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MONTECITO SANITARY DISTRICT

STAFF REPORT – 3A

DATE: January 17, 2024

TO: A&O Committee

FROM: John Weigold, General Manager

SUBJECT: Discuss Seismic Risk Assessment and Evaluation

RECOMMENDATION

It is recommended that the Board:

i) Receive a presentation from Buehler (formerly SSG) regarding the findings from their Seismic Risk Assessment and Evaluation; and
ii) Taking such additional, related action that may be desirable.

SUMMARY

Per the Recycled Water Study by Carollo Engineers and more specifically per Technical Memo 5, the "Mini Master Plan", it was recommended the District perform seismic evaluations as well as petrographic testing on various infrastructure at our District's wastewater treatment plant (WWTP).

The District procured the services of Buehler to perform this seismic risk assessment and evaluation. The both in-ground concrete treatment process structures and above-ground structures of the WWTP were assessed through a comprehensive review of in-situ conditions, analysis of as-built drawings, concrete core sampling, and structural evaluation using ASCE 41 methodologies. The field work was executed from July through August of 2024. Concrete and lab testing was conducted September through November of 2024 with the draft Final Report being presented to Staff in December 2024.

Key findings of this report will be discussed as part of the presentation.

ATTACHMENTS:

Buehler Report – Montecito Sanitary Seismic Risk Evaluation

Seismic Risk Assessment and Evaluation

for

Montecito Sanitary District Facilities

1042 Monte Cristo Ln, Montecito, CA

ideas engineered | visions realized

Executive Summary

The structures of both in-ground liquid containing and above-ground structures of the Montecito Sanitary District Wastewater Treatment Plant have been assessed through a comprehensive review of in-situ conditions, analysis of as-built drawings, concrete core sampling, and structural evaluation using ASCE 41 methodologies. A site plan with locations of the respective structures can be found in Figure 16. The following summarizes the key findings and technical recommendations based on this evaluation:

<u>In-Ground Concrete Structures</u>

Fifty-seven total core samples were extracted from the Aeration Basins, Clarifiers, Chlorine Contact Chamber, Aerobic Digester, and the subterranean levels of the controls building and influent pump station. All samples revealed the presence of vertical crack planes in the concrete walls with the exception of the Aerobic Digester. These cracks are atypical for the expected structural behavior under standard operational conditions and raise concerns regarding the structural performance and long-term durability of these components. It is unusual and unexpected for the same issue to be present in structures built over twenty years apart.

Further petrographic analysis of the concrete cores confirmed the presence of Alkali-Silica Reaction (ASR). ASR occurs when reactive aggregates within the concrete mix undergo a chemical reaction between alkali in the cement and silica in the aggregate, leading to the formation of a gel that expands upon exposure to moisture. This expansion results in the development of cracks and compromises the concrete's structural integrity. The ongoing presence of moisture in the subsurface environment ensures that ASR will continue to progress, exacerbating the cracking and potentially leading to structural failure over time.

As no remediation techniques are capable of halting the progression of ASR once it has initiated, the recommended approach for the in-ground concrete structures is either complete replacement in a new location on the property or substantial rebuilding of the affected walls. A comparative analysis of complete replacement or rehabilitation is beyond the scope of this evaluation. It is understood that this study will be explored by the District in subsequent engineering efforts. A rehabilitation approach can be found in Figure 17 and Figure 18. If rehabilitation is pursued, the recommended scope of work includes the demolition of all non-exterior walls, while maintaining the existing tank slabs, which can be overlaid with a new concrete slab. A separation barrier should be installed between the new and existing concrete to mitigate potential issues related to ASR propagation. The final decision to implement replacement or rehabilitation should consider operation during construction, capital cost, life cycle cost, among other factors.

<u>Above-Ground Building Structures</u>

The structural evaluation of the Controls Building and Offices, Digester Blower Building, and steel canopies involved both visual condition assessments and an ASCE 41 evaluation process. Based on this assessment, it is concluded that all buildings can remain operational; however, certain mitigation measures are necessary for optimal performance and to meet seismic design standards. Below is a description of the rehabilitation recommendations.

Controls Building and Offices:

- Add a new lateral system for the North, South, and East exterior wall lines. See Figure 19.

- Implement out-of-plane roof tie connections to enhance structural stability.

Digester Blower Building:

- Install out-of-plane roof tie connections.

Blower Building:

- Add out-of-plane wall anchors and diaphragm cross ties.

South Canopy:

- Add a portal frame or braced frame in the longitudinal direction to improve stability.
- Repair corroded and damaged structural members.
- Connect the South Canopy to the West Canopy or adjust the seismic gap to meet required clearances.
- Expose and assess buried base plates and anchors for corrosion. Provide 3” concrete cover for protection.
- See Figure 20 for schematic rehabilitation.

North Canopy:

- Add a portal frame or braced frame in the longitudinal direction to improve stability.
- Repair any corroded and damaged structural components.
- Expose and assess buried base plates and anchors for corrosion. Provide 3” concrete cover for protection.
- See Figure 20 for schematic rehabilitation.

West Canopy:

- Connect to the South Canopy or adjust seismic gap.

In conclusion, the existing in ground concrete structures require replacement or extensive remediation which will be evaluated by the District in subsequent engineering evaluations. The existing above ground buildings can continue operating, but structural retrofits are recommended to address identified vulnerabilities and extend the service life of the plant’s facilities. These improvements will also ensure compliance with modern seismic and safety standards.

Contents

1. Project Scope and Summary

Montecito Sanitary District Wastewater Treatment Plant is located off Highway 101 along the coast at 1042 Monte Cristo Ln, Montecito, CA 93108. While the District was established in 1947, the facilities began construction in 1961 with much of the plant infrastructure completed after the 1982 plant expansion.

The objective of this evaluation is to provide the District with an overall assessment of the existing structures and determine a general future service life along with determining a level of seismic risk. The evaluation of the structures is categorized into three primary groups:

1. In-ground concrete structures (Aeration Basins, Clarifiers, Chlorine Contact Tank, Aerobic Digester, and subterranean pump room and wet and dry well)
2. Office and maintenance buildings and roof covers
3. Equipment and distribution systems

The structures are reviewed considering two factors to service life, technical and economical. If an unacceptable state in respect to technical service life is encountered, then economic service life is not considered.

- Technical Service Life: Defined as the duration of service until an unacceptable state is reached, which includes structural safety concerns or excessive material degradation

- Economic Service Life: Refers to the period until replacement becomes more economically viable than ongoing maintenance costs.

An additional service life factor, outside the scope of this review, that should be considered by the District is functional service life, which addresses the potential obsolescence of current systems due to new processes or technologies.

Evaluation methodology of the structures involved a multi-faceted approach:

1. In-ground Concrete Structures:
    - Desktop review of as-builts
    - ACI 350 Code Requirements for Environmental Concrete Structures analysis to assess structural performance
    - Visual observation of the existing structure
    - Core sampling of in-situ concrete slabs and walls, conducted by Earth Systems Pacific in collaboration with concrete demolition sub-contractors
    - Ground Penetrating Radar (GPR) surveys to determine locations of existing reinforcement
    - Testing of concrete cores for compressive strength, chloride ion content, and petrographic examination to assess potential alkali-silica reactivity.
2. Buildings:
    - Visual observation of the existing structure
    - Desktop review of the as-built drawings
    - ASCE 41 Seismic Evaluation and Retrofit of Existing Buildings Tier 1 analysis to assess structural performance in relation to seismic risk.

- ASCE 41 Tier 2 analysis to assess structural deficiencies found in the Tier 1 evaluation. See Figures 1 and 2 for ASCE 41 evaluation process.
- Based on the plants Risk Category of III, the Basic Performance Objective for the buildings is Limited Safety has been assigned for Tier 1 and 2 analyses.

3. Equipment and Distribution Systems:
   - Visual observation of the existing conditions.
   - Desktop review of as-built drawings
   - ASCE 41 Tier 1 analysis

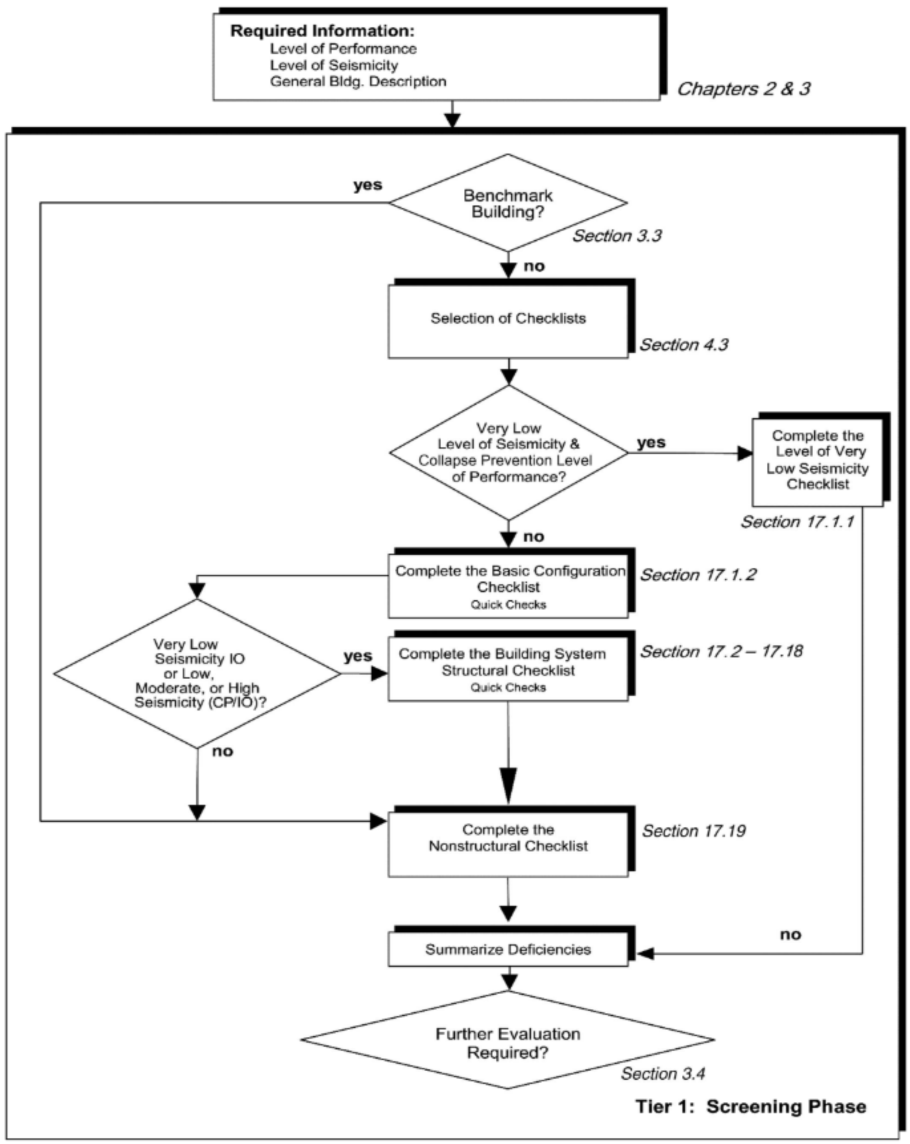

Figure 1

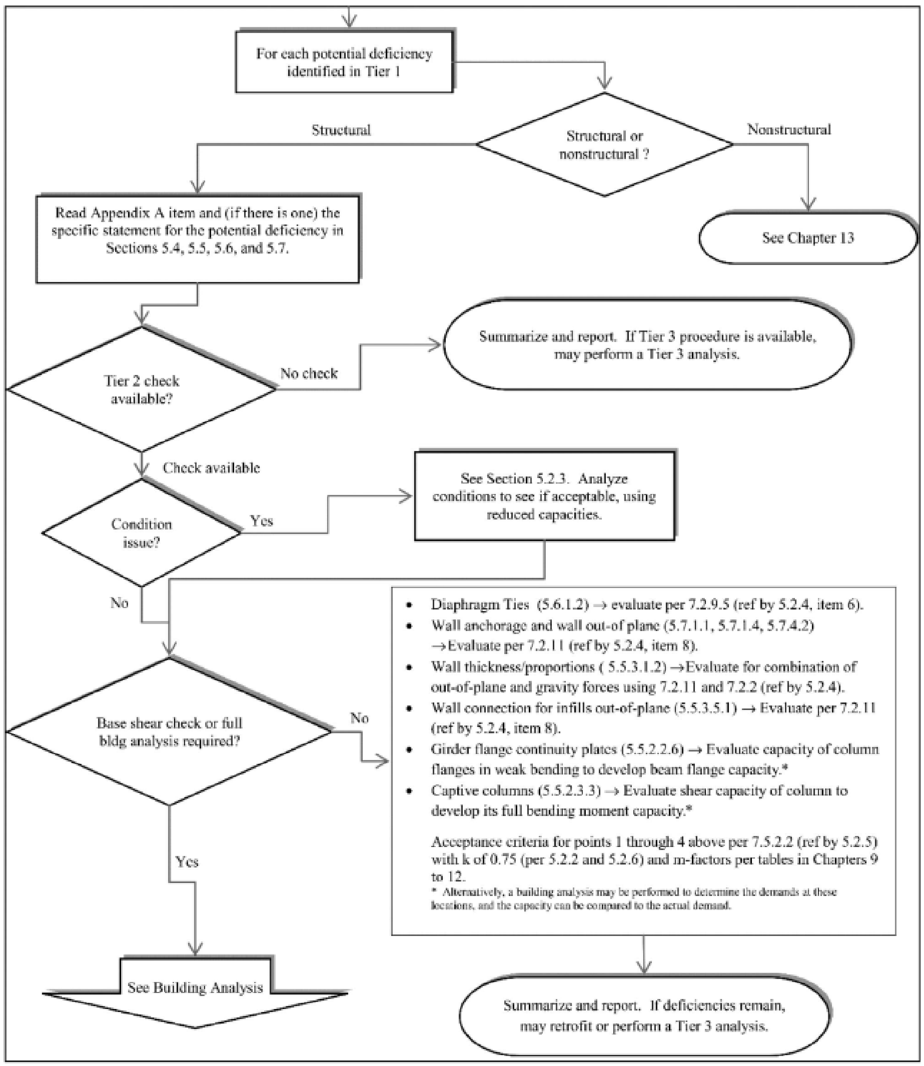
For each potential deficiency identified in Tier 1
Structural or nonstructural ?
Structural
Nonstructural
See Chapter 13
Read Appendix A item and (if there is one) the specific statement for the potential deficiency in Sections 5.4, 5.5, 5.6, and 5.7.
Tier 2 check available?
No check
Summarize and report. If Tier 3 procedure is available, may perform a Tier 3 analysis.
Check available
Condition issue?
Yes
See Section 5.2.3. Analyze conditions to see if acceptable, using reduced capacities.
No
Base shear check or full bldg analysis required?
No
Yes
• Diaphragm Ties (5.6.1.2) → evaluate per 7.2.9.5 (ref by 5.2.4, item 6).
• Wall anchorage and wall out-of plane (5.7.1.1, 5.7.1.4, 5.7.4.2) →Evaluate per 7.2.11 (ref by 5.2.4, item 8).
• Wall thickness/proportions (5.5.3.1.2) →Evaluate for combination of out-of-plane and gravity forces using 7.2.11 and 7.2.2 (ref by 5.2.4).
• Wall connection for infills out-of-plane (5.5.3.5.1) → Evaluate per 7.2.11 (ref by 5.2.4, item 8).
• Girder flange continuity plates (5.5.2.2.6) → Evaluate capacity of column flanges in weak bending to develop beam flange capacity.*
• Captive columns (5.5.2.3.3) → Evaluate shear capacity of column to develop its full bending moment capacity.*
Acceptance criteria for points 1 through 4 above per 7.5.2.2 (ref by 5.2.5) with k of 0.75 (per 5.2.2 and 5.2.6) and m-factors per tables in Chapters 9 to 12.
* Alternatively, a building analysis may be performed to determine the demands at these locations, and the capacity can be compared to the actual demand.
See Building Analysis
Summarize and report. If deficiencies remain, may retrofit or perform a Tier 3 analysis.

Figure 2

1.1. Engineering Team

The Engineering & Analysis Design Team collaborating to prepare this Risk Assessment includes the following Firms and Individuals:

Buehler Engineering	Structural Engineering	
Michael Parolini, S.E.	Principal	mparolini@buehlerengineering.com
Joseph Klimczyk, S.E.	Senior Associate Engineer	jklimczyk@buehlerengineering.com
Cheyne Kight, S.E.	Senior Associate Engineer	ckight@buehlerengineering.com
Reis Consulting	Risk Consultant	
Evan Reis, S.E.	Structural Risk Consultant	evanreis1@gmail.com
Earth Systems	Geotechnical Engineering & Materials Testing & Geology	
Fred Potthast, G.E.	Principal Engineer	fpotthast@earthsystems.com
Rob Down, P.E.	Managing Principal	rdown@earthsystems.com
Darrin Hasham, C.E.G.	Associate Geologist	dhasham@earthsystems.com
Samuel Venable, P.E.	Staff Engineer	svenable@earthsystems.com

1.2. Definitions

ASCE: American Society of Civil Engineers, (asce.org

ASR: Alkali Silica Reaction, a chemical reaction that occurs in concrete with alkaline cement paste reacts with reactive silica in aggregates.

GPR: Ground penetrating radar, a type of radar that uses pulses of radio waves of a frequency suitable for investigating solid materials and underground features. Used in this case to locate reinforcing bar locations and orientations in solid concrete materials

Seismic Design Category (SDC): a classification assigned to a structure based on its Risk Category and severity of the design earthquake ground motion at the site, as defined in the ASCE 7 ranging from A to F (A equates to minimal seismic design requirements, while F is the most stringent)

Structural Irregularities/Structural Deficiencies: Structural characteristics that negatively affect the behavior or strength of the structure under static (i.e. gravity) and dynamic (i.e. seismic/wind) loading.

1.3. Reference Standards

ASCE 7 – Minimum Design Loads and Associated Criteria for Buildings and Other Structures

ASCE 41 – Seismic Evaluation and Retrofit of Existing Buildings

ACI 318 – Building Code Requirements for Environmental Concrete Structures

ACI 350 – Code Requirements for Environmental Concrete Structures

ACI 350.3 – Seismic Design of Liquid-Containing Concrete Structures

AISC 360 – Specification for Structural Steel Buildings

AISC 341 – Seismic Provisions for Structural Steel Buildings

TMS 402 – Building Code Requirements and Specification for Masonry Structures

1.4. Seismicity and Geologic Hazard Risk

As part of the project scope, Earth System Pacific performed a Geotechnical Engineering and Geologic Hazard analysis. The findings of this analysis can be found in the report in Appendix A for review.

The site is situated in a high seismic area that has potential for strong ground shaking due to potential fault movements. For a new structure designed on the site, a Seismic Design Category of E would be applied. This designation limits acceptable lateral systems and requires more rigorous connections to ensure ductile performance from the structures in a seismic event.

Table 2: Summary of Site-Specific Seismic Design Parameters – 2022 CBC (ASCE 7-16 21.2.1.1 – Method 1)

Seismic Site Class (ASCE 7-16)	D
Occupancy (Risk) Category	III
Seismic Design Category	E
Mapped and Code Based Ground Motion Values	
Spectral Response Acceleration, Short Period – S_s	2.161 g
Spectral Response Acceleration at 1 sec. – S_1	0.791 g
Site Specific Site Coefficient – F_a	1.10
Site Specific Site Coefficient – F_v	2.5
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	2.168 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	2.200 g
Design Earthquake Ground Motion Values	
Short Period Spectral Response – S_{DS}	1.445 g
One Second Spectral Response – S_{D1}	1.467 g
Site Modified Peak Ground Acceleration - PGA_M	0.891 g
Site Specific T_S (S_{D1}/S_{DS}) (seconds)	1.015 s

Figure 3

In addition to seismicity, the geohazard testing evaluated several additional potential hazards, including surface ground rupture, liquefaction and seismically induced settlement, slope stability, flooding, and tsunami-induced seiches. The analysis indicates a low likelihood of these hazards occurring, thereby supporting the recommendation for future development of the site.

2. In-Ground Concrete Structures

Over the course of three days in August, 2024, concrete cores were taken from the walls and slabs of Aeration Basin 1 and 2, Clarifiers 2 and 4, Chlorine Contact Chamber 1, Aerobic Digester, and the Dry Well. Safety supervision, fall protection, and air monitoring were provided by the Montecito Sanitary District Staff. Coring was performed under the supervision of Earth Systems Pacific staff. Prior to coring, a GPR survey was performed by Earth Systems to reduce the potential for coring through existing reinforcement. Following coring operations, all cores were filled with a high strength non-shrink cementitious grout.

2.1. Aeration Basins

Description

Aeration Basins 1 and 2 were part of the original plant construction in 1961. They are constructed of 12-inch-thick reinforced concrete walls and a slab-on-grade that varies in thickness from 8-inches at the midspan to 12-inches at the walls. Combined the basins measures 128.5-feet in length and 63.5-feet in width with walls that are 16.25-feet tall. The basins are buried except for the upper 18-24 inches. Spanning across the basins are three 12-inch wide by 13.5-inch-deep concrete beams which brace the walls.

The vertical reinforcement, which varies in the structure, of the walls meets the minimum reinforcement ratio of 0.0030, as recommended by the ACI 350 (Code Requirements for Environmental Engineering Concrete Structures). Reinforcement ratio is defined as the area of steel to the area of the concrete structural member. The typical horizontal reinforcing, #4 at 18-inches on center, has a reinforcing ratio of 0.0018 which is less than one-third the recommended ratio of 0.0060. The reinforcement in the floor slab, #5 at 18-inches on center results in a reinforcing ratio of 0.002 which is less than the ACI 350 recommendation of 0.005. These ratios are based on minimums for temperature, shrinkage, and water tightness and are not minimums for strength design.

The as-built drawings are provided in Appendix B for reference. The as-builts indicate a minimum concrete strength of 3,000-psi, but do not include any additional requirements for water-cement ratio, aggregate size limitations, or permeability reduction. These factors along with the low reinforcing ratios can lead to additional cracking when compared to mix designs meeting requirements of current ACI 350 and ACI318 standards.

See Appendix D for photos of the Aeration Basins.

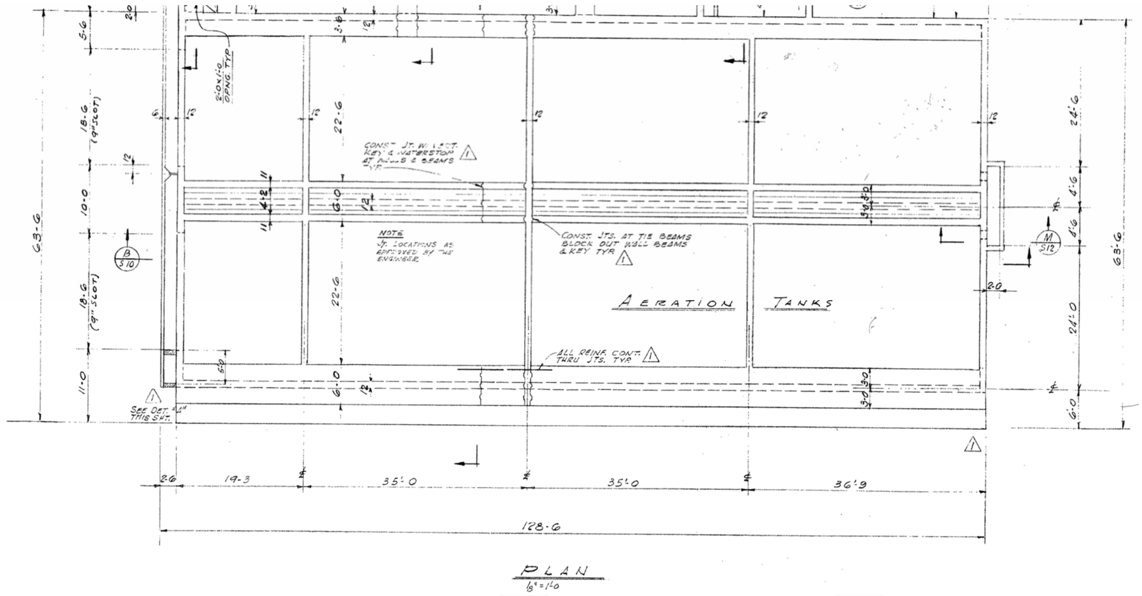

Figure 4

Wall and Slab Reinforcing Ratios

Aeration Basins				
Location	Reinforcing & Wall Thickness	Vertical Ratio (V), Horizontal Ratio (H)	ACI 350 Recommended Ratio	Percentage of Recommended
Aeration Basin 2				
North Wall	#7 at 12" V EF, #4 at 18" H EF, 12" thick wall	0.008V, 0.0018H	0.003V, 0.006H	278%V, 31%H
South Wall	#7 at 12" V EF, #4 at 18" H EF, 12" thick wall	0.008V, 0.0018H	0.003V, 0.006H	278%V, 31%H
West Wall	#6 at 9" EF V, #4 at 12"H EF, 12" thick wall	0.008V, 0.0028H	0.003V, 0.006H	272%V, 46%H
East Wall	#5 at 9" EF V, #6 at 12"H EF, 12" thick wall	0.0057V, 0.0061H	0.003V, 0.006H	191%V, 102%H
Slab	#5 at 18", 8" thick	0.0022	0.005	43%
Aeration Basin 1				
South Wall	See Aeration Basin 2 North wall.			
North Wall	#5 at 11" V EF, #4 at 18" H EF, 12" thick wall	0.0047V, 0.0018H	0.003V, 0.006H	157%V, 31%H
West Wall	#6 at 9" EF V, #4 at 12"H EF, 12" thick wall	0.008V, 0.0028H	0.003V, 0.006H	272%V, 46%H

East Wall	#5 at 9” EF V, #6 at 12”H EF, 12” thick wall	0.0057V, 0.0061H	0.003V, 0.006H	191%V, 102%H
Slab	#5 at 18”, 8” thick	0.0022	0.005	43%

2.1.1. Aeration Basin 1

Figure 5

Observation/Condition Assessment

Aeration Basin 1 was observed on August 6, 2024. Cores were taken in the North, East, and West walls and in three locations in the slab. The locations were selected to ensure protection of Basin 2 that remained operational during coring.

Cracking on the surface of the walls is prevalent. The crack pattern is generally random with a combination of vertical, horizontal, and diagonal cracks. Cracks in the walls range from hairline to 0.030-inches. There is no significant offset across the cracks.

North Wall: The north wall has diagonal cracks starting from the bottom corners indicating possible shrinkage cracks due to restraint at the foundation. The diagonal face of the haunch at the top of the walls has a consistent horizontal crack

South Wall: There are frequent cracks and no evidence of moisture seeping through cracks from Aeration Basin 2 which was in operation at the time of observation.

East & West Walls: Numerous cracks across surface. Not significant in width at the surface, but very frequent with an irregular pattern. There is heavy corrosion in the steel plate at the weir at the west wall.

Concrete Beams: Across the top of the basin are three concrete beams spanning from wall to wall. Each of the beams has significant cracks which can be observed in photos 08, 09, and 10 of Appendix D.

Slab: Generally, the slab in good condition. No major cracks visible at the surface, however some dry tank contents were present along the floor during observation which limited visibility.

Walkways: The surface of the walkways and tops of the walls has frequent cracking. Cracks range from hairline to 0.10-inches.

The most significant observation was made during coring. All cores indicated there is a vertical crack plane in the walls that is 4 to 5-inches from the wall surface. This is discussed further in the Summary for the In-Ground Concrete Structures.

Figure 6

Testing Result Summary

Concrete Compression Strength: 4,730 psi – 6,660 psi

Chloride Ion Content: Not Elevated, Unlikely to have detrimental effects.

Petrographic Examination: Alkali-silica reaction present

2.1.2. Aeration Basin 2

Observation/Condition Assessment

Aeration Basin 2 was observed on August 28, 2024. Cores were taken in the South, East, and West walls and in three locations in the slab. The divider wall between Aeration Basin 1 and 2 was not sampled. The locations were selected to ensure protection of Basin 1 that remained operational during coring.

Cracking on the surface of the walls is prevalent. The crack pattern is generally random with a combination of vertical, horizontal, and diagonal cracks. Cracks in the walls range from hairline to 0.030-inches.

North Wall: The north wall has exposed aggregate at the face of the wall at the east end. At a distance of 3 to 6-feet above the top of slab there were multiple vertical cracks with apparent moisture in the crack. It was inconclusive if this moisture was coming from the operating Aeration Basin 1.

Figure 7

South Wall: Similar to the north wall of Aeration Basin 1, the south wall of Basin 2 has diagonal cracks starting from the bottom corners indicating possible shrinkage cracks due to restraint at the foundation.

East & West Walls: Numerous cracks across surface. Not significant in width at the surface, but very frequent with an irregular pattern. There is heavy corrosion in the steel plate at the weir at the west wall.

Concrete Beams: Across the top of the basin are three concrete beams spanning from wall to wall. The western most beam has an exposed piece of reinforcing at the top of the beam. All beams have cracks along the exposed faces.

Slab: Generally, the slab is in good condition. Hairline cracks are present, but no major cracks visible at the surface, however some dry tank contents were present along the floor during observation which may have concealed additional cracks.

Walkways: The surface of the walkways and tops of the walls indicates significant cracking.

Similar to Aeration Basin 1, the vertical crack plane is present in the sampled concrete walls.

Testing Result Summary

Concrete Compression Strength: 4,330 psi – 6,870 psi

Chloride Ion Content: Not Elevated, Unlikely to have detrimental effects.

Petrographic Examination: Alkali-silica reaction present

2.2. Clarifiers

Description

Clarifiers 1 and 2 were part of the original plant construction designed in 1961. The structures are 83.5-feet long and a combined 26.5-feet wide. The exterior walls vary in thickness from 12-inches to 20-inches and vary from 12-feet to 13-feet in height. The concrete specification matches the 3,000-psi specification from the Aeration Basins. Based on the time of construction, it is anticipated that the concrete composition is very similar to the Aeration Basins.

Clarifiers 3 and 4 were part of the 1982 plan expansion and were installed north of Clarifiers 1 and 2. The new slab was doweled to the existing clarifier structure at the predesigned expansion slab detail on D/S-11 of the 1961 as-builts. The concrete for Clarifiers 3 and 4 is specified as 4,000-psi with aggregate that shall be non-reactive. The reinforcement is specified as ASTM A615 Grade 60.

As part of the expansion, the interior surfaces of the concrete of all four clarifiers were coated with a waterproofing product. The coating has failed and cracking, bubbling, and flaking of coating is prevalent throughout the clarifiers.

Wall and Slab Reinforcing Ratios

Clarifiers				
Location	Reinforcing & Wall Thickness	Vertical Ratio (V), Horizontal Ratio (H)	ACI 350 Recommended Ratio	Percentage of Recommended
Clarifier 1				
North Wall	#5 at 9” V EF, #4 at 18” H EF, 10” thick wall	0.006V, 0.0022H	0.003V, 0.006H	230%V, 37%H
South Wall	See Aeration Basin 1 North Wall			

East/West Wall	#5 at 15” EF V, #4 at 18”H EF, 12” thick wall	0.0034V, 0.0018H	0.003V, 0.006H	115%V, 31%H
Clarifier 2				
North Wall	#5 at 18” V EF, #4 at 18” H EF, 20” thick wall	0.0017V, 0.0011H	0.003V, 0.006H	57%V, 19%H
South Wall	See Clarifier 1 North Wall			
East/West Wall	#5 at 15” V EF, #4 at 18”H EF, 12” thick wall	0.0034V, 0.0018H	0.003V, 0.006H	115%V, 31%H
Slab	#5 at 18” T&B,10” thick	0.0034	0.005	69%
Clarifier 3				
North Wall	#5 at 12” V EF, #5 at 12” H EF, 12” thick wall	0.0043V, 0.0043H	0.003V, 0.006H	144%V, 72%H
South Wall	See Clarifier 2 North Wall			
East/West Wall	#5 at 12” V EF, #5 at 12”H EF, 12” thick wall	0.0043V, 0.0043H	0.003V, 0.006H	144%V, 72%H
Clarifier 4				
North Wall	#5 at 12” V EF, #5 at 12” H EF, 12” thick wall	0.0043V, 0.0043H	0.003V, 0.006H	144%V, 72%H
South Wall	See Clarifier 3 North Wall			
East/West Wall	#5 at 12” V EF, #4 at 18”H EF, 12” thick wall	0.0043V, 0.0043H	0.003V, 0.006H	144%V, 31%H
Slab	#5 at 12” T&B, 12” Thick	0.0043	0.005	86%

See Appendix B for as-built structural drawings and Appendix E for photos of the Clarifiers.

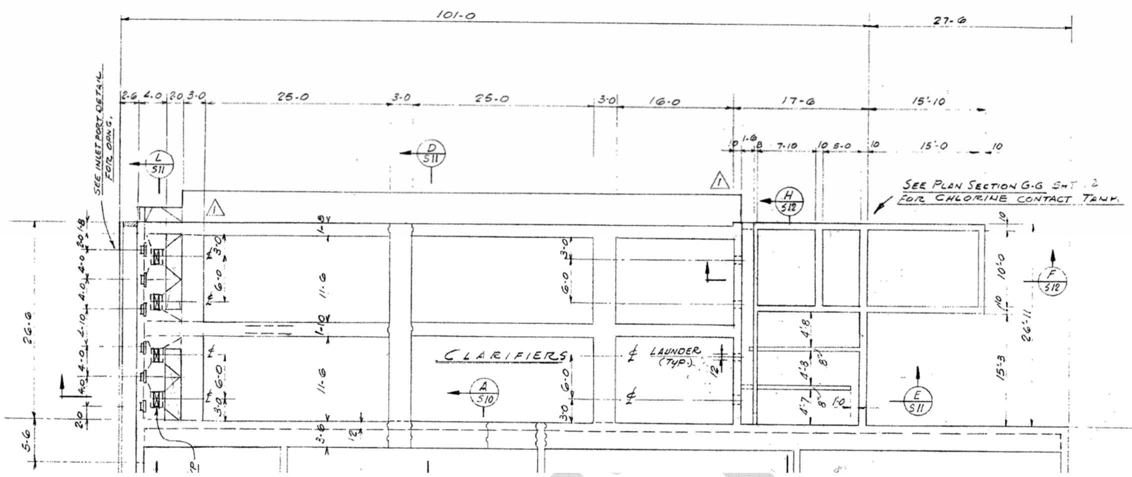

Figure 8

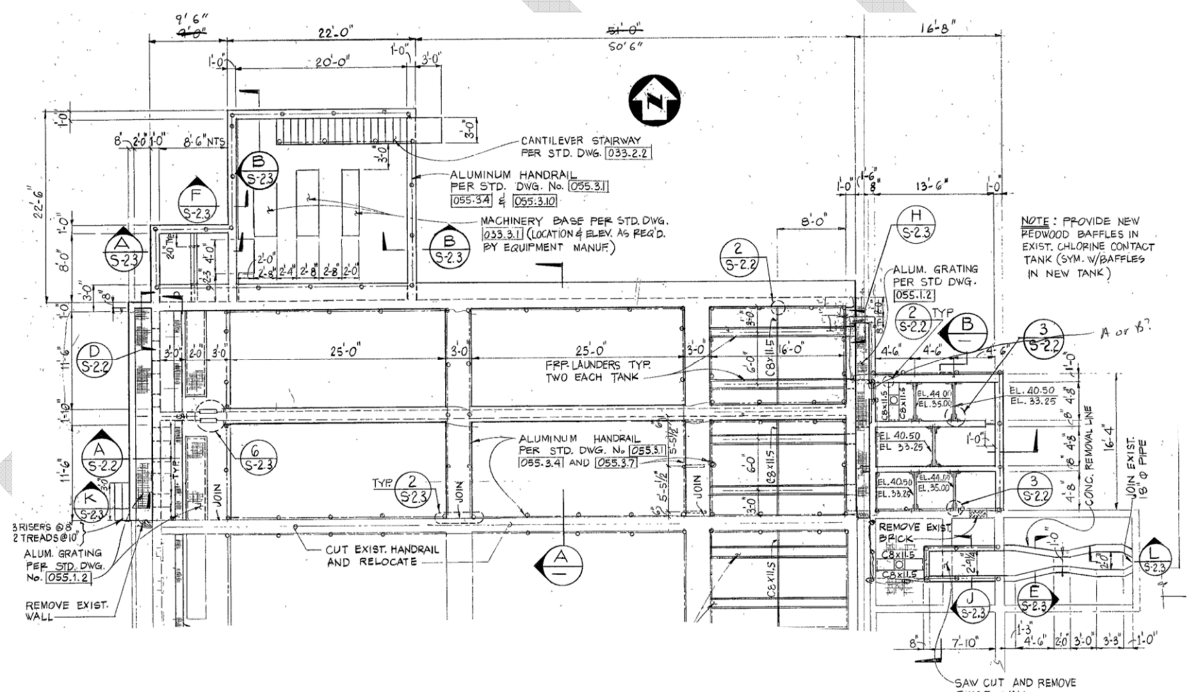

Figure 9

2.2.1. Clarifier 2

Figure 10

Observation/Condition Assessment

Clarifier 2 was observed on August 28, 2024. Cores were taken at the top of the south divider wall between Clarifiers 1 and 2, in the face of the north divider wall between Clarifier 2 and 3, and at the slab.

North Wall: Visual observation indicated cracks in the north wall are frequent and in many different directions. The largest cracks, 0.025-inches, are horizontal at around 4 to 6-feet above the slab level.

South Wall: Some small cracks visible, but coating is concealing most of the wall. If frequent cracking on the surface is present, it's anticipated they are small as the crack has not propagated across the coating.

East Wall: Cracking in the east wall is frequent. There are some large horizontal cracks exceeding 0.03-inches on the face of the wall. There was no apparent liquid transmission through the crack from the operating chlorine contact tank on the opposite side of the wall.

West Wall: Due to the depth of the channel at the west side of the clarifier the wall was only observable from 6-feet away. No major cracking is visible.

Slab: The slab of clarifier 2 has extensive cracks. They vary from hairline to 0.02-inches. The coating on the slab is no longer present as the sludge collector mechanism has rubbed it away.

Walkways: There is extensive cracking in the walkways around the clarifiers.

Cracking internal to the concrete walls was found in the concrete cores sampled.

Testing Result Summary

Concrete Compression Strength: 2,880 psi – 4,670 psi

Chloride Ion Content: Not Elevated, Unlikely to have detrimental effects.

Petrographic Examination: Alkali-silica reaction present

2.2.2. Clarifier 4

Figure 11

Observation/Condition Assessment

Clarifier 4 was observed on August 7, 2024. Cores were taken in the north wall and in the slab. The failed coating on the interior of the walls limited the observation of the concrete surface.

Walls: Some hairline cracks are present in the walls. Cracks have not propagated through the coating as it did in Clarifier 2.

Slab: Generally, the slab is in good condition. There are some small hairline cracks, but no major cracks are present.

Walkways: North of the Clarifier 2 the walkways have some cracking, but it is not as extensive as the original clarifier construction.

The vertical crack plane was found in the concrete cores sampled.

Testing Result Summary

Concrete Compression Strength: 3,320 psi – 6,090 psi

Chloride Ion Content: Not Elevated, Unlikely to have detrimental effects

Petrographic Examination: Alkali-silica reaction present

2.3. Chlorine Contact Chambers

Description

Chlorine Contact Chamber 1 was part of the original plant construction designed in 1961. The concrete specification matches the 3,000-psi specification from the Aeration Basins. Based on the time of construction, it is anticipated that the concrete composition is very similar to the Basins.

Chorine Contact Chamber 2 was added with Clarifiers 3 and 4 during the 1982 expansion. The concrete is specified as 4,000-psi with non-reactive aggregate.

The structures are 32.5-feet long and a combined 44.5-feet wide. The exterior walls vary in thickness from 10-inches to 12-inches and are 12-feet in height. The slab is 10-inches at Chamber 1 and 12-inches at Chamber 2. The interior walls are 8-inches thick.

See Appendix F for Chlorine Contact Tank photos.

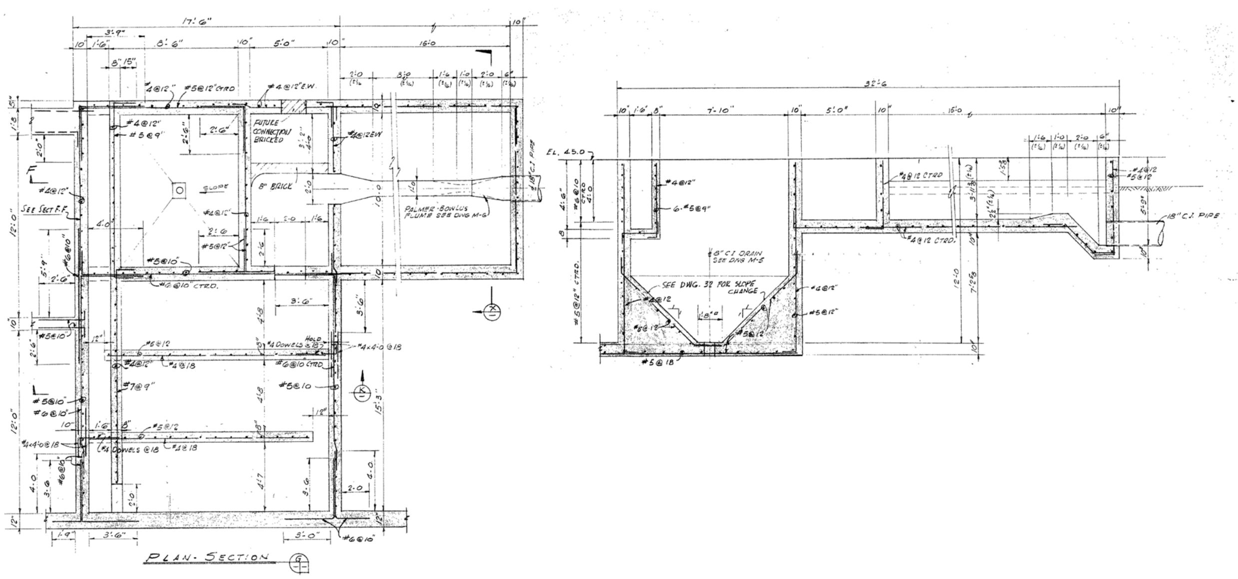

Figure 12

Wall and Slab Reinforcing Ratios

Chlorine Contact Tanks				
Location	Reinforcing & Wall Thickness	Vertical Ratio (V), Horizontal Ratio (H)	ACI 350 Recommended Ratio	Percentage of Recommended
Tank 1				
Exterior Walls	#5 at 10” V, #6 at 10” H, 8” thick wall	0.0039V, 0.0055H	0.003V, 0.006H	129%, 92%
Slab	#5 at 18” EF, 10” thick	0.0034	0.005	34%
Tank 2				
Exterior Walls	#5 at 12” EF V, #5 at 12” H EF, 12” thick wall	0.0043V, 0.0043H	0.003V, 0.006H	144%, 72%
Slab	#5 at 12” T&B, 12” thick	0.0043	0.005	86%

2.3.1. Chlorine Contact Tank 1

Figure 13

Observation/Condition Assessment

Chlorine Contact Tank 1 was observed on August 30, 2024. Cores were taken in interior divider wall only.

Walls: Cracking in the walls is very frequent. Large cracks 0.03-inches to 0.05-inches were observed inside the basin.

Slab: Due to limited time for testing, the slab was not dry and had standing water that was 2 to 3-inches deep. No major cracks were observed in the slab.

Testing Result Summary

Concrete Compression Strength: 4,000 psi – 5,420 psi

Chloride Ion Content: Not Elevated, Unlikely to have detrimental effects.

Petrographic Examination: Alkali-silica reaction present

2.4. Aerobic Digester

Description

The Aerobic Digester was built as part of the 1976 plan improvements and was modified in 1992. The structure is 20-feet wide by 53-feet long. The tank is 18-feet tall with 11.75 feet buried and 7.25-feet above finished grade.

Aerobic Digester Tank				
Location	Reinforcing & Wall Thickness	Vertical Ratio (V), Horizontal Ratio (H)	ACI 350 Recommended Ratio	Percentage of Recommended
Exterior Walls	#6 at 12” V EF, #6 at 12” H EF, 12” thick wall	0.0061V, 0.0061H	0.003V, 0.006H	204%V, 102%H
Slab	#8 at 12” EF, 12” thick	0.011	0.005	219%

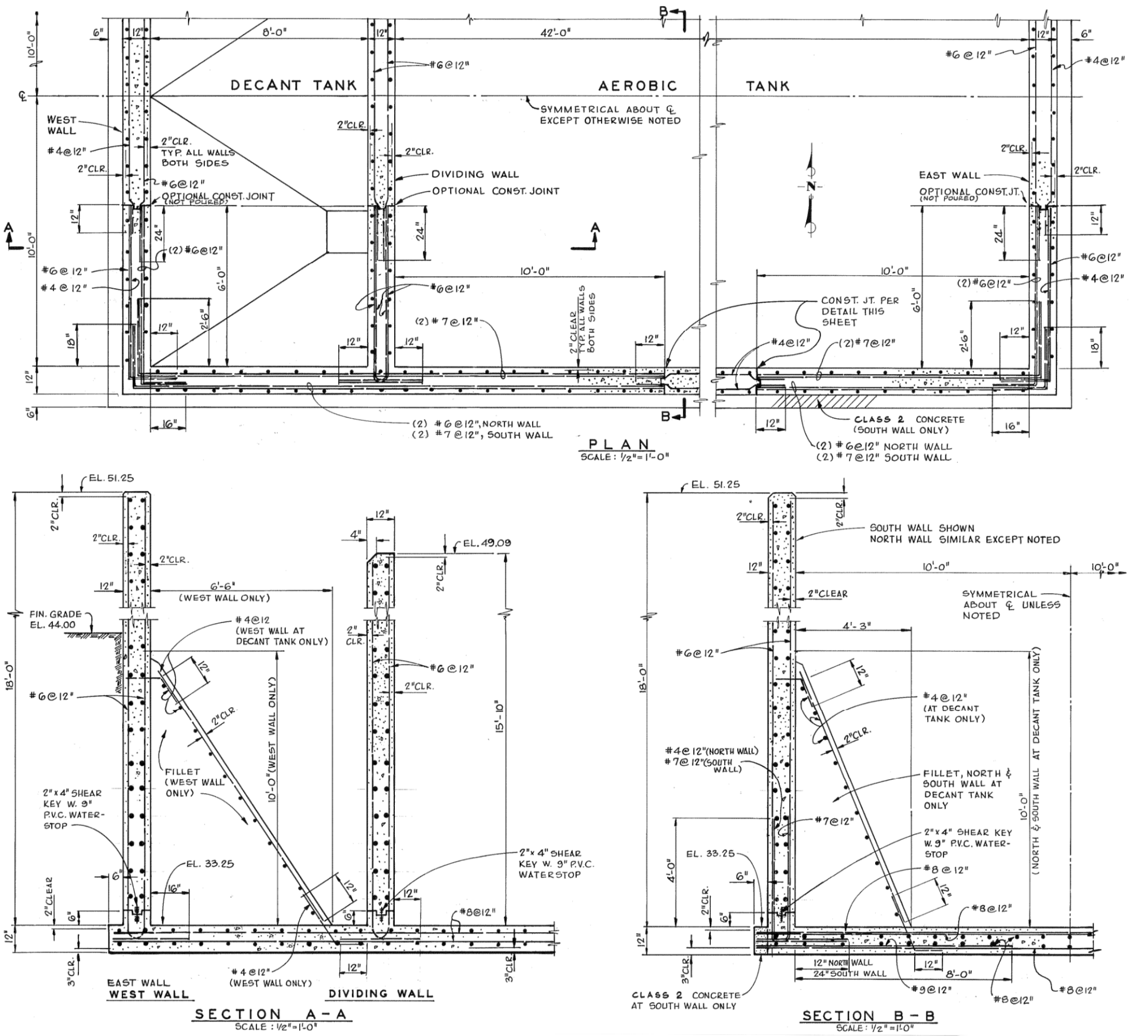

Figure 14

Observation/Condition Assessment

Concrete of the south section of the Aerobic Digester was observed and sampled on August 28, 2024. Samples were taken from two walls in the sections above grade. Cores were taken in the slab in two locations.

Walls: Generally, the walls were in good condition with some cracks. The quantity of cracks was minimal when compared to the other concrete structures.

Slab: During the observation, the bottom of the slab had dry sludge making the entirety of the slab difficult to observe. The areas that were exposed were in generally good condition with minimal to no cracking.

The vertical crack plane that was present in the other concrete structures was not identified in the Aerobic Digester walls or slab.

Figure 15

Testing Result Summary

Concrete Compression Strength: 6,280 psi – 6,930 psi

Chloride Ion Content: Not Elevated, Unlikely to have detrimental effects.

Petrographic Examination: Test Result Pending

2.5. RAS/WAS Pump Room

The RAS/WAS pump room consists of concrete retaining walls that were built as part of the 1982 expansion with Clarifiers 3 and 4. The south wall is common to Clarifier 4. The entry to the area is via concrete stairs at the north wall. There is a metal roof canopy covering the structure to protect from excessive water intrusion. The pumps are set on concrete housekeeping pads and are anchored.

Observation/Condition Assessment

The walls of the RAS/WAS were not sampled for testing, but concrete composition is anticipated to be similar to the results from Clarifier 4.

Walls: The south wall has multiple horizontal cracks. The largest is 0.025 to 0.03-inches in width. No moisture was noticed coming through the cracks.

The north, east, and west walls also have random surface cracks. The cracks are hairline to 0.02-inches in thickness.

Slab: The floor slab has some cracks but are not significant in width.

Metal roof cover: The roof cover is generally in good condition. The bearing seat of the aluminum beam has some corrosion which should be cleaned and painted to reduce future section loss in the steel. Neoprene should be placed between the aluminum and steel to prevent future corrosion of the dissimilar metals.

2.6. Wet Well and Dry Well and Pump Room

The wet well, dry well, and pump room are located below grade under the workshop of the control building. This buried structure was part of the original plant construction. The first level below grade is the pump room which is a 7-inch concrete slab supported by concrete beams. The levels below are the wet well and dry well which are supported on a 12-inch slab on grade. The exterior retaining walls are 12-inch concrete walls. The interior of the walls are painted.

Observation/Condition Assessment

Cores in the west wall and the slab on grade of the dry well were cored for concrete testing.

Pump Room Slab: The slab is generally in good condition with minimal cracks.

Slab on grade: Generally, the slab in good condition with some cracks. Coating on the floor requires replacement in the dry well and wet well.

Walls: Cracks in the walls are frequent and measure from hairline to 0.04-inches in width. During coring of the wall, a similar vertical crack plane was identified. In the dry well there is deteriorating concrete where the pump piping penetrates the north wall.

Testing Result Summary

Concrete Compression Strength: 4,800 psi – 10,640 psi

Chloride Ion Content: Not Elevated, Unlikely to have detrimental effects.

Petrographic Examination: Alkali-silica reaction present

2.7. In-Ground Structure Summary and Recommendations

The presence of the vertical crack plane in the concrete walls and the regularity in which it occurs is concerning when considering the future service life of the existing buried and liquid containing structures. This crack was present in all buried structures except for the Aerobic Digester. From a structural perspective, this crack is problematic because the wall section no longer acts as one continuous element. Rather the forces are resisted by two thin walls. Any composite action between the two sections relies on aggregate interlock across the crack plane. Significant failure has likely not occurred because the stresses on the structure are balanced by the external retaining pressure and the internal pressure from the contents.

Operationally the quantity and width of the cracks in the walls poses a challenge as the structures are liquid containing. As the cracks get larger there is potential for liquid migration through the walls which may potentially corrode the reinforcing over time. Seismic induced ground motion would also be detrimental to the walls and may cause a failure leaving one or more structures unusable.

While the cracks along the walkways and tops of the tank structures are not as big of a structural concern as the walls, they are a symptom of the underlying issue of the concrete's chemistry. Test results have indicated that there is a strong presence of Alkali-Silica Reactivity (ASR). Once ASR is present in concrete it cannot be remediated. Test results show that there is reactive aggregate remaining in the concrete and with the available moisture from the liquid containing structures has a high probability of continuous reaction. Additional reactions will increase expansion and cracking in the concrete leading to potential failure.

While there has not been a failure in terms of a partial collapse or noticeable loss of tank contents, the cracking is indicative that the in-ground tank structures are at the end of their Technical Service Life. The recommendation for the future service life of the Aeration Basins, Clarifiers, Chlorine Contact Tank, RAS/WAS Pump Pit and Dry and Wet Wells is full replacement of the existing structures.

An alternative to full replacement is to selectively demolish portions of the existing structure and cast new concrete slabs and walls inside of the existing structures. This would allow the existing piping to the plant to remain in operation. To keep the plant operational during construction, consideration of sequencing would be required which may extend the construction timeframe for full rehabilitation of the plant. See Figure 17 for conceptual plan of structural rehabilitation.

The Aerobic Digester is in good condition. It is recommended to add a water-proofing coating to the surface of the Aerobic Digester as an additional protection to the concrete surface and reinforcing. Testing for ASR is scheduled for this structure.

Rough order of magnitude of cost for rehabilitation as show in Figure 17: $4,000,000

Scope of work included in rough order of magnitude cost: Demolition of existing concrete, new concrete slabs, walls, and walkways.

Scope not included: Modification of existing piping or addition of new piping and mechanical equipment (pumps, blowers, etc.). Modification to existing electrical.

3. Buildings

3.1. Control Building and Offices

The control building and office was part of the original plant construction in 1961. A generator room and chemical storage room were added during the 1982 expansion. In 1988 an office was added to the south end of the building. The entire building, except the pump room and wells, are supported on shallow continuous footing foundations and slab on grade.

The original building was built with full height partially grouted CMU block, partial height precast tilt-up concrete walls and cast-in-place concrete columns. The roof is framed with poured in place gypsum supported on steel bulb tees which span to steel wide flange beams.

The 1982 addition consists of fully grouted CMU block walls that enclosed areas under the existing roof framing.

The 1988 addition consists of all light framed construction. The roof is framed by plywood sheathing on 2x rafters which are supported by wood stud walls. The walls are sheathed with 3/8-inch plywood sheathing which act as shear walls.

Observation/Condition Assessment

Generally, the building is in good condition. There are some minor cracks in the concrete, but nothing of major structural concern.

The north wall of the shop and the east wall of the entire building consists of 5.5-inch pre-cast concrete walls that extend to 8-feet above the finish floor. The walls do not attach to the roof diaphragm.

ASCE 41 Tier 1&2 Analysis Summary

While there are two lines of lateral systems in each direction, the clerestory windows above the precast panel do not allow for diaphragm shear transfer of lateral loads to the concrete wall. The configuration of the CMU lateral system results in an offset in the buildings center of mass and center of rigidity which results in an extreme torsional irregularity. This irregularity may result in failure of the 10-inch square concrete columns at the north and east walls when subject to seismic loading and displacement.

The horizontal and total reinforcing in the CMU walls does not meet the minimum requirements. The vertical reinforcing alone is adequate along with the shear stress check of Section 5.5.3.1.3 of the ASCE 41.

The anchorage of the CMU walls is inadequate for out of plane wall loads for the eave wall condition. The current attachment relies on torsion of the wide flange beams to support them for out of plane loads. The limited horizontal reinforcing in the CMU walls does not allow the CMU walls to span to perpendicular walls. ‘

See Appendix C for the ASCE 41 Tier 1 and Tier 2 analysis.

Risk Analysis and Economic Service Life

Current codes define the Maximum Considered Earthquake (MCE) as the most severe earthquake that may be experienced by a site. It has a probability of exceedance of 2% in 50 years, or an average return interval of 2475 years. Buildings designed to current codes are designed for an event that has an intensity of two-thirds that of the MCE and an average return period of roughly 500 years.

The FEMA P-58 risk analysis performed by Reis Consulting indicates that the deficiencies could lead to a collapse risk greater than that of a building designed and built to today's codes and standards by comparison. A building designed to modern codes would have a collapse risk of 0.2% in 50 years, while the Controls Room and Offices Building accumulates the same collapse risk of 0.2% in 2.8 years.

If the building is not seismically retrofitted, the damage estimated repair cost as a percentage of building replacement cost for a 500-year event is 28%.

Recommendations

To comply with current building codes, a lateral system should be added to exterior wall lines at the north, south, and east walls that have a direct attachment to the diaphragm. This could be done by infilling the clerestory windows over the existing precast panel in one or two wall panels on each line or by adding a steel frame, either interior or exterior to the building structure.

While the vertical reinforcement and partially grouted CMU is adequate for the shear stress check, the minimum horizontal wall reinforcing requirement is not met. Fiber Reinforced Polymer can be applied to the face of the CMU to add additional strength.

To provide adequate CMU wall anchorage, it is recommended to add out of plane ties for the CMU walls. This can be done by adding epoxy anchors to the tops of the walls and steel plate attachments to the diaphragm. Steel waler beams can also be used to brace the tops of the walls. See Figure 19 for a conceptual rehabilitation plan.

Several piping and distribution systems are unbraced for lateral loads. It is recommended to add bracing to distribution systems back to the building.

Rough order of magnitude of cost for rehabilitation: $1,000,000

Scope of work: Addition of lateral systems, reinforce CMU with FRP, add wall ties, brace distribution systems.

A seismic retrofit with the recommendations would bring the building's risk within the current code level risk of collapse, 0.2% in 50 years. With proper maintenance, the technical life of the building would be that of a new building.

3.2. Digester Blower Building

The blower building is constructed of CMU Walls and a wood framed roof. The CMU walls are fully grouted and are reinforced with #5 bars @ 16-in on-center vertical and #4 bars @ 16" on-center horizontal. The wood framed roof consists of open web trusses and a plywood sheathed diaphragm. Wood blocking is provided at unsupported sheathing edges to form a blocked diaphragm. The diaphragm is fastened to the CMU walls via 10d nails @ 2" on-center to a wood nailer and fastened to the roof framing via 6d nails @ 3" on-center.

The structure houses several pieces of large electrical equipment. It was observed on site that none of the exterior doors contain panic hardware. Per building code requirements, any exit door from a room with equipment rated 800-amperes or more shall be equipped with panic hardware or fire-exit hardware.

Observation

The existing structure is generally in good condition. There are no major cracks or indications of deterioration.

ASCE 41 Tier 1&2 Analysis Summary

No adequate out-of-plane lateral roof ties were able to be discerned in the field nor in the as-built plans. Additionally, no attachment of the CMU walls to the concrete slab of the building is apparent in the as-built plans. The in-plane capacity and diaphragm capacity of the structure is adequate.

Risk Analysis and Economic Service Life

The risk analysis performed by Reis Consulting indicates that the deficiencies could lead to a collapse risk greater than that of a building designed and built to today's codes and standards by comparison. A building designed to modern codes would have a collapse risk of 0.2% in 50 years, while the Blower Building has a collapse risk of 0.2% in 4.5 years.

Recommendations

To provide adequate CMU wall anchorage, it is recommended to add out of plane ties for the CMU walls and cross ties to the diaphragm.

For fire safety it is recommended to add panic hardware on the exit doors.

Rough order of magnitude of cost for rehabilitation: $50,000

Scope of work: Add wall ties and cross ties.

A seismic retrofit with the recommendations would bring the building's risk within the current code level risk of collapse, 0.2% in 50 years. With proper maintenance, the technical life of the building would be that of a new building.

3.3. Steel Canopy Structures

Three (3) total canopy structures were observed and documented as a part of this assessment. They consist of two shorter canopy structures to the east which serve to shelter parked vehicles and stored chemicals, and a single taller canopy structure to the west which serves to shelter the belt-press and sludge dewatering system.

The steel canopy structures consist of cold formed steel roof purlins and hot-rolled steel columns and beams. The structures' lateral system includes wide flange moment frames comprised of wide-flange columns and beams spaced at approximately 24-feet on-center in the transverse direction. The two eastern canopies' lateral system in the longitudinal direction consists of what would appear to be a single moment frame located on one side of the structure. The single moment frame consists of a cold-formed steel "strut" to stiffen the structure. The structure contains horizontal tension rod bracing within the roof depth of the structure to act as the structure's diaphragm.

Observation

It was observed that several of the anchor bolts anchoring the columns to the foundation are either exposed or buried in soil and several anchor bolts are corroded. To prevent further corrosive action and provide protection of the structural system, it is recommended that the column bases and anchor bolts be encased in concrete with a minimum of 3" of concrete cover around the anchor bolts.

Much of the steel framing of the two steel canopy structures are corroding and exhibit signs of chemical exposure, likely due to the proximity of the process chemicals that are being stored and being located in a marine environment. It is recommended that any steel framing that shows extensive signs of structure section loss to be replaced in kind, and all structural steel framing be painted with a fresh coat of paint.

ASCE 41 Tier 1&2 Analysis Summary

The larger canopy structure to the west appears to have been constructed more recently than the shorter canopy structures to the east. It was observed that there is minimal structural separation between the newer canopy structure and the older canopy structure, and the structures appear to be touching in some instances.

The pad footings are not interconnected with ties and are not restrained by a slab on grade. Tier 2 evaluation has indicated that the pad footings are acceptable for vertical and lateral resistance of anticipated loads.

The lateral system of the longitudinal direction of the north and south canopies consists of cold rolled channel material. These elements are not adequate to resist design level seismic loads. The lateral system of the transverse direction meets the demand load of the Tier 1 analysis.

Recommendations

To prevent further corrosion and protect the anchor bolts, expose buried anchorage and protect with concrete or reduce the finish grade around the anchorage.

To ensure adequacy to resist vertical loads from self-weight and roof live loads, replace corroded members and paint structures.

Add seismic gap or tie the taller canopy to the shorter canopy to prevent racking during an earthquake.

Add two lateral system frames (braced frame or portal frame) in the longitudinal direction (East-West) for each of the North and South Canopies. See Figure 20 for conceptual rehabilitation layout.

Rough order of magnitude of cost for rehabilitation: $100,000

4. Non-Structural Components

The following is a list of the observed deficiencies from a Tier 1 analysis in the anchorage and support of non-structural components. Generally, all equipment (pumps, generator, blowers, electrical gear, etc.) observed was bolted to a concrete structure. The most common issue encountered was a lack of bracing in piping and distribution systems.

Aeration Basin:
- Corrosion in the steel plate at the weir at west walls

Clarifier:
- The base plates of the telescoping valves are corroded.
- The base plate of the light pole at the north end is corroded.

Chlorine Contact:
- No non-structural deficiencies noted.

Aerobic Digester:
- Piping from aerobic digester to site wall does not have flexible coupling. There is a flexible coupling in from the pipe at the blower building.

RAS/WAS:
- Piping extending up from pumps is not braced back to pump pit walls.
- Bearing supports of roof beam is corroding.

Controls Building / Office:
- Shop
    - Lights at north wall are not braced and cannot swing 45 degrees without hitting the wall.
    - Suspended strut supports for conduit are not braced.
    - The 10” pipe at the east wall running north/south has vertical supports but is missing a strap that will resist lateral forces.

- Blower Room
  - The piping off of the blowers is vertically supported at the roof, but not laterally braced.
- Influent Pump Room
  - Overhead ducts and pipes are vertically supported, but not laterally braced.
- Dry well
  - Overhead piping is vertically supported at the slab, but not laterally braced.
- Wet Well
  - Most of the metals in the room have corrosion and staining.

Digester Blower Building:
- Piping is not braced to the roof.

South Canopy: - No non-structural deficiencies noted.

North Canopy: - No non-structural deficiencies noted.

West Canopy: - No non-structural deficiencies noted.

MONTECITO SANITARY DISTRICT - SITE PLAN

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Figure 16

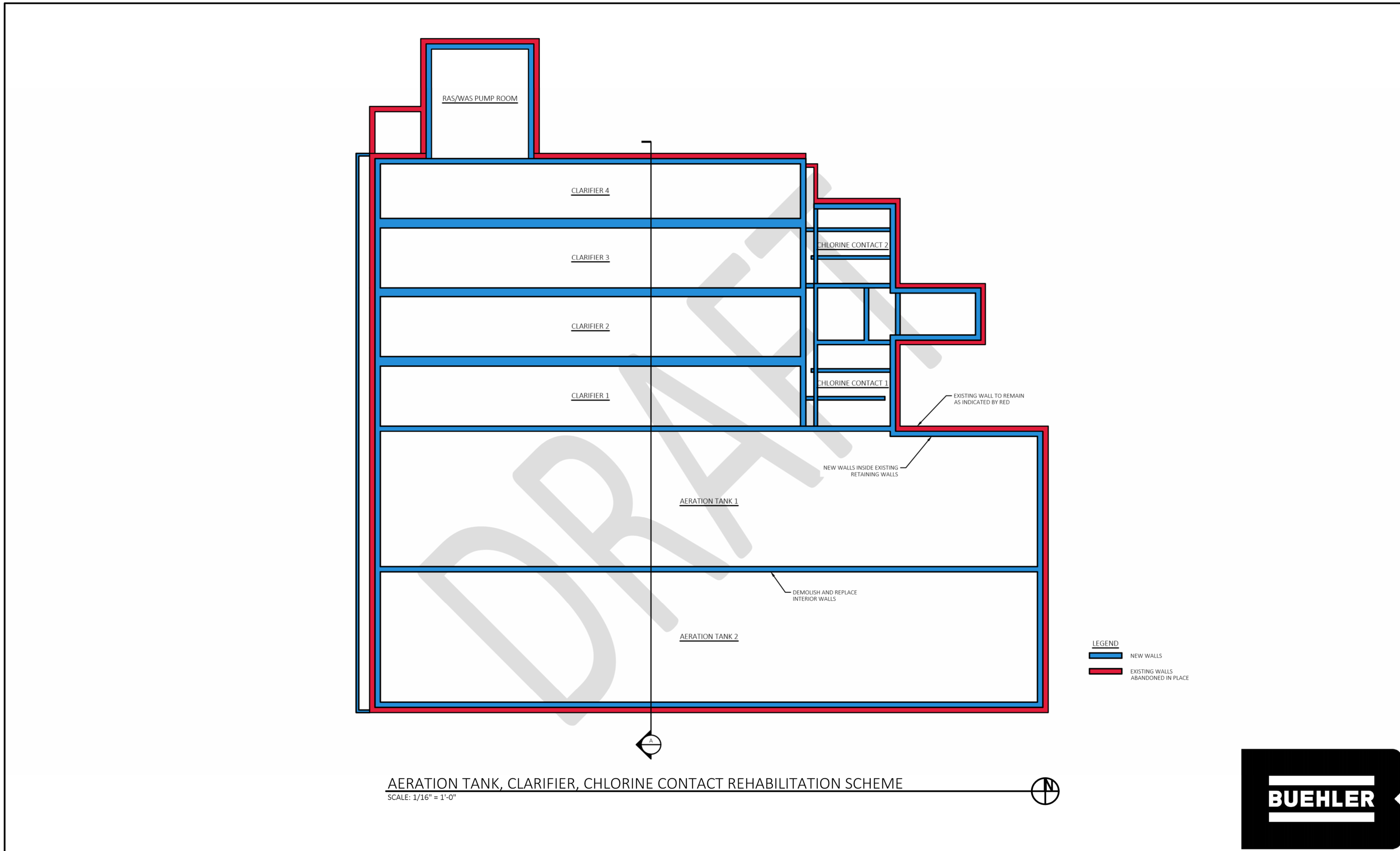
RAS/WAS PUMP ROOM
CLARIFIER 4
CLARIFIER 3
CLARIFIER 2
CLARIFIER 1
CHLORINE CONTACT 2
CHLORINE CONTACT 1
EXISTING WALL TO REMAIN AS INDICATED BY RED
NEW WALLS INSIDE EXISTING RETAINING WALLS
AERATION TANK 1
DEMOLISH AND REPLACE INTERIOR WALLS
AERATION TANK 2
LEGEND
NEW WALLS
EXISTING WALLS ABANDONED IN PLACE
DRAFT
AERATION TANK, CLARIFIER, CHLORINE CONTACT REHABILITATION SCHEME
SCALE: 1/16" = 1'-0"
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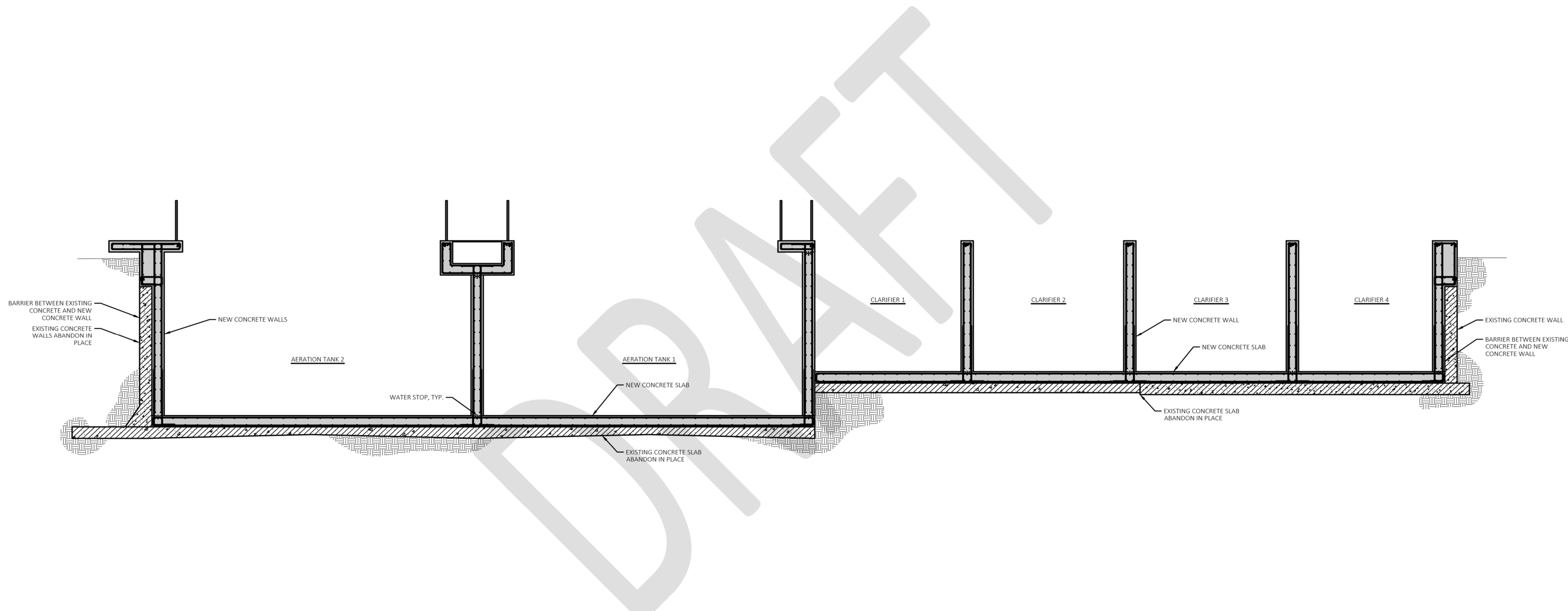
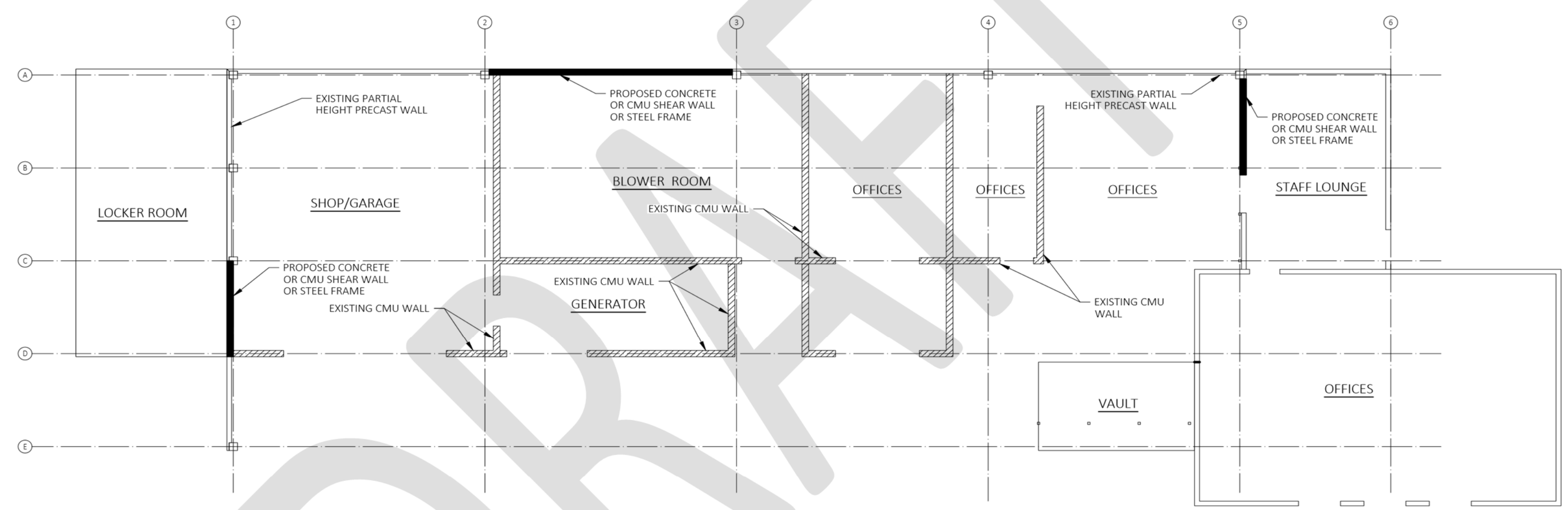
Figure 17



SECTION A: AERATION TANK, CLARIFIER, CHLORINE CONTACT REHABILITATION
SCALE: 1/16" = 1'-0"

Figure 18



CONTROLS BUILDING AND OFFICE CONCEPTUAL REHABILITATION PLAN
SCALE: 1/16" = 1'-0"

Figure 19

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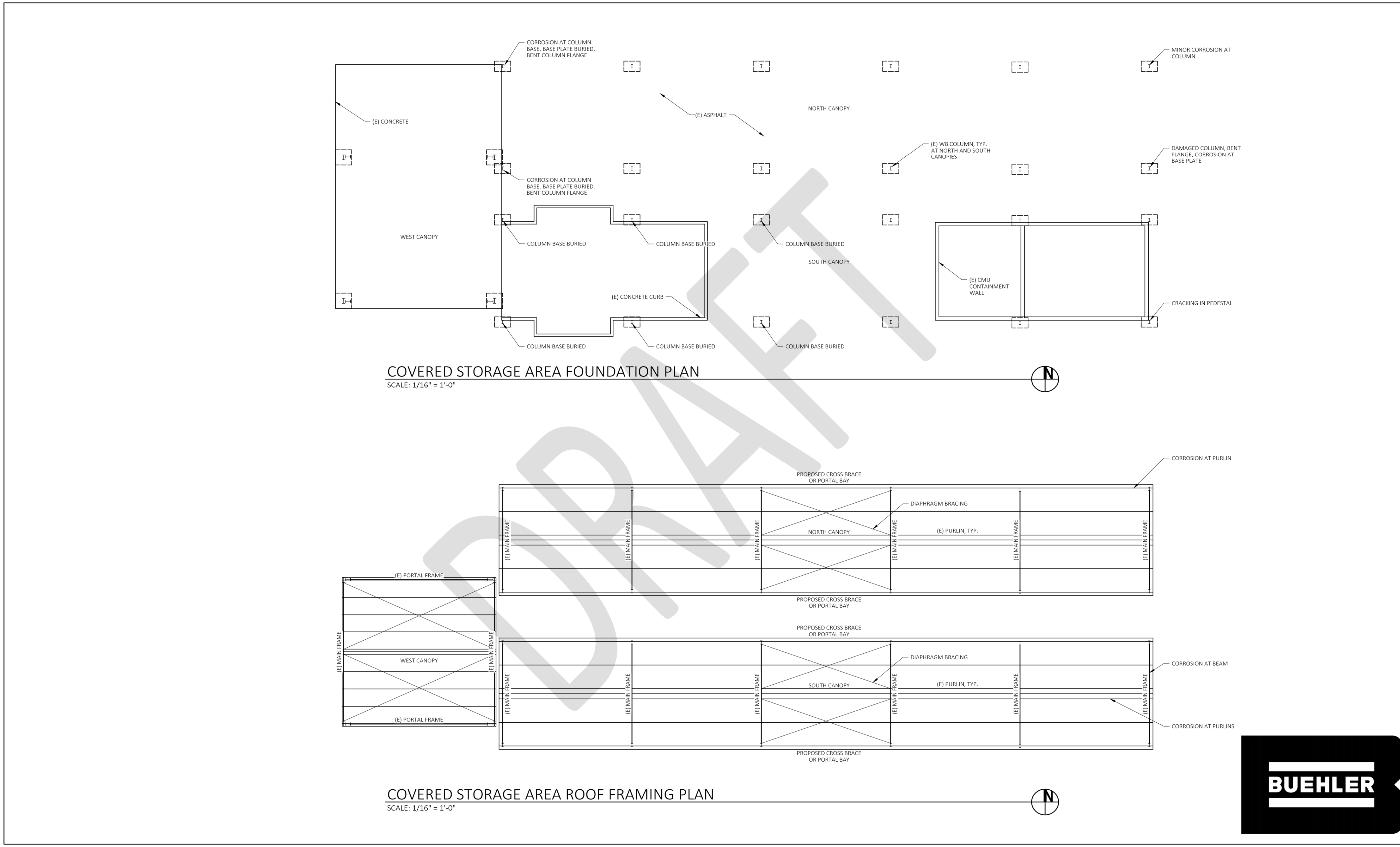

Figure 20

Experience and Qualifications of Buehler Engineering, Inc.

Buehler Engineering, Inc. was founded in 1946 under the original name of Walter A. Buehler, Structural Engineer. The firm has been engaged in structural design of a wide variety of projects over the life of the firm. The firm currently has a total staff of 142, including 55 registered structural engineers. The firm maintains computer facilities for the analysis and design of engineering structures. Engineering services are provided for the design and analysis of building and other structures and for structural investigations.

Limitations

The services of Buehler Engineering, Inc. performed for this project have been provided at a level that is consistent with the general level of skill and care ordinarily provided by engineers practicing in structural engineering. Sketches are schematic in nature for general cost estimating purposes. Work is necessarily done under the constraints of time and budget. The conclusions and information presented in this report are dependent on information provided by others. No warranty is expressed or implied.

Submitted:

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Michael Parolini, S.E.
Principal
For Buehler Engineering, Inc.