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the original drilling and construction of these wells. However, because the need for re-drilling is believed to be predictable, this activity would be scheduled to occur outside the peak use period for pelicans at the Salton Sea. In addition, a pipeline leak could result in a spill of brine. The double-walled pipeline and remotely controlled shut-off valves are expected to fully contain a spill associated with the pipeline to OB-3. Therefore, there should be no impacts to brown pelicans as a result of the failure of the primary pipeline to well pad OB-3. The pipelines associated with the other production well pads do not cross any waterways. A leak from one of these pipelines would not be expected to impact brown pelicans.

Injection well construction, operation and maintenance are not anticipated to impact California brown pelicans given the physical distance between these facilities and occupied habitat. Average noise levels from these activities would not be expected to exceed 50 dBA, and may be less, depending on the sound attenuation associated with the various physical barriers between these well pads and the Obsidian Butte roost area.

Construction, operation and maintenance of the water supply pipeline are not anticipated to impact California brown pelicans given the scale and location of this facility relative to the habitat for this species. Average noise levels at the roost islands associated with construction would be similar to or less than that for the general of the plant (54 dBA). Given that this facility would not be visible to the pelicans roosting at Obsidian Butte, construction activities would not be expected to have any adverse effects. Operation and maintenance would not be expected to require activities that would result in disturbance as they would be similar to those associated with operating the plant in general; consequently, no adverse effects to brown pelicans are anticipated.

The service water pond would be open and contain reverse osmosis reject in addition to the canal water used for various plant needs. Although the selenium concentration of this reject is expected to be on the order of 8 μg/L (as compared to a canal concentration of approximately 2 μg/L), this would comprise a small proportion of the total volume in the pond. Therefore, the overall pond concentration would be lower. This would be a lined structure and located adjacent to the cooling towers. This configuration is likely to discourage any use by brown pelicans. No additional impacts are expected as a result of the presence and use of the service water pond.

Construction and maintenance of the transmission lines may impact the California brown pelican given the small distance between the L-Line Interconnection and roosting/foraging habitat used by the pelicans at the pond located at the intersection of Lack and Lindsey Roads. Construction activities are likely to result in noise and disturbance levels that would preclude temporarily the use of this pond by brown pelicans. Based on the activities required, average noise levels at the pond could be on the order of 80 dBA. Brown pelicans in this area are not accustomed to high levels of activity, and they have been observed to move off as a result of a vehicle approaching in close proximity (Carol Roberts, CFWO, pers. obs.; Charlie Pelizza, SBSSNWR, pers. comm.). During this construction activity, this roost area is not expected to be available to brown pelicans. Maintenance of the lines is predominantly washing to remove dust build-up. This would be a short term activity and would not be preclude use of the site to the extend that adverse effects would occur.

The presence of new transmission lines may result in impacts through collision with these lines. Shields (2002) reported on several strikes that occurred in Venezuela when a power line was strung between a pelican breeding colony and the foraging area for that colony. However, the risk of strikes does appear to vary with the circumstances. The Mare Island monitoring (PG&E 1992) only recorded two pelican strikes (American white pelicans, *Pelecanus erythrorhynchos*, in that case) out of a total of 80 mortalities on that transect and 1,028 total mortalities in the study area. The L-Line Interconnection is of concern because it would be located in close proximity to roosting/foraging areas used by brown pelicans. Brown pelicans have been known to collide with power lines in the Imperial Valley (SBSSNWR, unpubl. data). In one case that involved multiple collisions of brown pelicans with a power distribution line adjacent to the Salton Sea, the situation was resolved by the addition of markers (orange balls) to the power line. No additional collisions were noted following the installation of these markers. Bird flight diverters are proposed for the majority of the project's new transmission lines, where they occur between wetlands or other pelican use areas, and this should greatly reduce the likelihood of collisions. APLIC (1994) indicates that average mortalities could be reduced by up to 90 percent with 10 cm bird flight diverters placed at 5 m intervals. By considering the specific situation and needs of the species involved, the project should reduce this further.

During overflight surveys conducted during project planning, a total of 20 brown pelicans were observed crossing the proposed L-Line Interconnection alignment, but their flight elevation was 150 feet or more above the height of the lines. These surveys were not conducted during the peak use months for the brown pelican at the Salton Sea, however. Given that destinations for brown pelicans away from the Salton Sea are expected to be some distance and transmission line segments between these areas and the Salton Sea would be marked, birds using the pond at the intersection of Lack and Lindsey Roads would be the most vulnerable to collision given the proximity of this site to the line. Numbers of pelicans using this area during the summer months are on the order of three to 20 birds at any one time (Charlie Pelizza, SBSSNWR, pers. comm.). However, the alignment of the line would not be in the direct path of pelicans moving between the pond and many other desirable locations (e.g., Alamo River Delta, New River Delta, Fish Partners Fish Farm), so collisions would not be likely without unusual circumstances such as strong winds or a sudden disturbance. Strong winds do occur in the area, but they are most common in the spring months when brown pelicans are few in number in the general area. Most human activity in the area would result from routine travel by geothermal plant staff or periodic fishing activity along the levee at Lack and Gruble Roads. Given the additional voluntary travel restrictions in the vicinity of the Lack and Lindsey Pond, pelicans would not be flushed from their roosts during conditions that would increase the risk of strikes (i.e., winds > 15 MPH or low light). In the unlikely event that pelicans left their roosts during the night to forage, this would most likely take them in the direction of the Salton Sea and away from the transmission line. Based on the information available to the Service at this time, including all of the measures that would be in place during operation of the plant and the associated transmission lines, we do not anticipate pelican strikes with the transmission lines during the operation of the plant.

Closure of the facility is not expected to result in impacts to the California brown pelican as the activities associated with closure would not result in an increased level of activity or noise beyond normal operation of the plant.

CUMULATIVE EFFECTS

Cumulative effects include the effects of future State, local or private actions that are reasonably certain to occur in the action area considered in this biological opinion. Future Federal actions that are unrelated to the proposed action are not considered in this section because they require separate consultation pursuant to section 7 of the ESA.

Several projects are planned in the action area that may affect listed species in the Imperial Valley and/or Salton Sea. However, a number of these projects require action on the part of a Federal agency, and thus would require independent review under section 7 of the ESA. Therefore, the impacts of such Federal projects are not considered to be cumulative effects. The Bureau of Reclamation (Reclamation) is the Federal lead agency on the Salton Sea Restoration Project, and the Service anticipates continuing to work with Reclamation to maximize the benefits and minimize the impacts associated with that project. Reclamation is the Federal lead agency on the Brawley wetlands demonstration project, and the Service intends to continue working with Reclamation to maximize the benefits and minimize the impacts associated with that project as it expands into other areas of the Imperial Valley.

Imperial Irrigation District - San Diego Water Authority Water Transfer

Although a formal consultation was completed for this project with Reclamation in 2002, recent changes to the project may result in the need to re-initiate consultation. Ultimately, the project impacts are intended to be permitted via a section 10(a)(1)(B) permit once a Habitat Conservation Plan has been completed. Should the modified project move forward, the salinity of the Salton Sea would increase more quickly and the elevation of the Sea would decline faster and farther than would occur without the project. This would have the effect of reducing the available pelican habitat in the vicinity of the project and reducing the likelihood that operation and maintenance of CEOE project facilities would impact California brown pelicans directly.

North Baja Powerline

The North Baja Powerline is a 6-mile powerline project in the southwest portion of the IID service area. The construction and maintenance of the powerline may result in the loss of riparian, wetland, and agricultural field habitats that may contribute to the impacts associated with the loss of these habitats from the proposed project under consultation. However, because of the linear nature of that power line project, habitat losses are not anticipated to occur in large blocks. Therefore, there would not be cumulative effects of that project in combination with the CEOE project under consultation here.

CONCLUSION

After reviewing the current status of the species, environmental baseline, effects of the proposed project's construction and operation/maintenance activities, and cumulative effects, it is the Service's biological opinion that the proposed Salton Sea Unit 6 Geothermal Power Plant is not likely to jeopardize the continued existence of the Yuma clapper rail and the California brown pelican. We reached this conclusion for the following reasons. The number of Yuma clapper rails potentially impacted by the project is small. The number of brown pelicans potentially impacted is larger, but the effects are limited to one large and one small roost area and one or two peak use periods (depending on the actual construction schedule). Most impacts are temporary in nature; permanent habitat loss is very limited. The project includes several avoidance and minimization measures that would reduce the incidental take associated with the project. No critical habitat has been designated for these species; therefore, none would be affected.

INCIDENTAL TAKE STATEMENT

Section 9 of the ESA and Federal regulation pursuant to section 4 (d) of the ESA prohibit the take of endangered or threatened species, respectively, without special exemption. Take is defined as to harass, harm, pursue, hunt, shoot, wound, kill, trap, capture, collect, or to attempt to engage in any such conduct. Harm is further defined by the Service to include significant habitat modification or degradation that results in death or injury to listed species by significantly impairing essential behavior patterns, including breeding, feeding, or sheltering. Harass is defined by the Service as intentional or negligent actions that create the likelihood of injury to listed species to such an extent as to significantly disrupt normal behavior patterns which include, but are not limited to, breeding, feeding or sheltering. Incidental take is defined as take that is incidental to, rather than the purpose of, the carrying out of an otherwise lawful activity. Under the terms of section 7(b)(4) and 7(o)(2) of the ESA, taking that is incidental to and not intended as part of the agency action is not considered to be prohibited taking under the ESA provided that such taking is in compliance with this Incidental Take Statement. This Incidental Take Statement does not address the restrictions or requirements of other applicable laws.

In conducting our analysis we have assumed that all of the avoidance and minimization measures will be implemented as described in the project description. The take described below is that which is anticipated with all of the measures in place concurrent with the construction and/or operation of the power plant and its associated facilities. If any of the avoidance and minimization measures are not implemented as described in the project description, our analysis of effects would require modification through re-initiation of the consultation to address changes not contemplated in this opinion.

The measures described below are nondiscretionary, and must be undertaken by the ACOE or made a binding condition of any grant, agreement or permit, as appropriate, for the exemption in section 7(o)(2) to apply. The ACOE has a continuing duty to regulate the activity covered by this incidental take statement. If the ACOE fails to require the applicant to implement the project (including the mitigation measures) as described above and to adhere to the terms and conditions of this incidental take statement through enforceable terms in the permit, the protective coverage of 7(o)(2) may lapse. This exemption does not take effect until the permit from the ACOE has been issued. To monitor the impact of the incidental take, the ACOE and/or the applicant must report the progress of the action and its impact on the species to the Service as specified in the incidental take statement. [50 CFR §402.14(I)(3)]

AMOUNT OR EXTENT OF TAKE ANTICIPATED

Yuma Clapper Rail

Based on available census results and approximate population levels in the project area described above, the Service anticipates that up to eight individual clapper rails may be harassed as a result of the protocol surveys conducted to monitor the impacts of project construction. These same rails may be harassed by the noise and disturbance associated with construction thus precluding them from foraging and nesting at the Union Pond and McKendry Pond sites during the peak construction period currently scheduled for November 2004 through January 2006. One Yuma clapper rail may be harmed by the loss of habitat south of McKendry Road resulting from road widening and pipeline construction. No incidental take of this species was identified as a result of long-term operation of the project; therefore, no take is exempted herein for operational impacts.

California Brown Pelican

The Service anticipates that up to 250 brown pelicans may be precluded from roosting and foraging near Obsidian Butte as a result of the harassment associated with the noise and activity of construction of Production Well Pad OB-3 and pile driving for the pipeline to well pad OB-3 and the steam turbine. These activities are anticipated to require a total of six months. Up to 20 brown pelicans may be precluded from roosting and foraging in the pond located at the intersection of Lack and Lindsey Roads as a result of the harassment associated with construction of the adjacent segment of the L-Line Interconnection. The mile-long segment in the vicinity of the pond will require approximately 45 days to complete. As a result of the avoidance and minimization measures incorporated into the project, no incidental take of this species is anticipated as a result of operation of the plant and its associated facilities. Therefore, no incidental take is exempted for operation activities.

REASONABLE AND PRUDENT MEASURES

The Service believes the following reasonable and prudent measures are necessary and appropriate to minimize the impacts of the incidental take of Yuma clapper rails and brown pelicans. These measures are based on the premise that the project, as proposed above, will be implemented in its entirety, including all specified avoidance and minimization measures outlined in the project description, unless otherwise modified below.

1. Measures shall be taken to minimize the harassment of listed species associated with construction noise and disturbance.

2. Measures shall be taken to minimize the harm of listed species associated with the loss of existing habitats.

TERMS AND CONDITIONS

To be exempt from the prohibitions of section 9 of the Act, the ACOE must comply with the following terms and conditions which implement the reasonable and prudent measures described above and outline reporting/monitoring requirements. These terms and conditions are non-discretionary and shall be included as binding conditions of any permit.

The following terms and conditions implement reasonable and prudent measure 1:

1.1 The project proponent shall conduct a noise study to determine the most effective means to reduce sound levels in habitats adjacent to the project site and production facilities. The location and composition of barriers or implementation of other sound attenuation methodologies shall be based on the noise study such that the measures have the maximum sound attenuation effect practicable (i.e., beyond reductions to 78 dBA L_{max}) in the habitat areas. The existing debris piles on Obsidian Butte shall be left in place throughout construction of the project to maintain the sound attenuation they provide to the pelican roost islands along the west side of Obsidian Butte.

1.2 Shielding on the pile driving equipment shall be oriented to maximize the noise reduction achieved in habitat to the north and northwest of the project site (i.e., Union Pond, McKendry Pond and Obsidian Butte).

1.3 The silencer on the steam blow equipment shall be oriented to maximize the noise reduction achieved in habitat to the north and northwest of the project site (i.e., Union Pond, McKendry Pond and Obsidian Butte).

1.4 Construction activities at the plant site that occur during the period from February 15 through August 31 shall be scheduled to minimize noise generation during the primary vocalization periods for the rail during the half hour before and hour after sunrise and the hour before and half hour after sunset such that noise levels at the edge and within occupied clapper rail habitat during these periods do not exceed 60 dBA L_{eq} hourly.

1.5 Well construction and drilling activities at Production Well Pads OB-1 and OB-2, and road widening and pipeline construction (including pile driving) along McKendry Road, shall be confined to the period from September 1 through February 14 to minimize disturbance to breeding Yuma clapper rails.

Construction and drilling activities at OB-3 shall be confined to the period from October 1 through February 14 to minimize the disturbance to roosting California brown pelicans and breeding Yuma clapper rails. If rails are not found to be breeding at McKendry Pond, construction and drilling activities at OB-3 shall be confined to the period from October 1 through May 31. If rails are not found to be breeding at McKendry Pond, road widening and pipeline construction activities (excluding pile driving) along McKendry Road may be completed at any time provided these activities are in compliance with Term and Condition 1.4 above.

1.6 Workers shall confine their activities to the plant or facility (well pad or pipeline) side of any existing (i.e., roads or levees) or constructed barriers to reduce the potential disruption associated with human presence adjacent to occupied listed species habitat. All workers shall be informed of this requirement as part of the Worker Environmental Awareness Program prior to conducting any project work.

1.7 Protocol surveys for Yuma clapper rails shall be conducted by qualified individuals with experience in conducting the Service protocol (Attachment 1) at Union Pond and McKendry Pond (and the adjacent part of the Vail 5 drain) prior to the start of any construction-related activities[1] within 0.5 miles of these sites, during each year that construction is occurring, and the year following the completion of construction. Individuals with a section 10 (a)(1)(A) permit from the Service to survey for Yuma clapper rails are qualified to complete this work. Qualifications shall be provided to the Service for approval at least 30 days prior to the start of the survey period.

The following terms and conditions implement reasonable and prudent measure 2:

2.1 The proposed land acquisition and habitat enhancements shall be in place prior to the start of the first rail breeding season following the initiation of fill operations associated with the construction of Production Well Pad OB-3.

Reporting Requirements

The applicant shall submit reports of the previous year's activities to the Service by March 31 of each year following years during which construction and/or monitoring activities occurred. This report shall include a summary of the status of project construction including reports on specific facilities, avoidance and minimization measures implemented in the previous year, and the results of any monitoring/survey activities conducted. The Service shall have access to the raw data from monitoring activities for review upon request. The reporting will occur annually

[1]Construction-related activities include grading, trenching, and other ground disturbing activities involved in site preparation in addition to the actual construction of permanent structures. Geo-technical studies and topographical surveys are not precluded by this Term and Condition and may occur prior to the completion of protocol surveys.

unless the Service approves a longer reporting interval. Following the completion of construction, the applicant shall submit to the Service reports of any monitoring activities conducted as a requirement of certification by the CEC.

The Service's SBSSNWR (760-348-5278) shall be notified within 24 hours of any upset condition at the power plant or its associated facilities that could impact federally-listed species, migratory birds, or SBSSNWR facilities.

The Service's Carlsbad Fish and Wildlife Office (760-431-9440) must be notified within three working days should any listed species be found dead or injured in or adjacent to the action area. A written notification must be made within five calendar days and include the date, time, and location of the discovered animal/carcass, the cause of injury or death, and any other pertinent information. Injured animals should be transported to a qualified veterinarian or permitted wildlife care facility and the Service informed of the final disposition of any surviving animal(s). All dead specimen(s)/carcass(es) shall be submitted to (1) educational/research institutions possessing the appropriate State and Federal permits, (2) Carlsbad Fish and Wildlife Office, or (3) Division of Law Enforcement (contact 310-328-1516 for further direction). Failing deposition to one of these entities, the carcass should be marked, photographed, and left in the field.

The reasonable and prudent measures, with their implementing terms and conditions, are designed to minimize the impact of incidental take that might otherwise result from the proposed action. If, during the course of the action, this level of incidental take is exceeded, such incidental take represents new information requiring re-initiation of consultation and review of the reasonable and prudent measures provided. The ACOE must immediately provide an explanation of the causes of the taking and review with the Service the need for possible modification of the reasonable and prudent measures.

CONSERVATION RECOMMENDATIONS

Section 7(a)(1) of the ESA directs Federal agencies to utilize their authorities to further the purposes of the ESA by carrying out conservation programs for the benefit of endangered and threatened species. Conservation recommendations are discretionary agency activities to minimize or avoid adverse effects of a proposed action on listed species or critical habitat, to help implement recovery plans or to develop information. The recommendations provided here do not necessarily represent complete fulfillment of the agency's 7(a)(1) responsibility for these species.

> The ACOE should work with CEOE to provide alternative roost structures for brown pelicans if monitoring of the Obsidian Butte and Lack and Lindsey Roads pond sites indicates that project construction activities are precluding the use of these sites.

For the Service to be kept informed of actions minimizing or avoiding adverse effects or benefitting listed species or their habitats, the Service requests notification of the implementation of any conservation recommendations.

RE-INITIATION NOTICE

This concludes formal consultation on CEOE's Salton Sea Unit 6 Geothermal Power Plant. As provided in 50 CFR §402.16, re-initiation of formal consultation is required where discretionary Federal agency involvement or control over the action has been retained (or is authorized by law) and if: (1) the amount or extent of incidental take is exceeded, (2) new information reveals effects of the agency action that may affect listed species or critical habitat in a manner or to an extent not considered in this opinion, (3) the action is subsequently modified in a manner that causes an effect to listed species or critical habitat that was not considered in this opinion, or (4) a new species is listed or critical habitat designated that may be affected by the action. In instances where the amount or extent of incidental take is exceeded, any operations causing such take must cease pending re-initiation.

If you have any questions about this consultation or the biological opinion, please contact Carol Roberts of my staff at (760) 431-9440 ext. 271.

Sincerely,

Therese O'Rourke
Assistant Field Supervisor

cc: Jeanette Baker, Army Corps of Engineers, San Diego Office
Jeff Hansen, MidAmerican Energy Holdings Company
Gavin Wright, Bureau of Land Management, El Centro Field Office
Natasha Nelson, California Energy Commission, Sacramento

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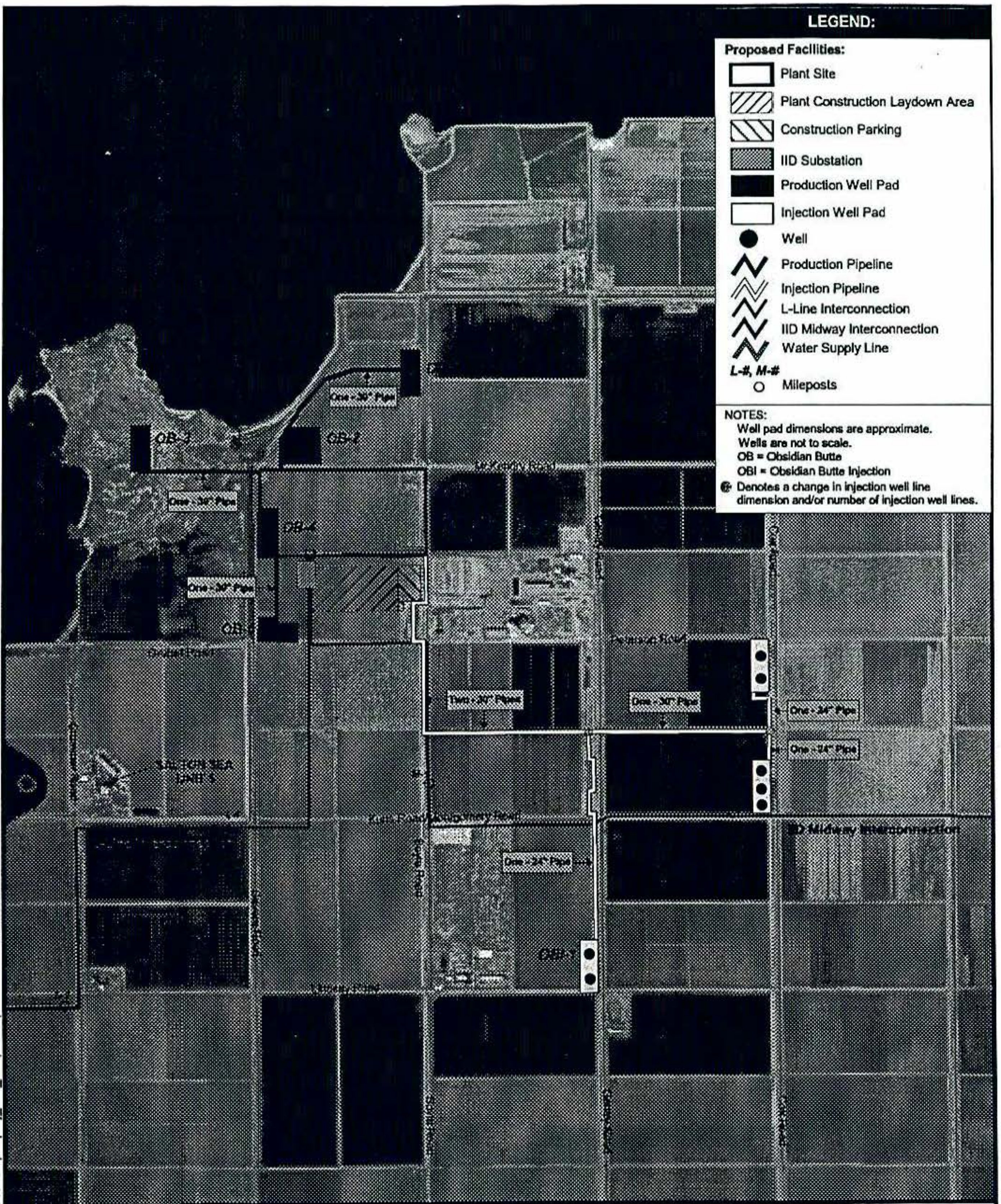

SOURCES: Imperial Irrigation District (transmission lines), CalEnergy (facility components), Aerial PhotoBank (2001 aerial).

SALTON SEA UNIT 6
PROPOSED PLANT SITE, WELL PADS, AND PIPELINES

URS

1000 0 1000 2000 Feet
SCALE: 1" = 2000' (1:24,000)

CHECKED BY:	DATE: 7-15-02	FIG. NO:
PM: PM	PROJ. NO: 58-00161046.03	3.1-4

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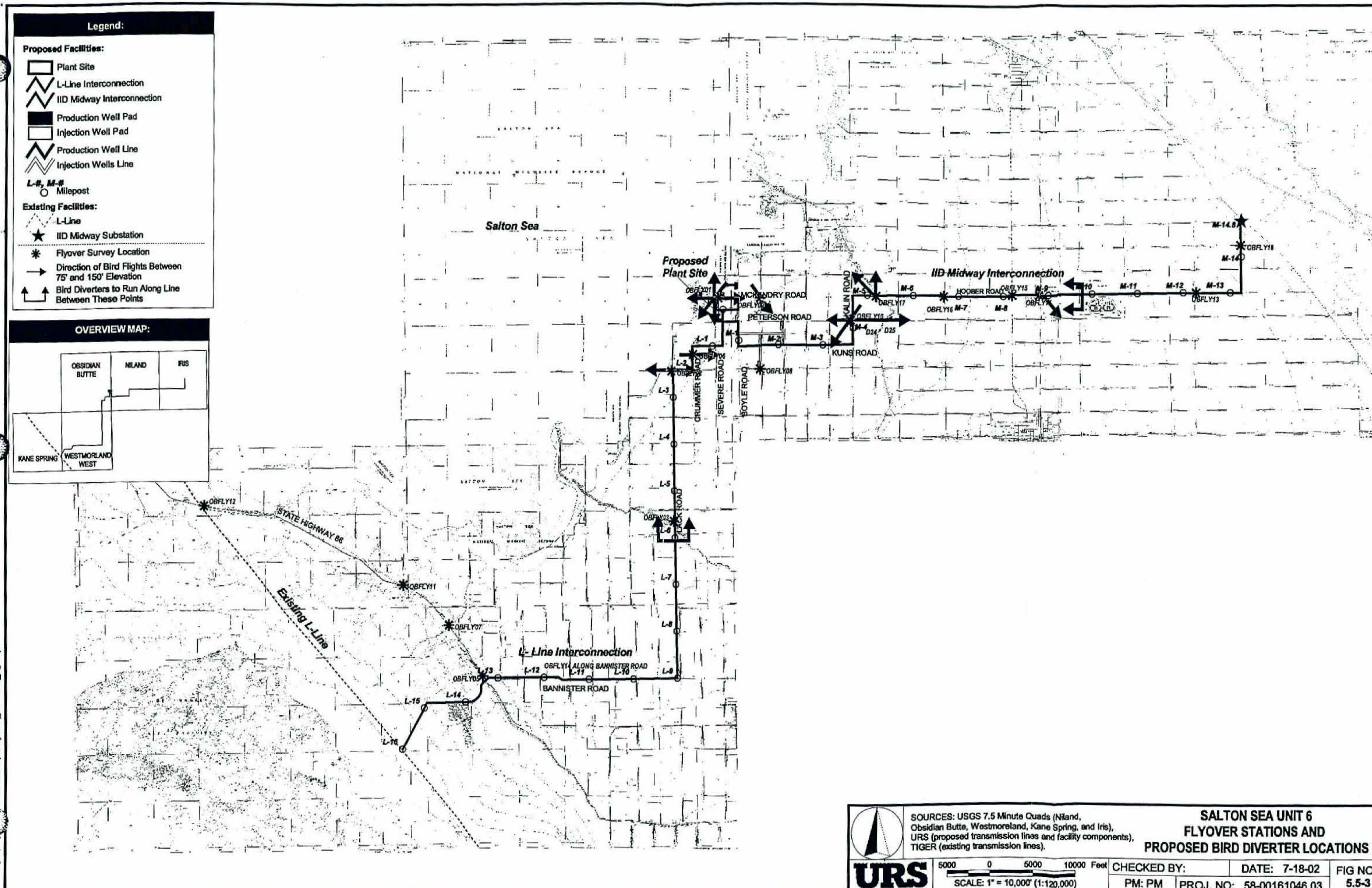
Legend:
Proposed Facilities:
Plant Site
L-Line Interconnection
IID Midway Interconnection
Production Well Pad
Injection Well Pad
Production Well Line
Injection Wells Line
L-#, M-# Milepost
Existing Facilities:
L-Line
IID Midway Substation
Flyover Survey Location
Direction of Bird Flights Between 75' and 150' Elevation
Bird Diverters to Run Along Line Between These Points
OVERVIEW MAP:
OBSIDIAN BUTTE
NILAND
IRIS
KANE SPRING
WESTMORLAND WEST
Salton Sea
Proposed Plant Site
IID Midway Interconnection
MCKENDRY ROAD
PETERSON ROAD
KALIN ROAD
KUNS ROAD
HOOBER ROAD
GRUMMER ROAD
SEVERE ROAD
BOYLE ROAD
JACK ROAD
BANNISTER ROAD
OBFLY14 ALONG BANNISTER ROAD
STATE HIGHWAY 86
Existing L-Line
L-Line Interconnection
OBFLY12
OBFLY11
OBFLY07
OBFLY05
OBFLY08
OBFLY13
OBFLY15
OBFLY16
OBFLY17
OBFLY18
M-14.8
M-14
M-13
M-12
M-11
M-7
M-8
M-6
M-3
M-2
L-1
L-2
L-3
L-4
L-5
L-6
L-7
L-8
L-9
L-10
L-11
L-12
L-13
L-14
L-15
L-16
D24
D25
SOURCES: USGS 7.5 Minute Quads (Niland, Obsidian Butte, Westmoreland, Kane Spring, and Iris), URS (proposed transmission lines and facility components), TIGER (existing transmission lines).
SALTON SEA UNIT 6
FLYOVER STATIONS AND
PROPOSED BIRD DIVERTER LOCATIONS
URS
5000 0 5000 10000 Feet
SCALE: 1" = 10,000' (1:120,000)
CHECKED BY:
PM: PM
DATE: 7-18-02
PROJ. NO: 58-00161046.03
FIG NO: 5.5-3

United States Department of the Interior
U.S. Fish and Wildlife Service
2321 West Royal Palm Road, Suite 103
Phoenix, Arizona 85021-4951
Telephone: (602) 242-0210 FAX: (602) 242-2513

In Reply Refer To:

AESO/SE

January 23, 2003

Memorandum

To: Yuma Clapper Rail Interested Parties

From: Acting Field Supervisor

Subject: Change to January 2000 Yuma Clapper Rail Survey Protocol

The Fish and Wildlife Service, Arizona Ecological Services Field Office appreciates receiving your input and suggestions concerning the proposed extension of the official survey period for the Yuma Clapper Rail Survey Protocol. The consensus of respondents was that the extension was appropriate. Please revise the January 2000 survey protocol as follows:

4. All official surveys must be conducted between March 15 and May 31. *(Rest of this paragraph remains unchanged)*

All other components of the survey remain the same. Surveyors can use the additional two weeks to complete their surveys as necessary. Surveys do not have to be extended into this additional period if the surveying agency does not wish to use it.

Please provide the results of the 2003 surveys to our office by July 1, 2003 so we may compile the results. Copies of the data sheets should be sent, along with a summary of results.

On the other topic of interest for the Yuma clapper rail, the FWS will be working with the University of Arizona to seek funding for the development of a training course for surveyors. Considerable interest in such training was expressed to us, and we believe training would be beneficial and assist in our efforts. Training before the 2003 field season is not likely; however, we hope to be operational for the 2004 season.

We would also like to schedule a meeting in August to discuss research and management issues that remain unresolved for the Yuma clapper rail. If there are issues you would like to see addressed, or know of funding sources that may be useful, please let us know by July 1, 2003.

U.S. Fish and Wildlife Service
2321 West Royal Palm Road, Suite 103
Phoenix, Arizona 85021-4951
Telephone: (602) 640-2720 FAX: (602) 640-2730

In Reply Refer To:
AESO/SE

February 15, 2000

Memorandum

To: Yuma Clapper Rail Interested Parties

From: Field Supervisor

Subject: Yuma Clapper Rail Revised Survey Protocol

The Fish and Wildlife Service has completed the revision of the Yuma clapper rail official survey protocol. This protocol is for use starting in the 2000 survey season. The Service would like to thank all those who provided materials, comment and other input to the draft protocols. Not all suggestions proved workable, but we have attempted to incorporate as many suggestions as possible.

The basic parameters of the protocol have not changed, however there are some significant differences to be aware of. The survey period now includes the first part of March and routes have been formally assigned. There is a new tape to use during survey and a training tape with rail calls to familiarize yourself with the calls you will hear. Copies of the survey tapes are included in this information package. The training tape will be mailed separately. Please review all the materials before going into the field.

Please provide copies of the survey results to this office by July 1, 2000.

If there are any questions concerning the survey, the survey materials, or other issues involving the Yuma clapper rail, please contact Lesley Fitzpatrick (x236) or Tom Gatz (x240). Thank you for your participation in the survey.

David L. Harlow

David L. Harlow

Attachments

Yuma clapper rail 2000 protocol.LAF

YUMA CLAPPER RAIL SURVEY PROTOCOL
JANUARY 2000

These instructions are for the official surveys for Yuma clapper rail (*Rallus longirostris yumanensis*) which are used to provide information on population trends of this endangered species. Significant changes have been made from earlier survey protocols and these instructions require the use of the new survey tape. These instructions will be in place for the 2000-2004 survey seasons, after which the Fish and Wildlife Service will review them in concert with the Yuma Clapper Rail Recovery Team. If there are questions about this survey protocol, or to obtain cassette tapes for use in the survey, please contact the Arizona Ecological Services Office at the address at the end of this document.

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1. Please review the list of official survey locations on pages 3 and 4. If your agency will be unable to survey any or all of the assigned locations, please contact the AESO as soon as possible so we can try and find volunteers to survey the location.

2. Before any survey for the Yuma clapper rail, review the training tape and the survey tape to become familiar with the various calls. The tapes repeat various "clatter" and "kek" calls and are 60 minutes long. This will allow you to complete several stops before having to rewind. Also, make sure your tape recorder and speaker produce good quality sound at 80 decibels, measured one meter from the speaker.

3. Use 1:24000 USGS topolographic maps for base maps. Sections of the map should be enlarged to show the survey location and route. Before beginning the survey, review maps of past surveys. Note especially the placement of "stops" from previous years. The same stops should be used, maintaining the same number. Any new stops added should have a unique number and be recorded on the map. GPS may be used to more carefully delineate stop locations.

4. All official surveys must be conducted between March 15 and May 15. The survey protocol calls for 2 surveys of each location or route per year. A third survey can be added if time and staff resources permit. There is a minimum of one week between surveys. Surveys should be conducted on the same routes used in previous years. Survey stops should be at 150-200 meter intervals unless local conditions warrant a different distance. Make sure the route and all stops are clearly recorded on the survey map.

5. Arrive at the survey location to begin surveying about 30 minutes before sunrise. Surveys should continue no later than 3 hours after sunrise. No evening surveys should be conducted for the official survey.

6. Upon reaching the location, fill in the weather information section of the cover sheet. If the wind speed is greater than 10 mph (a breeze that keeps leaves and small twigs in constant motion or extends a light flag), do not conduct official surveys. Responses to the calls are difficult to hear over the rustling of marsh vegetation.

7. For the survey, get as close to the marsh vegetation as possible at each stop. Note the time in the "time start" column. Wait quietly for one minute to listen for rails. Then play the tape, directing the speaker toward the marsh and at approximately 80 decibels volume. At each stop, play the tape for 2 minutes, turn it off for 2 minutes, turn it on for 2 minutes and turn it off then listen for one minute (total survey time 7 minutes). Keep to the 2 minute intervals as carefully as possible. Listen for rail responses during the entire period and record responses on the data sheet.

8. Record responses from each rail on a different line. If you do see/hear a pair, record the individuals separately and check the "was rail paired" column. All rails seen or heard at stops during the survey are to be counted. If you hear the same rail twice, only count it as one bird. Rails heard or seen at other times while on site during the survey are incidental and are recorded at the bottom of the data sheet. Since some observers are interested in other species, there is a column to record other species of birds observed during the survey on the data sheet.

9. After the survey has been completed, record on the cover sheet any events or disturbances that may have affected the survey results (other loud birds, boat or vehicle noise, etc.). Also, record the weather conditions. Make any other notes of observations of other species (as appropriate).

10. Please make sure the cover and data sheets are clearly filled out. The information can be used to define rails/station (all rails seen/heard), rails/stop (rails seen/heard at each stop or an average) and rails/hour (each stop has 7 minutes of survey time) after the surveys have been completed. The official survey will continue to look at rails per station.

11. Completed reports are due to AESO by July 1 of the survey year. Reports will include cover and data sheets and a map showing the survey route. Send completed survey forms and maps to:

    Yuma Clapper Rail Coordinator
    USFWS-AESO
    2321 W. Royal Palm Rd. Suite 103
    Phoenix, Arizona 85021
    602/640-2720 FAX 602/640-2730

YUMA CLAPPER RAIL SURVEY
COVER SHEET
(JANUARY 2000)

Date:________________

Location Information:
Location Name________________________________ Route____________

Map Name______________________ Township/Range/Section______________

Observer(s)_____________________________________

Weather:
Start %Cloud Cover___Temp______Wind Speed_______
End % Cloud Cover___Temp______Wind Speed_______

Data Summary:

1) Total individual rails seen or heard while surveying __________
2) Number of other rails seen or heard (incidentals) __________

Total rails per route or location equals #1+#2 __________
For rails/hour, each stop is 7 minutes

Observations:

Events during survey that may have affected results:

Other Observations/Comments:

Yuma Clapper Rail Survey Data Sheet

January 2000

Location____________________ Route____________________ Date_______________

Weather-start____________________ end____________________ Observer____________________

Stop #	Time Start	Time Stop	Clatter Call	Kek Call	Other Call	Was Rail Seen?	Was Rail Heard?	Was Rail Paired?	Other Species?	Habitat Type Where Rail Was Detected

[illegible] total:

Total rails recorded on survey _________

Incidental observations of rails in survey area ___________

ENGP Attachment DRR 35 Confidential OHP Built Environmental Resources Directory and Archaeological Determinations of Eligibility

Attachment DRR 35, OHP Built Environmental Resources Directory and Archaeological Determinations of Eligibility has been provided under a request for confidential designation.

ENGP Attachment DRR 49 LCI Report No. LE111, 2011

Draft Geotechnical Report

Black Rock 5-6
Calipatria, California

Prepared for:

Cal Energy
3075 Gentry Road
Calipatria, CA 92252

LANDMARK
Geo-Engineers and Geologists

Prepared by:

Landmark Consultants, Inc.
780 N. 4th Street
El Centro, CA 92243
(760) 370-3000

August 2011

LANDMARK
Geo-Engineers and Geologists

780 N. 4th Street
El Centro, CA 92243
(760) 370-3000
(760) 337-8900 fax

77-948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665
(760) 360-0521 fax

August 8, 2011

Mr. Robert Eberle
CalEnergy Operating Corporation
7030 Gentry Road
Calipatria, CA 92233

Draft Geotechnical Report
Black Rock 5-6 Geothermal Power Plant
West of Garst Road and North of Sinclair Road
Calipatria, California
LCI Report No. LE11138

Dear Mr. Eberle:

This draft geotechnical report is provided for planning and design of the Black Rock 5-6 Geothermal Power Plant located approximately ¼ mile north of Sinclair Road on the west side of Garst Road northwest of Calipatria, California. Our geotechnical investigation was conducted in response to your request for our services in accordance with the scope of work developed by WorleyParsons, CalEnergy's power plant engineering consultant. The enclosed report describes our soil engineering investigation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design of the project.

This executive summary presents ***selected*** elements of our findings and recommendations only. It ***does not*** present all details needed for the proper application of our findings and recommendations. Our findings, recommendations, and application options are related ***only through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them.

The general findings of this study follow:

1. Soft of loose sandy clays and silt predominate the upper 60 to 65 feet of the site.

2. A medium dense to dense silty sand layer was encountered from 60 to 70 feet below existing ground surface (bgs).

3. Groundwater is encountered at 3.5 feet (north) to 5.0 feet (south) bgs.

4. The site is considered "unsuitable" for wastewater disposal due to high groundwater and a very low infiltration capacity of the surface clays.

5. Liquefaction settlement within the upper 50 feet of soil profile is estimated at 2.5 to 6.0 inches (Boring B-13 has an estimated settlement potential of 9.0 inches and Boring B-24 has an estimated settlement potential of 7.0 inches).

6. Ground subsidence sinkholes have historically occurred along the existing geothermal fluids transport pipeline that transverses the north end of this site.

7. The surface slays exhibit "high" swell potential (Expansion Index of 100 to 130).

8. The site elevation is generally 6.0 feet below the minimum building elevation established by Imperial County (Elevation -220). The Salton Sea levee, located along the north boundary of this site may provide adequate flood protection to satisfy the County building ordinance.

9. Subsurface agricultural tile drainage pipelines exist at this site, used to control groundwater depth and to remove excess salts from irrigation water which infiltrates the soil.

10. The native soil is severely corrosive to metals and contains sufficient sulfates and chlorides to require special concrete mixes (6.5 sack cement factor with a 0.45 maximum water cement ratio and Type V cement) and protection of embedded steel components (3 inch minimum concrete cover when concrete is placed in contact with native soil and 5 inch cover when exposed to geothermal brines).

We appreciate the opportunity to provide our findings and professional opinions regarding geotechnical conditions at the site. If you have any questions or comments regarding our findings, please call our office at (760) 370-3000.

Respectfully Submitted,
Landmark Consultants, Inc.

PROFESSIONAL GEOLOGIST STEVEN K. WILLIAMS CERTIFIED ENGINEERING GEOLOGIST CEG 2261 STATE OF CALIFORNIA

Steven K. Williams, PG, CEG
Senior Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER JULIAN R. AVALOS No. 73339 EXPIRES 12-31-12 CIVIL STATE OF CALIFORNIA

Julian R. Avalos, PE
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER JEFFREY O. LYON No. 31921 EXPIRES 12-31-12 CIVIL STATE OF CALIFORNIA

Jeffrey O. Lyon, PE
President

Distribution:
Client (4)

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APPENDICES

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical investigation of the Black Rock 5-6 Geothermal Power Plant located approximately ¼ mile north of Sinclair Road on the west side of Garst Road about 6 miles northwest of Calipatria, California (See Vicinity Map, Plate A-1). The proposed geothermal plant will include a turbine-generator structure, cooling tower, wellhead separators, scrubbers and demisters, water tanks, brine ponds, gas removal systems, production test units, rock mufflers, control and operation buildings, electrical substation, geothermal fluids pipelines and supports, various ancillary structures and associated internal roadways. A raw water pond (process water) is planned to be constructed at the south side of the proposed geothermal plant site. Geothermal wells are planned to be located near the plant site. All geothermal fluids will be piped (above ground) to the power production site and piped away from the site for reinjection. A site plan for proposed development is shown on Plate A-2 (Appendix A).

The non-power generation structures (control and operating building, etc.) are planned to consist of slab-on-grade foundations with steel-frame construction. Expected footing loads are estimated at 1 to 2 kips per lineal foot for the small structures. Expected plant components, separators, scrubber, demisters, and turbine/generator loads range from 60 to 2,000 kips.

Site development will include deep ground improvement to control settlements, deep foundations for structures, building support pad construction, utility installation, on-site water and wastewater systems installation, roadway construction and concrete flatwork placement.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 120 feet of subsurface soil at selected locations within the site for evaluation of physical/engineering properties, conforming to the scope of work developed by WorleyParsons, CalEnergy's power plant engineering consultant. From the subsequent field and laboratory data, professional opinions were developed and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction.

The scope of our services consisted of the following:

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected samples.
- Review of the available literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.
- Preparation of this report presenting our findings, professional opinions, and recommendations for the geotechnical aspects of project design and construction.

This report addresses the following geotechnical issues:

- Subsurface soil and groundwater conditions
- Site geology, regional faulting and seismicity, and site seismic design criteria
- Liquefaction potential and its mitigation
- Expansive soil and methods of mitigation
- Aggressive soil conditions to metals and concrete

Professional opinions with regard to the above issues are presented for the following:

- Site grading, earthwork and embankment construction
- Building pad and foundation subgrade preparation
- Allowable soil bearing pressures and expected settlements
- Deep foundation alternatives
- Soil improvement alternatives
- Concrete slabs-on-grade
- Lateral earth pressures
- Excavation conditions and buried utility installations
- Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
- Seismic design parameters
- Pavement structural sections
- On site sewage disposal system

Our scope of work for this report did not include an evaluation of the site for the presence of environmentally hazardous materials or conditions, groundwater mounding, or landscape suitability of the soil.

1.3 Authorization

Mr. Robert Eberle with CalEnergy provided written authorization to proceed with our work by Purchase Order agreement (PO# IBR-82689) on June 11, 2011. We conducted our work according to our written proposal dated May 27, 2011.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

A Test Location Plan, showing type, location, number, and depth of subsurface exploration points, was provided by WorleyParsons of Folsom, California. GPS coordinates were provided by WorleyParsons for the CPT and boring locations and located in the field by handheld GPS unit. The boring and CPT locations are shown on the Site and Exploration Plan (Plate A-2). The plant site is currently in Bermuda grass crop. The field investigation was conducted between a harvesting and rewatering cycle.

The subsurface exploration also included twenty-eight (28) borings to depths of 20 to 120 feet below existing ground surface. The borings were performed on June 28 through July 6, 2011 using 2R Drilling of Ontario, California. The drilling program consisted of three (3) borings to a depth of 120 feet below ground surface, thirteen (13) borings to a depth of 100 feet, two (2) borings to a depth of 75 feet, one (1) boring to a depth of 50 feet, three (3) borings to a depth of 30 feet, and six (6) borings to a depth of 20 feet at specified locations within the project site.

The borings were advanced with a CME 55 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. Sampling was conducted at 2.5 foot intervals in the upper 20 feet and at 5 foot intervals below 20 feet.

A professional geologist or engineer observed the drilling operations and maintained logs of the soil encountered with sampling depths. Soils were visually classified during drilling according to the Unified Soil Classification System and relatively undisturbed and bulk samples of the subsurface materials were obtained at selected intervals. The relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler. In addition, Standard Penetration Tests (SPT) was performed in accordance with ASTM D1586. The samples were obtained by driving the samplers ahead of the auger tip at selected depths using a 140-pound CME automatic hammer with a 30-inch drop. The number of blows required to drive the samplers the last 12 inches of an 18-inch drive depth into the soil is recorded on the boring logs as "blows per foot". Blow counts (N values) reported on the boring logs represent the field blow counts.

No corrections have been applied to the blow counts shown on the boring logs for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter. Pocket penetrometer readings were also obtained to evaluate the stiffness of cohesive soils retrieved from sampler barrels. After logging and sampling the soil, the exploratory borings were backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill.

Subsurface exploration also included electronic Cone Penetrometer Testing (CPT), performed on June 28, 2011 using Middle Earth Geotesting, Inc. of Orange, California to advance five (5) CPT soundings to approximate depths of 50 to 120 feet below existing ground surface. CPT soundings provide a continuous profile of the soil stratigraphy with readings every 2.5cm (1 inch) in depth. Direct sampling for visual and physical confirmation of soil properties has been used by our firm to establish direct correlations with CPT exploration in this geographical region.

The CPT exploration was conducted by hydraulically advancing an instrumented Hogentogler 10cm^2 conical probe into the ground at a rate of 2cm per second using a 23-ton truck as a reaction mass. An electronic data acquisition system recorded a nearly continuous log of the resistance of the soil against the cone tip (Qc) and soil friction against the cone sleeve (Fs) as the probe was advanced. Empirical relationships (Robertson and Campanella, 1989) were then applied to the data to give a continuous profile of the soil stratigraphy. Interpretation of CPT data provides correlations for SPT blow count, phi (ϕ) angle (soil friction angle), undrained shear strength (S_u) of clays and over-consolidation ratio (OCR). These correlations may then be used to evaluate vertical and lateral soil bearing capacities and consolidation characteristics of the subsurface soil.

Shear wave velocity was determined for the subsurface soils to a depth of 120 feet at CPT-2 and CPT-3. Shear wave velocities ranged from 142 m/sec to 292 m/sec and averaged 211 m/sec (692 ft/sec) for the upper 120 feet. The site soils have been classified as Site Class D (stiff soil profile).

Offsite subsurface exploration was performed at the proposed borrow site on June 28, 2011 by using a backhoe to excavate seven (7) test pits to an approximate depth of 5 to 10 feet below the existing ground surface. The proposed borrow site is located at the southwest corner of Pound Road and English Road about 3.5 miles northeast of the plant site. The test pit locations are shown on the Borrow Area Site and Exploration Plan (Plate A-3).

Bulk samples were obtained at selected depths in the test pits. The test pits were located by taped or paced measurements and should be considered approximate.

Interpretive logs of the CPT soundings and logs of the test borings and test pits were produced after review of field and laboratory test data and are presented on Plates B-1 through B-40 in Appendix B of this report. Keys to the interpretation of CPT soundings, logs of test borings and test pits are presented on Plate B-41 and B-42.

2.2 Laboratory Testing

Laboratory tests were conducted on selected bulk (auger cuttings) and relatively undisturbed soil samples obtained from the soil boring to aid in classification and evaluation of selected engineering properties of the site soils. The tests were conducted in general conformance to the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory testing program consisted of the following tests:

- Plasticity Index (ASTM D4318)
- Particle Size Analyses (ASTM D422)
- Unit Dry Densities (ASTM D2937)
- Unconfined Compressive Strength (ASTM D2216)
- Moisture Contents (ASTM D2216)
- Specific Gravity (ASTM D854)
- Expansion Index (ASTM D4829)
- Moisture-Density Relationships (ASTM D1557)
- Chemical Analyses
  (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods)
- One-Dimensional Consolidation (ASTM D2435)
- R-Value (ASTM D2844)

The laboratory test results are presented on Plates C-1 through C-31 in Appendix C.

Engineering parameters of soil strength, compressibility and relative density utilized for developing design criteria provided within this report were either extrapolated by correlations from SPT blow counts, CPT data, or from physical and engineering properties obtained from the laboratory testing program.

Section 3
DISCUSSION

3.1 Site Conditions

The project site is located approximately ¼ mile north of Sinclair Road on the west side of Garst Road approximately 6 miles northwest of Calipatria, California. The project site is located in an active agricultural field currently in Bermuda grass crop. The project site is bounded on the east by a concrete irrigation ditch, powerline, and Garst Road. An agricultural field and Sinclair Road are located to the south of the site. The JJ Elmore Geothermal Power Plant is located adjacent to the southwest corner of the project site. Geothermal brine pipelines (production and/or injection) are located to the north and west of the project site. The Salton Sea is located north of the project site and has an approximately 8 foot high embankment (levee) separating the sea from the project site.

The project site lies at an elevation of approximately 226 feet below mean sea level (MSL) (El. 774 local datum) in the Imperial Valley region of the California low desert. In general, Imperial County regulations require all structures to be constructed above the -220 MSL contour. This will result in placing approximately 6 feet of fill at main structure locations.

The surrounding properties lie on terrain which is flat (planar), part of a large agricultural valley, which was previously an ancient lake bed covered with fresh water to an elevation of 43± feet above MSL. Annual rainfall in this arid region is less than 3 inches per year with four months of average summertime temperatures above 100 °F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located in the Imperial Valley portion of the Salton Trough physiographic province. The Salton Trough is a topographic and geologic structural depression resulting from large scale regional faulting. The trough is bounded on the northeast by the San Andreas Fault and Chocolate Mountains and the southwest by the Peninsular Range and faults of the San Jacinto Fault Zone. The Salton Trough represents the northward extension of the Gulf of California, containing both marine and non-marine sediments since the Miocene Epoch.

Tectonic activity that formed the trough continues at a high rate as evidenced by deformed young sedimentary deposits and high levels of seismicity. Figure 1 shows the location of the site in relation to regional faults and physiographic features.

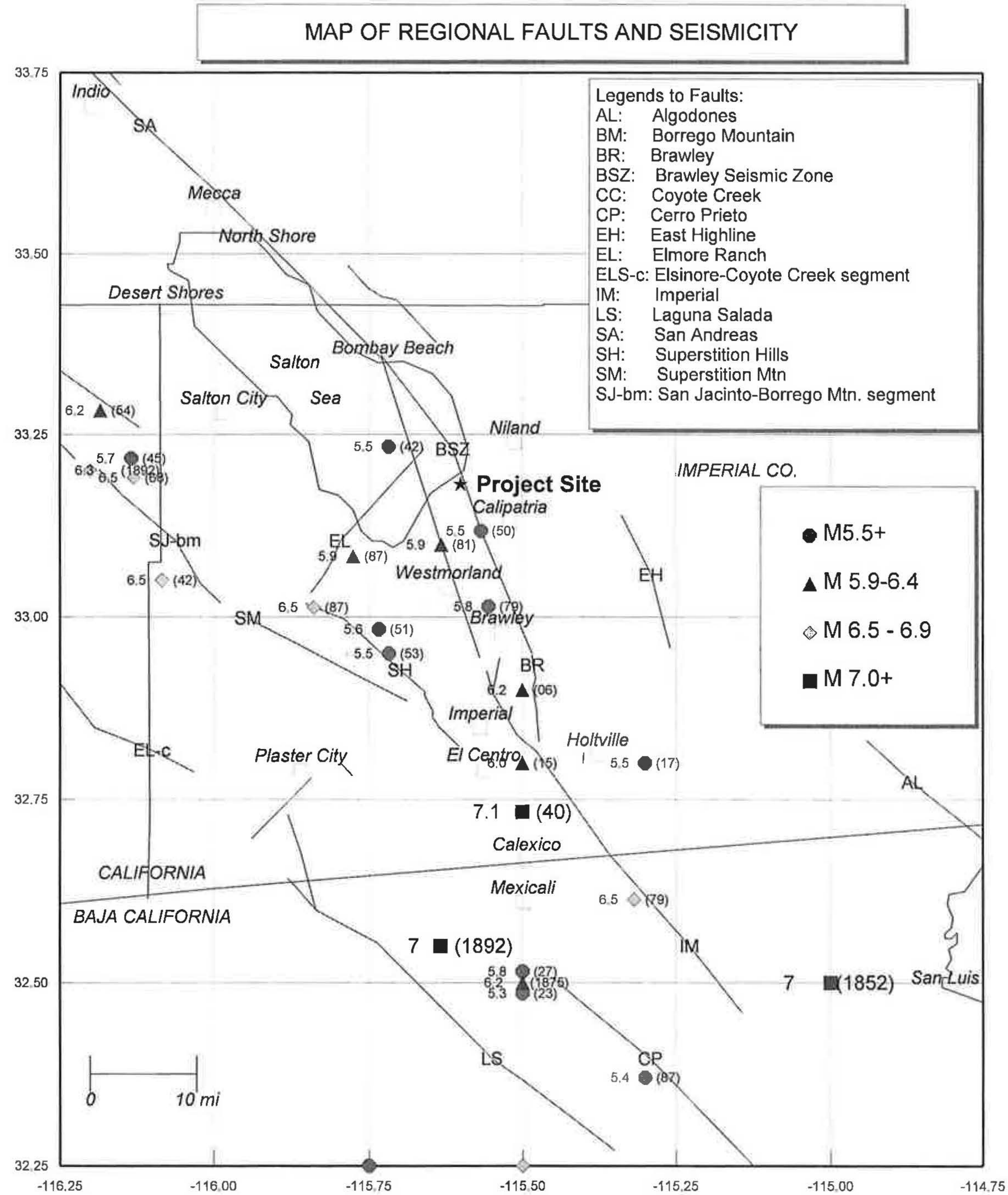
The Imperial Valley is directly underlain by lacustrine deposits, which consist of interbedded lenticular and tabular silt, sand, and clay. The Late Pleistocene to Holocene lake deposits are probably less than 100 feet thick and derived from periodic flooding of the Colorado River which intermittently formed a fresh water lake (Lake Cahuilla). Older deposits consist of Miocene to Pleistocene non-marine and marine sediments deposited during intrusions of the Gulf of California. Basement rock consisting of Mesozoic granite and Paleozoic metamorphic rocks are estimated to exist at depths between 15,000 - 20,000 feet.

The project site lies within the Brawley Seismic Zone (BSZ), a pull-apart basin between the southern terminus of the San Andreas Fault and the northern trace if the Imperial Fault. The BSZ is composed of numerous cross-cutting high angle normal faults. The seismicity of the BSZ is described in more detail in Section 3.3.

3.3 Seismicity and Faulting

<u>Faulting and Seismic Sources:</u> We have performed a computer-aided search of known faults or seismic zones that lie within a 62 mile (100 kilometer) radius of the project site as shown on Figure 1 and Table 1. The search identifies known faults within this distance and computes deterministic ground accelerations at the site based on the maximum credible earthquake expected on each of the faults and the distance from the fault to the site. A narrative of the most significant seismic sources affecting the project site follows.

- **<u>Brawley Seismic Zone:</u>** The current model for seismic and tectonic activity south of the San Andreas Fault is associated with interaction of transform faulting and spreading centers. The model depicts the Pacific Plate moving to the northwest relative to the North American Plate, along a series of subparallel, northwest trending, right lateral, en echelon faults, that results in the land being pulled apart at spreading centers. The northwest trending faults terminate at these centers, though continued transform movements are shifted across the spreading zone to the adjacent transform fault.



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Faults and Seismic Zones from Jennings (1994), Earthquakes modified from Ellsworth (1990) catalog.

Figure 1. Map of Regional Faults and Seismicity

Table 1
FAULT PARAMETERS & DETERMINISTIC
ESTIMATES OF PEAK GROUND ACCELERATION (PGA)

Fault Name or Seismic Zone	Distance (mi) & Direction from Site		Fault Type		Fault Length (km)	Maximum Magnitude Mmax (Mw)	Avg Slip Rate (mm/yr)	Avg Return Period (yrs)	Date of Last Rupture (year)	Largest Historic Event >5.5M	(year)	Est. Site PGA (g)
Reference Notes: (1)			(2)	(3)	(2)	(4)	(3)	(3)	(3)	(5)		(6)
Imperial Valley Faults												
Brawley Seismic Zone	0.4	ENE	B	B	42	**6.4**	25	24		5.9	1981	**0.45**
East Highline Canal	15	E	C	C	22	**6.3**	1	774				**0.13**
Imperial	17	SSE	A	B	62	**7.0**	20	79	1979	7.0	1940	**0.18**
Brawley	17	SSE	B	B	14	**7.0**	20	---	1979	5.8	1979	**0.17**
Cerro Prieto	47	SSE	A	B	116	**7.2**	34	50	1980	7.1	1934	**0.09**
San Jacinto Fault System												
- Elmore Ranch	4.8	NW	B	A	29	**6.6**	1	225	1987	5.9	1987	**0.33**
- Superstition Hills	17	SW	B	A	22	**6.6**	4	250	1987	6.5	1987	**0.14**
- Superstition Mtn.	21	SSW	B	A	23	**6.6**	5	500	1440 +/-			**0.12**
- Borrego Mtn	25	WSW	B	A	29	**6.6**	4	175		6.5	1942	**0.11**
- Anza Segment	31	W	A	A	90	**7.2**	12	250	1918	6.8	1918	**0.13**
- Coyote Creek	36	W	B	A	40	**6.8**	4	175	1968	6.5	1968	**0.09**
- Hot Spgs-Buck Ridge	43	WNW	B	A	70	**6.5**	2	354		6.3	1937	**0.07**
- Whole Zone	21	SSW	A	A	245	**7.5**	---	---				**0.20**
Elsinore Fault System												
- Laguna Salada	35	SSW	B	B	67	**7.0**	3.5	336		7.0	1891	**0.10**
- Coyote Segment	37	SW	B	A	38	**6.8**	4	625				**0.09**
- Earthquake Valley	48	W	B	A	20	**6.5**	2	351				**0.06**
- Julian Segment	50	WSW	A	A	75	**7.1**	5	340				**0.08**
- Whole Zone	37	SW	A	A	250	**7.5**	---	---				**0.13**
San Andreas Fault System												
- Coachella Valley	14	NW	A	A	95	**7.4**	25	220	1690+/-	6.5	1948	**0.25**
- San Gorgonio-Banning	56	NW	A	A	98	**7.4**	10	---	1690+/-	6.2	1986	**0.09**
- Whole S. Calif. Zone	14	NW	A	A	440	**7.9**	---	---	1857	7.8	1857	**0.33**

Notes:

1. Jennings (1994) and CDMG (1996)
2. CDMG (1996), where Type A faults -- slip rate >5 mm/yr and well constrained paleoseismic data
   Type B faults -- all other faults.
3. WGCEP (1995)
4. CDMG (1996) based on Wells & Coppersmith (1994)
5. Ellsworth Catalog in USGS PP 1515 (1990) and USBR (1976), Mw = moment magnitude,
6. The deterministic estimates of the Site PGA are based on the attenuation relationship of:
   Boore, Joyner, Fumal (1997)

This zone of crustal rifting and intense seismic activity is known as the Brawley Seismic Zone in the Imperial Valley.

The Brawley Seismic Zone (BSZ) extends northward beyond the termination of the mapped Imperial/Brawley faults to beneath the Salton Sea, where it terminates upon intersecting the San Andreas Fault near Bombay Beach. The Brawley Seismic Zone was the source of the 1981 5.9M_W Westmorland earthquake sequence that involved activity on at least seven distinct fault planes within the zone. The faults in the Brawley Seismic Zone are considered to be short enough that earthquakes much larger than 6-6.5M_W are unlikely. The California Geological Survey considers the Brawley Seismic Zone to have a Mmax magnitude of 6.4, with a very short 24-year average return interval, and a geologic slip rate of 25 mm/year (CDMG, 1996). Caltrans (1996) assigns a maximum magnitude of 6¼ to the BSZ.

- **San Andreas Fault:** The widely publicized San Andreas Fault lies about 14 miles north of the project site. The San Andreas Fault is a right-lateral transform fault extending for more than 600 miles northward from Bombay Beach at the east edge of the Salton Sea (13.5 miles northwest of the plant site) to off the coast of northern California at Cape Mendocino.

  The estimated characteristic earthquake is magnitude 7.4M_W for the Coachella Valley Segment that comprises the southern 115 km of the fault zone. This segment has the longest elapsed time since rupture of any portion of the San Andreas Fault. The last rupture occurred about 1690 AD, based on USGS dating of trench surveys near Indio (WGCEP, 1995). The segment has ruptured several times in the last millennium with an average recurrence interval of about 220 years. The San Andreas Fault may rupture in multiple segments producing higher magnitude earthquakes (Magnitude 7.6 to 7.9).

- **San Jacinto Fault:** The San Jacinto Fault has historically produced more large 6 to 7 magnitude earthquakes than any other fault in southern California. The San Jacinto Fault is a right-lateral strike-slip fault. The average slip rate is reported at 4-12 mm/yr. The fault is divided into five segments based on geologic characteristics and seismicity patterns. To the southwest of the Salton Sea and trending to the northwest in line with the San Jacinto Fault are two subparallel fault segments known as the Superstition Hills and Superstition Mountain Faults, both are roughly 15 miles long.

On November 24, 1987, a magnitude 6.5M_W earthquake ruptured the Superstition Hills Fault, triggering liquefaction within 8 miles of the project site along the Alamo River south of Calipatria, California.

- **Imperial and Brawley Faults:** The Imperial Fault is a right-lateral fault that connects the oceanic-type spreading centers located at the Brawley Seismic Zone and the Cerro Prieto geothermal area in Mexico. The Imperial Fault is about 60 miles in length. It has produced at least two large historic earthquakes. The largest events were the 7.0M_w on May 18, 1940 and 6.5M_w on October 15, 1979.

  The Brawley Fault trends to the north from an intersection with the Imperial Fault at a location about four miles northeast of the City of El Centro. This fault has a surface expression approximately 9 miles long. The Imperial and Brawley faults have ruptured synchronously during past earthquakes. Co-seismic slip was noted on both the Imperial and Brawley Faults after the April 4, 2010 7.2M_W El Mayor-Cucupah Earthquake. The northern termini of the Brawley and Imperial Faults are located at the City of Brawley (14.5 miles southeast of the plant site).

Seismic Hazards.

- **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the Imperial, Brawley, and San Andreas Faults and the Brawley Seismic Zone.
- **Surface Rupture.** The project site does not lie within a State of California, Alquist-Priolo Earthquake Fault Zone. Surface fault rupture is considered to be unlikely at the project site because of the well-delineated fault lines through the Imperial Valley as shown on USGS and CGS maps.

  Liquefaction. Liquefaction is a potential design consideration because of underlying saturated sandy substrata. The potential for liquefaction at the site is discussed in more detail in Section 3.7.

Other Secondary Hazards.

- **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps of the region and no indications of landslides were observed during our site investigation.
- **Volcanic hazards.** The site is located in close proximity (1 to 2 miles) to a known volcanically active area (Obsidian Buttes and Red Hill). The risk of volcanic hazards is considered low.
- **Tsunamis, seiches, and flooding.** The site lies adjacent to the Salton Sea. The project area lies near the Salton Sea that has a water level above the project area ground elevation. The Salton Sea is retained by low levees. The threat of a minor tsunami, sieches (standing waves), or other seismically-induced flooding is possible. Depending on the surface elevation of the Salton Sea, water may slosh over the levees or short reaches of the levees may breach.
- **Expansive soil.** In general, much of the near surface soils in the Imperial Valley consist of silty clays and clays which are moderate to highly expansive. The expansive soil conditions are discussed in more detail in Section 3.5.
- **Underground Carbon Dioxide Gas.** The site lies near a large reservoir of carbon dioxide gas as evidenced by open craters with bubbling gas and mud pot domes. Measured gas pressures at nearby sites range from 15 to 25 psi at 50 to 75 feet below ground surface. The presence of carbon dioxide gas was not noted in any borings during the site exploration.

3.4 Site Acceleration and UBC Seismic Coefficients

Seismic Risk: The project site is located in the seismically active Imperial Valley of southern California and is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. The proposed site structures should be designed in accordance with the 2010 California Building Code (CBC) for a "Maximum Considered Earthquake" (MCE) and with the appropriate site coefficients.

Site Acceleration: Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Accelerations also are dependent upon attenuation by rock and soil deposits, direction of rupture and type of fault; therefore, ground motions may vary considerably in the same general area.

CBC Seismic Coefficients: The 2010 California Building Code (CBC) general ground motion parameters are based on the Maximum Considered Earthquake for a ground motion with a 2% probability of occurrence in 50 years. The U.S. Geological Survey "Earthquake Ground Motion Tool", version 5.0.9a (USGS, 2009) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters shown in Table 2. The site soils have been classified as Site Class D (stiff soil profile) in accordance with an average shear wave velocity of 690 ft/sec in the upper 100 feet of soil profile.

Design earthquake ground motions are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE ground motions. Design earthquake ground motion data are provided in Table 2. A ground motion value of 0.40g ($S_{DS}/2.5$) was determined for liquefaction and seismic settlement analysis in accordance with ASCE 7-05 Section 11.8.3. The parameter S_{DS} is derived from the maximum considered earthquake spectral response acceleration for short periods (CBC Section 1613.5.4).

3.5 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on June 28 through July 6, 2011 consist of approximately 5 feet of near surface silty clays. Loose to medium dense silty sands and silts were encountered from about 5 to 35 feet below ground surface with a stiff clay layer at about 20 to 25 feet. Interbedded stiff clays, loose to dense silty sands and silts are encountered at a depth of 25 to 120 feet, the maximum depth of exploration. The subsurface logs (Plates B-1 through B-33) depict the stratigraphic relationships of the various soil types.

The native surface clays exhibit high to very high swell potential (Expansion Index, EI = 110 to 132) when tested in accordance with ASTM D4829. The clay is expansive when wetted and can shrink with moisture loss (drying).

Table 2
2010 California Building Code (CBC) and ASCE 7-5 Seismic Parameters

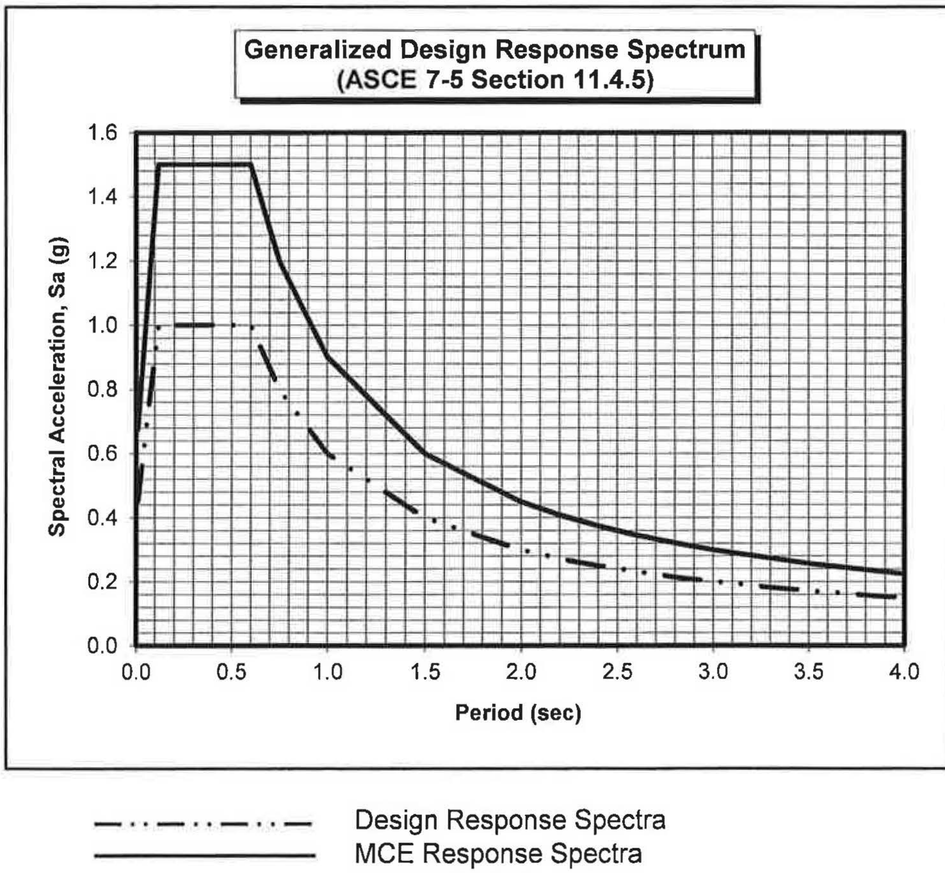
		CBC Reference
Site Class:	D	Table 1613.5.2
Latitude:	33.1813 N	
Longitude:	-115.6002 W	

Maximum Considered Earthquake (MCE) Ground Motion

Short Period Spectral Response	S_s	1.50 g	Figure 1613.5(3)	
1 second Spectral Response	S_1	0.60 g	Figure 1613.5(4)	
Site Coefficient	F_a	1.00	Table 1613.5.3 (1)	
Site Coefficient	F_v	1.50	Table 1613.5.3 (2)	
Adjusted Short Period Spectral Response	S_{MS}	1.50 g	$= F_a * S_s$	Equation 16-36
Adjusted 1 second Spectral Response	S_{M1}	0.90 g	$= F_v * S_1$	Equation 16-37

Design Earthquake Ground Motion

Short Period Spectral Response	S_{DS}	1.00 g	$= 2/3*S_{MS}$	Equation 16-38
1 second Spectral Response	S_{D1}	0.60 g	$= 2/3*S_{M1}$	Equation 16-39
	To	0.12 sec	$=0.2*S_{D1}/S_{DS}$	
	Ts	0.60 sec	$=S_{D1}/S_{DS}$	



2010 CBC

Period T (sec)	Sa (g)	MCE Sa (g)	80% Sa (g)
0.01	0.45	0.68	0.36
0.03	0.55	0.83	0.44
0.10	0.90	1.35	0.72
0.12	1.00	1.50	0.80
0.15	1.00	1.50	0.80
0.20	1.00	1.50	0.80
0.30	1.00	1.50	0.80
0.40	1.00	1.50	0.80
0.50	1.00	1.50	0.80
0.60	1.00	1.50	0.80
0.75	0.80	1.20	0.64
1.00	0.60	0.90	0.48
1.50	0.40	0.60	0.32
2.00	0.30	0.45	0.24
2.20	0.27	0.41	0.22
2.40	0.25	0.38	0.20
2.60	0.23	0.35	0.18
2.80	0.21	0.32	0.17
3.00	0.20	0.30	0.16
3.50	0.17	0.26	0.14
4.00	0.15	0.23	0.12

Development of building foundations, concrete flatwork, and asphaltic concrete pavements should include provisions for mitigating potential swelling forces and reduction in soil strength, which can occur from saturation of the soil. Typical measures considered to remediate expansive soil include:

- capping silt/clay soil with a non-expansive sand layer of sufficient thickness to reduce the effects of soil shrink/swell,
- design of foundations that are resistant to shrink/swell forces of silt/clay soil.

3.6 Groundwater

A 2-inch diameter PVC piezometer was installed in Borings B-9 and B-19 to a depth of 20 feet at the project site. Groundwater was encountered in the piezometers at a depth of 3.5 feet (B-9) and a depth of 6 feet (B-19) on July 6, 2011, 6 days after placement of the piezometers. Each boring was left open for 24 hours to allow groundwater depth readings. Groundwater was encountered in the borings at about 3.5 to 5.25 feet after 24 hours. Groundwater levels may fluctuate with precipitation, irrigation of adjacent properties, removal of the subsurface tile drainage pipeline, and site grading.

Soils encountered below 2 to 3 feet are likely to pump under construction wheel loads. Light earthmoving equipment should be anticipated for use in these areas.

Subsurface agricultural tile drainage pipelines (4-inch diameter plastic or clay perforated pipelines encapsulated by sand/gravel envelopes) exist at a depth of 5 to 7 feet below this site and are used to remove salts accumulating from agricultural irrigation and crop production. Abandoning and plugging the subsurface drainage pipelines can allow groundwater levels to rise variably across the site. Cutting the subsurface tile drain pipelines with utility trenches will likely result in some localized trench flooding. Base line collectors (6 or 8 inch diameter) should be crushed in-place and trench backfill compacted (85-90%). The 4-inch lateral pipeline drains are not required to be removed or crushed in-place. The pipelines should be plugged at utility trench crossings. Copies of the tile drainage system plats as obtained from Imperial Irrigation District records are provided in Appendix F.

3.7 Liquefaction

Liquefaction occurs when granular soil below the water table is subjected to vibratory motions, such as produced by earthquakes. With strong ground shaking, an increase in pore water pressure develops as the soil tends to reduce in volume. If the increase in pore water pressure is sufficient to reduce the vertical effective stress (suspending the soil particles in water), the soil strength decreases and the soil behaves as a liquid (similar to quicksand). Liquefaction can produce excessive settlement, ground rupture, lateral spreading, or failure of shallow bearing foundations.

Four conditions are generally required for liquefaction to occur:

(1) the soil must be saturated (relatively shallow groundwater);
(2) the soil must be loosely packed (low to medium relative density);
(3) the soil must be relatively cohesionless (not clayey); and
(4) groundshaking of sufficient intensity must occur to function as a trigger mechanism.

All of these conditions exist to some degree at this site.

Methods of Analysis: Liquefaction potential at the project site was evaluated using the 1997 NCEER Liquefaction Workshop methods. The 1997 NCEER methods utilize direct SPT blow counts or CPT cone readings from site exploration and earthquake magnitude/PGA estimates from the seismic hazard analysis. The resistance to liquefaction is plotted on a chart of cyclic shear stress ratio (CSR) versus a corrected blow count $N_{1(60)}$ or Qc_{1N}. A ground acceleration of 0.40g was used in the analysis with a 5.0-foot groundwater depth.

Liquefaction induced settlements have been estimated using the 1987 Tokimatsu and Seed method. Fines content of liquefiable sands and silt increase the liquefaction resistance in that more cycles of ground motions are required to fully develop pore pressures. The CPT tip pressures (Qc) were adjusted to an equivalent clean sand pressure (Q_{CINcs}). The adjusted tip pressures were converted to equivalent clean sand blow counts ($N_{1(60)cs}$) prior to calculating settlements (see Table 3). A computed factor of safety less than 1.0 indicates a liquefiable condition.

The soil encountered at the points of exploration included saturated silts and silty sands that could liquefy during a strong seismic event.

Table 3: Corrected Blow Counts

Black Rock 5-6 Geothermal Power Plant (July 2011)

	Boring Number																											
Depth (ft)	B-1	B-2	B-3	B-4	B-5	B-6	B-7	B-8	B-9	B-10	B-11	B-12	B-13	B-14	B-15	B-16	B-17	B-18	B-19	B-20	B-21	B-22	B-23	B-24	B-25	B-26	B-27	B-28
2.5	23	20	30	26	24	16	24	10	13	22	36	28	20	22	22	23	16	20	26	16	26	36	13	30	24	16	24	26
5	19	13	12	15	20	10	22	19	9	10	19	12	15	17	12	21	10	24	20	25	15	12	39	20	15	10	17	13
7.5	7	15	12	16	7	7	4	4	11	3	13	16	5	7	5	9	7	7	11	7	4	7	4	7	3	5	13	9
10	17	25	21	13	15	18	8	27	17	28	22	8	2	11	11	27	17	32	8	10	39	8	25	8	17	15	13	15
12.5	24	29	21	29	14	18	6	16	16	6	20	29	14	29	31	25	27	14	14	14	31	33	24	12	25	21	19	14
15	49	39	39	27	14	43	14	20	14	12	32	25	16	23	10	21	23	19	31	41	52	21	40	18	54	12	21	19
17.5		37	19	21	7	7	20	13	18	7	26	11	8	13	31	11	9	9	21	2	13	36	7	16	16	6	32	29
20	25	26	31	10	54	28	12	15	22	8	27	12	4	8	12	21	27	9	25	11	19	23	8	8	6	14	49	49
25	18			14	26	7	4	12		10	9	17	10	14	10	11	2	2	14	2	11	8	7	9	25			
30	27			25	13	25	5	5		23	21	10	8	16	20	28	9	8	10	8	16	20	12	16	7			
35					16		5	5		12	6	12	8	15	11	6	6	8	14	8	11	5	8	14	13			
40					5		18	6		6	19	5	9	14	5	15	6	10	11	3	8	6	6	6	3			
45					9		4	9		5	10	13	8	23	10	23	30	9	8	35	3	6	3	11	10			
50					7		6	4		11	37	12	3	17	18	12	11	14	14	8	12	7	84	7	21			
55							14	20		17	11	33	9	13	12	14	13	21	11	11	11	12	1	9	28			
60							4	8		11	28	32	13	11	8	15	8	16	23	30	19	14	23	27	19			
65							26	15		18	15	11	9	13	9	10	10	67	26	15	56	29	13	41	13			
70							34	12		27	16	20	38	12	25	42	15	5	12	8	11	35	11	7	14			
75							16	22		9	18	14	14	10	14	23	13	9	13	23	10	17	6	9	5			
80							11	19		13			8	17	11	10	35	20	18	17	3	6	14	21	20			
85							6	8		14			12	5	10	13	26	30	11	21	13	4	42	20	41			
90							40	31		32			4	17	30	17	8	5	4	13	24	41	3	37	44			
95							30	30		25			31	39	21	32	13	18	6	10	13	26	5	5	13			
100							6	9		12			10	16	9	8	13	11	12	8	6	15	7	14	11			
105																	1				6			9				
110																	30				9			9				
115																	45				7			7				
120																	14				1			7				

Liquefaction can occur within interbedded silty sands, sandy silts, and silt layers between depths of 5 to 50 feet. The likely triggering mechanism for liquefaction appears to be strong groundshaking associated with the rupture of the San Andreas Fault, Elmore Fault and Brawley Seismic Zone. The liquefaction settlement analysis is summarized in Table 4 below.

Table 4: SUMMARY OF LIQUEFACTION ANALYSES

Boring Location	Depth To First Liquefiable Zone (ft)	Potential Induced Settlement (in)
CPT-1	9.5	4¼
CPT-2	5.5	3½
CPT-3	4.5	4
CPT-4	5.0	3
CPT-5	5.5	2¼
B-5	6.5	3½
B-7	7.5	4¾
B-8	7.5	4½
B-10	5	5
B-11	7	2¼
B-12	5	3¼
B-13	8	9½
B-14	7.5	5
B-15	5	4¾
B-16	7.5	3¼
B-17	6	3½
B-18	7.5	5½
B-19	7.5	4
B-20	7.5	6
B-21	7.5	3½
B-22	6	5
B-23	7	2¾
B-24	7	7
B-25	7.5	4½

Liquefaction Effects: Based on research from Ishihara (1985) and Youd and Garris (1995) ground rupture or sand boil formation is possible at the plant site because of the relatively thin layer of the overlying unliquefiable soil. Sand boils are conical piles of sand derived from the upward flow of groundwater caused by excess porewater pressures created during strong ground shaking. Sand boils are not inherently damaging by themselves, but are an indication that liquefaction occurred at depth (Jones, 2003). Liquefaction induced lateral spreading is not expected to occur at this site due to the planar topography. According to Youd (2005), if the liquefiable layer lies at a depth greater that about twice the height of a free face, lateral spread is not likely to develop. No slopes or free faces occur at this site except for the shallow brine ponds.

Mitigation: At major power plant structural components liquefaction settlements can be mitigated by one of the following methods:

1) Structural flat-plate mats, either conventionally reinforced or tied with post-tensioned tendons.
2) Deep foundations (drilled piers or auger cast piles) founded at a minimum depth of 62 feet.
3) Soil improvement by soil-cement mixing or soil replacement to create non-liquefying soils (35 feet minimum depth).

3.8 Percolation Testing

Percolation tests were performed on July 1, 2011 in the proposed leach field and alternative field areas for the new septic tank leach field planned north of the control and operations building. The native soil consists of clay (CH) to a depth of 5 feet. Groundwater was encountered at a depth of 4.5 feet at the proposed location of the leach field.

Tests were made in accordance with the Taft Method as described in the U.S. Department of Health "Manual of Septic Tank Practice." The percolation test holes consisted of 6-inch diameter holes extending to a depth of 3 feet below the natural ground surface. A hand auger was used to excavate the bottom 12 inches of the hole to prevent sidewall smearing. A 2-inch layer of pea gravel was placed on the bottom of each test hole and the hole was filled with water 8 to 12 inches above the gravel.

After a 24 hour pre-saturation period, a 6-hour test was performed, refilling the test holes to a hydrostatic level of 8 to 12 inches in the bottom of each hole after each 30 minute reading during a 6 hour test period.

The measured drop in water surface indicates that the onsite clay at a depth of 3 feet have a very low infiltration capacity. Additionally, groundwater is shallow, encountered to a depth of 4.5 feet below existing ground surface. The test results follow:

Primary Test Hole	Location	Stabilized Drop Min./Inch	Percolation Rate Gal./S.F./Day
P-1	Northeast corner	>240	0.32
P-2	Southeast corner	>240	0.32
P-3	Southwest corner	>240	0.32
P-4	Northwest corner	>240	0.32

The infiltration rates are less than the minimums established by the California Plumbing Code and Imperial Count Health Department and due to the shallow groundwater conditions will require that an "engineered disposal system" be used at this site. This potentially may consist of a pump tank, pump and mounded leach field. Additionally, advanced treatment may be used for wastewater and the effluent discharged to a plant site "process water" area.

Section 4
RECOMMENDATIONS

4.1 Site Preparation and Backfill

Clearing and Grubbing: All surface improvements, crop or vegetation including grass, brush, and weeds on the site at the time of construction should be removed from the construction area. The crop may be stripped by cutting with a blade or earthmover to 1 inch below ground surface. Organic strippings should not be used as fill within plant structural areas. The agricultural subsurface tile drainage system shall be abandoned by cutting and plugging laterals at the boundary of the plant site (see Tile Drainage Maps – Appendix F).

Building Pad Preparation: The existing surface soil within the administration office, control rooms, and other light building foundation areas should be removed to 36 inches below the building pad elevation or existing grade (whichever is lower) extending five feet beyond all exterior wall/column lines (including adjacent concreted areas). Exposed subgrade should be bladed or excavated smooth.

A geotextile separation fabric and geogrid layer should be placed over the graded surface and a minimum of 12 inches of aggregate base placed over the geotextile and geogrids prior to allowing any construction equipment onto the building pad. The surface of the aggregate base shall be compacted to a minimum of 90% of ASTM D1557 maximum density prior to placing a subsequent 6 inch lift of aggregate base. The geotextile shall a 6 oz. non-woven fabric equivalent to Mirafi 160N or Propex 4506. Geogrids shall be either Tensar TriAx 5 or Tenax MS330. The 6 inch lift of aggregate base shall be compacted to at least 95% of ASTM D1557 maximum density.

An engineered building support pad consisting a minimum of 3.0 feet of granular soil, placed in maximum 8-inch lifts (loose), compacted to a minimum of 95% of ASTM D1557 maximum density at 2% below to 4% above optimum moisture, should be placed below the control building and warehouse slabs.

Imported fill soil shall be non-expansive, granular soil meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35% passing the No. 200 sieve. Imported granular fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 95% of ASTM D1557 maximum dry density at optimum moisture ±2%.

In areas other than the building pad which are to receive area concrete slabs, the native soils should be removed to a minimum depth of 18 inches and then scarified to 6 inches, uniformly moisture conditioned to 5 to 10% over optimum, and recompacted to 85-90% of ASTM D1557 maximum density just prior to concrete placement.

Utility Trench Backfill: Trench backfill for utilities should conform to San Diego Regional Standard Drawing S-4 (Appendix D), using either Type A, B or C backfill.

Type A backfill for HDPE pipe consists of a 4 to 6 inch bed of ¾-inch crushed rock below the pipe and pipezone backfill (to 12" above top of pipe) consisting of crusher fines (sand). Sewer pipes (SDR-35), water mains, and stormdrain pipes of other that HDPE pipe may use crusher fines for bedding. The crusher fines shall be compacted to a minimum of 90% of ASTM D1557 maximum density. Pipe deflection should be checked to not exceed 2% of pipe diameter. Native clay/silt soils may be used to backfill the remainder of the trench. Clays shall be compacted to a minimum of 85% of ASTM D1557 maximum density and silts shall be compacted to a minimum of 87% of ASTM D1557 maximum density, except that the top 12 inches of the trench shall be compacted to at least 90% of ASTM D1557 maximum density.

Type B backfill for HDPE pipe requires 6 inches of ¾-inch crushed rock as bedding and to springline of the pipe. Thereafter, sand/cement slurry (3 sack cement factor) should be used to 12 inches above the top of the pipe. Native clay and silt soils may be used in the remainder of the trench backfill as specified above.

Type C backfill for HDPE pipe shall consist of a geotextile filter fabric encapsulating ¾-inch crushed rock. The crushed rock thickness shall be 6 inches below and to the sides of the pipe and shall extend to 12 inches above the top of the pipe. The filter fabric shall cover the trench bottom, sidewalls and over the top of the crushed rock. Native clay and silt soils may be used in the remainder of the trench backfill as specified above.

Type C backfill must be used in wet soils and below groundwater for all buried utility pipelines unless dewatered (by well points) to at least 24 inches below the trench bottom prior to excavation. Type A backfill may be used in the case of a dewatered trench condition.

On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill above pipezone, but may be difficult to uniformly maintain at specified moistures and compact to the specified densities. Native backfill should only be placed and compacted after encapsulating buried pipes with suitable bedding and pipe envelope material.

Imported granular material is acceptable for backfill of utility trenches.

Backfill soil of utility trenches within paved areas should be placed in layers not more that 6 inches in thickness and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density.

Observation and Density Testing: All site preparation and fill placement should be observed and tested by a representative of a qualified geotechnical engineering firm. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "*geotechnical engineer of record*" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the recommendations for site development.

4.2 Shallow Foundations, Structural Mats and Settlements

Spread footings: Shallow spread footings and continuous wall footings are suitable to support the building structures associated with offices, control rooms and warehouse. The bottoms of footings shall be founded on at least 18 inches of properly prepared and compacted granular soil as described in Section 4.1. The foundations shall be designed for an allowable soil bearing pressure of 2,000 psf when foundations are supported on 18 inches of granular soil. The allowable soil pressure may be increased by one-third for short term loads induced by winds or seismic events.

In areas where multiple spread footings are considered to support piping or process equipment, a reinforced aggregate fill composite mat may be considered. The bottom layer of the reinforced aggregate composite mat shall be constructed as described in Section 4.1 of this report. At the surface of the 18 inch thick stabilizing layer, a second layer of geogrids shall be placed and two 6-inch lifts of aggregate base placed and compacted to at least 95% of ASTM D1557 maximum density.

The reinforced composite mat shall extend to a minimum of 10 feet beyond the outer edges of the spread footings. The allowable bearing pressure at the surface of the 2.5 foot thick composite mat shall be 2,500 psf with 33% increases allowed for wind and seismic loads.

Structural Mats: Structural mats may be used to mitigate differential movement between structural components of the plant. Mats may be designed for a modulus of subgrade reaction (Ks) of 50 pci when placed on 24 inches of compacted native clay soil and a minimum of 8 inches of Caltrans Class 2 aggregate base (compacted to 95%).

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement.

An allowable friction coefficient of 0.35 may also be used at the base of the foundations to resist lateral loading.

Settlement estimates (in inches) developed for foundations of varying dimension (embedded 18 inches into the ground) supported by 1.5 feet of reinforced structural fill and loaded to 2,000 psf follows:

Table 5: Settlement Estimates (inches)

Load, psf	**Size of Footing or Mat (ft.)**			
	3 x 3	**5 x 5**	**7 x7**	**10 x 10**
2,000	0.60	0.90	1.20	1.50

4.3 Cooling Tower Foundations and Settlements

Soil Improvements and Underlayment: The existing soils underlying the cooling tower should be improved by soil mixing or soil replacement (sand/cement) with 48 inch diameter shafts. The minimum surface area replacement ratio shall be 20 percent. The soil mix formula shall be developed by the specially contractor to provide a minimum strength of 100 psi.

Table 6: Estimated Settlement - 135 ft. x 420 ft. Foundation Overlaying Soil Mixed Columns

Treatment Depth (ft)	Load (psf)	Settlement Estimates (in)
No Treatment	750	3.0
10	750	2.2
20	750	2.0
30	750	1.7
*35	750	1.5

* Minimum depth required to mitigate liquefaction settlement.

Following soil mixing, the area should then be brought to finish grade with 36 inches of crushed aggregate base.

4.4 Soil Mixing (Rigid Mats)

The use of soil improvement like soil mixing with cement may be used to reduce settlement of structural mats to tolerable limits.

Structural mat foundations placed over the improved soil are anticipated to be used to support various structural elements of the plant. Mats overlaying soil mixed columns should be underlain by 36 inches of crushed aggregate base. Soil improvement which is extended to a minimum depth of 35 feet below ground surface will mitigate liquefaction settlements.

The deep soil mixing serves to reduce settlement by replacing the compressible clay soils below the structures with very stiff soil-cement columns, creating a stiffer composite soil matrix. The soil-cement mix design should be provided by a licensed specialty contractor to provide a minimum strength of 100 psi. Soil improvement treatment depth is expected to reduce settlements according to Table 7:

Table 7: Estimated Settlements – Mats Overlaying Soil Mixed Columns
Typical Heavy Equipment Foundation Footprint

Treatment Depth (ft)	Foundation Size (ft)	Load (psf)	Settlement Estimates (in)
10	40x80	1000	1.6
20	40x80	1000	1.4
30	40x80	1000	1.0

It is unlikely that significant differential settlement will occur on foundations supported by improved soil.

4.5 Deep Foundations (Drilled Piers or Auger Cast Piles)

Drilled piers or auger cast piles (cast-in-place grout with steel cage reinforcement) have been used successfully to provide deep foundations for heavily loaded and critical elements of geothermal power plants. Capacities for 24, 30 and 36 inch diameter shafts are provided below.

Vertical Capacity: Vertical capacity for 24, 30 and 36 inch diameter shafts are shown on Figure 3. Capacities for other shaft sizes can be determined in direct proportion to shaft diameters. End bearing and skin friction parameters have been used to determine the allowable shaft capacity. The allowable capacities include a factor of safety of 2.5.

The allowable vertical compression capacities may be increased by 33 percent to accommodate temporary loads from wind or seismic forces. The allowable vertical shaft capacities are based on the supporting capacity of the soil. The structural capacity of the piers should be verified by the structural engineer.

Drilled pier or auger cast piles should have a minimum embedment depth of 62 feet within a medium dense to dense sand layer.

Settlement: Total settlements of less than ¼ inch are anticipated for single piles designed according to the preceding recommendations. If pier/pile spacing is a least 2.5 pier/pile diameters center-to-center, no reduction in axial load capacity is considered necessary for group effect.

Lateral Capacity: The allowable lateral capacity for 24, 30 and 36 inch diameter shafts are given in the Table 8. The allowable horizontal deflection at the shaft head has been assumed to be one-half inch (0.50 inch).

Table 8: Lateral Capacities – Drilled Piers / Auger Cast Piles

Shaft Diameter (in.)	**24**		**30**		**36**	
Head Condition	Free	Fixed	Free	Fixed	Free	Fixed
Allowable Head Deflection (in.)	0.5	0.5	0.5	0.5	0.5	0.5
Length (ft.)	**62**	**62**	**62**	**62**	**62**	**62**
Lateral Capacity (kips)	32	73	47	108	65	148
Maximum Moment (foot-kips)	152.5	-412.50	261.7	-720.0	417.5	-1125
@Depth from Pier Head (ft.)	7.6	0	9.2	0	10.5	0

The geotechnical engineer should observe the drilling operations and evaluate pier/pile capacities.

4.6 Short Drilled Piers for Pipe Rack Supports

Drilled piers for pipe racks supports have been used successfully on geothermal power plants. Recommendations for 24 and 36 inch diameter shafts are provided in Figure 3 and Table 10.

Vertical Capacity: Vertical capacity for 24 and 36 inch diameter shafts are presented in Figure 3. Capacities for other shaft sizes can be determined in direct proportion to shaft diameters. End bearing and skin friction parameters have been used to determine the allowable shaft capacity. The allowable capacities include a factor of safety of 2.5.

The allowable vertical compression capacities may be increased by 33 percent to accommodate temporary loads from wind or seismic forces. The allowable vertical shaft capacities are based on the supporting capacity of the soil. The structural capacity of the piers should be verified by the structural engineer.

Settlement: Total settlements of less than ¼ inch are anticipated for single piles designed according to the preceding recommendations. If pier/pile spacing is a least 2.5 pier/pile diameters center-to-center, no reduction in axial load capacity is considered necessary for a group effect. Due to the short length of these piers (15 feet), liquefaction settlements will not be mitigated. A minimum pier length of 40 feet is required to mitigate liquefaction settlements.

Lateral Capacity: The allowable lateral capacity for 24 and 36 inch diameter shafts are given in the Table 9. The allowable horizontal deflection at the shaft head has been assumed to be one-half inch (0.50 inch). Shear loads were applied one (1) foot above ground surface elevation.

Table 9: Lateral Capacities – Short Drilled Piers

Shaft Diameter (in.)	**24**		**36**	
Head Condition	Free	Fixed	Free	Fixed
Allowable Head Deflection (in.)	0.5	0.5	0.5	0.5
Length (ft.)	**15**	**15**	**15**	**15**
Lateral Capacity (kips)	19	52	24	108
Maximum Moment (foot-kips)	87.5	-355.8	107.5	-1008.3
@Depth from Pier Head (ft.)	7.2	0	7.2	0

The geotechnical engineer should observe the drilling operations and evaluate capacities of drilled piers.

4.7 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plates C-7 and C-8). The native soils were found to have severe levels of sulfate ion concentration (3,079 to 5,981 ppm). Sulfate ions in high concentrations can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling.

A minimum of 6.5 sacks per cubic yard of concrete (4,500 psi) of Type V Portland Cement with a maximum water/cement ratio of 0.45 (by weight) should be used for concrete placed in contact with native soil on this project (sitework including sidewalks, housekeeping slabs and foundations). Admixtures may be required to allow placement of this low water/cement ratio concrete.

The native soil has a moderate to severe level of chloride ion concentration (280 to 1,150 ppm). Chloride ions can cause corrosion of reinforcing steel, anchor bolts and other embedded metallic conduits. Mitigation of the corrosion of steel can be achieved by using steel pipes coated with epoxy corrosion inhibitors, asphaltic and epoxy coatings, cathodic protection or by encapsulating the portion of the pipe lying above groundwater with a minimum of 3 inches of densely consolidated concrete. ***No metallic pipes or conduits should be placed below foundations.***

Foundation designs shall provide a minimum concrete cover of three (3) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or five (5) inches of concrete cover when exposed to geothermal brine. If the 3 or 5-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy dipped for corrosion protection or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the concrete. Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.

4.8 Excavations

All site excavations should conform to CalOSHA requirements for Type B soil. The contractor is solely responsible for the safety of workers entering trenches. Temporary excavations with depths of 4 feet or less may be cut nearly vertical for short duration.

Excavations deeper than 4 feet will require shoring or slope inclinations in conformance to CAL/OSHA regulations for Type B soil. Surcharge loads of stockpiled soil or construction materials should be set back from the top of the slope a minimum distance equal to the height of the slope.

All permanent slopes should not be steeper than 3:1 to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination.

Groundwater was encountered at a depth of 3.5 and 6.0 feet on July 6, 2011. The contractor is cautioned to evaluate soil moisture and groundwater conditions at the time of bidding. Groundwater depths may not be apparent in short term open excavations (up to 4 feet deep) due to the equivalency of atmospheric evaporation rates to groundwater migration through the fine grained upper clay soils.

4.9 Embankment Construction and General Site Fill

Site preparation and embankment construction: All areas to receive new fill for the embankments should be stripped of all vegetation. The surface 12 inches of native soil shall be uniformly moisture conditioned to 5 to 10% above optimum moisture by discing and compacted in 6 inch maximum lifts to a minimum of 90% of ASTM D1557 maximum density.

The embankment slopes may be constructed no steeper than 3:1 (unless lined with concrete or HDPE/PVC sheeting) with a minimum crown width of 15 feet. Embankments should be overbuilt by a minimum of 6 inches and subsequently cut to the plan line and grade to remove loose material along the slope faces.

Native cohesive soil from the borrow site located at the northwest corner of English Road and Hazard Road is anticipated to be used as general and embankment fill. The borrow area fill soils consist of cohesive silty clay (CL) or clay (CH). The clay soils are considered adequate for engineered fill. The general and embankment fill should be pulverized/disced to less than ¾ inch maximum clod size, uniformly moisture conditioned to 2 to 7% over optimum, placed in 6 inch maximum lifts and compacted to a minimum of 90% of ASTM D1557 maximum density.

4.10 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the San Andreas and the Brawley Seismic Zone. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.4 of this report.

4.11 Pavements

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements.

Based on the current State of California CALTRANS method, and R-value of 5 for the subgrade soil and assumed traffic indices, Table 10 provides our estimates for asphaltic concrete (AC) pavement sections.

TABLE 10
RECOMMENDED PAVEMENTS SECTIONS

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.) **	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
4.0	3.0	8.0	5.0	6.0
5.0	3.0	9.0	5.5	6.0
6.0	3.0	14.0	6.0	6.0
6.5 (*)	4.0	14.0	7.0	8.0
8.0	4.0	18.0	8.0	11.0
10.0	4.5	26.0	9.0	13.0
11.0	5.5	28.0	10.0	15.0

** Aggregate base thickness may be reduced by 1/3 when a geotextile fabric and geogrids are used at the bottom of the aggregate base layer.

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum (½ inch maximum for parking areas), medium grading with PG 64-16 oil, compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D1559) or Hveem Density (Cal 366).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (minimum 4% above optimum if clays) native clay soil compacted to a minimum of 90% of the maximum dry density determined by ASTM D1557.
4) Portland cement concrete for pavements should have Type V cement, a minimum compressive strength of 4,500 psi at 28 days, and a maximum water-cement ratio of 0.45.
5) Typical Street Classifications (Imperial County)

Cul-de-Sacs:	TI = 5.0
Local Streets:	TI = 6.0
Minor Collectors:	TI = 6.5 (*) Plant Roadways
Major Collectors:	TI = 8.0
Minor Arterial:	TI = 10.0
Primary Arterial:	TI = 11.0

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The recommendations and conclusions within this report are based on current information regarding the design and construction of the Black Rock 5-6 Geothermal Power Plant located approximately ¼ mile north of Sinclair Road on the west side of Garst Road about 6 miles northwest of Calipatria, California. The conclusions and recommendations of this report are invalid if:

- Structural loads change from those stated or the structures are relocated.
- The Additional Services section of this report is not followed.
- This report is used for adjacent or other property.
- Changes of grade or groundwater occur between the issuance of this report and construction other than those anticipated in this report.
- Any other change that materially alters the project from that proposed at the time this report was prepared.

Findings and recommendations in this report are based on selected points of field exploration, geologic literature, laboratory testing, and our understanding of the proposed project. Our analysis of data and recommendations presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. If detected, these conditions may require additional studies, consultation, and possible design revisions.

This report contains information that may be useful in the preparation of contract specifications. However, the report is not worded is such a manner that we recommend its use as a construction specification document without proper modification. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in Imperial County at the time the report was prepared. No express or implied warranties are made in connection with our services.

This report should be considered invalid for periods after two years from the report date without a review of the validity of the findings and recommendations by our firm, due to potential changes in the Geotechnical Engineering Standards of Practice.

The client has responsibility to see that all parties to the project including, designer, contractor, and subcontractor are made aware of this entire report. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

5.2 Additional Services

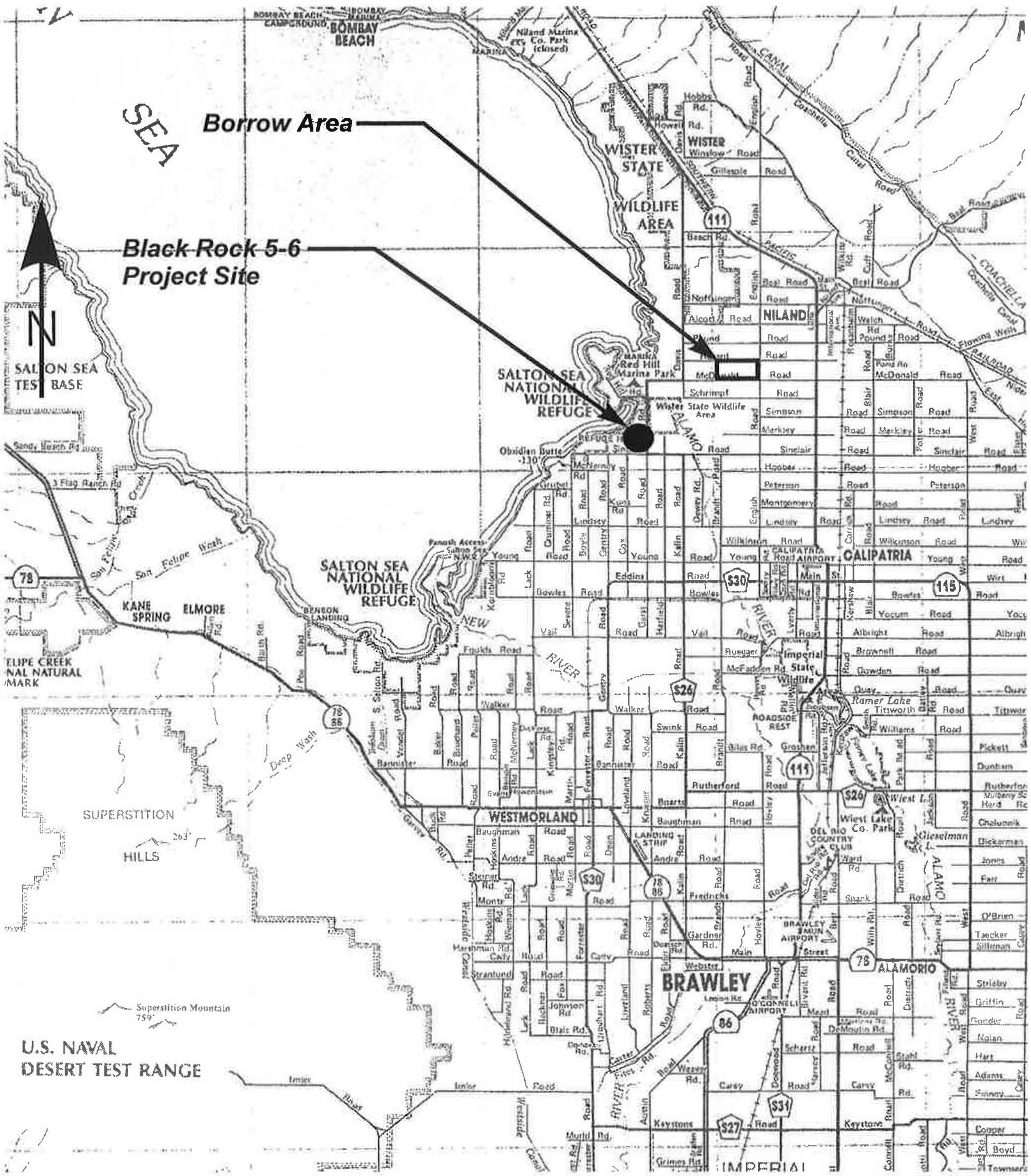
We recommend that a qualified Geotechnical consultant be retained to provide the tests and observations services during construction. *The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

The recommendations presented in this report are based on the assumption that:

- Consultation during development of design and construction documents to check that the geotechnical recommendations are appropriate for the proposed project and that the geotechnical recommendations are properly interpreted and incorporated into the documents.
- Landmark Consultants will have the opportunity to review and comment on the plans and specifications for the project prior to the issuance of such for bidding.
- Continuous observation, inspection, and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, building pad and subgrade preparation, and backfilling of utility trenches.
- Observation of foundation excavations and reinforcing steel before concrete placement.
- Other consultation as necessary during design and construction.

We emphasize our review of the project plans and specifications to check for compatibility with our recommendations and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

APPENDIX A



LANDMARK
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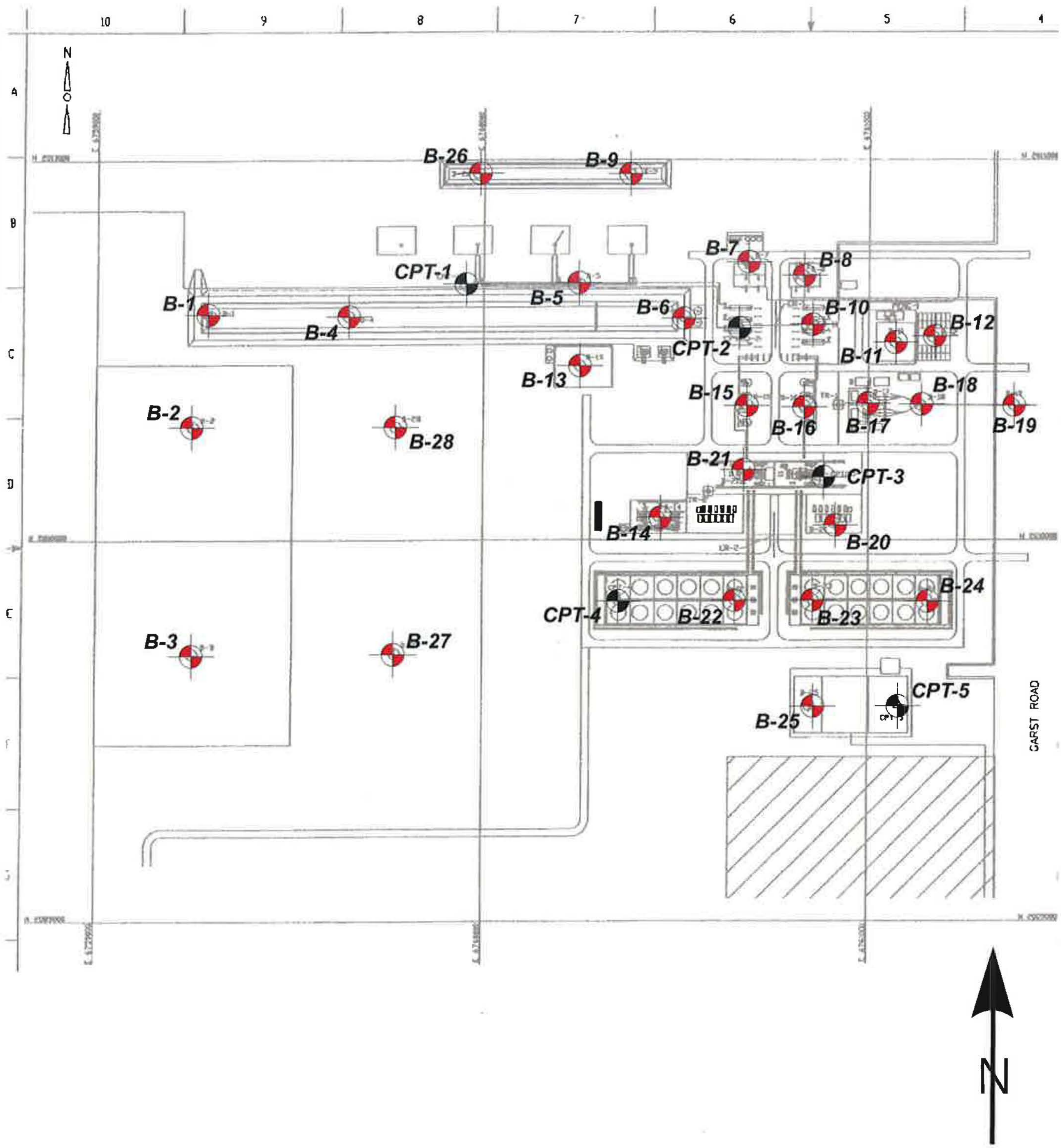
Project No.: LE11138

Vicinity Map

Plate A-1



LANDMARK
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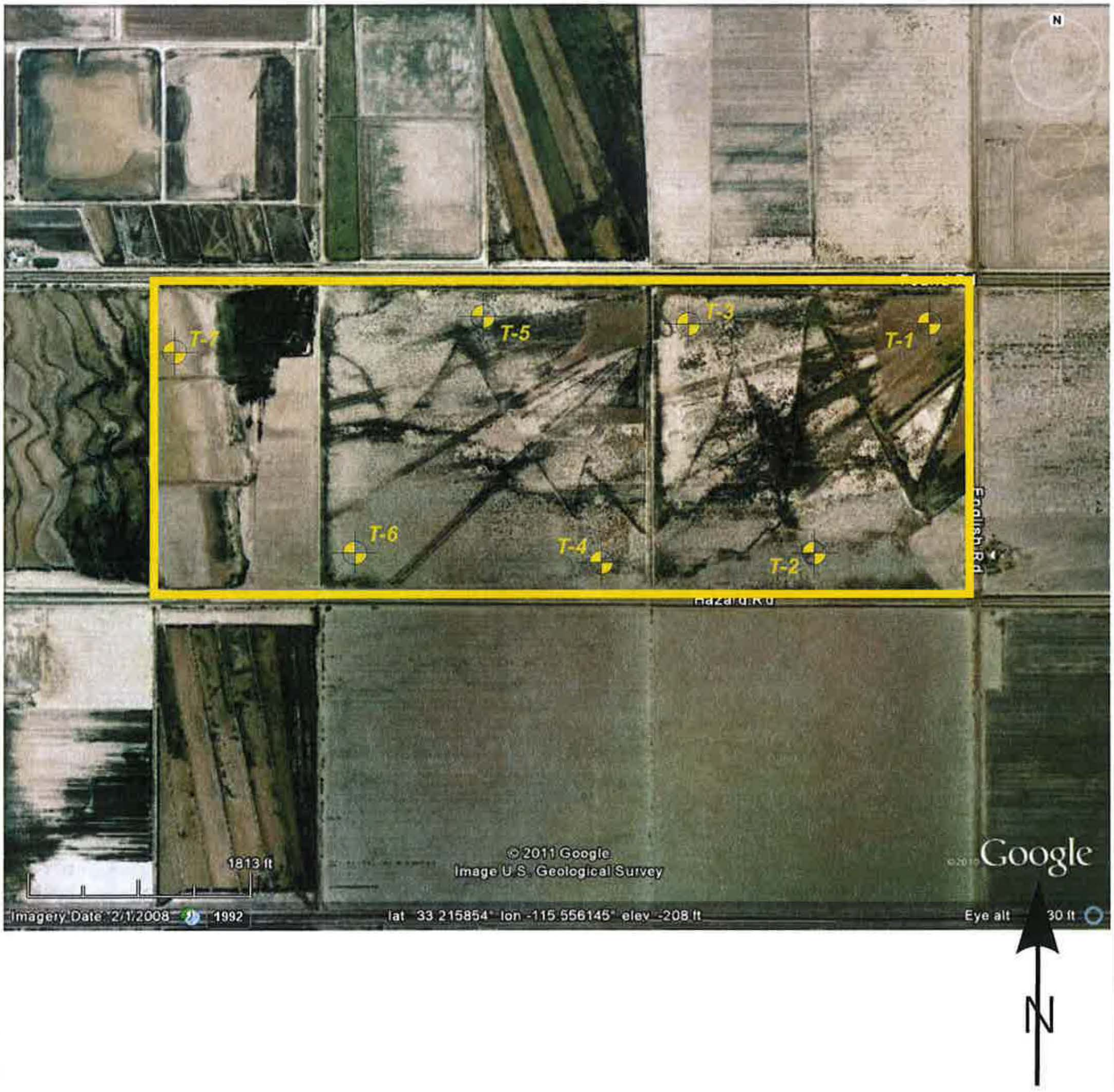
Project No.: LE11138

Black Rock 5-6
Site and Exploration Plan

Plate
A-2



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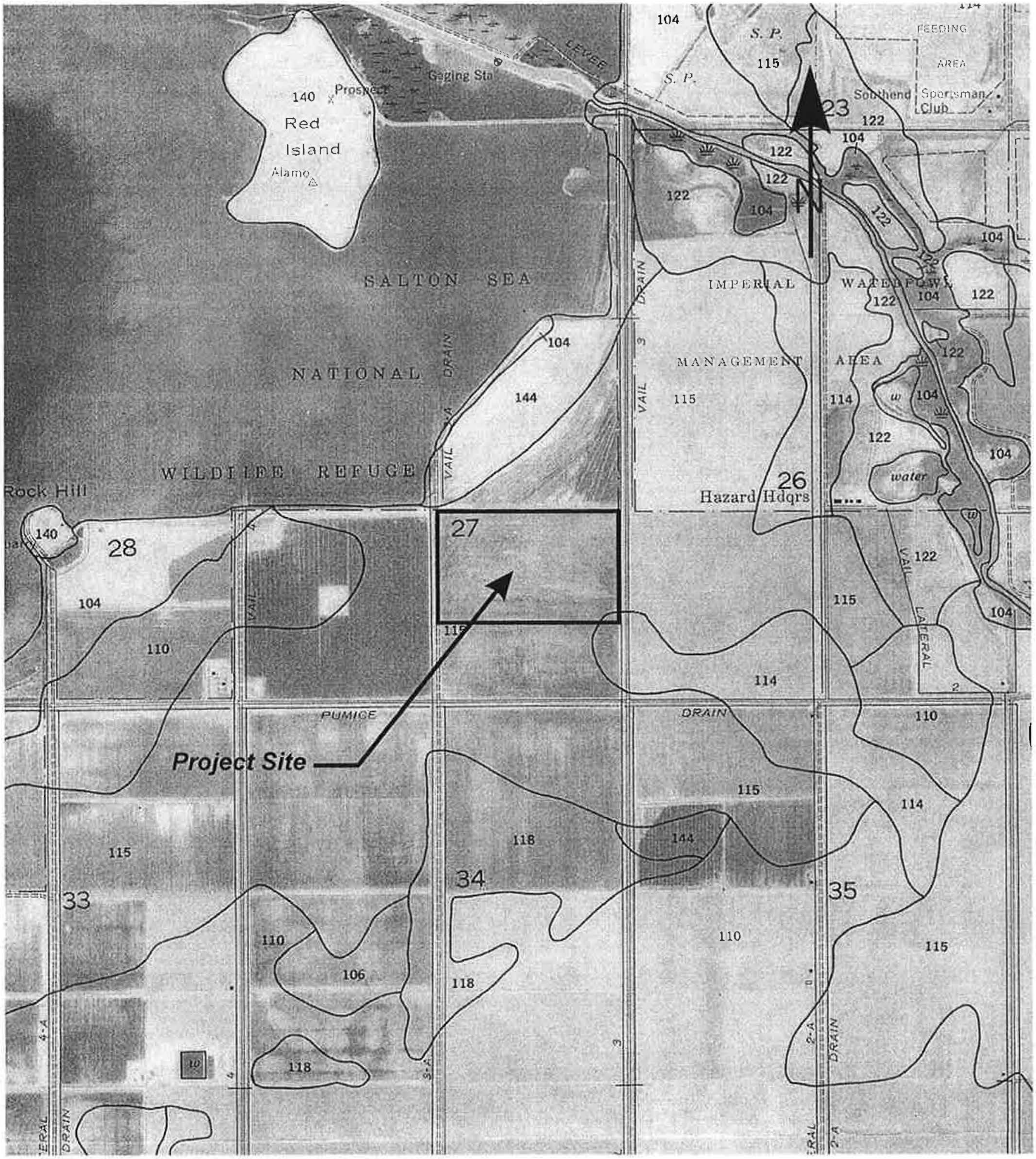
Borrow Area
Site and Exploration Plan

Plate
A-3



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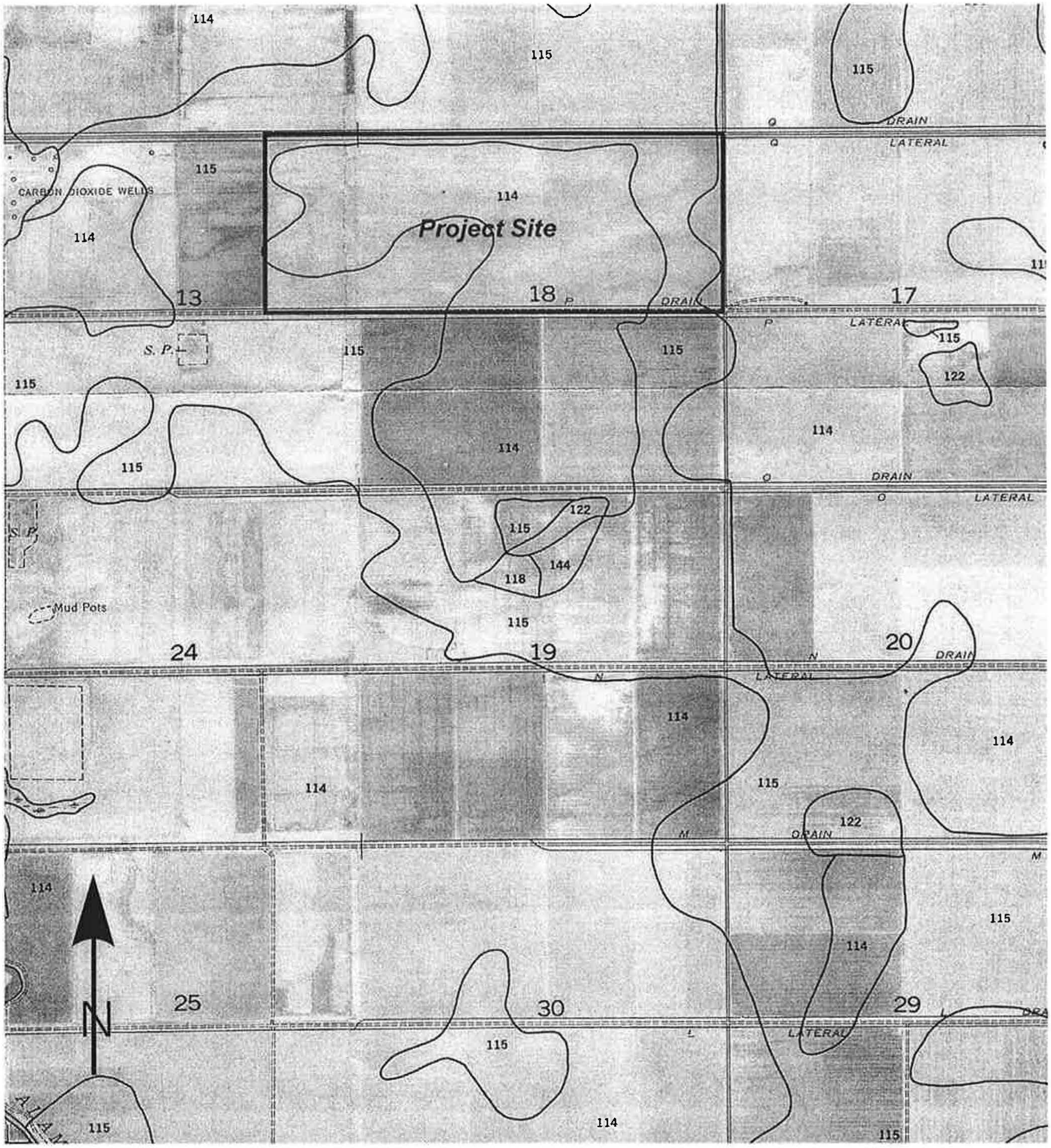
Black Rock 5-6 Soil Survey Map

Plate
A-4



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Borrow Area
Soil Survey Map

Plate
A-5