Agenda Item No. D-6

DATE SUBMITTED	11/4/25	COUNCIL ACTION	(X)
SUBMITTED BY	Public Services	PUBLIC HEARING REQUIRED	( )
		RESOLUTION	( )
DATE ACTION REQUIRED	11/4/25	ORDINANCE 1ST READING	( )
		ORDINANCE 2ND READING	( )
		CITY CLERK'S INITIALS	( )

IMPERIAL CITY COUNCIL
AGENDA ITEM

SUBJECT: DISCUSSION/ACTION:

1. Adopt Plans and Specifications and Authorize Public Bidding for the 2025 City of Imperial Wastewater Treatment Plant Demolition Project; Bid-2025-19

DEPARTMENT INVOLVED: Public Services

BACKGROUND/SUMMARY:

The City of Imperial wastewater treatment plant underwent major upgrades in 2021 to provide a membrane bioreactor (MBR) activated sludge process. Due to budget constraints at the time, the improvements excluded replacement of the existing influent lift station (headworks). A Preliminary Engineering Report was prepared by WEBB/AQUA, dated July 22, 2025. The report recommends Option 2, which would allow the city to keep the existing office structure in place. This demolition project would prepare the site for the installation of the new headworks. The project also includes the installation of a modular office building for use by staff during the demolition and eventual remodeling of the existing office building.

The project will bid in accordance with the Public Contract Code (PCC). The project plans and specifications will be on file with the City Clerk at City Hall located at 420 S. Imperial Ave, Imperial, CA 92251.

FISCAL IMPACT: NOT TO EXCEED

Funds to cover associated costs will be expended from enterprise funds. Project is in the FY 25-26 Capital Improvement Plan.

CIP Project No. 849, Wastewater Treatment Plant Demolition, $500,000

FINANCE INITIALS VMS

STAFF RECOMMENDATION:

approve request

DEPT. INITIALS JG

MANAGER'S RECOMMENDATION: approve

CITY MANAGER's INITIALS Otm

MOTION:

SECONDED:	APPROVED	( )	REJECTED	( )
AYES:	DISAPPROVED	( )	DEFERRED	( )
NAYES:				
ABSENT:	REFERRED TO:			

TECHNICAL MEMORANDUM

TO:	David Dale, PE, PLS, Public Services Director, City of Imperial Jenell Guerrero, Public Services Manager, City of Imperial Chris Kemp, Chief Wastewater Operator, City of Imperial
FROM:	Justin Logan, PE, Principal, AQUA Engineering Mitchell Weldon, PE, Project Manager, AQUA Engineering
DATE:	July 22, 2025
SUBJECT:	Preliminary Engineering Report – Imperial WWTP Influent Pump Station
PROJECT NO.:	002857.C

This report memorializes existing conditions, design guidelines, potential options, and final recommendation for the City of Imperial (Imperial) Wastewater Treatment Plant (WWTP) influent pumping facility upgrade. WEBB/AQUA recommends a dry-pit pumping facility utilizing the basement of the existing sludge pumping building. We conclude by presenting implications for construction sequencing and commissioning.

INTRODUCTION

Background

The City of Imperial owns and maintains the Imperial wastewater treatment plant, located at 701 E 14th St, in the north end of the city along its eastern boundary. The facility discharges under NPDES Permit CA0104400.

The plant underwent major upgrades in 2021 to provide a membrane bioreactor (MBR) activated sludge process. Due to budget constraints, 2021 improvements excluded replacement of the existing influent lift station. Figure 1 shows the location of the existing pump station and associated electrical house (e-house) on the wastewater plant site.

Imperial is pursuing a new influent pumping facility for several purposes:

- Increase process hydraulic capacity
- Improve influent screening to protect influent pumps and downstream equipment
- Improve system redundancy and reliability
- Align with current operational requirements
- Provide a low maintenance profile and convenient access for maintenance activities
- Minimize, contain, and treat foul air to reduce odors from raw wastewater

Report Objectives

This Preliminary Engineer Report (PER) addresses the following objectives:

- Identify key infrastructure constraints
- Outline options for new plant lift station
- Provide cost estimates for Project options
- Outline critical construction sequencing and commissioning approaches

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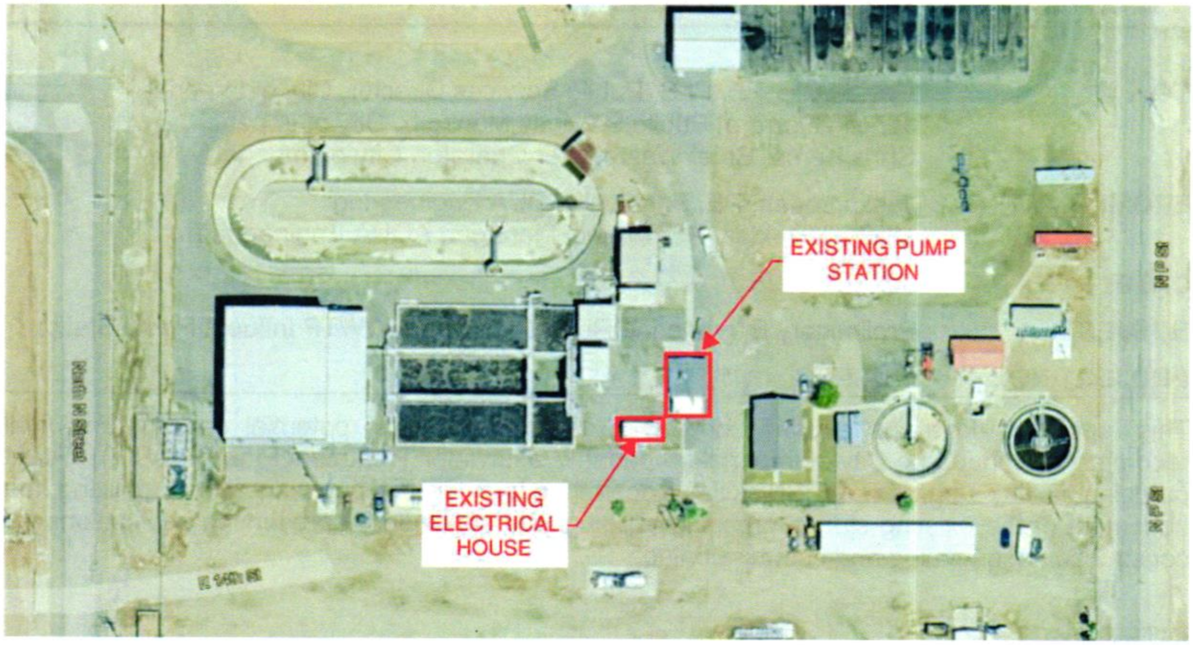
Phone: 801.299.1327 | Fax: 801.299.0153

Figure 1. Existing Pump Station and E-House on Wastewater Plant Site



CURRENT CONDITIONS ANALYSIS

Current Deficiencies

The current influent pump station and downstream headworks have encountered several major concerns:

- The current pump station is the facility's hydraulic bottleneck.
- Due to a lack of screening, rags impact influent pumps and have damaged downstream fine screening units. Figure 2 shows damage to the perforated plate underneath one unit's auger.
- Hydrogen sulfide (H2S) release and high chloride levels corrode downstream infrastructure.
- Collection system diversion manholes upstream of the existing pump station have deteriorated significantly.
- Access to existing pumps and wetwell is limited by the existing building design.

Design Flows

Equipment will be sized to handle current and future flows:

- Current Average Day flow: 2.4 MGD
- Current Peak Hour Flow: 5.3 MGD
- Future Average Day Flow: 3.0 MGD
- Future Peak Hour Flow: 6.3 MGD

Designing for future flows primarily impacts equipment, pump, and pipe sizing as well as the design of hydraulic structures (channels, wetwell). Electrical equipment must be adequately sized to handle future power demands.

Figure 2. Damage to Fine Screening Unit

Existing Connections

Flow enters the existing wetwell from several directions:

- To the south, a series of manholes diverts flows from several sewer lines. It appears these manholes are severely deteriorated, and it is assumed the project will re-route influent sewer around these manholes.
- To the east, two 24-inch interceptors converge in a common manhole, with flow directed in a single line west to the wetwell. This east invert is the deepest invert into the wetwell, per available information.
- To the north, an 8-inch filtrate line enters the wet well. This line currently accepts all percolation water for the existing drying beds, the centrate water from the dewatering system, and flows from the dewatering building drains.

Existing force main piping to the fine-screening facility includes buried 14-inch C-900 PVC and an exposed ductile iron pipe (DIP) header outside of the fine screening building. At a future 6.3 mgd peak hour flow, pipe velocities in 14-inch pipe would exceed 9 FPS. With a preferred velocity below 5 FPS, upsizing of piping will be necessary. Bypass pumping will be required to finalize interconnections with new piping.

Geotechnical

A geotechnical study was completed by Landmark Consultants, Inc. on April 30, 2025. Key findings include the following considerations:

- Bores encountered groundwater at depths of 9-ft. A dewatering system will be required during construction. The structure must be designed to resist flotation.
- Structural engineering must incorporate seismic design criteria for a Site Class D with S_1 value of 0.65.
- Overexcavation of 2-ft is required for below grade structures, with subgrade replaced by drainage rock covered with a geotextile filter fabric.
- Severe sulfate ion concentrations necessitate the use of a Type V cement.
- A concrete mix design with minimum 4500 psi compressive strength is required.
- Chloride ion concentrations are also severe. No metallic water pipes or conduits should be placed below foundations.
- A 4-inch edge distance to reinforcing bar within concrete should be maintained, otherwise embedded steel components shall be epoxy coated for corrosion protection.

In general, geotechnical observations do not differ from previous understanding of site soil conditions. Site soils are corrosive – buried infrastructure must be appropriately selected and adequately protected to ensure a long service life.

DESIGN GUIDELINES FOR SCREENING AND PUMPING

The following subsections detail design considerations common to the development of both headworks facility options.

Materials of Construction

High chlorides and hydrogen sulfide in Imperial's influent wastewater contribute to accelerated corrosion of metals. Influent testing has yielded the following concentrations:

Ion	Reading 1	Reading 2
Chloride	310 mg/L	380 mg/L
Sulfide	30 mg/L	34 mg/L

For all metal fabrications, appropriate alloys must be selected to avoid rapid corrosion. 304 stainless steel suffices for chloride levels below 100 mg/L but experiences "pitting" above this concentration, which has been observed at the treatment facility. Therefore, stainless steel shall be at minimum 316, which better resists chloride pitting due to the addition of 2-3% molybdenum. During final design, Webb/AQUA will evaluate critical components for available material selections that offer corrosion protection exceeding that of 316 SS (e.g. duplex stainless steel).

Within the wet well and screening facility, nonmetallic options shall be provided when appropriate:

- FRP covers for screening channels and wetwell
- Vinyl ester unistrut for conduit support

Screening

Design of new screening facilities, prior to influent pumps, should adhere to the following requirements:

- Screening must remove solids to reduce pump ragging and adequately protect downstream equipment.

  - Recommend screening openings of ¼-inch
- Screening must have built-in redundancy.
  - Two units are recommended, each sized at 100% of flow. For the design flows and other constraints, upsizing from 50% to 100% redundancy does not appreciably increase project cost.
  - Provide a passive overflow channel with manual bar screen in case of failure of both screens.
- Screening must provide for convenient disposal of solids
  - Screenings must discharge at grade level for ease of subsequent hauling.
  - Compactor unit provided to reduce total disposal volume and decrease required hauling frequency.
- Screening facilities must minimize resulting odors:
  - Screening unit will include washer and compactor to return excess organics to the influent wet well. Washing of screenings will reduce odors.
  - Screening channels and downstream wetwell shall be covered. An odor control system shall remove air from beneath the covers and scrub it prior to release to atmosphere.
- Protection of exposed infrastructure
  - Provide a metal awning with siding on the west side to protect equipment from accelerated UV degradation, dust, and debris.

Pumping

WEBB/AQUA recommends that pump design adhere to the following guidelines:

- Design pumps to be solids-handling. While upstream screening will reduce ragging concerns, there is a chance of solids agglomeration in the wet well.
- Provide N+1 redundancy to maintain system operation in the case of a single pump failure.
- Select pump materials appropriate for corrosive service:
  - Stainless steel impeller recommended in lieu of cast iron.
  - Heavy-duty external coatings for submersible pumps
  - WEBB/AQUA is aware of abrasive materials (grit) in the influent. Excess abrasives may justify a different material selection for the impeller. Final material selections will be made in consultation with pump manufacturers.
- Operate pumps on variable frequency drives to reduce pump starts and minimize wastewater detention times in the wetwell.

Corrosion Protection

To protect downstream equipment (fine screens) from corrosion, the following equipment is recommended:

- Channel and wetwell covers
- Odor control system
- Wetwell aeration
- Coatings

Developed design options include an odor control system to scrub foul air from underneath screening channel and wetwell covers. The system will use either a carbon media scrubber or biotrickling filter to reduce potential facility odors. By exchanging air at the interface with raw sewage, the system will also encourage further H2S volatilization and removal.

A coarse bubble aeration system is also proposed to provide several benefits:

- Wetwell mixing to promote volatilization of H2S and subsequent removal by the odor control system.
- Addition of oxygen to mitigate anaerobic conditions which favor the bacteria that reduce sulfate to sulfide
- Promotion of sulfur-oxidizing bacteria to convert aqueous sulfides to sulfate ions.

The City of Imperial has recently purchased a Fog Log (Risen Water) aerator for use in H2S reduction in the collection system. The vendor recommends 2 Fog Log units to adequately mix the proposed wetwell.

Finally, concrete and piping will have appropriate coatings to resist sulfide corrosion. WEBB/AQUA recommends a 100% solids thick film coating for the concrete exposed to the interior airspace of the screening facility (Tnemec Perma-Glaze Series G435 or equal).

Screening Vault Requirements

Influent sewer depths necessitate a below-ground structure for screening channels and the pump wetwell. Several ancillary building design recommendations are summarized below:

- Minimize equipment placed below grade. Motors, control panels, and other equipment should be placed above grade to reduce exposure to a corrosive environment.

- Provide improved maintenance access to wetwell:

    - Stairwell to screening floor
    - Improved clearances within screening floor
    - FRP cover system with hoists to remove panels for unobstructed wetwell access.

- Provide exhaust fans and supply vents to reduce humidity and remove gases that have bypassed channel covers and the odor control system. Ventilation design may provide continuous ventilation or only when personnel are present.

- Provide combustible gas detectors (CGDs) to comply with NFPA 820.

- Provide a metal awning to cover critical equipment at grade level. The awning will reduce UV exposure and provide a more comfortable work environment for maintenance activities. Provide siding on the west side of the awning to protect equipment from afternoon sun as well as prevailing winds and associated debris.

OPTION 1 – SUBMERSIBLE PUMPS

Description

This option consists of a new screening facility and wetwell to be located at the Operations Building site. See Attachment 1 for a basic site plan, plan views, and section view of the concept.

The following points summarize the screening configuration:

- Influent sewer depths necessitate a below-grade screening floor. Three channels will direct influent wastewater to the wetwell.
- Two channels are screened by mechanical screening equipment, which discharge screenings to the upper level at grade.
    - The third channel has a manual bar screen and is normally isolated with an overflow weir gate, so that it only operates under emergency conditions.
- Per design guidelines above, the screens are sized such that each screen can independently handle the future peak hour flow (6.3 MGD). This approach will minimize bypass through the manual screen.

The screening channels discharge directly to the wetwell, which is preliminarily sized at 20'x8'x8'. Within the wetwell sit 4 submersible centrifugal, non-clog pumps. These pumps are sized for 3 duty, 1 standby (3+1) configuration to handle 6.3 MGD. The pumps will have discharge elbow bases and guide rails for retrieval. They can be retrieved from the wetwell using a monorail installed at grade level above the wetwell.

The screening channels and wetwell will be enclosed with FRP covers. The airspace underneath the covers will be ventilated to remove odors, with foul air passing through a carbon media or biological odor control system located at grade. Additionally, a type of aeration will be added to the influent channels or wet well to help reduce sulfides in the wastewater and strip them out of solution in an effort to reduce downstream corrosion potential. This will also be removed with the foul air to the odor control system.

Screenings will discharge at grade to washer/compactor units. Washing reduces organic content of screenings, with resulting benefits for odor control and screening volume.

Discharge piping from the pumps will exit the facility at the grade level and combine in an above-ground discharge header adjacent to the structure. The discharge line to the existing headworks will be buried and is preliminarily sized at 18-inches.

Equipment

For mechanical screening, a multi-rake screen was evaluated in the preliminary evaluation for the following reasons:

- Ability to convey solids to a high discharge point.
- High degree of inclination reduces horizontal footprint of the unit.
- Washer/compactor units are available as an integrated package.
- Multiple vendors are available to provide a competitive bidding process.

Odor control evaluated carbon media, chemical, and biological scrubbers. Preliminary evaluation yielded the following conclusions:

- Air stripping will be provided in the influent channels or wet well. This technology will be determined during the design phase.
- A chemical scrubber has increased maintenance profile and operating costs compared to the other options and is not considered further.
- A carbon media scrubber has a lower capital cost than a biological system but faces recurring costs of media replacement.
- A biological system (biotrickling filter) has large capital costs but recurring O&M expenses is limited to power and water.

Further evaluation and selection of odor control technology is anticipated during final design.

Site Constraints and Electrical Design Considerations

Option 1 necessitates full demolition of the existing Operations Building.

Due to its proximity to the new facility, the existing electrical house can be re-used for housing electrical and controls equipment. The existing PLC can be switched over to control the new pumps, with only minor modifications necessary to add control capabilities over the fourth proposed pump. Preliminary evaluation of power demands suggests the distribution panel feeding the existing e-house is of sufficient size to power new equipment for Option 1.

The three existing pump VFDs can also be maintained in the e-house. WEBB/AQUA recommends evaluating procurement of two new VFDs – one for the 4th pump, and an additional to facilitate pump clean water testing and switchover operations. During final design, the existing shelf spare VFD will be evaluated for use as one of the new VFDs required. Following switchover, two of the existing VFDs can be rewired one by one to the new pumps, and the final existing VFD can serve as a spare.

Cost

A preliminary cost estimate has been developed equivalent to AACE Level 3 "Budgetary" cost estimate. The expected range of accuracy for this level of cost estimate is -20% to +30%. The overall project cost (including design and construction management) with a 30% contingency is estimated at $6,450,440. See Attachment 1 for a cost estimate broken out by division.

This cost includes an estimated cost for demolishing the Operations Building. WEBB/AQUA understands that Imperial will soon receive pricing for Operations Building removal – that pricing can then be figured into the City's final evaluation.

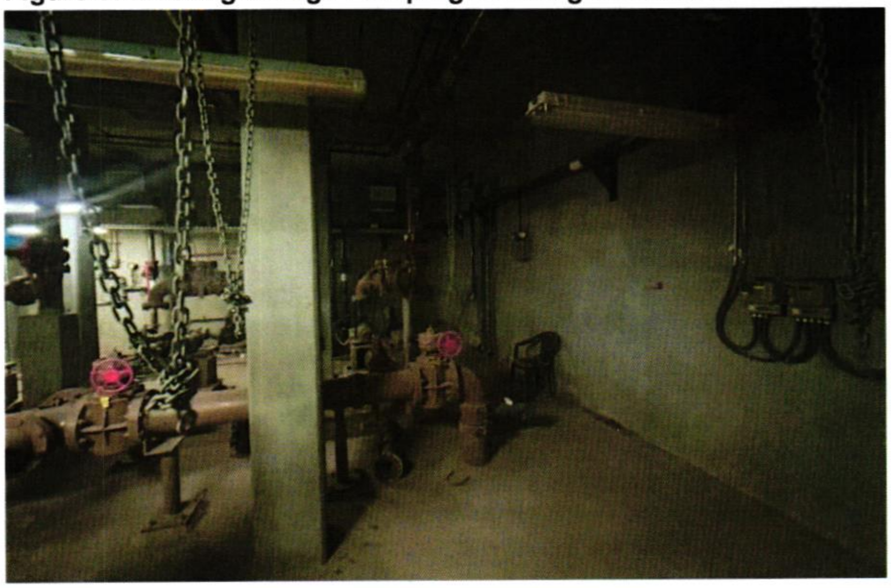
OPTION 2 – DRY-PIT PUMPS

Description

Option 2 consists of a screening facility and wetwell similar to Option 1. However, this option offers a dry-pit pumping configuration, which will allow direct access to pumps and pump motors for regular maintenance activities. See Attachment 2 for a basic site plan, plan views, and section view of the concept.

To reduce the capital costs associated with a dry pit, WEBB/AQUA proposes siting the wetwell adjacent to the west side of the existing Sludge Pumping Building. The basement floor of the existing building would be used to house the dry pit pumps and discharge header. Figure 3 shows the wall adjacent to which the new pumps would be installed. Most equipment in the Sludge Pumping Building is no longer in use and can be removed to provide space for the dry pit pumps.

Figure 3. Existing Sludge Pumping Building Basement



By placing the new structure adjacent to the west side of the sludge pumping building, site demolition work should be minimized.

Other main differences compared to Option 1 are listed below:

- Demolition work will be required within the existing Sludge Pumping Building to remove piping and equipment that is no longer in use.
- WEBB/AQUA recommends upgrades to the existing HVAC system at the Sludge Pumping Building to ensure NFPA-compliant controls and alarming are in place. Some existing HVAC equipment is within the basement level, and it is recommended to replace this with equipment mounted on the exterior concrete lid.
- No monorail or grating on the top level of the screening facility is required for pump retrieval.
- Greater lengths of influent sewer piping and additional manholes will be required to route influent sewer to this location on site.
- Force main piping to the fine screens will be of greater length.

Equipment

Screening and odor control equipment remains the same as Option 1. Overall screening facility and wetwell sizing will be approximately the same.

Given sewer invert elevations, dry-pit pumps must be capable of pulling a suction lift. As such, a self-priming model is recommended. Pump impellers and volute design shall be non-clog to handle solids which bypass the coarse screens.

New, exterior-mounted HVAC equipment is recommended for the sludge pumping building basement, to provide sufficient air exchanges in accordance with NFPA 820 as well as cooling to condition the space during hot weather.

Site Constraints and Electrical Design Considerations

Option 2 does not require the demolition of the existing Operations Building. Demolition of the west secondary clarifier (Clarifier #1), including removal of below-grade concrete, is required. However, per City workshop held on May 30, 2025, this clarifier demolition is likely to be pursued in either case. Therefore, the cost of clarifier and associated infrastructure demolition is not considered in the evaluation of Option 2.

With the demolition of Clarifier #1, there are no known existing facilities that would substantially impact the installation of the new screening and wetwell structure.

The electrical room on the top floor of the sludge pumping building would house electrical and controls equipment for Option 2. Preliminary power demand evaluation suggests that the 300-amp service to this electrical room is adequate for the new equipment. However, if pump power requirements increase significantly above the initial estimate (25 hp per pump), a new breaker and larger wire to the service may be required. The pending removal of the UV equipment loading from this service will increase power available for the new equipment.

Compared to the e-house, Option 2 will require additional electrical and controls wiring. The existing PLC (for the now-defunct RAS, WAS, and NPW pump systems) may have a reusable cabinet, but it will require new internals. Per Chris Kemp, the existing Motor Control Center (MCC) is in good condition and has ample buckets for new equipment. Re-use of these two pieces of equipment is estimated to save $30,000 in construction costs. The condition of other existing gear identified for potential reuse will be verified during final design.

Existing VFDs in the e-house could theoretically be moved and re-installed, but two new VFDs are likely required anyway for testing and startup (if the shelf spare VFD is installed, only one new VFD is required). The sequencing and switchover process adds additional complications – compared to an estimated $20,000 per VFD panel, reuse of the existing VFDs may not be worth additional constructability concerns. WEBB/AQUA recommends acquiring four new VFDs (three if the existing shelf spare may be used) instead. These differences between Option 2 and Option 1 are reflected in greater estimated Option 2 EI&C costs.

Access modifications to the electrical room and stairwell to the sludge pumping building basement will be made so that access to the dry-pit pumps does not require access to the electrical room.

Cost

Compared to Option 1, major items reducing and increasing costs for Option 2 are presented below:

Cost Reduction
- No monorail for pump retrieval
- Reduced pump costs
- No demolition of Operations Building

Cost Increase
+ Demo work within Sludge Pump Building
+ HVAC upgrades to Sludge Pump Building
+ Additional electrical gear including VFDs
+ Additional controls work including new PLC internals and HVAC controls
+ Additional discharge header piping
+ Additional piping supports and coating
+ Additional influent sewer piping and manholes (deep trenching)
+ Additional force main piping

Estimated cost for this option (with 30% contingency) is **$6,897,100**, or $446,660 greater than Option 1. Increased costs are largely driven by the additional electrical and controls scope anticipated for this option.

RECOMMENDATION

Option 2, the dry-pit pumping arrangement is recommended for several reasons:

(1) Equipment access: pumps and motors are accessible without hoisting from wet well.
(2) Maintenance:
   a. Ease of inspection and access promotes improved preventative maintenance.
   b. Greater number of maintenance activities can be performed by operators on-site without need for shipping to a manufacturer's service shop.
   c. Critical issues such as seal failure do not lead to total motor failure.
(3) Avoids demolition of the existing Operations Building.

The operational advantages of Option 2 should be considered against the greater anticipated costs in guiding the City's final decision.

COMMISSIONING STRATEGY

A multi-step commissioning procedure will be implemented to ensure that new equipment is functioning properly before hand-over to Imperial staff is completed. Commissioning of equipment will proceed in three main steps:

(1) Pre-commissioning work: activities to be completed before the Contractor is permitted to begin Commissioning. The intent is to test isolated equipment and components. Primary activities for this phase include:
   a. Factory testing
   b. Component and stand-alone equipment testing
   c. Energization of electrical power distribution equipment
   d. Pipe pressure testing
   e. Loop testing
   f. Operational Readiness Tests to verify that all parts of a system are in working order and functioning properly.
   g. Draft O&M Manuals Submitted and Approved.

(2) Phase 1 Commissioning: the first phase of Commissioning will include operator training as well as comprehensive testing with clean water.
    a. The steps will include approval of Operational Readiness Tests and the Functional Acceptance Test (FAT).
    b. The purpose of the FAT is to test all equipment, instruments and software as an integrated system using plant water wherever applicable.
        i. The successful completion of the Functional Acceptance Test will allow the Contractor to request Operational Acceptance.
    c. FAT tests will be conducted using clean water, which can then be recycled back to the wetwell.

(3) Phase 2 Commissioning is designed to functionally test the facility as an integrated system under normal operating conditions using wastewater. The testing includes the Reliability Acceptance Test (RAT) that will be conducted over a period of time that demonstrates the operational reliability of the system.
    a. For Coarse Screening and Influent Pumping, a 30-day RAT period is recommended.
    b. Wastewater will be introduced by starting the bypassing and diversion process of the existing influent diversion manholes, with flow entering the new sewer piping to the influent screening channels.
    c. After successful completion of the RAT and all Manufacturers' Certificates of Proper Operation have been submitted to Engineer, and after the Contractor has submitted all Operation and Maintenance Manuals, the Contractor may request the Owners' acceptance that the system is Substantially Complete.

CONSTRUCTION SEQUENCING

Evaluation of construction sequencing requirements is summarized below:

- Construction of the new screening and wetwell structure does not impact existing facilities and has no preceding tasks.
- New sewer and force main piping should be laid prior to switchover, apart from final interconnections .
- Temporary bypass pumping will be required when upsizing the force main discharge header to the fine screens.
- Temporary bypass pumping will be required during sewer line diversions, from a manhole upstream of the diversion point.
- If VFDs are reused, While the existing pump station and pump VFDs remains in operation, one or two new VFDs will first be installed for the new pumps. Clean water testing may occur using the pumps connected to the VFDs. Then, after flow has been diverted to the new pump station, switchover of existing VFDs may occur sequentially.
- If four new VFDs are provided, then clean water testing may occur using the complete set of pumps. Switchover will not require sequential reinstallation of VFDs.
- Following completion of the RAT, demolition activities at the existing pump station may begin.

FACILITY PERMITTING

Webb/AQUA recommends that the new pump station be considered as a replacement of existing facilities with no resulting change to the plant capacity. As a replacement, only a CEQA exemption would be required for the project.

When the City proposes to increase the permitted capacity of the Imperial WWTP and processes a permit amendment, Webb/AQUA will investigate if a CEQA document was prepared for the City's General Plan that would handle growth inducement impacts. If a CEQA document exists, the opportunity may exist to use it to satisfy the CEQA requirements relating to growth, and the CEQA document for the new pump station can simply address the localized impacts of construction.

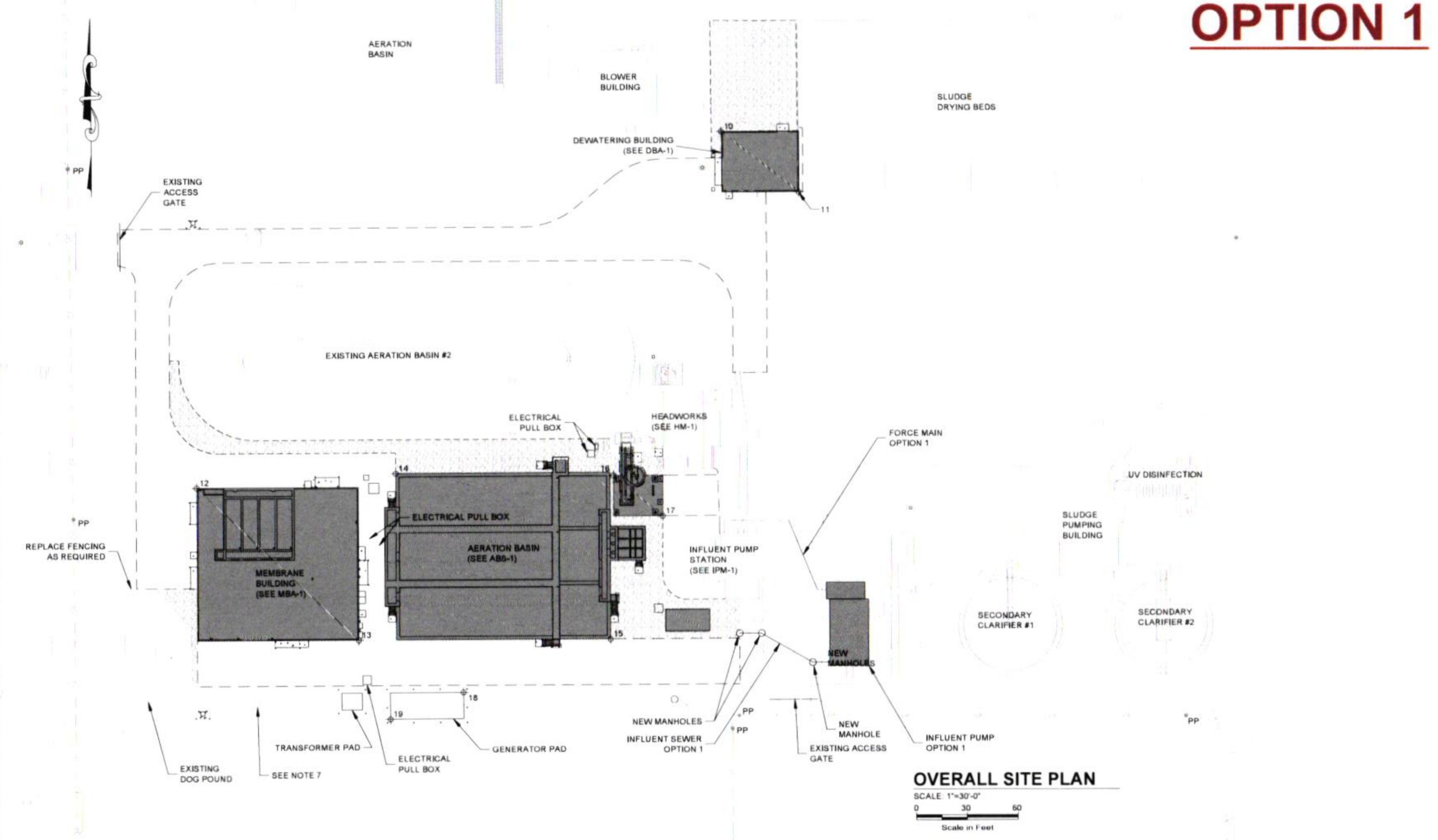
ATTACHMENTS

- Attachment 1: Option 1 Site Plan, Mechanical Plan and Section, and Cost Estimate
- Attachment 2: Option 2 Site Plan, Mechanical Plan and Section, and Cost Estimate
- Attachment 3: "Limited Geotechnical Report, Proposed Headworks at Imperial WWTP. LCI Report No. LE25072." Landmark Consultants, Inc. April 30, 2025.

ATTACHMENT 1

Option 1 Site Plan, Mechanical Plan and Section, and Cost Estimate

OPTION 1



OVERALL SITE PLAN

SCALE: 1"=30'-0"

0 30 60

Scale in Feet

CITY OF IMPERIAL

HEADERWORKS CONCEPT

SITE PLAN OPTION 1

DRAWING IS TO SCALE IF BAR MEASURES 1" = FULL SCALE 1/2" = HALF SCALE

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		ORIGINAL		
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		REVISIONS		

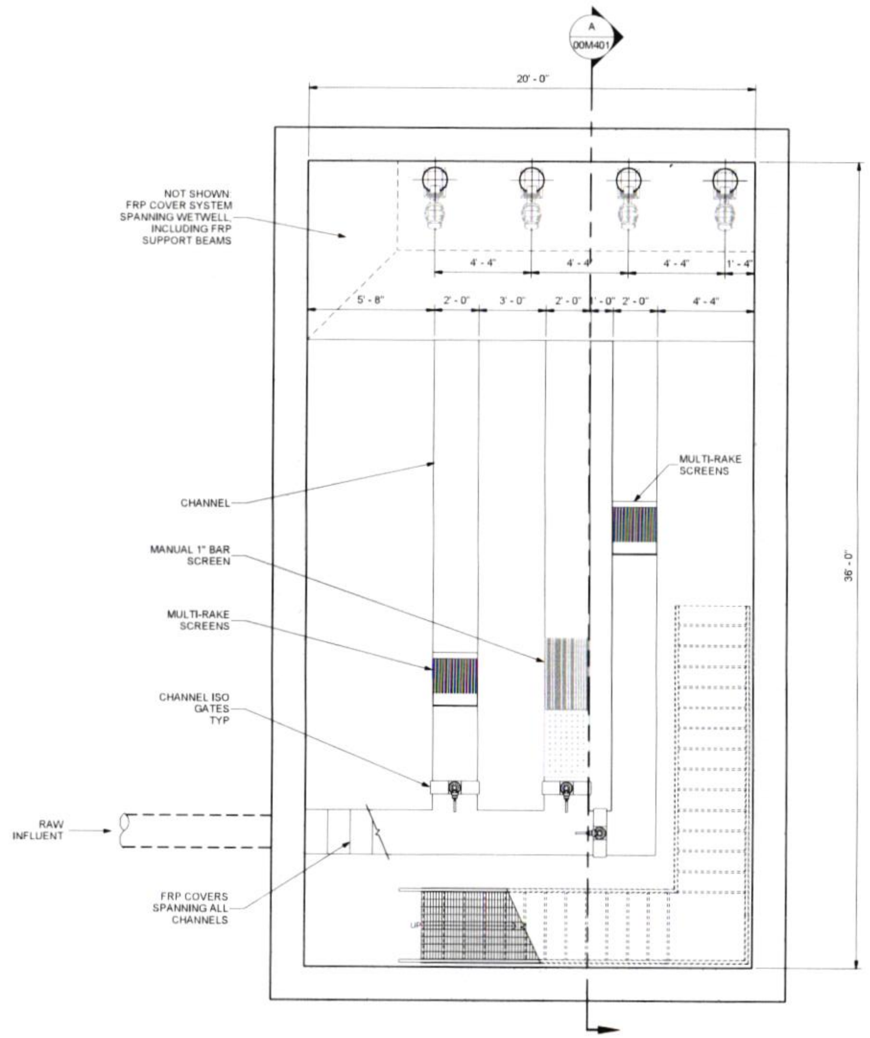
AQUA ENGINEERING

DRAWING NO.

C-1

SHEET

OPTION 1



PLAN
3/8" = 1'-0"

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REVISIONS				
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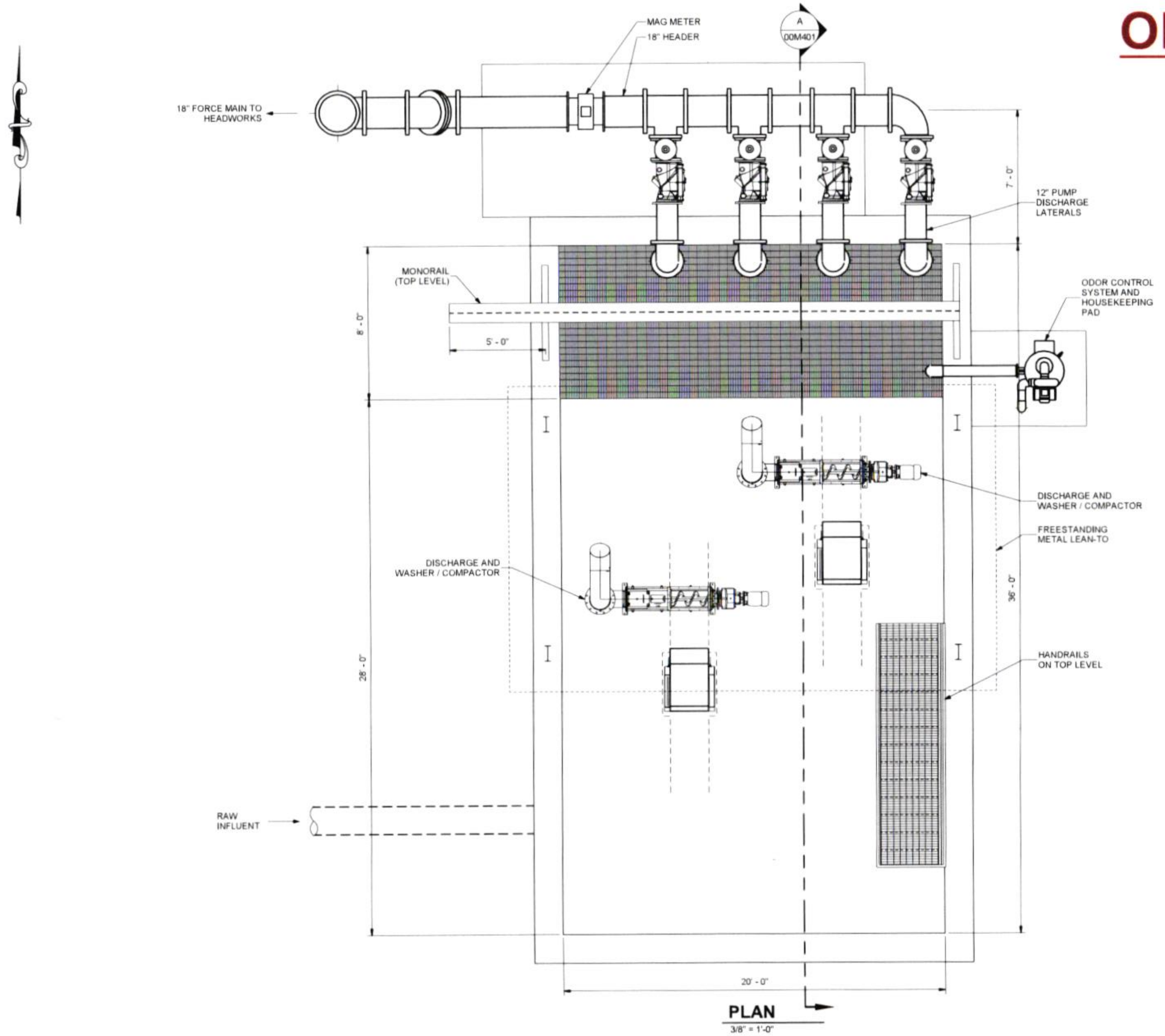
IMPERIAL HEADWORKS CONCEPT
Location

AQUA ENGINEERING

DRAWING NO.
00M201

SHEET

OPTION 1



PLAN
3/8" = 1'-0"

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REVISIONS

NO	DATE	DESIGN	DRAWN	CHECKED

IMPERIAL HEADWORKS CONCEPT
Location

AQUA ENGINEERING

DRAWING NO.
00M202

SHEET

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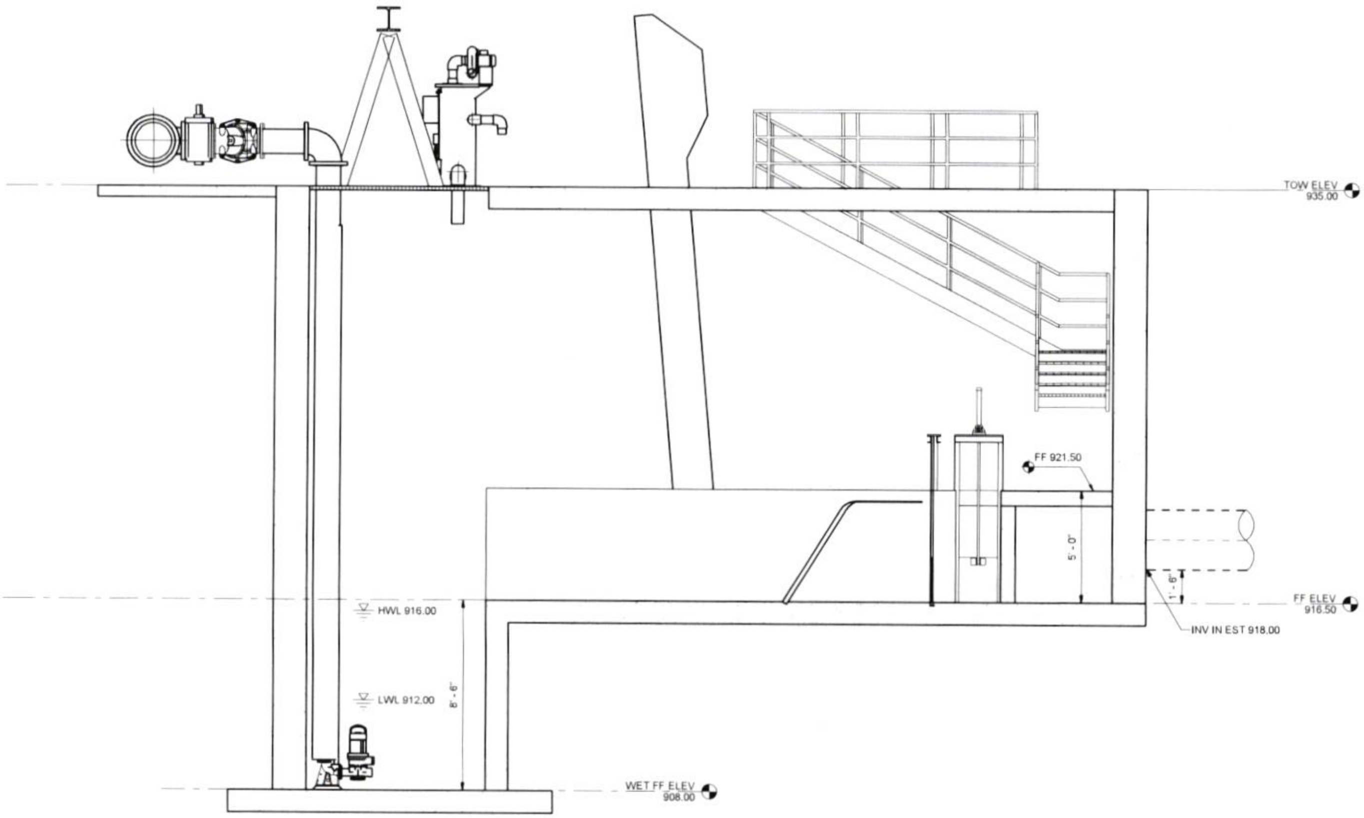
OPTION 1



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00M201
SECTION
3/8" = 1'-0"

DRAWING IS TO SCALE IF BAR MEASURES: 1" = FULL SCALE, 1/2" = HALF SCALE

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REVISIONS				
NO.	DATE	DESIGN	DRAWN	CHECKED

IMPERIAL HEADWORKS CONCEPT
Location

AQUA ENGINEERING

DRAWING NO.
00M401

SHEET

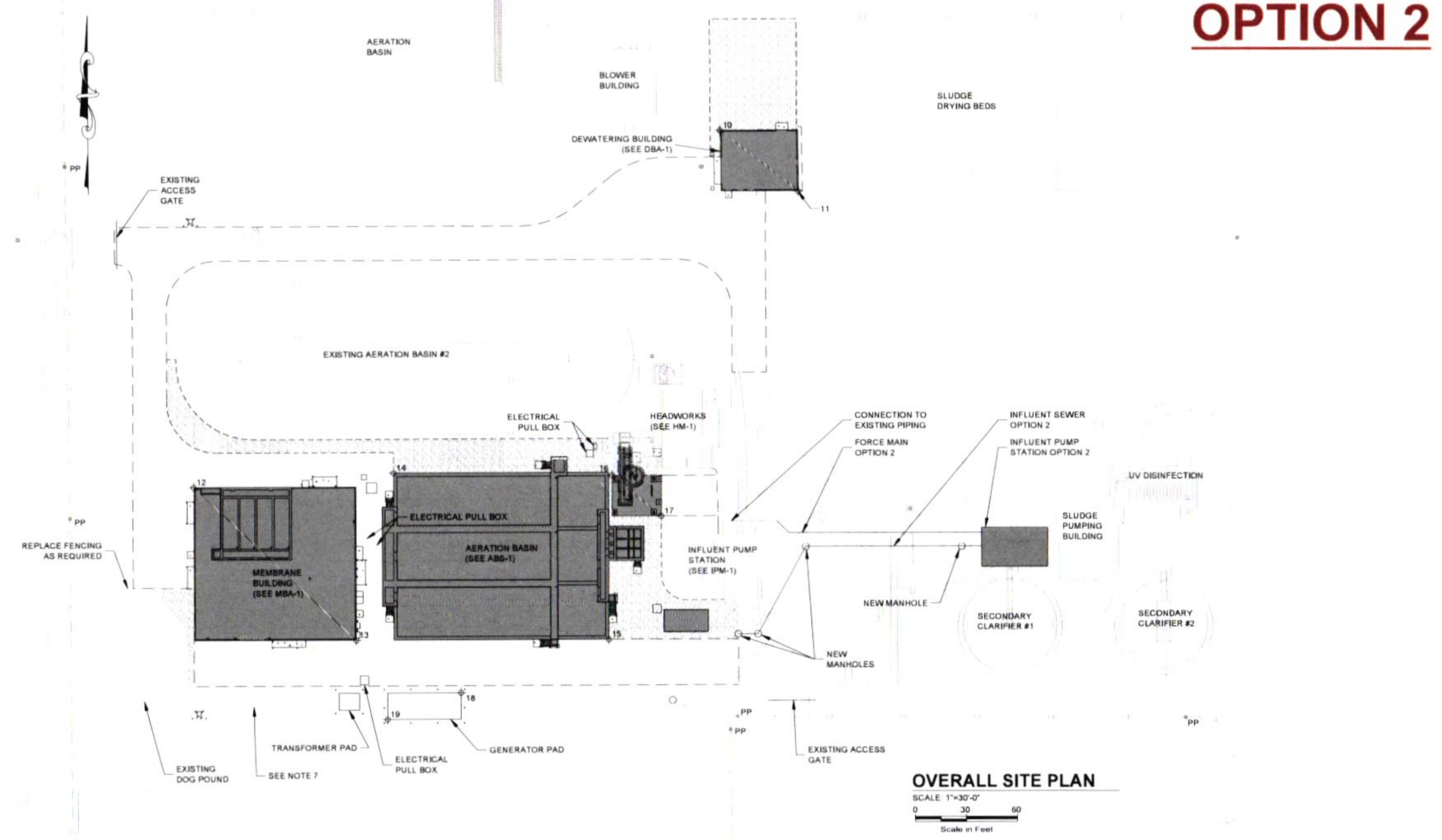
Option 1 (Submersibles) Cost Estimate

Division	Cost, $
Div. 2 Existing Conditions	$53,430
Div. 3 Concrete	$281,840
Div. 4 Masonry	$0
Div. 5 Metals	$96,800
Div. 6 Woods, Plastics, and Composites	$81,000
Div. 7 Thermal and Moisture Protection	$3,750
Div. 8 Openings	$0
Div. 9 Finishes	$115,000
Div. 10 Specialties	$0
Div. 13 Special Construction	$20,000
Div. 22 Plumbing	$5,000
Div. 23 HVAC	$21,500
Div. 26 Electrical	$610,830
Div. 31 Earthwork/Civil	$351,370
Div. 32 Exterior Improvements	$150,850
Div. 33 Utilities	$56,000
Div. 40 Process Integration	$183,080
Div 41. Material Processing and Material Handling	$44,620
Div. 43 Liquid and Solids Handling Equipment	$456,810
Div 44. Pollution Control Equipment	$55,000
Div 46. Water and Wastewater Equipment	$1,078,050
SUBTOTAL A - Inflated	**$3,640,000**
Div. 1 General Conditions (7.5%)	$273,000
Bond/Insurance	$54,600
Contingency (30%)	$1,092,000
Contractor's Overhead & Profit (10%)	$505,960
TOTAL CONSTRUCTION COST, 2023 DOLLARS	**$5,565,600**
Engineering Planning and Design	$417,420
Engineering Bidding and Construction	$417,420
Materials Testing	$50,000
TOTAL PROJECT COST, 2023 DOLLARS	**$6,450,440**

ATTACHMENT 2

Option 2 Site Plan, Mechanical Plan and Section, and Cost Estimate

OPTION 2



OVERALL SITE PLAN

SCALE: 1"=30'-0"

0 30 60

Scale in Feet

DRAWING IS TO SCALE IF BAR MEASURES 1" = FULL SCALE 1/2" = HALF SCALE (0 1/2 1)				
ORIGINAL				
NO	DATE	DESIGN	DRAWN	CHECKED
0	-	-	-	-
REVISIONS				

CITY OF IMPERIAL

HEADWORKS CONCEPT
SITE PLAN OPTION2

AQUA ENGINEERING

DRAWING NO.

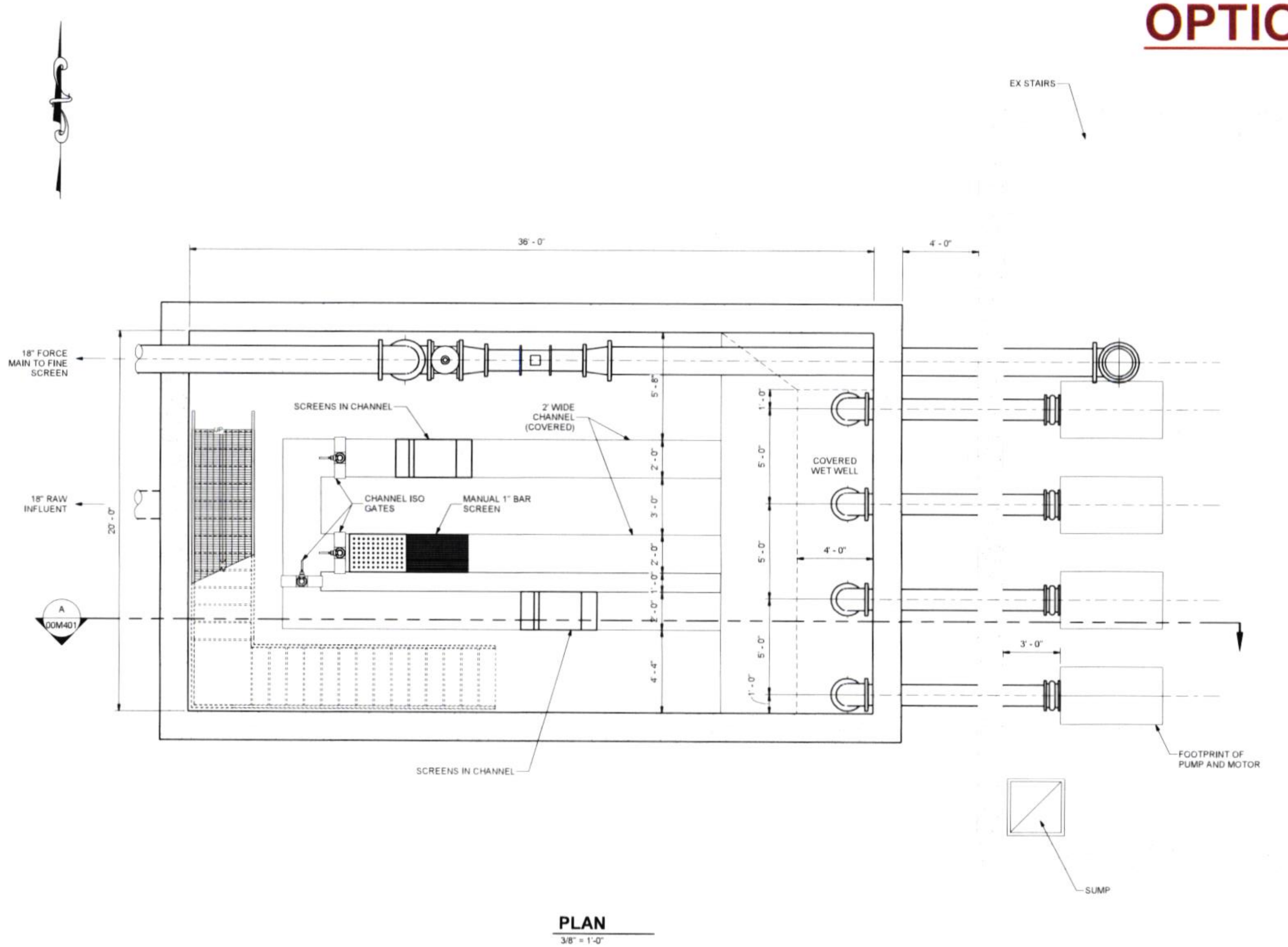
C-1

SHEET

5/28/2025

C:\USERS\KENNETH.BAETZ\DOCS\ACCDOCS\AQUA ENGINEERING\02857 C - IMPERIAL WWTP HEADWORKS PRELIM REPORT\PROJECT FILES\01 SITE CIVIL\C-1 - STANDARD\C-1_OPTION SITE PLAN.DWG

OPTION 2



PLAN

3/8" = 1'-0"

DRAWING IS TO SCALE IF BAR MEASURES 1" = FULL SCALE 1/2" = HALF SCALE

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ORIGINAL				
0	08/05/2021	Designer	Author	Checker
REVISIONS				
NO	DATE	DESIGN	DRAWN	CHECKED

IMPERIAL HEADWORKS CONCEPT

Location

AQUA ENGINEERING

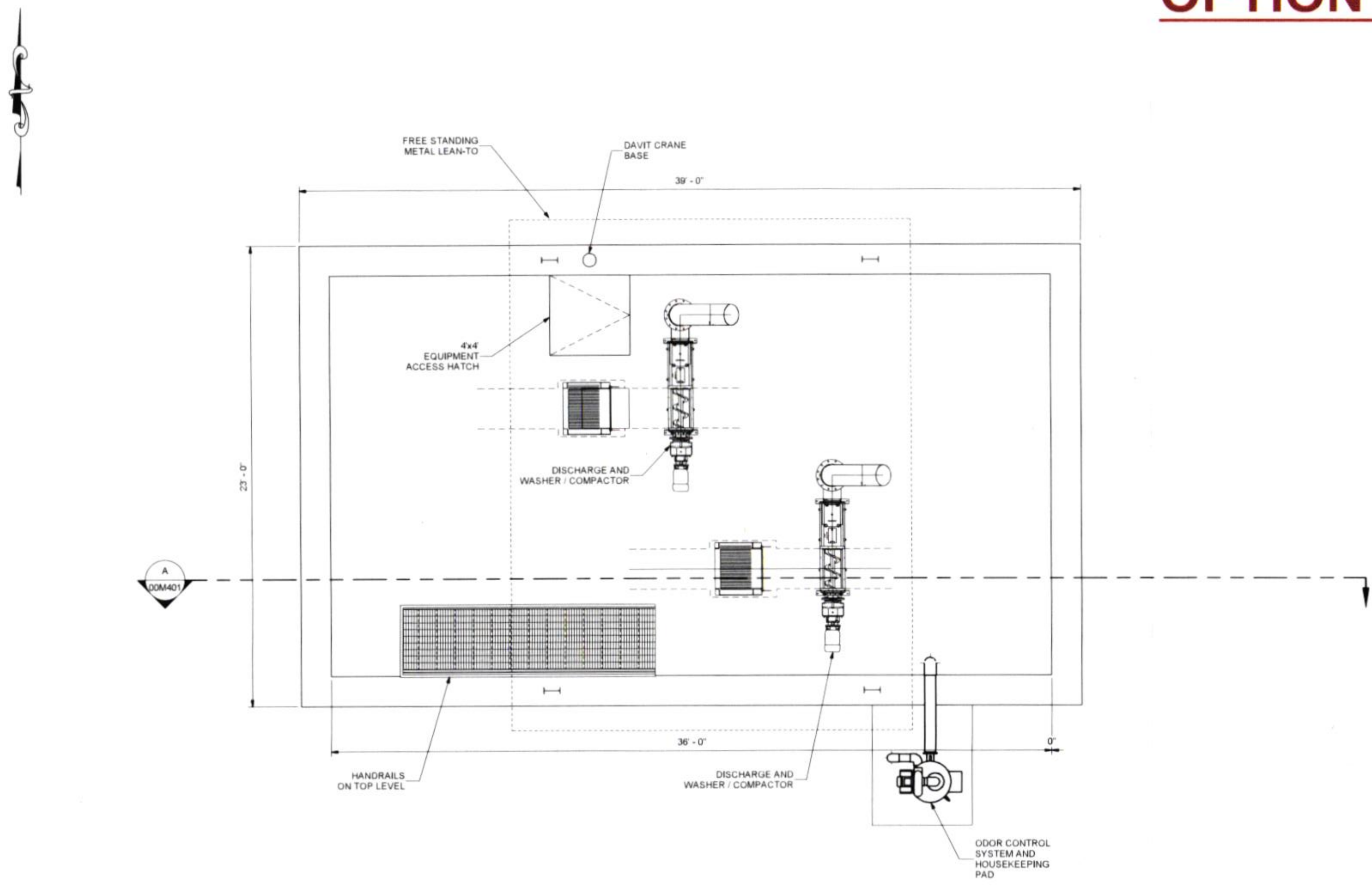
DRAWING NO.

00M201

SHEET

5/29/2025 11:29:24 AM BIM 360://002857.C - IMPERIAL WWTP HEADWORKS Prelim Report/City of Imperial_OPTION 2.rvt

OPTION 2



PLAN
3/8" = 1'-0"

ORIGINAL				
NO	DATE	DESIGN	DRAWN	CHECKED
C	08/08/2021	Designer	Author	Checker
REVISIONS				
NO	DATE	DESIGN	DRAWN	CHECKED

DRAWING IS TO SCALE IF BAR MEASURES 1" = FULL SCALE 1/2" = HALF SCALE

0 1/2 1

IMPERIAL HEADWORKS CONCEPT
Location

AQUA ENGINEERING

DRAWING NO.
00M202

SHEET

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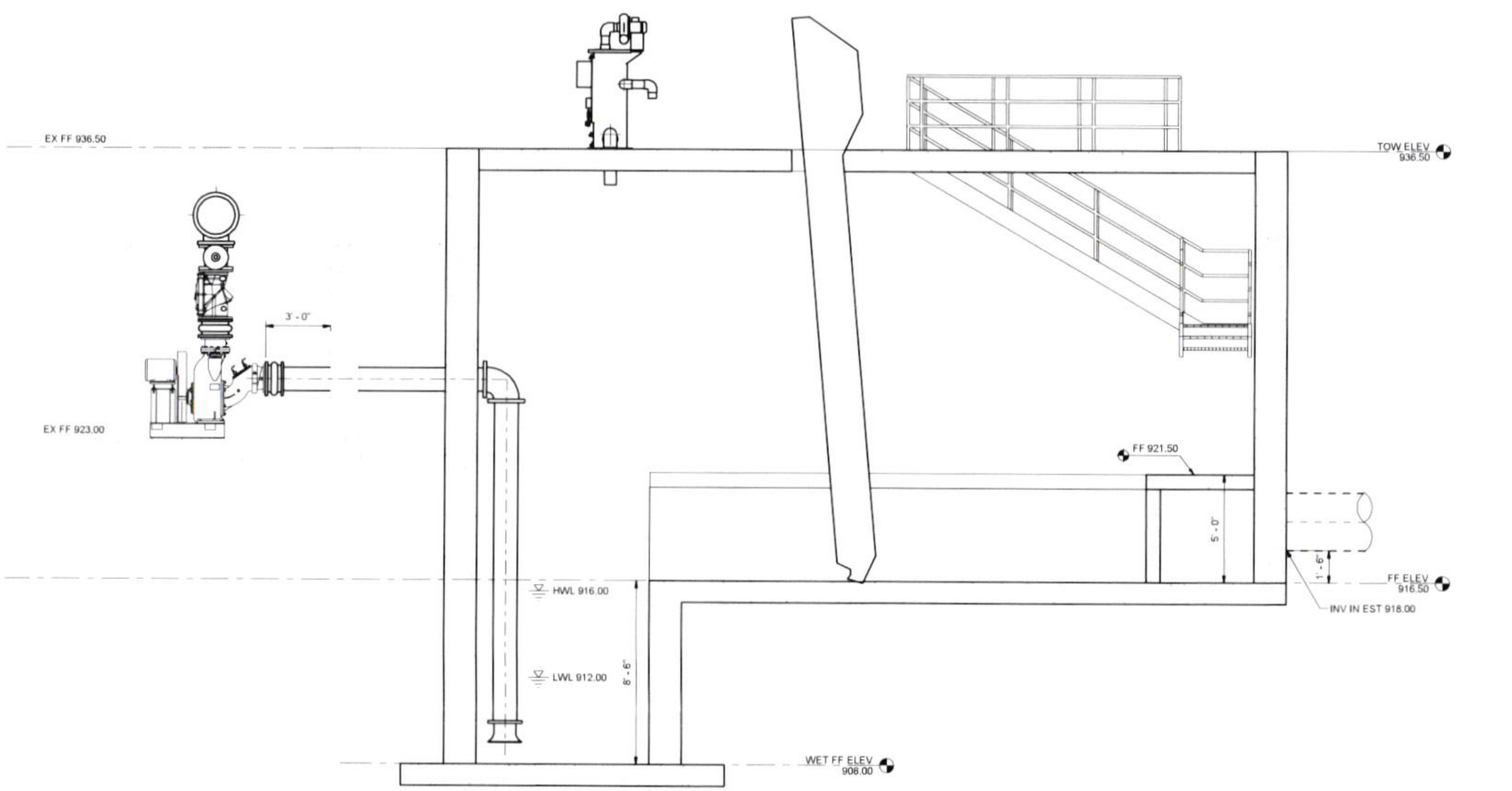
OPTION 2



A SECTION
00M201 3/8" = 1'-0"

NO	DATE	DESIGN	DRAWN	CHECKED
ORIGINAL				
0	06/05/2021	Designer	Author	Checker
REVISIONS				
NO	DATE	DESIGN	DRAWN	CHECKED

DRAWING IS TO SCALE IF BAR MEASURES 1" = FULL SCALE 1/2" = HALF SCALE

IMPERIAL HEADWORKS CONCEPT
Location

AQUA ENGINEERING

DRAWING NO.
00M401

SHEET

Option 2 (Dry Pit) Cost Estimate

Division	Cost, $
Div. 2 Existing Conditions	**$52,970**
Div. 3 Concrete	**$285,840**
Div. 4 Masonry	$0
Div. 5 Metals	$96,800
Div. 6 Woods, Plastics, and Composites	$81,000
Div. 7 Thermal and Moisture Protection	$3,750
Div. 8 Openings	$0
Div. 9 Finishes	**$120,000**
Div. 10 Specialties	$0
Div. 13 Special Construction	$20,000
Div. 22 Plumbing	$5,000
Div. 23 HVAC	**$41,500**
Div. 26 Electrical	**$916,940**
Div. 31 Earthwork/Civil	$351,370
Div. 32 Exterior Improvements	$150,850
Div. 33 Utilities	**$119,500**
Div. 40 Process Integration	**$208,280**
Div 41. Material Processing and Material Handling	**$4,620**
Div. 43 Liquid and Solids Handling Equipment	**$302,500**
Div 44. Pollution Control Equipment	$55,000
Div 46. Water and Wastewater Equipment	$1,078,050
SUBTOTAL A - Inflated	**$3,894,000**
Div. 1 General Conditions (7.5%)	$292,050
Bond/Insurance	$58,410
Contingency (30%)	$1,168,200
Contractor's Overhead & Profit (10%)	$541,270
TOTAL CONSTRUCTION COST, 2023 DOLLARS	**$5,954,000**
Engineering Planning and Design	$446,550
Engineering Bidding and Construction	$446,550
Materials Testing	$50,000
TOTAL PROJECT COST, 2023 DOLLARS	**$6,897,100**

ATTACHMENT 3

"Limited Geotechnical Report, Proposed Headworks at Imperial WWTP. LCI Report No. LE25072."
Landmark Consultants, Inc. April 30, 2025.

Limited Geotechnical Report

Proposed Headworks
Imperial WWTP
Imperial, California

Prepared for:

Albert A. Webb & Associates
3788 McCray Street
Riverside, CA 92506

LANDMARK
Geo-Engineers and Geologists

Prepared by:

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

April 2025

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April 30, 2025

Mr. Brian Knoll, PE
Albert A. Webb Associates
3788 McCray Street
Riverside, CA 92506

Geotechnical Report
Proposed Headworks at Imperial WWTP
720 E. 14th Street
Imperial, California
LCI Report No. LE25072

Dear Mr. Knoll:

This geotechnical report is provided for design and construction of the proposed improvements to the existing Imperial Wastewater Treatment plant located at 720 E. 14th Street in northeast Imperial, California. Our geotechnical exploration was conducted in response to your request for our services. The enclosed report describes our soil engineering site evaluation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

This executive summary presents ***selected*** elements of our findings and professional opinions. This summary ***may not*** present all details needed for the proper application of our findings and professional opinions. Our findings, professional opinions, and application options are ***best related through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them. The findings of this study are summarized below:

The findings of this study indicate that the site is, in general, predominantly underlain by stiff to very stiff silty clay/clay (CL-CH) to a depth of 41.5 feet. Interbedded sandy silt/silty sand (ML/SM) and clayey silt (ML) layers of about 5 to 8 feet were encountered at a depth of 3 to 8 feet, 14 to 22 feet and 33 to 38 feet below ground surface.

The clay soils are very aggressive to concrete and steel. Concrete mixes shall have a maximum water cement ratio of 0.45 and a minimum compressive strength of 4,500 psi (minimum of 6.25 sacks Type V cement per cubic yard).

All reinforcing bars, anchor bolts and hold down bolts shall have a minimum concrete cover of 4.0 inches unless epoxy coated (ASTM D3963/A934). Hold-down straps are not allowed at the foundation perimeter. No pressurized water lines are allowed below or within the foundations.

We did not encounter soil conditions that would preclude development of the proposed project provided the professional opinions contained in this report are considered in the design and construction of this project.

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.

Steven K. Williams, PG, CEG
Senior Engineering Geologist

Peter E. LaBrucherie, PE, GE
Principal Geotechnical Engineer

Julian R. Avalos, GE
Senior Geotechnical Engineer

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Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed improvements to the existing Imperial Wastewater Treatment plant located at 720 14th Street in northeast Imperial, California (See Vicinity Map, Plate A-1). The proposed improvements will consist of the removal of an existing clarifier and operations building, and the construction of an underground headworks structure which will consist of an approximate 30-foot deep concrete headworks structure.

Site development will include headworks excavations, underground utility installation including trench backfill and concrete foundation/wall construction.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 40 feet of subsurface soil at the proposed headworks site for evaluation of physical/engineering properties and liquefaction potential during seismic events. Professional opinions were developed from field and laboratory test data 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.
- 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 and professional opinions regarding the geotechnical aspects of project design and construction.

This report addresses the following geotechnical parameters:

- Subsurface soil and groundwater conditions
- Site geology, regional faulting and seismicity, near source factors, and site seismic accelerations
- Expansive soil and methods of mitigation
- Aggressive soil conditions to metals and concrete

Professional opinions with regard to the above parameters are provided for the following:

- Site grading and earthwork
- Foundation subgrade preparation
- Allowable soil bearing pressures and expected settlements
- 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

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

1.3 Authorization

Mr. Brian Knoll of Webb Associates provided authorization by written agreement to proceed with our work on April 4, 2025. We conducted our work according to our written proposal dated April 4, 2025.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on April 8, 2025 using 2R Drilling of Ontario, California to advance one (1) boring to a depth of 41.5 feet below existing ground surface. A temporary piezometer pipe was placed to a depth of 20 feet below ground surface within this boring (B-1). An additional piezometer was placed north of the proposed headworks location (B-2). The borings/piezometers were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernible site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A geotechnician 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) were 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.

After excavation of the borings, a 2-inch diameter PVC piezometer was installed in each boring for groundwater level readings. The piezometers consisted of 10 feet of slotted well screen (0.010 screen size) covered with a filter sock and 10 feet of solid riser pipe. The annular space was backfilled using the native auger cuttings. The piezometers were completed with traffic rated steel manhole covers and concrete aprons.

The subsurface log is presented on Plate B-1 in Appendix B. A key to the log symbols is presented on Plate B-2. The stratification lines shown on the subsurface log represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

No laboratory tests were conducted for this investigation.

Section 3
DISCUSSION

3.1 Site Conditions

The Imperial Wastewater Treatment Plant facility is rectangular in plan view and is located at 720 E. 14^{th} Street in northeast Imperial, California. The existing west clarifier and operations building will be removed to allow construction of the proposed headworks structure.

The MBR facility (built within the last 4 years) is located to the west of the proposed headworks structure location. An oxidation basin is located adjacent to the north side of the MBR facility location plant. Existing headworks, an aeration basin and influent pump stations are located adjacent to the east side of the MBR facility area. The existing operation building, clarifiers, sludge pumping station building and the UV disinfection structure are located at the southeast side of the wastewater plant. Sludge drying beds are located to the north side of the wastewater plant. Existing underground power lines and raw water supply lines cross the wastewater plant in east to west and north to south directions.

Adjacent properties are flat-lying and are approximately at the same elevation with this site. The Imperial Public Works maintenance yard and a 2.0 MG above ground treated water steel storage tank lies to the south side of the site. P Street and the Date Canal are located along the east side of the project site with agricultural land beyond. The Union Pacific Railroad tracks are located along the west side of the project site, with the Imperial Irrigation District Headquarters Yard and IID substation beyond. The Morningside Residential subdivision lies to the north side of the wastewater plant facility.

The project site lies at an elevation of approximately 65 feet below mean sea level (MSL) (El. 935 local datum) in the Imperial Valley region of the California low desert. The surrounding properties lie on terrain which is flat (planar), part of a large agricultural valley, which was previously an ancient lakebed 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 deposited 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 (present) lake deposits are probably less than 100 feet thick and derived from periodic flooding of the Colorado River which intermittently formed a freshwater 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.

3.3 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on April 8, 2025 consist of dominantly stiff to very stiff silty clay/ clay (CL-CH) to a depth of 41.5 feet. Interbedded sandy silt/silty sand (ML/SM) and clayey silt (ML) layers of about 5 to 8 feet were encountered at a depth of 3 to 8 feet, 14 to 22 feet and 33 to 38 feet below ground surface.

The native surface clays likely exhibit moderate to high swell potential (Expansion Index, EI = 51 to 110) when correlated to Plasticity Index tests (ASTM D4318) performed on the native clays. The clay is expansive when wetted and can shrink with moisture loss (drying).

3.4 Groundwater

Groundwater was encountered in the borings at about 13 feet during the time of exploration but stabilized within the installed piezometers at approximately 9 feet below ground surface at Boring B-1 and 10 feet below ground surface at Boring B-2 on April 9, 2025, approximately 24 hours after installation. Dewatering should be anticipated for wet well construction and piping installed below a depth of 9 feet. There is uncertainty in the accuracy of short-term water level measurements, particularly in fine-grained soil. Groundwater levels may fluctuate with precipitation, irrigation of adjacent properties, drainage, and site grading. The referenced groundwater level should not be interpreted to represent an accurate or permanent condition. Our work scope did not include a groundwater surface mounding study resulting from applied landscape water.

3.5 Faulting

The project site is located in the seismically active Imperial Valley of southern California with numerous mapped faults of the San Andreas Fault System traversing the region. The San Andreas Fault System is comprised of the San Andreas, San Jacinto, and Elsinore Fault Zones in southern California. The Imperial fault represents a transition from the more continuous San Andreas fault to a more nearly echelon pattern characteristic of the faults under the Gulf of California (USGS 1990). We have performed a computer-aided search of known faults or seismic zones that lie within a 36-mile radius of the project site (Table 1).

A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults. The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along active or potentially active faults. An active fault is one that has ruptured during Holocene time (roughly within the last 11,000 years). A fault that has ruptured during the last 1.8 million years (Quaternary time), but has not been proven by direct evidence to have not moved within Holocene time is considered to be potentially active. A fault that has not moved during Quaternary time is considered to be inactive. Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2000a) indicates that the nearest mapped Earthquake Fault Zone is the Imperial fault located approximately 2.4 miles northeast of the project site.

3.6 General Ground Motion Analysis

The project site is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Acceleration magnitudes 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.

2022 CBC General Ground Motion Parameters: The California Building Code (CBC) requires that a site-specific ground motion hazard analysis be performed in accordance with ASCE 7-16 Section 11.4.8 (ASCE, 2016) for structures on Site Class D with S_1 greater than or equal to 0.2 and Site Class E sites with S_s greater than or equal to 1.0 (CBC, 2023). **This project site has been classified as Site Class D and has a S_1 value of 0.65, which would require a site-specific ground motion hazard analysis.** However, ASCE 7-16 Section 11.4.8 Supplement 3 provides exceptions which permit the use of conservative values of design parameters for certain conditions for Site Class D and E sites in lieu of a site specific hazard analysis. The exceptions are:

- Site Class D sites: A ground motion hazard analysis is not required where the value of the parameter S_{M1} determined by Equation 11.4-2 is increased by 50% for all applications of S_{M1} in ASCE 7-16. The resulting value of the parameter S_{D1} determined by ASCE 7-16 Equation 11.4-4 shall be used for all applications of S_{D1} in ASCE 7-16.

- Site Class E sites: A ground motion hazard analysis is not required:
  a. Where the equivalent lateral force procedure is used for design and the value of C_S is determined by ASCE 7-16 Equation 12.8-2 for all values of T, or
  b. Where (i) the value of S_{ai} is determined by ASCE 7-16 Equation 15.7-7 for all values of T_i and (ii) the value of the parameter S_{D1} is replaced with $1.5S_{D1}$ in ASCE 7-16 Equation 15.7-10 and ASCE 7-16 Equation 15.7-11.

Based on the project site being classified as Site Class D, the structural engineer should increase the parameter S_{M1} provided in Table 2 by 50% for all applications of S_{M1} in ASCE 7-16. If a site-specific ground motion hazard analysis is required for the project, our office should be consulted to perform a site-specific ground motion hazard analysis. **Design earthquake ground motion parameters are provided in Table 2.**

The 2022 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps Web Application (SEAOC, 2025) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for soil site class effects (PGA_M) value to be used for liquefaction and seismic settlement analysis in accordance with 2022 CBC Section 1803.5.12.2 is estimated at 0.84g for the project site.

3.7 Seismic and Other Hazards

- **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the Imperial, Brawley, and Superstition Hills faults.
- **Surface Rupture.** The California Geological Survey has established Earthquake Fault Zones in accordance with the 1972 Alquist-Priolo Earthquake Fault Zone Act. The Earthquake Fault Zones consists of boundary zones surrounding well defined, active faults or fault segments. The project site does not lie within an A-P Earthquake Fault Zone; therefore, surface fault rupture is considered to be low at the project site.
- **Liquefaction.** Liquefaction is a potential design consideration because of underlying saturated sandy substrata. Although the Imperial Valley has not yet been evaluated for seismic hazards by the California Geological Survey seismic hazards zonation program, liquefaction is well documented in the Imperial Valley after strong seismic events (McCrink, et al, 2011 and Rymer et al, 2011). ***The evaluation for the potential for liquefaction induced settlements at the site is not included in the scope of work for this project.***

Other Potential Geologic 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 not located in proximity to any known volcanically active area and the risk of volcanic hazards is considered very low.

- **Tsunamis and seiches.** The site is not located near any large bodies of water, so the threat of tsunami, seiches, or other seismically-induced flooding is unlikely.
- **Flooding.** The project site is located in FEMA Flood Zone X, an area determined to be outside the 0.2% annual chance floodplain (FIRM Panel 06025C1725C).
- **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 3.3.

Section 4
DESIGN CRITERIA

4.1 Site Preparation

Clearing and Grubbing: All surface improvements, debris or vegetation including grass and weeds on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic strippings should be stockpiled and not used as engineered fill. All trash, construction debris, concrete slabs, old pavement, landfill, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign material by the grading contractor and removed under our supervision. Any excavations resulting from site clearing should be sloped to a bowl shape to the lowest depth of disturbance and backfilled under the observation of the geotechnical engineer's representative.

Below Grade Structures Site Preparation: The headworks structure is planned to be constructed at the location of the existing operations building and is anticipated to be founded at approximately 30 feet below existing grade. The subsurface silty clays at the proposed bottom of excavation are saturated; consequently, the subgrade has a high potential for pumping under equipment loads. Therefore, the subgrade for the new headworks structure should be overexcavated 24 inches and replaced with drainage rock (ASTM C33, Size 57 or 467). The bottom of the excavation should be covered with a geotextile filter fabric (Mirafi 180 or better) lapped at sides and ends in accordance with manufacture's installations guidelines. The 2.0 ft thick layer of drainage rock should be end-dumped onto the filter fabric and spread evenly by excavators or dozers. Upon completing placement of the drainage rock a small vibratory compactor (walk-behind or equivalent) should be used to densify the crushed rock layer. Following densification of the drainage rock, a second layer of filter fabric should be placed over the drainage rock.

Excavation for the headworks structure (approximately 30 feet depth) will encounter the groundwater table (9 feet bgs). Therefore, seepage and pumping subgrade conditions should be anticipated. An adequately designed dewatering system, such as well points or sumps, will be required to control groundwater seepage and prevent running ground conditions. During construction groundwater should be maintained a minimum of 2 feet below the bottom of the excavation. The responsibility for dewatering and selection of an appropriate system for dewatering is beyond the scope of this report.

Utility Trench Backfill: Prior to placement of utility bedding, the exposed subgrade at the bottom of trench excavations should be examined for soft, loose, or unstable soil. Loose materials at trench bottoms resulting from excavation disturbance should be removed to firm material. If extensive soft or unstable areas are encountered, these areas should be over-excavated to a depth of at least 2 feet or to a firm base and be replaced with additional bedding material.

Backfill Materials: Pipe zone backfill (i.e., material beneath and in the immediate vicinity of the pipe) should consist of a 4 to 8 inch bed of ⅜-inch crushed rock, sand/cement slurry (3 sack cement factor), and/or crusher fines (sand) extending to a minimum of 12 inches above the top of pipe. If crushed rock is used for pipe zone backfill for utilities, the crushed rock material should be completed surrounded by a non-woven filter fabric such as Mirafi 140N or equivalent. The filter fabric shall cover the trench bottom, sidewalls and over the top of the crushed rock. The filter fabric is recommended to inhibit the migration of fine material into void spaces in the crushed rock which may create the potential for sinkholes or depressions to develop at the ground surface.

Pipe bedding should be in accordance with pipe manufacturer's recommendations. Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local codes and/or bedding requirements for specific types of pipes. 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.

Compaction Criteria: Mechanical compaction is recommended; ponding or jetting should not be allowed, especially in areas supporting structural loads or beneath concrete slabs supported-on-grade, pavements, or other improvements. All trench backfill should be placed and compacted in accordance with recommendations provided above for engineered fill.

The pipe zone material (crusher fines, sand) shall be compacted to a minimum of 95% 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. Soils used for trench backfill shall be placed in maximum 6-inch lifts (loose), compacted to a minimum of 90% of ASTM D1557 maximum density at a minimum of 4% above optimum moisture.

Imported granular material is acceptable for backfill of utility trenches. Granular trench backfill used in building pad areas should be plugged with a solid (no clods or voids) 2-foot width of native clay soils at each end of the building foundation to prevent landscape water migration into the trench below the building.

Backfill soil of utility trenches within paved areas should be uniformly moisture conditioned to a minimum of 4% above optimum moisture, placed in layers not more than 6 inches in thickness and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density, except that the top 12 inches shall be compacted to 95% (if granular trench backfill).

Observation and Density Testing: All site preparation and fill placement should be continuously observed and tested by a representative of a qualified geotechnical engineering firm. Full-time observation services during the excavation and scarification process is necessary to detect undesirable materials or conditions and soft areas that may be encountered in the construction area. 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 geotechnical parameters for site development.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have footings extended to a minimum of 30 inches below grade. The existing soil beneath the structure foundation preparation needed only to extend 18 inches below and beyond the footing.

4.2 Foundations and Settlements

Structural concrete mat foundations are suitable to support the headworks structure. The mats shall be founded on a layer of properly prepared and compacted soil as described in Section 4.1.

The relatively light headworks structure may use soil unloading as a means to control settlement. The general, in-situ soil load is approximately 120 pcf and by removing 30 feet of soil, 3,600 psf of foundation loading can be offset (e.g. a 5,000 psf foundation load can be reduced to 1,400 psf net soil loading).

The foundations may be designed using an allowable net soil bearing pressure of 3,000 psf when foundations are supported a minimum 20 feet below ground surface.

Flat Plate Structural Mats: Flat plate structural mats may be used to mitigate expansive soils at the project site. The structural mat shall have a double mat of steel (minimum No. 4's @ 12 inches O.C. each way – top and bottom) and a minimum thickness of 10 inches. Mat edges shall have a minimum edge footing of 12 inches width and 24 inches depth (below the building pad surface).

Structural mats may be designed for a modulus of subgrade reaction (Ks) of 50 pci when placed on compacted native soil or a subgrade modulus of 250 pci when placed on 24 inches of crushed rock (below grade structures).

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 250 pcf (300 pcf for imported sands or crushed rock) 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 0.25 (0.35 for imported sands or crushed rock) may also be used at the base of the footings to resist lateral loading.

Foundation movement under the estimated static (non-seismic) loadings and static site conditions are estimated to not exceed 1 inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed.

4.3 Slabs-On-Grade

Structural Concrete: Structural concrete slabs are those slabs (foundations) that underlie structures or patio covers (shades). These slabs that are placed over native clay soil should be either a uniformly thick structural mats (10 inches or greater) or should be designed in accordance with Chapter 18 of the 2022 CBC and shall be a minimum of 5 inches thick due to expansive soil conditions. Concrete floor slabs shall be monolithically placed with the footings (no cold joints) unless placed on 3.0 feet of granular fill soil.

American Concrete Institute (ACI) guidelines (ACI 302.1R-15 Chapter 5, Section 5.2.3) provide recommendations regarding the use of moisture barriers beneath concrete slabs. The concrete floor slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. All laps and seams should be overlapped 6-inches or as recommended by the manufacturer. The vapor retarder should be protected from puncture. The joints and penetrations should be sealed with the manufacturer's recommended adhesive, pressure-sensitive tape, or both. The vapor retarder should extend a minimum of 12 inches into the footing excavations. The vapor retarder should be covered by 4 inches of clean sand (Sand Equivalent SE>30) unless placed on 3.5 feet of granular fill, in which case, the vapor retarder may lie directly on the granular fill with 2 inches of clean sand cover.

Placing sand over the vapor retarder may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor retarder may also move and mound prior to concrete placement, resulting in an irregular slab thickness. For areas with moisture sensitive flooring materials, ACI recommends that concrete slabs be placed without a sand cover directly over the vapor retarder, provided that the concrete mix uses a low-water cement ratio and concrete curing methods are employed to compensate for release of bleed water through the top of the slab. The vapor retarder should have a minimum thickness of 15-mil (Stego-Wrap or equivalent).

Structural concrete slab reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 4 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking. Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings. All steel components of the foundation system should be protected from corrosion by maintaining a 4-inch minimum concrete cover of densely consolidated concrete at footings (by use of a vibrator). The construction joint between the foundation and any mowstrips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint. Epoxy coated embedded steel components (ASTM D3963/A934) or permanent waterproofing membranes placed at the exterior footing sidewall may also be used to mitigate the corrosion potential of concrete placed in contact with native soil.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement. Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

<u>Non-structural Concrete:</u> All non-structural independent flatwork (sidewalks and housekeeping slabs) shall be a minimum of 4 inches thick and should be placed on a minimum of 4 inches of compacted (90%) concrete sand or aggregate base, dowelled to the perimeter foundations where adjacent to the building to prevent separation and sloped 2% (sidewalks) or 1 to 2% (housekeeping slabs) away from the building. A 15-mil polypropylene vapor barrier shall be placed over native soils prior to placing sand underlayment. Area slabs with shade structures shall have an 18-inch deep perimeter footing and shall have interior grade beams at 15 feet on center. Planters that trap water between sidewalks and foundations are not allowed.

A minimum of 24 inches of moisture conditioned (5% minimum above optimum) and 8 inches of compacted subgrade (85 to 90%) should underlie all independent flatwork. Flatwork which contains steel reinforcing (except wire mesh) should be underlain by a 10-mil (minimum) polyethylene separation sheet and at least a 2-inch sand cover. All flatwork should be jointed in square patterns and at irregularities in shape at a maximum spacing of 8 feet or the least width of the sidewalk.

4.4 Concrete Mixes and Corrosivity

Past projects at the project site have indicated severe levels of sulfate ion concentration (approximately 4,000 ppm). Sulfate ions in high concentrations can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling. The following table provides American Concrete Institute (ACI) recommended cement types, water-cement ratio and minimum compressive strengths for concrete in contact with soils:

Concrete Mix Design Criteria due to Soluble Sulfate Exposure

Sulfate Exposure	Water-soluble Sulfate (SO_4) in soil, ppm	Cement Type	Maximum Water-Cement Ratio by weight	Minimum Strength f'c (psi)
Negligible	0-1,000	–	–	–
Moderate	1,000-2,000	II	0.50	4,000
Severe	2,000-20,000	V	0.45	4,500
Very Severe	Over 20,000	V (plus Pozzolon)	0.45	4,500

Note: from ACI 318-11 Table 4.2.1

A minimum of 6.25 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 hardscape and foundations). Admixtures may be required to allow placement of this low water/cement ratio concrete. Thorough concrete consolidation and hard trowel finishes should be used due to the aggressive soil exposure.

The native soil (past projects) has very severe levels of chloride ion concentration (5,000 to 10,000 ppm). Chloride ions can cause corrosion of reinforcing steel, anchor bolts and other buried metallic conduits. Resistivity determinations on the soil indicate very severe potential for metal loss because of electrochemical corrosion processes.

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 4 inches of densely consolidated concrete. ***No metallic water pipes or conduits should be placed below foundations.***

Foundation designs shall provide a minimum concrete cover of four (4) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 4-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy coated for corrosion protection (in accordance with ASTM D3963/A934) or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the exterior footings. ***Hold-down straps should not be used at foundation edges due to corrosion of metal at its protrusion from the slab edge.*** Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete. ***Copper water piping should not be placed under floor slabs.***

4.5 Excavations

All site excavations should conform to CalOSHA requirements for Type B soil (if site is dewatered or Type C soils for non-dewatered excavations). 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. These temporary deep excavations will require slope inclinations no steeper than 1½(H):1(V) unless trench shoring is used. If excavations are planned below groundwater (10 feet below ground surface), all excavation slopes should be excavated according to OSHA Standards for Type C soils. Dewatering of the excavation site will be required prior to start of excavation (2 ft. below bottom of excavation).

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.

All discussions in this section regarding stable excavation slopes assume minimal equipment vibration and adequate setback of excavated material and construction equipment from the top of the excavation. We recommended that the minimum setback distance be equal to the depth of excavation and at least 10 feet from the crown of the slope. If excavated materials are stockpiled adjacent to the excavation, the weight of the material should be considered as a surcharge load for slope stability.

Excavation for the headworks (approximately 30 feet depth) will encounter the groundwater table (9 feet bgs). Therefore, seepage and pumping subgrade conditions should be anticipated. An adequately designed dewatering system, such as well points or sumps, will be required to control groundwater seepage and prevent running ground conditions. The responsibility for dewatering and selection of an appropriate system for dewatering is beyond the scope of this report.

4.6 Lateral Earth Pressures

Earth retaining structures, such as retaining walls, should be designed to resist the soil pressure imposed by the retained soil mass. Walls with granular drained backfill may be designed for an assumed static earth pressure equivalent to that exerted by a fluid weighing 60 pcf (native) and 45 pcf (granular) for unrestrained (active) conditions (able to rotate 0.1% of wall height), and 100 (native) and 60 pcf (granular) for restrained (at-rest) conditions. These values should be verified at the actual wall locations during construction.

Walls below groundwater may be designed with a static earth pressure equivalent to that exerted by a fluid weighing 35 pcf (native) and 25 pcf (granular) for unrestrained (active) conditions, and 60 (native) and 35 pcf (granular) for restrained (at-rest) conditions. Hydrostatic water pressure of 62.4 pcf shall be added to the provided values for structures below groundwater. Native soils unit weight considered are as follows:

bulk unit weight = 125 pcf
saturated unit weight = 133 pcf
submerged unit weight = 71 pcf.

When applicable (Seismic Design Category D, E or F), retaining wall structures where the backfill is greater than 6 feet high shall be designed in addition to the static loading (active or at-rest condition) with an additional seismic lateral pressure increasing linearly with depth and the resultant acting as a point load at 0.4H above the base of the wall. The term H is the height of the backfill against a retaining wall in feet. The seismic load increment, shall be determined using the following equations for different wall type and backfill conditions:

Basement (restrained) walls with level backfill:	$\Delta K_{ae} = \frac{1}{2}\gamma H^2(0.68\ PGA_M/g)$
Cantilever (unrestrained) wall with level backfill:	$\Delta K_{ae} = \frac{1}{2}\gamma H^2(0.42\ PGA_M/g)$
Cantilever (unrestrained) wall with sloping backfill*:	$\Delta K_{ae} = \frac{1}{2}\gamma H^2(0.70\ PGA_M/g)$

*Applicable for sloping backfill that is no steeper than 2:1 (horizontal:vertical).

Where:

ΔK_{ae} = Seismic Lateral Force (plf) based on seismic pressure
γ = 125 pcf
H = Height of retained soil (ft)
g = A PGA_M value of 0.84g has been determined for the project site.

Surcharge loads should be considered if loads are applied within a zone between the face of the wall and a plane projected behind the wall 45 degrees upward from the base of the wall. The increase in lateral earth pressure acting uniformly against the back of the wall should be taken as 50% of the surcharge load within this zone. Areas of the retaining wall subjected to traffic loads should be designed for a uniform surcharge load equivalent to two feet of native soil.

Walls should be provided with backdrains to reduce the potential for the buildup of hydrostatic pressure. The drainage system should consist of a composite HDPE drainage panel, or a 2-foot-wide zone of free draining crushed rock placed adjacent to the wall and extending 2/3 the height of the wall. The gravel should be completely enclosed in an approved filter fabric to separate the gravel and backfill soil. A perforated pipe should be placed perforations down at the base of the permeable material at least six inches below finished floor elevations. The pipe should be sloped to drain to an appropriate outlet that is protected against erosion. Walls should be properly waterproofed. The project geotechnical engineer should approve any alternative drain system.

4.7 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 Brawley, Superstition Hills, and Imperial Faults. 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 3.6 and Table 2 of this report.

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The findings and professional opinions within this report are based on current information regarding the proposed improvements to the existing Imperial Wastewater Treatment plant located at 720 E. 14th Street in northeast Imperial, California.

The conclusions and professional opinions 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 professional opinions 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 professional opinions 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 professional opinions by our firm, because of 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 professional opinions presented in this report are based on the assumption that:

- Consultation during development of design and construction documents to check that the geotechnical professional opinions are appropriate for the proposed project and that the geotechnical professional opinions 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.
- 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 professional opinions and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

Section 6
REFERENCES

American Concrete Institute (ACI), 2015, ACI Manual of Concrete Practice 302.1R-15.

American Concrete Institute (ACI), 2019, ACI Manual of Concrete Practice 318-19.

American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads for Buildings and Other Structures: ASCE Standard 7-16.

California Building Standards Commission, 2023, 2022 California Building Code. California Code of Regulations, Title 24, Part 2, Vol. 2 of 2.

Caltrans, 2020, Caltrans Geotechnical Manual.

California Geological Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, 98p.

California Geological Survey (CGS), 2025a, Fault Activity Map of California https://maps.conservation.ca.gov/cgs/fam/.

California Geological Survey (CGS), 2025b, Alquist-Priolo Earthquake Fault Zone Maps. http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map (FIRM), Imperial County, California and Incorporated Areas. Dated September 26, 2008.

Morton, P. K., 1977, Geology and mineral resources of Imperial County, California: California Division of Mines and Geology, County Report No. 7, 104 p.

Norris and Webb, 1990, Geology of California, 2nd Edition, John Wiley and Sons.

Structural Engineers Association of California (SEAOC), 2025, Seismic Design Maps Web Application, available at https://seismicmaps.org/

USDA Natural Resources Conservation Service, 2025, Web Soil Survey Website. https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

Wire Reinforcement Institute (WRI/CRSI), 2003, Design of Slab-on-Ground Foundations, Tech Facts TF 700-R-03, 23 p.

TABLES

Table 1
Summary of Characteristics of Closest Known Active Faults

Fault Name	Approximate Distance (miles)	Approximate Distance (km)	Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
Imperial	2.4	3.8	7	62 ± 6	20 ± 5
Superstition Hills	3.2	5.1	6.6	23 ± 2	4 ± 2
Brawley *	5.0	8.1			
Superstition Mountain	7.5	12.1	6.6	24 ± 2	5 ± 3
Rico *	9.8	15.7			
Route 247*	13.4	21.5			
Northern Centinela*	13.7	21.9			
Yuha*	15.4	24.7			
Shell Beds	17.1	27.3			
Yuha Well *	17.2	27.4			
Painted Gorge Wash*	20.0	32.1			
Vista de Anza*	20.3	32.6			
Laguna Salada	20.4	32.6	7	67 ± 7	3.5 ± 1.5
Elmore Ranch	20.5	32.8	6.6	29 ± 3	1 ± 0.5
Borrego (Mexico)*	21.3	34.1			
Ocotillo*	24.4	39.0			
Cerro Prieto *	24.9	39.8			
Pescadores (Mexico)*	26.8	42.9			
San Jacinto - Borrego	27.2	43.6	6.6	29 ± 3	4 ± 2
Elsinore - Coyote Mountain	27.6	44.1	6.8	39 ± 4	4 ± 2
Cucapah (Mexico)*	28.1	45.0			
San Andreas - Coachella	35.6	56.9	7.2	96 ± 10	25 ± 5

* Note: Faults not included in CGS database.

Table 2
2022 California Building Code (CBC) and ASCE 7-16 Seismic Parameters

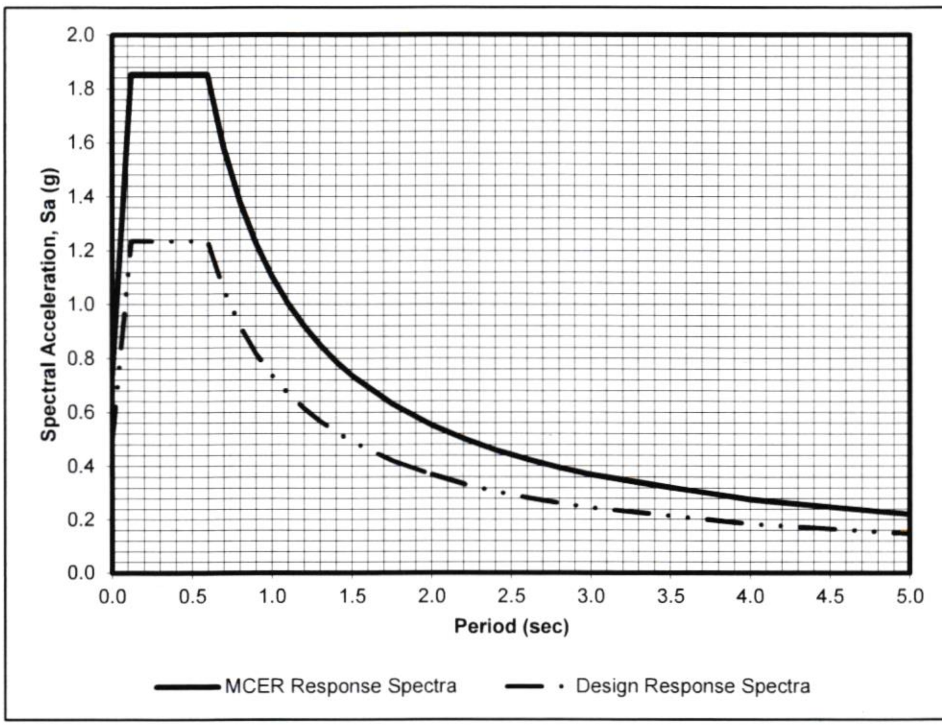
			ASCE 7-16 Reference
Soil Site Class:	**D**		Table 20.3-1
Latitude:	32.8538	N	
Longitude:	-115.5624	W	
Risk Category:	II		
Seismic Design Category:	D		

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	S_s	1.854 g	ASCE Figure 22-1	
Mapped MCE_R 1 second Spectral Response	S_1	0.650 g	ASCE Figure 22-2	
Short Period (0.2 s) Site Coefficient	F_a	1.00	ASCE Table 11.4-1	
Long Period (1.0 s) Site Coefficient	F_v	1.70	ASCE Table 11.4-2	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	S_{MS}	1.854 g	$= Fa * S_s$	ASCE Equation 11.4-1
MCE_R Spectral Response Acceleration Parameter (1.0 s)	S_{M1}	1.105 g	$= Fv * S_1$	ASCE Equation 11.4-2

Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	S_{DS}	1.236 g	$= 2/3*S_{MS}$	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	S_{D1}	0.737 g	$= 2/3*S_{M1}$	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	C_{RS}	0.955		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	C_{R1}	0.924		ASCE Figure 22-18
	T_L	8.00 sec		ASCE Figure 22-12
	T_O	0.12 sec	$=0.2*S_{D1}/S_{DS}$	
	T_S	0.60 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	PGA_M	0.84 g		ASCE Equation 11.8-1

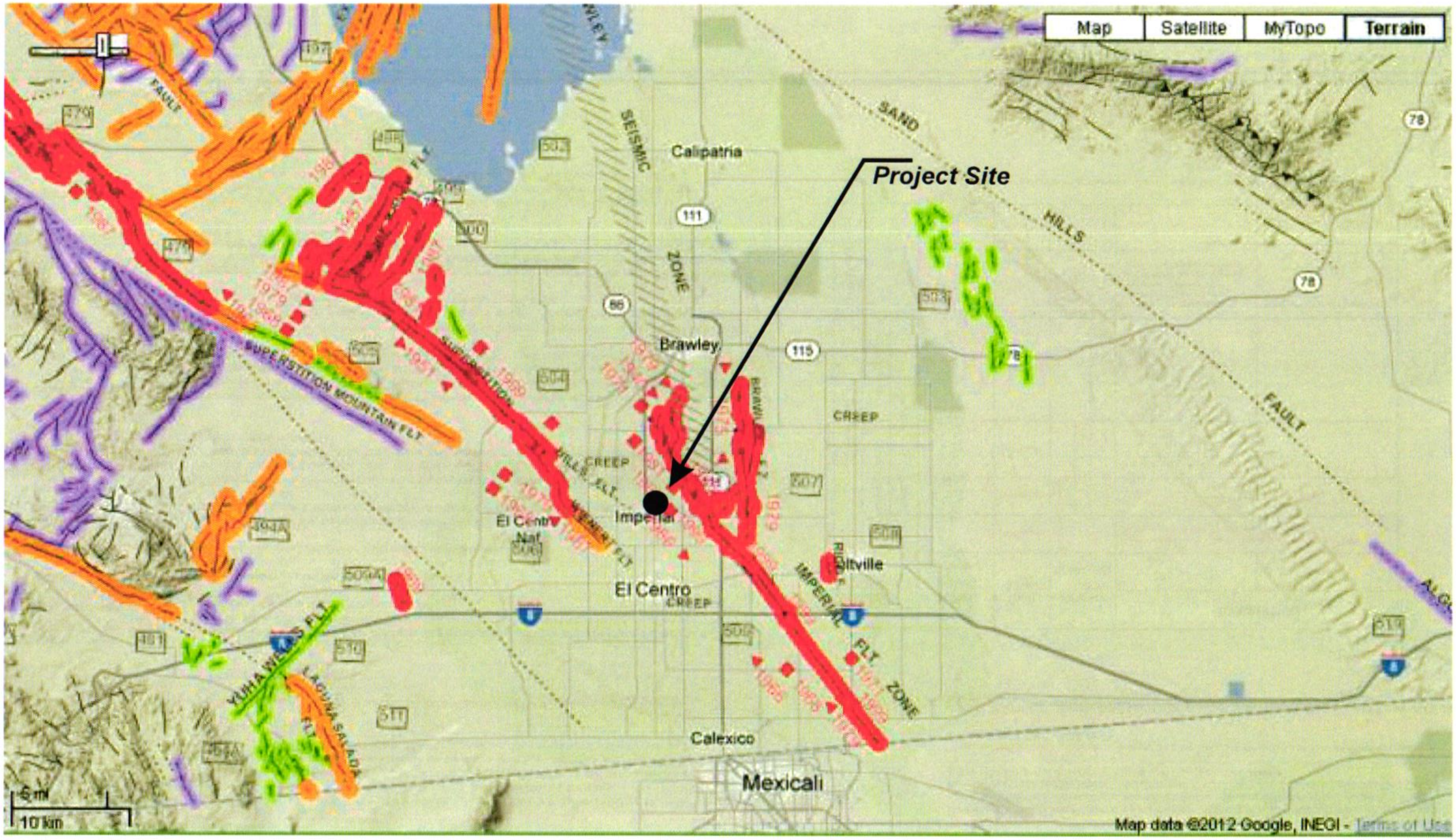


Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.49	0.74
0.12	1.24	1.85
0.60	1.24	1.85
0.70	1.05	1.58
0.80	0.92	1.38
0.90	0.82	1.23
1.00	0.74	1.11
1.10	0.67	1.00
1.20	0.61	0.92
1.30	0.57	0.85
1.40	0.53	0.79
1.50	0.49	0.74
1.75	0.42	0.63
2.00	0.37	0.55
2.20	0.33	0.50
2.40	0.31	0.46
2.60	0.28	0.43
2.80	0.26	0.39
3.00	0.25	0.37
4.00	0.18	0.28
5.00	0.15	0.22

FIGURES

Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

LANDMARK Geo-Engineers and Geologists Project No.: LE25072	Regional Fault Map	Figure 1



Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

LANDMARK
Geo-Engineers and Geologists
Project No.: LE25072

Map of Local Faults

Figure 2

EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along fault break.

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip.

OTHER SYMBOLS

Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geologist to delineate zones to encompass faults with Holocene displacement.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
Quaternary	Late Quaternary	Holocene	11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
Quaternary	Late Quaternary	Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Quaternary	Early Quaternary	Pleistocene	1,600,000*			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

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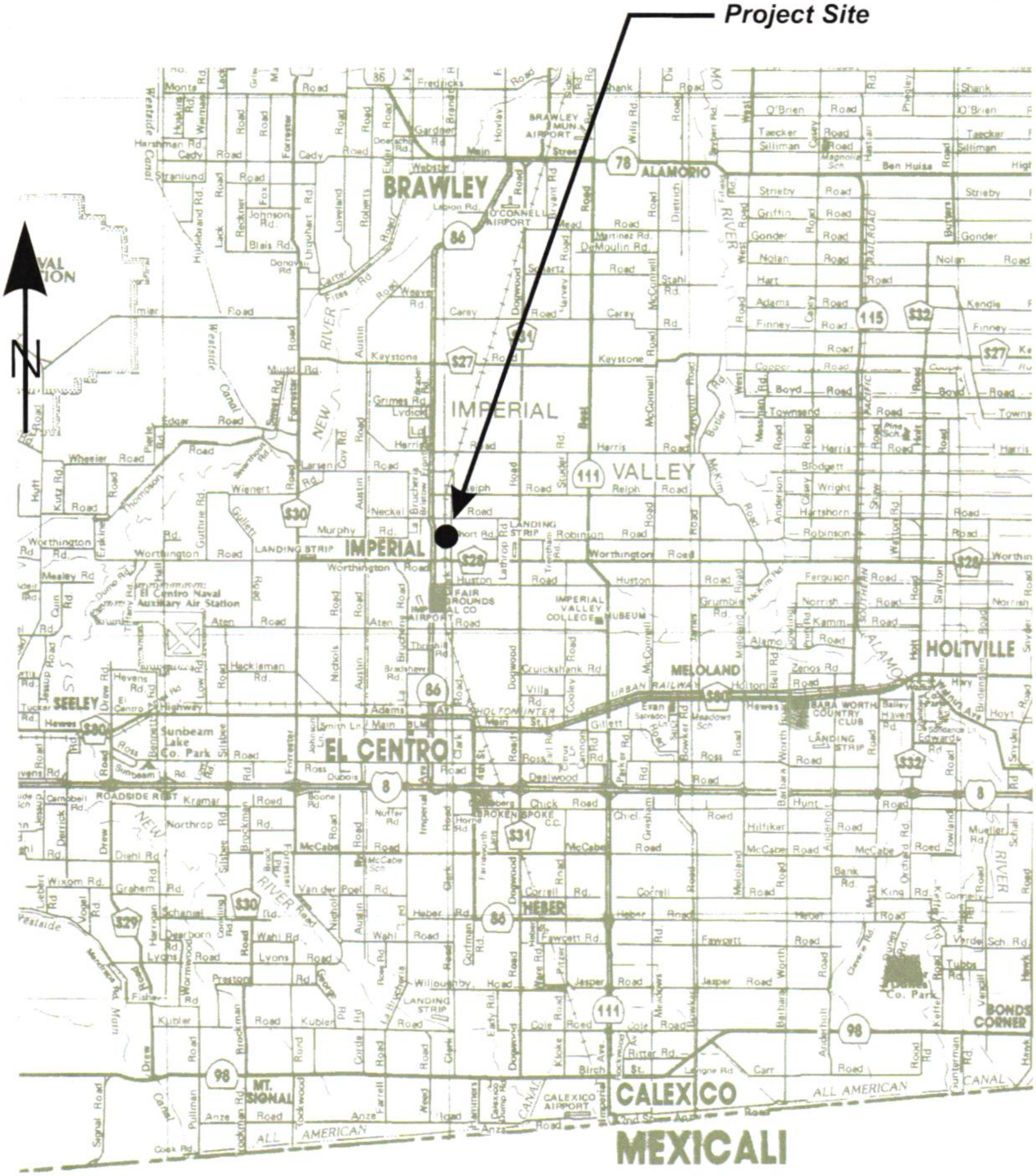
Fault Map Legend

Figure 3

APPENDIX A



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Vicinity Map

Plate A-1

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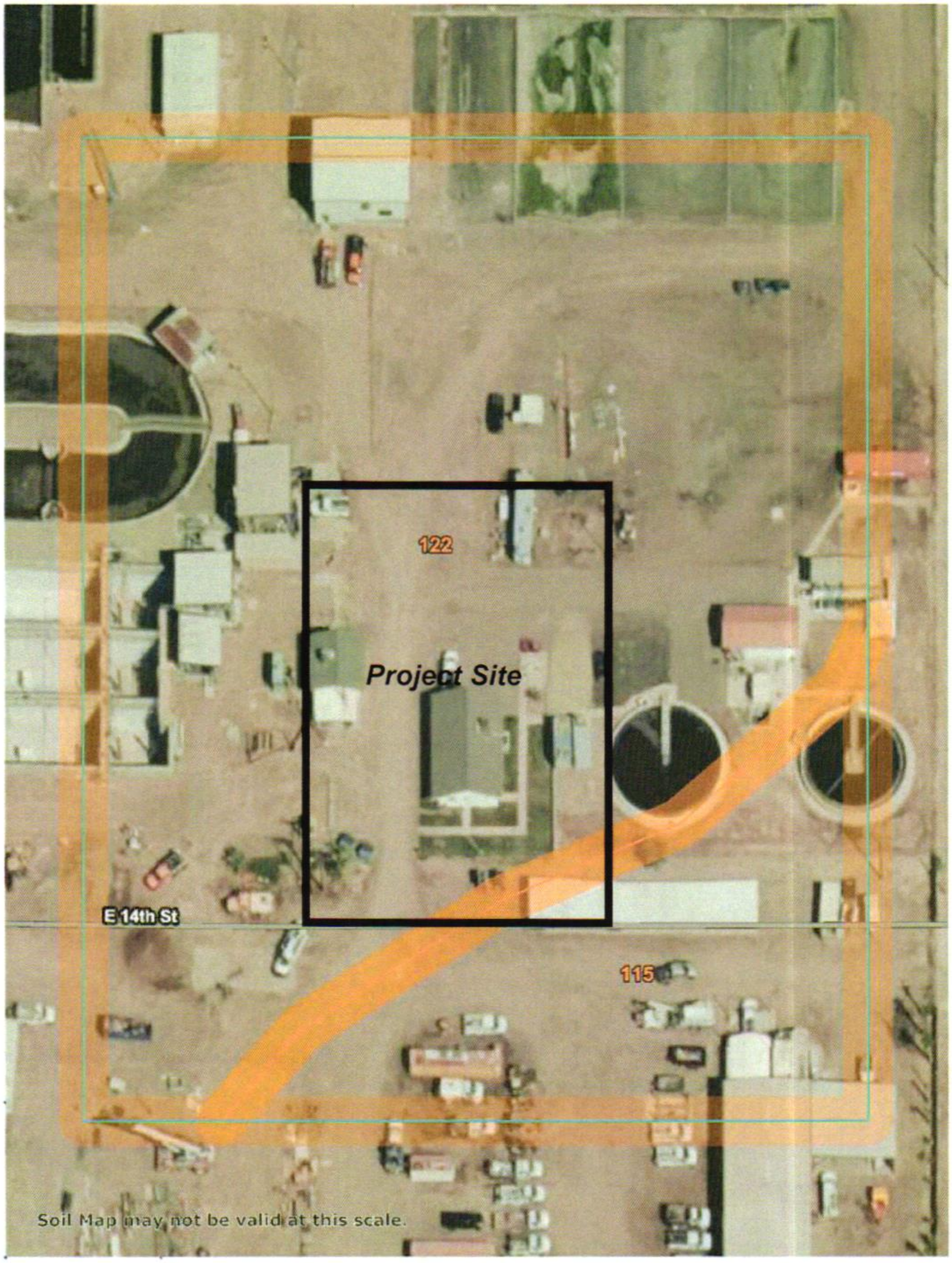
Site and Exploration Map

Plate
A-2



115° 33' 46" W

115° 33' 41" W

Map Scale: 1:884 if printed on A portrait (8.5" x 11") sheet.

Meters: 0, 10, 20, 40, 60

Feet: 0, 40, 80, 160, 240

Map projection: Web Mercator Corner coordinates: WGS84

N

USDA Natural Resources Conservation Service

Web Soil Survey
National Cooperative Soil Survey

4/28/2025
Page 1 of 3

N

LANDMARK Geo-Engineers and Geologists Project No.: LE25072	Soil Survey Map	Plate A-3

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Imperial County, California, Imperial Valley Area
Survey Area Data: Version 17, Sep 10, 2024

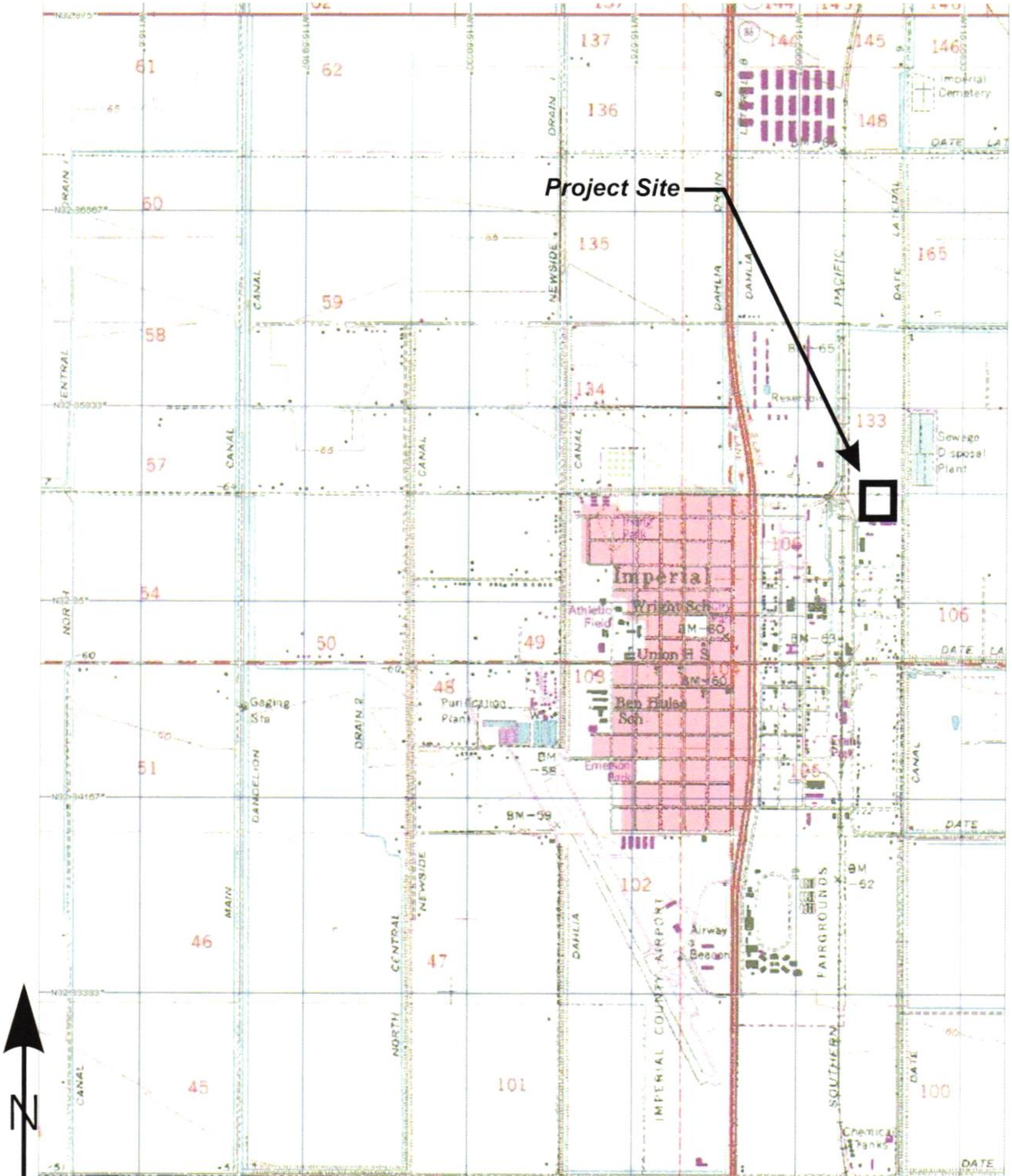
Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 17, 2021—May 22, 2021

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
115	Imperial-Glenbar silty clay loams complex, 0 to 2 percent slopes, wet	0.8	19.8%
122	Meloland very fine sandy loam, wet	3.2	80.2%
Totals for Area of Interest		**4.0**	**100.0%**



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Topographic Map

Plate A-4

APPENDIX B

LOG OF BORING No. B-1

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–2	●				Recycled AC Aggregate (12-in) FAT CLAY (CH): Dark brown, very moist, high plasticity.			
5			11		SANDY SILT/SILTY SAND (ML-SM): Brown, moist, medium dense, fine grained sand.			
10			8	2.0	SILTY CLAY (CL): Reddish brown, very moist, stiff, medium plasticity.			
15			4		SANDY SILT/SILTY SAND (ML-SM): Brown, saturated, loose, fine grained sand.			
20			8					
25			12	2.5	SILTY CLAY/CLAY (CL-CH): Dark brown, very moist, very stiff, medium to high plasticity.			
30			11	3.5				
35			5	0.5	CLAYEY SILT (ML): Brown, saturated, soft, low plasticity.			
40			10	2.0	SILTY CLAY/CLAY (CL-CH): Dark brown, very moist, stiff, medium to high plasticity.			

Total Depth = 41.5 ft.
Groundwater encountered at a depth of 13.0 ft. at time of drilling.
Groundwater level stabilized at 9 feet after 24 hours
Backfilled with excavated soil

DATE DRILLED: 4/8/25 TOTAL DEPTH: 41.5 feet DEPTH TO WATER: 9.0 ft.

LOGGED BY: A. Morales TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.

SURFACE ELEVATION: Approximately -65' HAMMER WT.: 140 lbs. DROP: 30 in.

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PLATE B-1

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS		SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger that No. 200 sieve	Gravels	Clean gravels (less than 5% fines)		GW	Well graded gravels, gravel-sand mixtures, little or no fines
	More than half of coarse fraction is larger than No. 4 sieve			GP	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines		GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
				GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands	Clean sands (less than 5% fines)		SW	Well graded sands, gravelly sands, little or no fines
	More than half of coarse fraction is smaller than No. 4 sieve			SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines		SM	Silty sands, sand-silt mixtures, non-plastic fines
				SC	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	Silts and clays			ML	Inorganic silts, clayey silts with slight plasticity
	Liquid limit is less than 50%			CL	Inorganic clays of low to medium plasticity, gravely, sandy, or lean clays
				OL	Organic silts and organic clays of low plasticity
	Silts and clays			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	Liquid limit is more than 50%			CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils				PT	Peat and other highly organic soils

GRAIN SIZES

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

200 40 10 4 3/4" 3" 12"

US Standard Series Sieve — Clear Square Openings

Sands, Gravels, etc.	Blows/ft. *
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

Clays & Plastic Silts	Strength **	Blows/ft. *
Very Soft	0-0.25	0-2
Soft	0.25-0.5	2-4
Firm	0.5-1.0	4-8
Stiff	1.0-2.0	8-16
Very Stiff	2.0-4.0	16-32
Hard	Over 4.0	Over 32

* Number of blows of 140 lb. hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 in. I.D.) split spoon (ASTM D1586).

** Unconfined compressive strength in tons/s.f. as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D1586), Pocket Penetrometer, Torvane, or visual observation.

Type of Samples:

Ring Sample — Standard Penetration Test — Shelby Tube — Bulk (Bag) Sample

Drilling Notes:

1. Sampling and Blow Counts
   - Ring Sampler - Number of blows per foot of a 140 lb. hammer falling 30 inches.
   - Standard Penetration Test - Number of blows per foot.
   - Shelby Tube - Three (3) inch nominal diameter tube hydraulically pushed.
2. P. P. = Pocket Penetrometer (tons/s.f.).
3. NR = No recovery.
4. GWT = Ground Water Table observed @ specified time.

LANDMARK Geo-Engineers and Geologists

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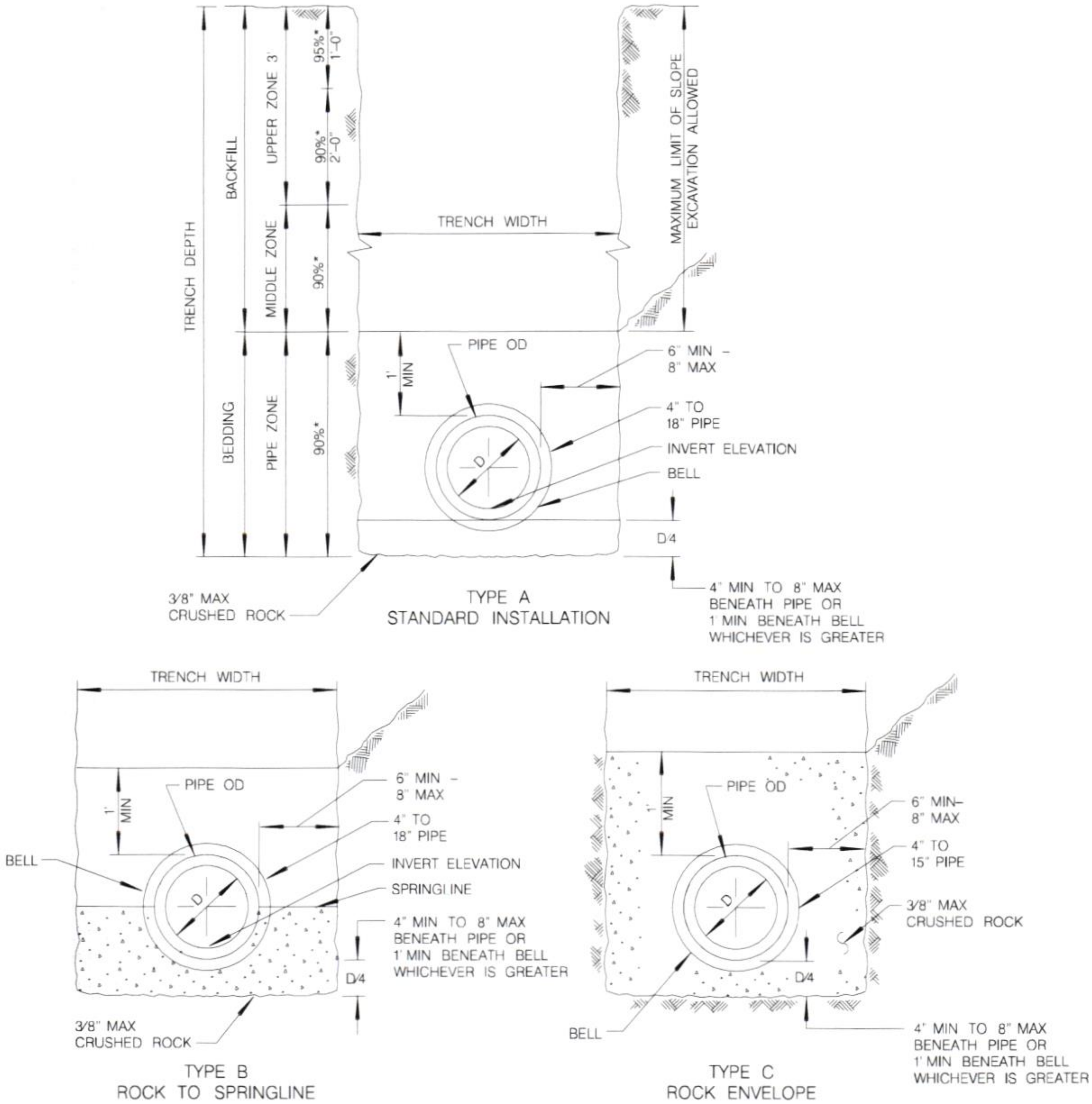
Key to Logs

Plate B-2

APPENDIX C



NOTES

1. FOR TRENCH RESURFACING IN IMPROVED STREETS, SEE STANDARD DRAWINGS SDG-107 AND SDG-108.
2. (*) INDICATES MINIMUM RELATIVE COMPACTION.
3. MINIMUM DEPTH OF COVER FROM THE TOP OF PIPE TO FINISH GRADE FOR PVC SDR 35 SEWER MAIN SHALL BE 5'. FOR SHALLOWER DEPTH, SPECIAL DESIGN IS REQUIRED. SEE SDS-101.
4. SEE TYPE A INSTALLATION FOR DETAILS NOT SHOWN FOR TYPES B AND C.
5. FOR PIPE SIZE ENCASEMENT LARGER THAN 15", MAXIMUM SIDE WALL CLEARANCE SHALL BE 12" OR AS SHOWN ON THE PLANS.
6. 6" METAL TAPE SHALL BE INSTALLED ABOVE PIPE 4" BELOW TRENCH CAP AND 12" BELOW FINISH GRADE IN UNIMPROVED STREETS.
7. 1' SAND CUSHION OR A 6" MINIMUM SAND CUSHION WITH 1" NEOPRENE PAD SHALL BE PLACED FOR CROSSINGS UTILITIES WHEN VERTICAL CLEARANCE IS 1' OR LESS. THE NEOPRENE PAD SHALL BE PLACED ON THE MOST FRAGILE UTILITY.

From: City of San Diego Standard Drawing SDS-110 (2016)

LANDMARK Geo-Engineers and Geologists **Project No.: LE25072**	**Pipe Bedding and Trench Backfill Recommendations**	**Plate C-1**