly of SP and SM soils. The presence of waste material could increase or decrease these average values but is not a major concern since relatively uniform placement of fill material lack the mechanism to cause bearing capacity failures. The impact of the presence of waste materials is accounted for in CPT and SPT test data. This data was not used in the simplified conservative evaluation since the shear strength parameters were not determined to be critical for the evaluation. However, the presence of waste materials is accounted for calculation of settlements using CPT and SPT test data. This calculation is presented in Appendix K. Effects of settlements over time after placement of landfill cap and additional fill will be addressed by maintenance activities that need to be performed in the future. The effects of immediate settlements during construction will be addressed in the final design. A 4 feet thick uniform landfill cap was considered a reasonable assumption for an evaluation of the allowable bearing pressure of the fill material. However, for clarity and completeness, additional discussion of the issues involved in the bearing capacity evaluation is provided in Section 4.5.1 (page 4-18).
Comment 8. Section 4.5.2, Settlements, Page 4-18: This section states that seismically induced settlements were estimated to be 1 foot maximum; however, Section 4.6.7 states that "the maximum estimated liquefaction-induced settlement is on the order of 1 foot, including an additional settlement of approximately 4 to 5 inches due to possible liquefaction/ consolidation of silty soils in Young Bay sediments. This statement implies seismically induced settlements are estimated to be 1 foot, 4 to 5 inches. Please revise the report to clarify the maximum estimated seismically induced settlements.	**Response 8.** Text in Section 4.5.3 (page 4-19) referring to "1 foot maximum" has been revised to "up to 18 inches." Additional settlements due to liquefaction/consolidation of silty soils in Young Bay sediments have been changed to approximately "4 to 6" inches in Section 4.6.7.1 (page 4-35).

Comment 9. Section 4.5.3, Settlements, Page 4-18: This section states that immediate and long-term settlement due to the landfill cap is not likely to exceed the seismically induced settlements (1 foot maximum); however, no evidence is provided to support this conclusion. Please revise the report to provide evidence to support the conclusion that immediate and long-term settlement due to the landfill cap load is not likely to exceed one foot.	**Response 9**. Calculations showing immediate and long term settlements due to the landfill cap have been included in Appendix K.
Comment 10. Section 4.6.5, Ground Response Analysis, Page 4-24: This section used a maximum earthquake magnitude of 7.1 on the Hayward Fault; however, it is not clear why 7.1 was selected. The discussion of the Hayward fault on page 4-22 lists several different maximum values. Please clarify why the earthquake magnitude 7.1 was used to estimate the peak ground acceleration due to the Hayward Fault.	**Response 10.** The last sentence of Page 4-21 states "The fault parameters were derived from the recent fault database compiled as part of the seismic hazard evaluation model developed for the State of California by the CDMG and USGS (Petersen et al., 1996)." Among different values for the Hayward Fault available from various sources, this report selected the parameters determined by the State of California fault database model (Petersen et al., 1996). The State database provides the most recent and widely accepted fault database model used in seismic hazard evaluations. For additional clarity, a discussion has been added to Section 4.6.5 (page 4-26) to explain the reason for selecting a maximum magnitude of 7.1 for the Hayward Fault to estimate the site PHGA. Also a sentence in the last paragraph of Section 4.6.2 (page 4-24) has been revised to read, "Many seismic hazard analyses used values of 7.25 **prior to development of the state of California fault database model."**

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Comment	Response
Comment 11. Section 4.6.5.2, One-Dimensional Site Response Analyses, Page 4-26: This section states the most credible earthquake (MCE) on the San Andreas Fault is used as the design event; however, it is not clear how the design event was selected. The report discusses the MCE on both the Hayward and San Andreas Faults. The report indicates that the Hayward has the highest probability of generating earthquakes within the next 30 years (Page 4-21) and that the peak horizontal ground acceleration due to the MCE on the Hayward fault is higher (Page 4-24). Therefore, it is not clear why the MCE on the San Andreas Fault was selected as the design event. Please revise the report to clarify this selection.	**Response 11.** Section 4.6.5 (page 4-25) states that a deterministic approach was used to estimate the earthquake shaking levels due to the Maximum Earthquakes (also defined as MCE in CCR Title 27) on Hayward and San Andreas faults at the project site. The Hayward fault has the highest probability of generating earthquakes within the next 30 years (page 4-22) and that the peak horizontal ground acceleration due to the MCE on the Hayward fault is higher than the San Andreas Faults (page 4-26). However, these factors did not impact the determination of the design event because: 1) A deterministic approach provides estimates of the site design ground motion parameters without any reference to probability of earthquake occurrence. 2) Historical seismicity data indicated that the site has possibly experienced a maximum rock acceleration of about 0.4 g in the past 200 years, which is higher than the estimated PHGA of 0.34g and 0.3g generated by the MCE on the Hayward and San Andreas Faults, respectively. The design event was determined by selecting a design earthquake ground motion with a magnitude and fault-to-site distance based on the MCE on the San Andreas Fault (an event of magnitude 7.9 at a distance of 19.5 km), and a peak rock horizontal acceleration of 0.4g. This resulted in a more conservative design event (an event with a higher intensity and longer duration) compared to the mean ground motions generated by the MCE on either the Hayward or San Andreas faults. Sections 4.6.5 (page 4-26) and 4.6.5.2 (pages 4-28 and 4-29) have been modified to include the discussion above. Additionally, Figure 4-13 was changed to include the site response spectrum due to the MCE on Hayward fault for comparison with the San Andreas fault MCE design spectrum.

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Comment 12. Section 4.6.5.2, One-Dimensional Site Response Analyses, Page 4-27: The shear-wave velocity of the foundation Franciscan Formation bedrock was assumed to be 4,000 feet/sec; however, the basis for this assumption is not clear. On Page 4-13, the report presents the shear-wave velocity range for the Franciscan Formation as 3,500 to 6,500 feet/second (ft/sec). Please clarify how 4000 ft/sec was selected for modeling and indicate why a value in the lower end of the shear-wave velocity range was selected.	**Response 12.** The shear-wave velocity of 4,000 ft/sec for the foundation Franciscan Formation bedrock was selected based on the recent shear wave velocity measurements in the upper 200 feet of the bedrock for the San Francisco-Oakland Bay Bridge (SFOBB) East Span Seismic Safety project (EMI, 2001a). This velocity was selected because the referenced project site is about 1.5 miles north of the disposal area. The shear wave velocity range for the Franciscan Formation presented on page 4-14 provides the wave velocity range over a large area and depth interval compared to the more site-specific measurements performed in the vicinity of the disposal area. Additionally, new one-dimensional site response (SHAKE 91) analyses were performed using a shear wave velocity of 5,000 ft/sec (average of 3,500 to 6,500 ft/sec range), which resulted in site ground motions similar to those obtained using the bedrock velocity of 4,000 ft/sec. The text in Section 4.6.5.2 (page 4-29) was modified to indicate that the bedrock velocity of 5,000 ft/sec was used in revised SHAKE91 analyses.

Comment	Response
Comment 13. Section 4.6.5.2, One-Dimensional Site Response Analyses, Page 4-27: It is not clear how the presence of waste material was taken into account in the five soil types used in the model. For clarity and completeness, please discuss the presence of waste material, how it is taken into account in the soil types listed, and the impact of the variable amounts of waste on ground motions at the site.	**Response 13.** Sufficient site-specific data for waste material properties are not available. The site-specific field exploration performed for this study concentrated on a narrow zone along the site perimeter, which possibly included some waste material. The waste material was modeled as the upper 20-foot-thick soil layer (fill) with material properties estimated based on the results of field exploration and laboratory testing performed for this study to determine soil classifications, unit weights, and shear wave velocities. Published relations were used to define variations of damping and shear modulus ratio as a function of shear strain for waste materials (upper fill layer). The waste in the upper fill layer is generally mixed with granular soils and therefore, the selected properties of the fill materials placed along the disposal area perimeter are expected to be relatively similar to properties of the mixed soil/waste fill in the landfill. Additionally, based on the published data (USEPA, 1995), waste material properties are not too different from the fill properties measured in this investigation. Therefore, the impact of variable amounts of waste on ground motions at the site is negligible due to the similarity of material properties of mixed soil/waste fill in the landfill, soil fill along the site perimeter, and relatively small thickness of the disposal area. Section 4.6.5.2 (page 4-30) has been revised to include the discussion above.
Comment 14. Section 4.6.5.2, One-Dimensional Site Response Analyses, Page 4-28: This section states that the site response analyses provided the site peak horizontal ground surface acceleration that was used in evaluation of the site liquefaction potential; however, the peak horizontal ground surface acceleration is not listed here. For clarity and completeness, please include the site peak horizontal ground surface acceleration in this section.	**Response 14.** The site peak horizontal ground surface acceleration for 3 sets of earthquake records are summarized in Section 4.6.5.2 (page 4-30). Set 3 as listed has 0.45g peak ground surface acceleration. Section 4.6.6.1 (page 4-32) also addressed that "The liquefaction potential of the subject site was analyzed utilizing a maximum peak site acceleration of about 0.45g for a magnitude 7.9 seismic event…" For clarity, the last paragraph in Section 4.6.5.2 (page 4-31) has been modified to read, "The site response analyses provided the maximum site peak horizontal ground surface acceleration of 0.45g. This was used in evaluation of the site liquefaction potential and seismically induced settlements."

Comment 15. Section 4.6.8, Seismic Slope Stability, Page 4-32: The proposed landfill cover is included in the analysis of slope stability; however, the thickness and final slope of the landfill cover have not yet been determined. It is not clear what assumptions about the landfill cover were used in the evaluation of slope stability. It is also not clear what relative impact the assumptions about the landfill cover had on the results of the slope stability analysis. For clarity and completeness, please revise the report to include a discussion of the assumptions that were used about the proposed landfill cover and discuss the impact the design of the cover had on the results of the slope stability analysis (i.e., if the cover design changes, will the slope stability change?).	**Response 15.** The proposed landfill cover is included in the analysis of slope stability. However, the thickness and final slope of the landfill cover have not yet been determined and the assumptions regarding the soil cover (soil type, geometry, and material properties) are provided in Appendix M. Section 4.6.8 (page 4-37) references Appendix M for information regarding the proposed landfill cover (soil type, geometry, and material properties). The impact of the landfill cover on the results of the slope stability analysis is discussed in Section 4.6.8 (page 4-37). The assumptions regarding the soil cover will impact the stability analyses. However, the impact is expected to be minor. More detailed analyses may have to be performed during the final cover design to verify the stability results.

Comment	Response
Comment 16. Section 4.6.8, Seismic Slope Stability, Page 4-34: This section states that "for post-earthquake stability conditions, according to the USACE Manual EM 1110-2-1913, the minimum acceptable factor of safety is 1.0. This criterion was satisfied for all cross sections except Cross Sections D-D' and I-I'." This statement appears to contradict Section 4.6.9 which states that "post-earthquake static factors of safety calculated were less than one in most of the cross sections analyzed." Please correct this discrepancy. Also, please clarify how the post-earthquake minimum acceptable factor of safety of 1.0 cited in this section would meet the Title 27 requirement of a factor of safety of 1.5 under dynamic conditions.	**Response 16.** As previously discussed in response to Comment 1 (Specific Comments by the EPA), The statement in Section 4.6.9 (page 4-41) of the report on post-earthquake static factors of safety has been revised to read, "Post-earthquake static factors of safety calculated were **greater** than 1 for all cross sections except two sections. Permanent slope deformations calculated ranged from **5 to 23** feet." This statement is based on stability analysis results presented in Table 4-14 and Page 4-34, which has been modified to include an increased value for the seismic permanent displacement. As discussed in Section 1.5.5.1 State and Federal Regulations, Title 27 requires a factor of safety for the critical slope of at least 1.5 under dynamic conditions (factor of safety calculated in a pseudo-static slope stability analysis). However, as indicated in this section, Title 27 adds that in lieu of achieving a factor of safety of 1.5 under dynamic conditions, a more rigorous analytical method that provides quantified estimate of the magnitude of movement (i.e., seismically-induced slope deformation) may be used. Title 27 only refers to evaluation of dynamic stability (stability during earthquake shaking) when landfill slopes are subjected to seismic loading. Site-specific seismic hazard, pseudo-static slope stability, and Newmark double integration deformation analyses (Newmark, 1965) were performed to provide estimates of seismically-induced slope deformations as required by Title 27 (see Section 4.6.8). In addition to Title 27 requirements, post-earthquake static slope stability analyses were also performed in accordance with the USACE Manual EM 1110-2-1913 guidelines for seismic stability evaluation of levees. For slopes comprised of or founded on materials that their strength properties change considerably when subjected to strong ground shaking (e.g., liquefiable soils), post-earthquake static stability analyses using residual strength properties are performed to evaluate the potential for slope failure after earthquake shaking terminates. For post-earthquake stability conditions, according to the USACE Manual EM 1110-2-1913, the minimum acceptable factor of safety is 1.0. The above discussion has been added as the last paragraph to Section 1.5.5.1 (page 1-15) for clarity.